MULTIPLE EARTHQUAKE EFFECTS ON DEGRADING REINFORCED CONCRETE STRUCTURES

BY

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DISSERTATION

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ABSTRACT

Multiple earthquakes occur at many regions around the world where complex fault systems exist. These fault systems usually do not relieve all accumulated strains at once when the first rupture takes place. Therefore high stresses form at different locations causing sequential ruptures until the fault system is completely stabilized. The sequential ruptures along the fault segment(s) lead to multiple earthquakes which are often hard to distinguish them as fore-, main- and after-shocks, or a sequence of earthquakes from proximate fault segments.

Field investigations reported failure of structural systems under repeated earthquakes, especially where structural retrofitting was not provided due to the short time frames between the successive shaking. In most failure cases the reported damage is mainly due to dramatic loss of stiffness and strength of structural elements as a result of material deterioration under repeated earthquake loadings. Deterioration effects are obvious in structures that experienced main-shock aftershock earthquake sequence and were able to withstand the main-shock however they collapsed in the smaller aftershock.

Limited research has addressed the seismic behavior of structures subjected to multiple earthquakes. Repeated shaking induces accumulated damage to structures that affects their level of stiffness and strength and hence their response. Given the complexity of depicting the degrading behavior of structures using the current numerical tools, previous researchers used simplified approaches to compensate for the absence of important numerical model features of stiffness and strength degradation, alongside pinching of load-displacement loops. Moreover ground motion sequences used in previous studies were randomized and hence the characteristics of ground motions effects on the response
were not accurately accounted for. Findings from previous research indicated that repeated shaking has a minimal effect on the response of structures in terms of peak displacements, maximum base shear and period elongation and hence it can be neglected for seismic evaluation of structures if the most damaging earthquake is to be considered. This research re-investigates the behavior of reinforced concrete frame systems under multiple earthquakes. The aforementioned damage features are modeled on the material level by using a plastic energy-based degrading concrete model and a steel model that considers reinforcing bars deterioration under large cyclic amplitude plastic excursions. Structural models of reinforced concrete degrading systems are subjected to selected earthquake sequence scenarios. Ground motion characteristics of individual records within the sequence, such as peak ground accelerations, predominant periods, and durations as well as the order of records application in the sequence, are parameterized and their effect on the response is monitored. Finally the effect of multiple earthquakes on current design guidelines is investigated and modifications are proposed accordingly. The case for developing design and assessment methodologies for structures to more than one earthquake is emphasized. The results presented in this study clearly indicate that the response of degrading structural systems is appreciably influenced by strong-motion sequences in a manner that cannot be predicted from simple analysis. It also confirmed that previous research that dismissed the effect of multiple earthquakes lacked the salient modeling features, and that including appropriate degrading constitutive relationships leads to reversing previous recommendations. The effect of multiple earthquakes on earthquake safety can be very considerable.
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CHAPTER 1 INTRODUCTION

1.1 Overview and Problem Statement

Reinforced concrete structures are vulnerable to multiple earthquake excitations. Researchers have focused mainly on the seismic vulnerability of structures under the most damaging earthquake, and hence neglected the effects of prior shaking. These effects include stiffness and strength degradation due to damage accumulation in construction materials under large amplitude cyclic excursions as well as P-Δ effects that are introduced due to residual displacements induced from previous shaking. The stiffness and strength degradation significantly affect the dynamic characteristics of damaged structures, and hence their response under subsequent earthquakes. Figure 1-1 shows damaged buildings after the Kobe earthquake main shock in 1995. The dynamic characteristics of the shown buildings as well as their strength capacities are unknown; therefore their resistivity to a small subsequent aftershock is not predictable.

Figure 1-1: Damaged buildings after the Kobe earthquake in Japan.
Earthquakes usually occur as a cluster in many regions around the world where complex fault systems exist. This is mainly due to that the initial rupture, which causes the first earthquake, does not usually relieve all accumulated strains; therefore high stresses at different locations at fault system keep on forming. The stresses cause sequential ruptures that decays when the fault system is completely stabilized; this phenomena leads to multiple earthquakes.

Multiple earthquakes are defined as: (1) fore-, main-, and after-shocks; or (2) sequence of independent earthquakes from proximate fault segments. An aftershock is a smaller earthquake that occurs after a previous large earthquake, within the same area of the main shock. If an aftershock is larger than the main shock, the aftershock is re-designated as the main shock and the original main shock is re-designated as a foreshock (Figure 1-2). Aftershocks and the main shock epicenters are located over the full area of fault rupture and they either occur along the fault plane itself or along other faults within the volume affected by the strain associated with their fore- and main shocks.

Typically, aftershocks are found up to a distance equal to the rupture length away from the fault plane (McGuire et al., 2005). Aftershocks have high damage potential to structures because: (1) they cannot be predicted in terms of their location (distance from source to affected structures), triggering, and energy content; moreover (2) they strike damaged structures of lower stiffness and strength capacities. Strong aftershocks can occur at any time after their main shocks. In the great Tohoku earthquake, a magnitude 7.9 aftershock occurred less than one hour after the magnitude 9.0 main shock. In Chile an aftershock of M7.2 took place 12 days later, while in New Zealand an aftershock of magnitude 6.3 occurred more than 4 months later than the main shock.
In an earthquake sequence, individual earthquakes generated from proximate fault segments that are not affected by strains associated with the generation of their counterparts are usually categorized as independent earthquakes. Independent earthquakes might have close rupture locations in a way that they might strike the same built environment within the seismic zone one after the other. The August 17 Kocaeli (M$_w$ 7.4) and November 12 Duzce (M$_w$ 7.2) Earthquakes in Turkey are good examples of independent earthquakes that are generated from two different fault segments, namely the western extension and eastern end of the North Anatolian fault system as shown in Figure 1-3 (Erdik, 2000; and Braunmiller et al., 2002).
Field investigations had reported the collapse of buildings under repeated earthquakes. The failure in many cases is due to the loss of stiffness and strength of structural systems that resulted from damage accumulation due to repeated shaking. This failure scenario is obvious for buildings that stayed intact during large main-shocks and collapsed days later in smaller aftershocks as shown in Figure 1-4. In the most recent earthquakes, repeated shaking was observed and failure of many structures in the manner described above was reported. These earthquakes include the Umbria-Marche (Italy 1997), Kocaeli and Duzce (Turkey 1999), Chile (2012), Christchurch (New Zealand 2011 and 2012) and Tohoku (Japan 2011 and 2012) earthquake sequences.
In the Umbria-Marche earthquakes, a sequence of seismic events took place at Central Italy on September the 26th 1997. The sequence included two main shocks of magnitudes $M_s$ 5.5 and 5.9. These main shocks occurred at times 00:33 and 09:42 GMT. A large aftershock of magnitude $M_s$ 5.5 occurred later on October 14 (Figure 1-5).

During the Umbria-Marche seismic sequences many buildings withstood the first shock and collapsed in subsequent shaking. Examples of such buildings are the Basilica di San Francesco Church and the Foligno Tower. The Basilica di San Francesco had its roof collapsed during the second main shock (at 09:42). The roof after the first and second shocks is shown in Figure 1-6. Four priests were killed by the falling rubble from the roof due to the second shock while they were inspecting the damage caused by the first shock.
The Foligno Tower is another famous structure that collapsed as a result of repeated shaking during the Marche-Umbria earthquake sequence. Unlike the Basilica of San Francesco, this tower withstood the first and second main shocks on September 26. The top part of the tower collapsed a couple of weeks later due to the aftershock that occurred on October 14 (Figure 1-7).

Figure 1-5: Affected area for the 1997 Umbria-Marche earthquake sequences. Epicenter locations of earthquakes are marked by stars (Prete et al., 1998).
On August 17, 1999 an earthquake of magnitude $M_w$ 7.4 struck Kocaeli and Sakarya provinces in northwestern Turkey. A few months later, Duzce earthquake ($M_w$ 7.2) occurred on November 12. The two earthquakes are generated from fault segments of the
North Anatolian fault (Edrik et al. 1999). Figure 1-3 shows the epicenter location of both earthquakes as well as their aftershocks.

Many buildings collapsed during Duzce earthquake (second earthquake on November 12), many of these buildings were previously damaged during the Kocaeli earthquake. Figure 1-8 shows a building that was tagged after the Kocaeli earthquake and collapsed during the Duzce earthquake.

During the February 27, 2010 earthquake in Chile many aftershocks of large magnitude were generated after the main shock (Mw 8.8). The aftershocks had significantly affected the damaged structures, however there is no information of structures that survived the main shock and collapsed during aftershocks in this study. Figure 1-9 shows the number of aftershocks of high magnitudes that hit Chile within a three day time frame after the main shock had took place.
The ongoing Christchurch earthquake sequence in New Zealand started on September 2010 by the Darfield main shock (Mw 7.1). This earthquake is generated 40 km from the highly populated Christchurch city. The following aftershock on February 2011 (Mw 6.2) was more devastating since it was generated directly under Christchurch; in addition it struck initially damaged structures after the September 2010 main shock. Significant damage has been reported and many buildings collapsed in the aftershock. The death toll was more than 180 persons. Less damage was reported by later aftershocks on June (Mw 6.0) and December (Mw 6.0) 2011 respectively (Kaiser et al., 2012).

Figure 1-9: Main shock and aftershocks time and magnitude in Chile.

Figure 1-10: The Christchurch earthquake sequence and magnitude.
The recent March 11, 2011 Tohoku earthquake in Japan (M$_w$ 9.0) was followed by significant number of large magnitude aftershocks (Figure 1-12). An aftershock of M$_w$ 7.4 occurred less than a one month later (on April 7), caused the collapse of many buildings, mainly in Sendai city. One of the houses in this city (shown in Figure 1-13) did not suffer any damage during the March 11 main shock and collapsed as shown in the April 7 aftershock.
As discussed by showing evidence, damage and collapse of structures resulted from the continuous accumulation of damage when these structures are subjected to multiple earthquakes. Current design codes for buildings and bridges do not consider multiple earthquake effects in design and assessment. Moreover this issue has been either neglected or not properly considered by previous researchers due to the complexity of developing and implementing degrading models. It is worth mentioning that current numerical tools that are capable of performing inelastic non-linear dynamic analyses of reinforced concrete structures (such as ABAQUS, OpenSees, and ANSYS) do not include the degrading features required to simulate the actual behavior under multiple earthquakes. Therefore, establishment of appropriate models is sought in order to complete this study. The lack of existence of a comprehensive literature that covers this
topic has raised difficulties in establishing a solid starting point to pursue from what was conducted previously.

1.2 Objectives and Scope of Research

Prior work aimed at investigating the response of structural systems subjected to multiple earthquakes has been conducted using simplified approaches. Numerical models that have been used in literature were either system level or component level based models. Studies based on the system level assessed the behavior of SDOF systems that incorporate a degrading hysteretic force-displacement relationship. For the component level based models, MDOF frame systems have been studied by utilizing moment-rotation relationships characterizing the behavior of plastic hinges, developed at beam-to-column connections. Assessment of structures using idealized system and component level models leads to inaccurate assessment of the degrading response under repeated earthquakes; since many features are not precisely represented. The lacking features in the system level models include higher modes effects, localized failure behavior, and actions redistribution between structural components or assembly of components. While for the component level models, the pre-specification of plastic hinge locations does not represent localized deformations in terms of plastic hinge length, yielding and buckling of steel, and crushing of concrete. Moreover, prior research lacked proper selection criteria of earthquake ground motion sequences.

To make a near-fully realistic assessment of the demands upon and performance of reinforced concrete structures subjected to repeated seismic loadings. A material level based model, as opposed to prior system and component based models, is established and
used in conducting the assessment. The main challenge of establishing a degrading material level based model is that, in all existing analysis tools proper damage features of concrete and steel materials are not implements. This is due to the complexity of implementing these features in the material models because they are computationally expensive and usually cause convergence problems when subjected to complex dynamic loading. Therefore, in order to complete this study, implementation of these complicated models was sought. The convergence of these models under complex, large amplitude earthquake loading conditions is examined; and optimization subroutines are utilized in the models to save their analysis time and avoid any divergence that might occur during the time integration.

Also, in this study representative ground motion sequences are selected from previous earthquake sequences that have been experienced recently and their ground motions have been recorded. Replicate and random motions are also used to facilitate the comparison between non-degrading and degrading models.

In addition to establishing a material level based model that depicts the degrading behavior of reinforced concrete structures and selecting representative set of earthquake sequences, the scope of this work is extended to conducting a parametric study on the response of reinforced concrete frame systems using three different design approaches. The results of the parametric study are used to accurately predict the response of reinforced concrete systems of different code-based design parameters, under repeated earthquakes. In addition, implications to design and assessment of reinforced concrete buildings in regions prone to multiple earthquake excitations is emphasized. The framework of this study is illustrated in Figure 1-14.
Figure 1-14: Proposed approach for the seismic assessment of reinforced concrete frames subjected to multiple earthquakes.

Realizing the above objective requires the following tasks and subtasks to be accomplished:

- **Task1: Conduct Comprehensive Literature Review**
  - Emphasize on the drawbacks in models simplification assumptions and ground motion sequences selection criteria
  - Discuss the conclusions drawn from previous work that contradict field investigation reports of collapsed structures under small aftershocks, however they withstood the bigger aftershocks

- **Task2: Select Representative Sets of Ground Motion Sequences**
Select synthetic and natural records to be used for the replicate motion case, records cover a wide range of input motion parameters such as PGA, frequency content, duration, and site conditions.

Select realistic earthquake sequences records recorded from previous earthquakes (e.g. Tohoku and Christchurch earthquake sequences).

**Task 3: Select and Implement Degrading Material Models**

- Select previously developed models of steel and concrete materials that capture accurately the degrading response of both materials under large amplitude repeated plastic excursions and are experimentally verified.
- Understand Zeus-NL source code skeleton and subroutines in order to implement new complicated degrading material models.
- Implement the concrete and steel material models in Zeus-NL software in separate reader and solver .exe files and examine their performance under simple axial cyclic loading.
- Develop optimized convergence criteria for the complicated material models to save computational costs during the expensive inelastic non-linear dynamic analyses.
- Examine the response of the material models under cycling bending actions and verify the response using closed form solutions.
- Examine the material models under large excursions and check their convergence using simple models.
– Compile both models in one reader and one solver executive files and check for any convergence problems under high amplitude displacements and during their softening behavior (under negative stiffness)

– Check the convergence when using a complicated frame model subjected to long duration, and high PGA earthquake record

• **Task4: Establish Simple Numerical Models**

  – Develop a simple model of a reinforced concrete pier with a lumped mass at the top
  – Use the existing non-degrading materials to model the pier (non-degrading pier model)
  – Utilize the newly implemented degrading material models (degrading pier model)
  – Conduct dynamic analyses of these piers under replicate earthquake motions using the artificial and natural records selected in task2

• **Task5: Interpret the Results**

  – Perform comparison of the non-degrading and degrading pier models results under the first earthquake
  – Investigate the differences in pier models response under the second earthquake (damaged case)
  – Highlight the effect of damage accumulation on the response based on the above comparisons

• **Task6: Establish Numerical Models of Different Design-Based Reinforced Concrete Frames**
– Design reinforced concrete frame systems using different design approaches
– Establish numerical models of these frames taking into consideration reinforcement detailing
– Conduct pushover analyses of frame systems to determine the frames stiffness, strength, and ductility

• **Task7: Conduct Analytical Investigation of Frame Response**
  – Perform dynamic analyses under replicate motions
  – Perform dynamic analysis under realistic recorded earthquake sequences
  – Monitor the response in terms of period elongation in each subsequent earthquake as well as development of plastic hinges, and inter-story drifts

• **Task8: Investigate the Implication of the Analytical Study on the Seismic Design of Reinforced Concrete Frames located in Regions prone to Multiple Earthquakes**
  – Study the response under repeated shaking
  – Assess and develop metrics to quantify the period elongation of the frames as a function of input motion parameters and structure characteristics
  – Compare the performance of frames with and without consideration of damage accumulation (i.e. performing the analyses using the whole earthquake sequence record once and then do one by one record separate analyses)
  – Perform comparison between the response of frames designed based on different approaches
  – Develop design guidelines based on the above findings to increase the safety of structures under multiple earthquakes
1.3 Significance of Proposed Research

Unlike previous work, this research highlights a new approach to design and assess reinforced concrete structures taking into account the effect of multiple earthquake shakings. This research is expected to have an impact on (1) seismic design/assessment of reinforced concrete structures that are vulnerable to repeated shaking; (2) seismic retrofit of reinforced concrete structures that have experienced earthquakes and are prone to subsequent shaking; (3) evacuation decisions and procedures of reinforced concrete buildings while being subjected to an earthquake sequence; and (4) social and economic loss estimation after a sequence of earthquakes.

1.3.1 Design and retrofit

Based on the parametric analysis conducted in this research work, which studies effect of structures design parameters such as stiffness, strength and ductility as well as detailing parameters and seismic code provisions on the response under repeated earthquakes; new proposed design guidelines are suggested to avoid unfavorable response of reinforced concrete structures prone to repeated earthquakes.

The study also helps structural engineers to efficiently retrofit damaged structures that had experienced individual earthquakes and expect future aftershocks. The retrofitting procedures are mainly depending on the characteristics of the damaged structure as well as the expected subsequent ground motions. Both characteristics can be estimated based on this study and hence retrofitting process can be more effective and less costly.

1.3.2 Structure evacuation decisions

Buildings evacuation decisions have a significant social and economic impacts on the buildings residents. Moreover, the vulnerability of damaged buildings under subsequent
shaking could be estimated based on this research work. This study investigates the failure probability of structures that have survived from the first shock, and hence it helps engineers to take correct decisions regarding buildings evacuation after an individual earthquake shock or during the fore-, main-, and after-shocks sequence.

1.3.3 Economic loss estimation

Estimation of economic costs under an earthquake has been extensively studied in previous work. The effects of multiple shaking was either neglected or added linearly on a single shock basis. Neglecting multiple earthquake effects underestimates the economic losses while accounting for multiple earthquakes on a single shock basis (assuming undamaged structures for individual earthquakes) does not represent the realistic situation. Based on this study, the amount of induced damage to structures under repeated shaking could be easily evaluated using the established models; and hence the economic losses could be estimated precisely.

1.4 Organization of the Thesis

A new approach for design and assessment of reinforced concrete structures utilizing material level numerical models is highlighted. Degrading concrete and steel material constitutive models that consider damage accumulation under repeated or long duration earthquake motions are implemented. Response history analyses are conducted under representative earthquake ground motion sequences.

This dissertation includes six different chapters. Chapter 1 introduces the problem statement and objective of this research. Chapter 2 discusses background and literature review in reference to evaluating structures seismic behavior considering the effects of
multiple earthquakes. This chapter describes the degrading models that have been used as well as the ground motions.

Chapter 3 focuses on the development of the analytical models and presents the sophisticated degrading material models used in the analyses. The degrading concrete and steel models are briefly overviewed. Simple pier models of lumped masses at the top are introduced. Moment resisting frames designed based on different approaches are presented; these frames will be modeled and used in the analysis. Ground motion selection criteria for multiple earthquakes are discussed.

Chapter 4 is an overview of the results. It discusses the monotonic, cyclic, and dynamic response of the simple pier model. Moreover, it presents the response of the frames when subjected to replicate and realistic sequences, from a global and local behavior perspective. It also compares between the results obtained using the existing traditional non-degrading models and their developed degrading counterparts.

Chapter 5 presents an interpretation and comparison of results for the non-degrading and degrading systems. It also discusses the effects of various parameters on the response such as force/capacity ratio, site conditions, and input motion predominant periods. The response was measured in terms of inter-story drifts, localized strains and stresses, plastic hinges development during the earthquake sequence, and period elongation of structures due to damage induced during earthquake sequences. This chapter also concentrates on the analytical investigation of frame response, with varying design parameters, and using a collection of ground motion sequences. The implication of the analytical results on the seismic design of reinforced concrete frames prone to repeated earthquake was
investigated. Chapter 6 summarizes the findings from current work followed by future research requirements.
CHAPTER 2 BACKGROUND AND LITERATURE REVIEW

2.1 Introduction

Post Chile, Tohoku (Japan), and Christchurch earthquakes, researchers have started paying attention to implement a new methodology to design and assess structures to resist more than one earthquake. In current design codes, design solutions for reinforced concrete and steel structures are sought to allow for appropriate stiffness, strength, and ductility to resist small (frequent), intermediate (less frequent), and large (rare) earthquakes for serviceability, damage protection, and collapse prevention limit states respectively. Design codes do not consider the effects of damage accumulation induced to structures due small and intermediate earthquakes on the damaged structure behavior during a large earthquake. Previous researchers have used simplified models to consider damage accumulations in prior studies. These models lacked important degrading features presented herein this study. Conclusions drawn from previous work indicated that structures prone to multiple earthquakes are not significantly affected by damage accumulation if they are designed to withstand the most damaging earthquakes. This conclusion contradicts with the field investigation reports presented in Chapter 1 which showed cases of buildings collapse under small aftershocks; however they were intact in the main shock.

In this Chapter, a comprehensive literature review has been conducted to establish a solid starting point to pursue the present study. The literature review focused on (1) understanding the approaches that have been used to determine the response of structures under repeated earthquakes; (2) studying carefully the assumptions that were made in
modeling the systems degrading behavior; and (3) assessing the results and conclusions of each study.

Due to their simplicity, SDOF systems incorporating different inelastic degrading hysteretic force-displacement relationships have been used extensively by many researchers such as Mahin (1980); Aschheim et al. (1999); Amadio et al. (2003); and Hatzigeorgiou et al. (2009). MDOF systems such as moment resisting steel (Fragiacomo et al., 2003; Ellingwood et al., 2007; and Garcia et al., 2011) and concrete frames (Hatzigeorgiou et al., 2010) were introduced as well. In prior work, up to this thesis, MDOF frame systems were modeled based on an approximate component level; i.e. degrading moment rotation relationships at beam column connections using an idealized rotational spring element. This element simulates the behavior of plastic hinges assumed to develop at these frame locations.

Different modeling approaches were followed to study the system level based models (SDOF systems) and component level based models (MDOF frame systems incorporating moment rotation relationships and beam-column joints). Moreover, different ground motions selection criteria have been used in each study; for example some studies used Gutenberg aftershock-main shock relationships to scale their aftershocks records (Ellingwood, 2007), other used realistic ground motions recorded at different stations during previous multiple earthquake scenarios (Fragiacomo et al., 2003), and lastly randomized sequences of ground motions were used by Amadio et al. (2003), Mahin (1980), Hatzigeorgiou et al. (2009 & 2010). Since different approaches and assumptions have been used in literature to model structures and to select their applied ground motion
sequences; therefore, discrepancies between results and conclusions for different studies have been clearly observed.

2.2 SDOF Systems

SDOF systems have been extensively studied for their simplicity in implementing degrading hysteretic force-displacement relationships. Non-linear degrading springs attached to a lumped mass has been used. This section will provide a brief outline on motions, models and results obtained from previous researchers who studied the response of SDOF systems under repeated earthquake.

Mahin (1980) has studied the “effects of duration and aftershocks on inelastic design earthquakes”. The hysteretic models used in this study were linear elastic perfectly plastic (non-degrading) models; however the analyses considered P-\(\Delta\) effects on stiffness degradation of the whole system during ground motion application. Mahin used the 1972 Managua earthquake main shock (PGA = 0.351g) and its two succeeding large aftershocks (PGAs = 0.120g and 0.277g respectively). Cumulative ductility spectra due to main- and aftershocks were presented in this study as shown in Figure 2-1. It was shown from this figure that the main shock had induced significant inelastic deformations when applied to the undamaged system (of initial period and strength). The first aftershock had relatively little effect while on the other hand the second aftershock had caused significant inelastic deformations and also doubled the ductility and energy dissipating demands of the system.
Mahin finally concluded that aftershocks do not have a significant impact on maximum displacements and damage of SDOF systems; he added as a side note that additional research is advised to be conducted taking into account the effect of stiffness and strength degradation of the systems on the response under long duration earthquakes and/or aftershocks.

Aschheim (1999) was the first researcher who introduced degrading systems in his study on “the effects of prior earthquake damage on response of simple stiffness-degrading structures”. The focus was mainly to assess effects of prior earthquake damage on the peak displacement response of over than 20,000 SDOF oscillators. Takeda model (Figure 2-2) was implemented and used in the hysteretic behavior of the SDOF systems. The model incorporated pinched hysteresis as well as stiffness and strength degradations.

Eighteen ground motions that represent different frequency content, duration, and the presence or absence of near-field directivity effects. The effects of residual displacements due to prior shaking were not considered in this study as was believed to have negligible
effect on the response. Prior damage was simulated by adjusting the initial stiffness as well as the current displacement ductility to reach a pre-specified level of prior ductility demand (PDD) and PDD values of 1, 2, 3, 4 and 8 were used.

![Modified Takeda model for SDOF systems incorporating pinching and strength degradation](image)

**Figure 2-2:** Modified Takeda model for SDOF systems incorporating pinching and strength degradation (used in Aschheim, 1999).

In conclusion, the analytical results based on this study demonstrated that prior earthquake shaking has a very minor influence on peak displacement response on average. In addition the study indicated that the displacement response of initially damaged SDOF systems match their undamaged counterparts after the system experiences the peak displacement during the earthquake. This conclusion was supported by the displacement response shown in Figure 2-3 in which four SDOF systems of PDD values 1, 2, 4 and 8 (which represent different initial damage indices) were subjected to an earthquake ground motion. It is revealed in the figure that the responses were not matching before the displacements reached the peak displacement at time approximately equal to 5.6 seconds, then the displacement responses were almost matching after that for different PDDs.
In a study conducted in 2003, Amadio et al. introduced a new design philosophy for buildings that account for more than a “single damageability limit state” after studying the response of SDOF systems with non-linear behavior under repeated earthquakes. Amadia used different hysteretic models which are shown in Figure 2-4. A comparison between initially undamaged and damaged systems was performed and the results revealed that the elastic-perfectly plastic system was the most vulnerable system under repeated earthquake motions.

A sequence of repeated identical earthquake motions were applied to the different hysteretic SDOF models mentioned in the above paragraph. A forty seconds time was believed to be sufficient for the system to damp out and ensure that the systems had come completely to rest. The ground motions used comprised design response spectra compatible records as well as actual records from previous earthquakes (Figure 2-5).

This study indicated that multiple earthquakes can imply a considerable accumulation of damage and a consequent reduction in the response modification (q) factor. As shown in
Figure 2-6, the q-factor ratio \( q_3/q_1 \), where \( q_1 \) and \( q_3 \) are the response modification factors calculated for the first and second earthquakes respectively, were reduced for different periods of SDOF systems. It was also indicated in this study that the SDOF systems is not a correct model to predict the actual response of structures since SDOF systems (1) do not include the complex history of opening the closure of plastic hinges; (2) do not account for the interaction between first and higher modes of vibration; and (3) the effect of axial forces on the external columns. Therefore, more complicated material and component based level models were suggested to be used in future studies to compensate the deficiencies of SDOF systems in depicting an accurate response.

![Hysteretic models](image)

**Figure 2-4:** Hysteretic models of the analyzed SDOF systems by Amadio (2003); (a) Bi-linear model; (b) degrading stiffness models without pinching; and (c) degrading stiffness and strength models with pinching.
**Figure 2-5:** Sequence of three G2 earthquake ground motions (Amadio et al., 2003)

**Figure 2-6:** $q$ ratios at different fundamental periods of vibration. $q_1$ and $q_3$ are force reduction factors calculated after applying the first and the third earthquakes respectively (Amadio et al., 2003).
2.3 MDOF Component Level-Based Frame Systems

Reinforced concrete and steel moment resisting frames have been limitedly studied under repeated earthquakes because their degrading models cannot be easily incorporated in the analyses. However, for simplicity all models established in literature, for frame systems, were component level based degrading models. The component models comprise moment-rotation relationships that account for stiffness and strength degradation as well as fracture at plastic hinges developed at the beam column connections. In these models locations of plastic hinges have been pre-specified at beam-column connections and the rest of the structure was assumed to behave elastically.

Ellingwood (2007) has studied the “performance evaluation and damage assessment of steel frame buildings under main shock-aftershock earthquake sequences. In this study identical earthquake sequences were used in addition to main shocks followed by aftershocks utilizing Gutenberg/Richter formula to estimate the magnitude of the aftershock based on the main shock magnitude. The enhanced uncoupled modal response history analysis method was used (Chopra & Goel, 2002). The frame connections were modeled by a moment-rotation relationship that takes in consideration the fracture of connection welds as shown in Figure 2-7.
The damage accumulation was analyzed in terms of a normalized damage ratio which in this study is equal to the ratio of the number of fractured connections to the total number of connections in the steel moment resisting frame. The main conclusion from this study is that replicate (identical) main shock-aftershock assumption underestimate the damage pattern in the aftershock because the damage pattern depends significantly on the amplitude and frequency content of the aftershock ground motion. Figure 2-8 provides the damage pattern of steel frame connections under main shock and main-aftershock sequence of replicate ground motions. As shown in the figure, the additional damage induced was not significant. A further study on the influence of replicate motions on the response had been conducted and the results summarized in Figure 2-9 show the effect of amplitude of aftershocks on the amount of additional damage induced to frames under aftershocks as a function of the damage induced due to main shocks.
A similar study was conducted by Hatzigeorgiou (2010) but on reinforced concrete structures. The study used a similar approach of component level based models assuming bi-linear moment-rotation relationship at beam column connections and including geometric non-linearity (P-Δ effects) as well. As shown from the analyses in Figure 2-10,
residual displacements played an important role on the stiffness degradation of the overall frames due to P-Δ effects however material deterioration was not accounted for.

![Graphs showing displacement over time for different frames.](image)

**Figure 2-10:** Permanent displacement of a reinforced concrete frame (Hatzigeorgiou, 2010).

### 2.4 Summary

Various analytical studies on the response of structures subjected to multiple earthquakes have been previously conducted. The limitations to the modeling procedures of both systems were highlighted in terms of accuracy in depicting the deteriorating response of structures. For the SDOF systems, the structure behavior was idealized as one spring element. This approximation neglects the localized behavior at the material, section,
element, and subassemblies levels; hence does not account for the forces redistribution during the earthquake. Moreover, higher mode contributions are neglected.

For the MDOF frame systems, results from prior studies were utilized in a number of frame analyses that included moment-rotation relationships which are idealized and not well representative of complex inelastic nature of the frame behavior. The idealization of plastic hinge behavior using lumped springs has many drawbacks since it pre-specifies the failure behavior and location of induced inelasticity. This approach does not depict: (1) actual length of plastic hinge developed (assumes plastic hinge length equals to zero); (2) flexural-shear and flexural-axial interactions; (3) crushing and cracking of concrete; (4) yielding and fracture of steel in tension, as well as buckling of reinforcement bars under high compression; and (5) stiffness recovery from tension to compression and vice versa due to stiffness degradation (pinching).

In this chapter, different methods are introduced, results and conclusions are discussed. It was observed from previous studies that different methodologies were followed per study hence inconsistent results and conclusions were obtained. This is mainly due to the approximations in modeling and their idealization; moreover different criteria of ground motion selection were used in each study.
CHAPTER 3 MODEL DEVELOPMENT

3.1 Introduction

In this chapter, steel and concrete constitutive models that account for damage accumulation in terms of stiffness and strength degradation as well as pinching are introduced and the implementation procedures are discussed. Simple models of typical reinforced concrete flexural piers, of different periods of vibration, are established using the existing non-degrading and newly implemented degrading material models. The response is investigated under monotonic, cyclic, and dynamic earthquake loading. The response of the degrading piers under repeated earthquakes is compared with the response of their non-degrading counterparts.

Furthermore, complicated MDOF reinforced concrete frame models are introduced. The models represent frame systems that comprise the lateral supporting system of a typical reinforced concrete building designed using different approaches. The design approaches, which represent the gravity, direct and capacity design concepts are overviewed. Ground motions used in this study are discussed. Selection criteria of ground motion are also presented.

3.2 Constitutive Material Models

3.2.1 Steel model

The stress-strain relationship of reinforcing bars steel material is based on the modified Menegotto-Pinto relationship (Menegotto and Pinto, 1973). The steel model simulates the following characteristics, which are shown in Figure 3-1: (1) elastic, yielding and hardening branches in the first excursion; (2) Bauschinger effect which consists of (a)
Reduction of the yield stress after a reverse which increases with the enlargement of the plastic strain component of the last excursion, (b) decrease of the curvature in the transition zone between the elastic and the plastic branches; (3) isotropic strain which consists of an increase of the envelope curve, proportional to the plastic strain component of the last excursion; (4) fracture of reinforcing bars when the ultimate strain is exceeded under any excursion; and finally (5) inelastic buckling of reinforcing bars after crushing of bar surrounding concrete. The buckling stress-strain path was simulated by a simplified model based on the equilibrium of a plastic mechanism of the buckled bar.

![Steel stress-strain relationship](image)

**Figure 3-1**: Main characteristics of steel stress-strain relationship (Gomes, 1997).

1. **Degrading features**
   
   a) **Bauschinger effect**

   As mentioned above, the Bauschinger effect includes reduction of the yield stress after a reverse which increases with the enlargement of the plastic strain component of the last excursion; and decrease of the curvature in the transition zone between the elastic and plastic branches. In order to account for the Bauschinger effect, the loading and unloading
paths for the steel stress-strain relationship (not incorporated buckling and fracture features) are developed using the equations of a bilinear envelope below

\[ \sigma_s^* = \beta \varepsilon_s^* + (1 - \beta) \frac{\varepsilon_s^*}{[1 + (\varepsilon_s^*)^R]^{1/R}} \]  

(1)

The normalized strain and stress, \( \varepsilon_s^* \) and \( \sigma_s^* \), are obtained by a variable substitution given in the first load, by

\[ \varepsilon_s^* = \frac{\varepsilon_s}{\varepsilon_{sa}} \]

(2)

\[ \sigma_s^* = \frac{\sigma_s}{\sigma_{sa}} \]

and after the first load reverse by

\[ \varepsilon_s^* = \frac{\varepsilon_s - \varepsilon_{sa}}{2\varepsilon_{sa}} \]

(3)

\[ \sigma_s^* = \frac{\sigma_s - \sigma_{sa}}{2\sigma_{sa}} \]

where

\( \varepsilon_{sa}, \sigma_{sa} \): are strain and stress at the yield point of the bilinear envelope

\( \varepsilon_{sa}, \sigma_{sa} \): are strain and stress at the inversion point (Figure 3-2).

\( \beta = E_{s1}/E_s \): ratio between hardening stiffness, \( E_{s1} \), and the tangent modulus of elasticity at the origin, \( E_s \)

\( R \): constant taking into account Bauschinger effect

Equation (1) represents a curved transition from a straight line asymptote with slope \( E_s \) beginning at point \( (\varepsilon_{sa}, \sigma_{sa}) \) to another asymptote with slope \( E_{s1} \).
The distance to the elastic curve, which simulates the Bauschinger effect, is a function of the parameter $R$ defined by

$$R = R_0 - \frac{a_1 \varepsilon}{a_2 + \varepsilon}$$

(4)

where $R_0$, $a_1$, and $a_2$ are material constants; and $\varepsilon$ is the absolute value of the plastic strain of the last excursion (Figure 3-2).

Two modifications of this model have been proposed to simulate the isotropic strain hardening of this model. The first one consists of a new variable substitution defined by

$$\varepsilon_s^* = \frac{\varepsilon_s - \varepsilon_{sa}}{\varepsilon_{s1} - \varepsilon_{sa}}$$

(5)

$$\sigma_s^* = \frac{\sigma_s - \sigma_{sa}}{\sigma_{s1} - \sigma_{sa}}$$

where: $\varepsilon_{s1}$, $\sigma_{s1}$ are the strain and stress, respectively, of the intercession point of the envelope line to the elastic path as indicated in Figure 3-2.
This modification has the objective of improving the accuracy of the model. The other modification consists in the change of the yield stress value to take into account the isotropic strain hardening. After a load reversal the yield line $\sigma_{so}$, that defines the hardening envelope line, is given by

$$\sigma_{so}^e = \sigma_{so} a_3 \left( \frac{\varepsilon_{smax}}{\varepsilon_{so}} - a_4 \right)$$  \hspace{1cm} (6)

where $\varepsilon_{smax}$ is the maximum absolute strain value before the load reverse; and $a_3, a_4$ are constants of the material.

b) Buckling

After crushing of the concrete cover that surrounds reinforcing bars under compression; longitudinal bars lose their confinement and hence buckle. This is always observed in reinforced concrete columns that are subjected to extreme loading conditions such as earthquakes (Figure 3-3). Buckling of reinforcing bars is a function of many parameters that include bar diameter, yield strength of steel, spacing between transverse reinforcement (stirrups), axial stress applied on reinforcing bar, and crushing strain of surrounding concrete. This section describes briefly the development of a stress-strain relationship of steel reinforcement that considers inelastic buckling of reinforcing bars taking into account the aforementioned parameters.

To take buckling into account, a simple model was developed. The model is based on the equilibrium of a buckled bar limited by two stirrups as shown in Figure 3-4. The equilibrium of the buckled bar in the deformed configuration is given by

$$p = \frac{2M_p}{w}$$  \hspace{1cm} (7)
where \( w \) is the transverse displacement and \( M_p \) is the plastic moment of the bar. For a circular section without axial load it is given by

\[
M_p = Z_p \sigma_{so} = 0.424 \pi R^3 \sigma_{so}
\]  

(8)

where \( Z_p \) is the plastic modulus of the section.

**Figure 3-3:** Buckled reinforcing bars of a reinforced concrete column after Kobe earthquake.

**Figure 3-4:** Equilibrium of buckled longitudinal steel bar.

The compatibility between the transversal displacement \( w \), the longitudinal displacement \( \delta \) and the rigid body rotation \( \theta \), can be expressed by
\[ w = \frac{L}{2 \sin \theta} \]  
\[ \delta = L(1 - \cos \theta) \]  
(9)

where \( L \) is the distance between two consecutive stirrups.

Expanding in series and ignoring the third- and higher- order terms gives

\[ w = \sqrt{\frac{\delta L}{2}} \]  
(10)

from equations (7) and (10)

\[ \sigma_s = \frac{2\sqrt{2} M_p}{A_s L} \frac{1}{\sqrt{\varepsilon_s}} \]  
(11)

To include this equation in the global stress-strain relationship the strains has to be referred to the zero stress point which corresponds to a translation of the curve from tension to compression, as shown in Figure 3-6. This can be affected by the variable substitution

\[ \varepsilon_{sr} = \varepsilon_s - \varepsilon_{sq} \]  
(12)

where \( \varepsilon_{sq} \) is the strain of the zero stress point from tension to compression (Figure 3-6).

The intersection point between the compression stress-strain relationship and buckling curve is determined using the Newton-Raphson Method. Optimization algorithms are utilized to reduce the number of iterations and speed the analysis.

The hysteretic behavior of the steel model after including the buckling effect is shown in Figure 3-5. This figure shows the response of the implemented steel model in Zeus-NL under cyclic loading of linearly increasing amplitude of sinusoidal axial strains.
Figure 3-5: Cyclic response of implemented steel model, including buckling of reinforcing bars.

c) Fracture

Reinforcing bar fracture under large excursions of strains that exceed that ultimate strain of steel is also considered. In this model, the fracture is modeled by the complete loss of stiffness and strength of reinforcing steel, i.e. applied strains result in zero stresses response. Figure 3-6 shows the implementation of rebar fracture when the applied reverse strain exceeds the fracture strain of steel.

Figure 3-6: Buckling and fracture implementation in the steel model.
ii  Monotonic and cyclic flexural behavior

The aforementioned steel model is implemented in Zeus-NL analytical tool and optimization algorithms are employed to speed the convergence of the model under high inelasticity. Simplified reinforced concrete pier models are established, an overview of the models characteristics in terms of concrete dimensions, steel reinforcement, concrete and steel material properties are discussed in the next section of this chapter. Two modeling approaches are considered, the first approach (Pier1) uses the existing bi-linear steel stress-strain (stl1) in Zeus-NL platform; while the second approach (Pier2) utilizes the aforementioned, newly implemented, degrading steel model (stl4). Both piers are developed based on the non-degrading Mander concrete stress-strain relationship, con2, (Mander, 1994).

![Figure 3-7: Simplified pier model, concrete dimensions and reinforcement.](image)

The simple pier model shown in Figure 3-7 is subjected to a monotonic lateral displacement. A constant axial compressive stress equals to 10% of concrete strength, $f'_c$. 


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is imposed on the pier. Resulted lateral forces are captured and the load-displacement response is plotted (Figure 3-8).

The comparison between Pier1 and Pier2 response is shown in Figure 3-8. The results obtained from Pier1 matched very well with Pier2 at the pre-peak range (zero to 0.056 m displacement). However, discrepancies are revealed at the post peak response. At the peak load, concrete strains are measured at the compression extreme fibers of the pier section. These strains reached the crushing strain level of concrete material. In Pier2, crushing of concrete unconfined the reinforcing bars under compression (at the top row) and allowed them to buckle (as modeled in stl4). This justifies why Pier2 shows lower resulting lateral forces than Pier1 for a displacement range of 0.056 m and 0.182 m. The sudden loss of strength in Pier2 model shown at approximately 0.182 m displacement is due to fracture of the nine reinforcing bars at the bottom row of the pier section reinforcement when strains at this row exceeded the ultimate strain (Figure 3-8). Similarly, sudden loss in strength is captured at higher displacement levels due to cascading fractures of other rows of reinforcement. This comparison shows the importance of including bar buckling and fracture features in case of having large inelastic strains imposed.
Figure 3-8: Load-displacement diagram under monotonic loading for Pier1 (using stl1 model) and Pier2 (stl4).

The same analysis is repeated for four different pier models utilizing stl4 and con2 models only. The models have the same properties in terms of materials, dimensions, reinforcement, and stirrups spacing. However, the piers are imposed to constant axial compressive stresses equal to 5%, 10%, 15%, and 20% of \( f'_c \). Comparison of the results is shown in Figure 3-9. Fracture of reinforcing bars takes place at higher displacements when the axial load is increased. On the other hand, buckling of bars is initiated at lower displacement values as the axial load increases.
In order to investigate stirrups spacing effect on buckling of reinforcing bars, similar analysis is undertaken. In this case, four piers of different stirrups spacing are subjected to the same loading conditions and an axial stress level of 10% $f'_c$. Results shown in Figure 3-10 indicate that spacing between stirrups affects the flexural strength of columns. Steeper softening response is observed in piers of larger spacing. This is due to the initiation of buckling of bars under compression for piers of larger stirrups spacing prior to their counterparts of smaller spacing.

**Figure 3-9:** Axial Load effect on the flexural response of a reinforced concrete pier.
Cyclic displacements ($\delta$) are applied in the transverse direction, at the top ends of Pier1 and Pier2. Displacement amplitudes are ±0.05, ±0.1, and ±0.15 meters for the first, second, and third set of cyclic displacements respectively. Each set of cyclic displacements consisted of three cyclic excursions to depict crack opening and closure effects on the pinching behavior of the pier. The axial load imposed stresses equal to 10% of concrete strength. Axial loads are constant throughout the cyclic analysis ($10% f'_c$).

**Figure 3-10:** Effect of stirrups spacing on beam flexural response.

**Figure 3-11:** Cyclic load pattern.
Figure 3-12 shows the comparison of the force-displacement hysteretic response under cyclic loading for Pier1 and Pier2, i.e. using stl1 and stl4 steel models, respectively. Pier1 depicted the loss in strength and stiffness due to buckling and fracture in reinforcing bars especially under large displacement excursions.

![Figure 3-12: Effect of buckling and fracture of reinforcing bars on the flexural hysteretic response of a reinforced concrete pier.](image)

### iii Dynamic response history analysis

A dynamic time history analysis is carried out for Pier1 and Pier2 models. A lumped mass is added at the top of each pier, and the self-weight of pier is neglected. The elastic fundamental period of vibration of both piers is adjusted to 0.46 seconds. Acceleration
time history records are applied at the piers support. The record is scaled in a manner that ensures crushing of concrete and buckling of steel during the analysis.

![Figure 3-13: Re-bars buckling and fracture effects on an SDOF reinforced concrete beam.](image)

The displacement response of Pier1 and Pier2 is plotted in Figure 3-13. From this figure, it is observed that the displacement response of both piers is matching very well at the first 32 seconds. This perfect match indicates that buckling and fracture of reinforcing bars is not initiated in the first 32 seconds. Results are not matching at the second half of the analysis and longer periods are captured for the Pier2 model.

### 3.2.2 Concrete model

A plastic-damage concrete model for concrete is implemented in the analysis tool (Zeus-NL). This concrete model was developed using the concepts of fracture-energy-based damage and stiffness degradation in continuum damage mechanics (Fenves 1998). Two damage hardening variable are introduced to account for different damage states under tensile and compressive stresses. A simple degradation model is introduced to simulate the effect of damage on elastic stiffness and its recovery during crack opening and
closure. Strength deterioration is modeled by using the effective stress (of cracked concrete) to control the evolution of the yield surface. The performance of the plastic-damage model is demonstrated with a numerical example that simulates a cyclically loaded reinforced concrete pier. The plastic-damage model is compared with Mander model (Mander, 1994) and effects of stiffness and strength degradation on the response are emphasized.

\textit{i Degradation features}

The constitutive relationship of the concrete plastic-damage model based on plasticity and continuum mechanics is developed. The concept of fracture-energy-based multiple-hardening variables is used to represent tensile and compressive damage independently. The constitutive system for the elasto-plastic response is described completely by the effective stress and damage variable, which leads to a decoupled algorithm for the effective stress computation and degradation evaluation. A thermodynamically consistent scalar model is used to simulate the stiffness degradation and recovery. This model was verified experimentally as shown in Figure 3-14.
Two damages variables are introduced in this model. One variable is for tensile damage \((d_t)\) and the other for compression \((d_c)\) as shown in Figure 3-15. The constitutive relationships for elastoplastic response are decoupled from the degradation damage response. The evolution of yield surface was controlled by the strength function of effective stress that accounts for strength deterioration due to existence of cracks parallel to the loading direction. A simple thermodynamically consistent stiffness recovery scheme is introduced for simulating crack opening and closure (pinching effects). The damaged elastic stiffness remained in this model isotropic; therefore, the plastic model
provided a separate evolution of tensile and compressive strength and stiffness degradations.

![Figure 3-15: Stiffness and strength degradation of concrete model (Fenves 1998)](image)

In this model the uniaxial stress-strain relationship developed by

\[ \sigma_g = f_{go} \left[ (1 + a_g) \exp(-b_g \varepsilon^p) - a_g \exp(-2b_g \varepsilon^p) \right] \]

\[ 1 - D_g = \exp(-d_g \varepsilon^p) \]

\[ \overline{\sigma}_g = f_{go} \left[ (1 + a_g) \left( \exp(-b_g \varepsilon^p) \right)^{-1} \left( \frac{d_g}{b_g} \right) - a_g \left( \exp(-2b_g \varepsilon^p) \right)^{2} \left( \frac{d_g}{b_g} \right) \right] \]  

(13)

\[ k_g = \frac{1}{g_g} \int_0^{\varepsilon^p} \sigma_g (\varepsilon^p) d\varepsilon^p \]

\[ g_g = \int_0^{\infty} \sigma_g (\varepsilon^p) d\varepsilon^p \]

where,

\( \sigma_g \) is the uniaxial stress

\( \varepsilon_p \) is the scalar plastic strain
\( f_{go} \) is the initial yield stress defined as the maximum stress without damage.

\( a_g, b_g \) and \( d_g \) are constants.

\( \sigma'_e \) is the effective stress.

Figure 3-16: Uniaxial cyclic behavior of implemented concrete model (con5).

Figure 3-16 shows the uniaxial stress-strain response of the concrete model that is implemented in the analytical tool under the name of con5. The plot reveals the stiffness and strength degradation of the material response under simple cyclic axial, linearly increasing, sinusoidal strain loading. The stiffness degradation is obvious, when the initial stiffness at zero stress-strain values is compared with the stiffness at any other stress-strain level. It is also obvious that smooth transitional reduction of stiffness takes
place during the cyclic loading unlike other models where stiffness suddenly drops and stays constant due to concrete cracking.

The strength deterioration, in tension and compression, is also captured using con5 concrete model as shown in Figure 3-16. The stiffness recovery (pinching effects) is revealed at the unloading curves where the stress state changes from tension to compression or vice versa.

**ii  Cyclic flexural behavior**

This section focuses on the degrading features of the concrete model. Two reinforced concrete piers, Pier3 and Pier4, of same characteristics are modeled using the non-degrading Mander model (con2) and the aforementioned plastic-damage model (con5) respectively. Both models use stl1 (bi-linear steel model) for reinforcement to highlight the effects of concrete degrading features only, assuming non-degrading steel, on the flexural response under cyclic loading. The response of both pier models is studied and compared. The reinforced concrete pier described in the previous section (for the steel model) is used this section; i.e. the pier used herein this section has the same dimensions, reinforcement configuration, and material properties.
The applied displacements and calculated forces on Pier3 and Pier4 are compared in Figure 3-17. It is observed that Pier4 captures stiffness degradation at large displacement excursions. Pier4 also depicts the strength deterioration of concrete under repeated cyclic loading with same imposed displacements amplitude.

3.3 Simple Pier Model

An inelastic finite element model is employed for the simple pier model using Zeus-NL platform. The cross sectional dimensions and reinforcement of the pier are shown in Figure 3-7; the height of the pier is 4m and the spacing between stirrups is 250 mm. The
model consists of five nodes that connect four 3D cubic elasto-plastic beam-column elements (Zeus-NL User Manual) with 200 monitoring points. Reinforced concrete rectangular section is used. Rectangular reinforced concrete section is used.

Concrete parameters in terms of strength (f'c), modulus of rupture (ft), crushing strain (e'c), and confinement (for confined regions within stirrups) are 40 MPa, 5 MPa, 0.003, and 1.2 respectively. Reinforcing steel parameters including Steel Young’s Modulus (E), yield stress (fy), ultimate strain (eu), strain hardening (\(\mu\)) are 2100000 MPa, 360 MPa, 0.2, and 10%.

Masses are assigned at the top node of the pier model using the lumped (concentrated) mass element (lmass). Four models of four different mass values are established to yield different periods of vibrations of each model (0.12, 0.22, 0.46, and 1.00 seconds). Two modeling approaches are introduced, the first used the non-degrading concrete and steel models (con2 and stl1) while the second utilized both the concrete and steel models introduced in previous sections (con5 and stl4).

3.4 Overview of Frame Models

3.4.1 Description of the structure

The structure under consideration is a 3-story, 2-bay (longitudinal) and 4-bay (transverse) reinforced concrete frame, assumed to be located in Aliso Viejo, California (Figure 3-18). The height of the first story is 4 m and the height of the second and third stories is 3 m. The lateral load resisting system is moment resisting frame designed using the ASCE-7 2005 code. The highlighted frame system in Figure 3-18, in the transverse direction is of our interest in this study. The Soil at this region is of type B. The frame elements (beams
and columns) are of aspect ratio greater than 5, and hence shear deformation was
neglected since the elements are flexural dominant.

Three concepts are introduced to design this frame namely gravity, direct, and capacity
design approaches. The building configuration and design procedures (design forces,
sizing of beams and columns, and reinforcement detailing) are provided in APPENDIX
A; while the sizes of beams and columns as well as the reinforcement detailing are
described in detail in APPENDIX B.

The frame consists of nine columns and six beams. The frame column and beam elements
are numbered as shown in Figure 3-19. As an example, C2 denotes the first story mid
column of the frame. The start and end nodes of each element is defined by S and E
respectively. For consistency, the start node is always defined as the node at the bottom end of columns and left end of beams. In addition, top and bottom fibers of each section are defined using the symbols T and B for top and bottom respectively. This notation is used in the rest of this dissertation.

3.4.2 Design approach

In this section, three moment resisting frames systems are designed under three different design approaches. The frames are modeled using Zeus-NL utilizing the degrading
concrete and steel models (con5 and stl4) explained previously. The three design approaches used in this study are (1) gravity load based design approach; (2) direct design approach for seismic frames; and (3) capacity design approach for seismic frames.

i Gravity designed frame

Gravity frame is designed under the action of the vertical dead and live loads only. Load combinations of factored dead and live loads are used in order to obtain the straining actions. Live loads under different loading configurations defined by the ASCE-7 code (described in APPENDIX A) are utilized to maximize the design straining actions at critical sections. The load combinations yield the envelop design straining actions shown in Figure 3-20.

Figure 3-20: Envelop straining action diagrams of the gravity frame under vertical loads; (a) bending moment, (b) shear force, and (c) axial force.

Same concrete and steel parameters mentioned in the previous section, for the pier model, are used for the frames. The concrete section dimensions of beams and columns of the gravity frame are shown in Figure 3-21. The detailed design procedures as well as the reinforcement detailing are provided in APPENDIX A and APPENDIX B respectively. In the fiber-based model, columns are represented by reinforced concrete rectangular sections; while beams are represented by reinforced concrete T sections to account for
slab contribution in flexural capacity/stiffness of beams. Detailing and configuration of
reinforcing steel bars, presented in APPENDIX B, are accurately modeled using the
aforementioned sections for beams and columns.

![Diagram of concrete dimensions of gravity designed frame.](image)

**Figure 3-21:** Concrete dimensions of gravity designed frame.

A detailed 2D fiber-based finite element model is established using 3D cubic elastoplastic beam-column elements implemented in Zeus-NL software to model beams and
columns. The software is capable of conducting static and dynamic analyses incorporating material and geometric non-linearity (employing the Eulerian formulation).

A mesh of four elements is used for each beam/column member. To capture the high
inelasticity induced near the beam-column joints accurately, smaller element sizes are
used at the start and end points of each member. The length of elements is 0.15L, 0.35L,
0.35L, and 0.15L starting from the start node to end node of the element respectively.

The mesh of the frame element is shown in Figure 3-22. This figure also describes the
fiber analysis approach Zeus-NL utilizes.
The fiber-based finite element modeling, used in this study, is an efficient and accurate tool for simulating the response of a complete structural system under static and dynamic loading conditions. Members of the frame are modeled using elasto-plastic beam-column elements, with 200 monitoring points. These elements follow the Euler-Bernoulli formulation (Izzuddin and Elnashai, 1993). Each element has two nodes, for 2D analysis, each node considers 3 Degrees of Freedom (DOFs), two displacement components and one rotation. Evaluation of stiffness matrix of the element is performed at two Gaussian points located at a distance approximately 0.3l from the mid-point of the member. The section at each integration point is further divided into fibers that form the basis of distributed inelasticity models. Section stiffness is evaluated at the Gaussian points based on the contribution of each fiber. Integration of the stiffness at the Gaussian points yields the tangent stiffness matrix for the element. The element stiffness matrices are assembled into the global stiffness matrix of the whole structure.
**Direct designed frame**

The seismic forces imposed on the frame system are calculated in APPENDIX A, based on the ASCE-07 code. Equivalent static lateral load method is used to simplify the design procedure. In the direct design approach used in this section, the dimensioning of individual columns and beams and determining their reinforcement is done to resist the locally evaluated actions (bending moments, shear and axial forces) with no due consideration to the action redistribution effects in the system as a whole. Therefore, the sections are solely designed to resist the imposed action shown in Figure 3-23.

*Figure 3-23:* Envelop straining action diagrams of the seismic frames under vertical and earthquake loads; (a) bending moment, (b) shear force, and (c) axial force.

Detailing of longitudinal and transverse (stirrups) reinforcement is based on the ACI 2008 design code for reinforced concrete structures in high seismic regions. The reinforcement detailing and curtailment are provided in APPENDIX B.

A 2D inelastic fiber-based model is established for the direct designed frame similar to the one established for the gravity frame discussed in the previous section. Same modeling approaches are employed in terms of mesh size, beam and column sections modeling and reinforcement detailing. The concrete dimensions of the column and beam sections are provided in Figure 3-24.
iii Capacity designed frame

In the capacity design approach, one set of actions represent the ultimate capacity of the members responsible for the energy absorption, whilst the rest of the design actions are calculated to maintain equilibrium (Elnashai and Di Sarno, 2008). Therefore, in the capacity designed frame studied herein this section; strong column weak beam method is deployed. The beams are considered the energy dissipative zone where flexural plastic hinges are favorably located at. Therefore, the design straining actions for beams are evaluated from the applied actions, shown in Figure 3-23. Design actions for columns are calculated based on the beam-column joints equilibrium taking into account all various sources of over-strength of beam sections, such as unintentional increase in material properties, rounding-off of member or reinforcement dimensions, post yield hardening, etc. Column design actions calculations are provided in APPENDIX A.

Figure 3-24: Concrete dimensions of seismically designed frame (direct design approach).
Concrete dimensions of columns and beams of the capacity designed frame are shown in Figure 3-25. It is shown in this figure that the size of beams of the capacity designed frame is the same as the beam sizes in the direct designed frame (Figure 3-24). However, the columns of the capacity designed frame are of bigger size, and this is due to imposing the concept of strong column weak beam in the capacity design approach. The fiber modeling approach of the capacity frame is similar to the modeling approach of the gravity and direct designed frames discussed before.

3.5 Input Ground Motion Sequence

The input ground motion sequences selected in this study are divided into: (1) replicate motion sequences; (2) random ground motion sequences; and (3) real ground motion sequences. In the replicate case, two identical earthquakes are applied to the structure, while in the random sequences, the structure is subjected to two different earthquakes, and their selection criteria are discussed later in this section. The real ground motion sequences, are the successive records that are measured at one station during an
earthquake sequence such as the sequences generated at the Tohoku and Christchurch earthquakes.

In all three cases, a time buffer ranging from 10 to 20 seconds is used between the earthquake records that are applied in series as shown in Figure 3-26. The reason of using this time buffer between successive records is to ensure that the structure is brought to rest before it experiences the subsequent earthquake. Hence the structural behavior under the subsequent earthquake is not influenced by any remaining dynamic movements due to the previous earthquake. All ground motion parameters used in this study are summarized in APPENDIX C.

![Figure 3-26: Time buffer between successive earthquakes to ensure that the structure is brought to rest; Loma Prieta earthquake recorded acceleration history.](image)

3.5.1 Replicate earthquake sequence

Two identical ground motions are applied in series to the simplified pier model and the complicated MDOF frame systems. The purpose of using replicate motions is to determine the effects of damage accumulation induced in the first earthquake on the behavior of the structure under the second earthquake with limiting the input motion parameters. For the simplified pier model, a short duration record of the Loma Prieta
earthquake, shown in Figure 3-26, is used to save the analysis time. The purpose of selecting this record is to have an insight on the comparison between the response of non-degrading (uses stl1 and con2 materials) and degrading (utilizes stl4 and con4) pier models. The response spectrum of the Loma Prieta record used for the simple pier model is shown in Figure 3-27. In this figure the fundamental periods of vibration of the four pier models (described in section 3.3) are also provided and their spectral accelerations are shown as well.

**Figure 3-27:** Response spectrum of the Loma Prieta earthquake record and the periods of vibration of the simplified pier models.

Replicate motions used for the MDOF frame systems (gravity, direct, and capacity designed frames) discussed in section are selected based on different frequency content and soil types. The first record used is the Loma Prieta earthquake which comprises the high frequency content and rock soil conditions. The second record is the Chi-Chi
earthquake record which is measured on soft soil conditions and contains low frequency content motions. Stations information of both records, in terms of soil type, earthquake magnitude, and source to site distance, are provided in APPENDIX C. A code compatible earthquake record, with a response spectrum that matches with the ASCE-7 design spectra, is finally used.

The acceleration time histories and response spectra of the records are shown in Figure 3-28 and Figure 3-29. The selection criterion of replicate ground motions is based on extreme point parametric analysis, where ground motion parameters such as predominant periods and soil conditions vary significantly from one record to another.
Figure 3-28: Selected earthquake motion for frame systems.

Figure 3-29: Response spectra of the selected motions along with the code design spectra.
3.5.2 Random earthquake sequence

Ground motions in this section are selected based on soil type and frequency content. The ground motions in this case are applied in series with different sequence combinations. Twenty seven records are selected that comprise soil types A (8 records), B (6), C (6), and D (6). The records earthquake names, magnitudes, and station information are provided in APPENDIX C. The records are defined by their serial numbers as follows, records number 1, 2, 3, 4, 5, 24, 25, 26, and 27 are of soil type A; records number 6, 7, 8, 9, 10, and 22 are of soil type B; records number 11, 12, 13, 14, 15, and 21 are of soil type C; while records number 16, 17, 18, 19, 20, and 25 are of soil type D. Measured shear velocities of soil types A, B, C, and D; are greater than 750 m/s; between 360 and 750 m/s; between 180 and 360; and less than 180, respectively. The response spectra of the records of soil types A, B, C, and D are shown in Figure 3-30, Figure 3-31, Figure 3-32, and Figure 3-33.
Figure 3-30: Response spectra of soil type A records.

Figure 3-31: Response spectra of soil type B records.
Figure 3-32: Response spectra of soil type C records.

Figure 3-33: Response spectra of soil type D records.
The ground motions are selected for the frame systems based on the motions predominant periods. Three records are assigned for each of the three frame systems (gravity designed, direct designed and capacity designed). One of the three records is selected based on the criterion that it has a predominant period that matches the first mode period of vibration of the structure; while the other two periods have predominant periods much lower and much higher than the structure first mode period of vibration. The predominant period of the records is evaluated as the periods that correspond to the period at which the relative velocity of a linear system with 5% damping is maximum within the entire period range. The velocity spectra of records and the records predominant periods are provided in APPENDIX C.

For the gravity designed frame, records number 3, 10, and 12 are selected. These records have 0.4, 1.0, and 1.6 sec predominant periods respectively. The record number 3 comprise the short period record where the predominant period of this record is much lower than the first fundamental period of vibration of the gravity designed frame (1.1 seconds). Records number 10 comprise the record at which the predominant period is matching with the period of the gravity frame while record number 12 predominant period is much longer that the period of the frame.

The first fundamental periods of vibration of the direct and capacity design seismic frames are 0.4 and 0.28 seconds, respectively. Similar selection criterion is used to select the ground motion records for the direct and capacity designed frames. Records number 2, 6, and 11; and 12, 26, and 27 are used for the analysis of the direct and capacity designed frames. The nine earthquake sequences are used for each frame, since only two successive earthquake records applied. Table 3-1 shows the earthquake sequences used
for the gravity, direct, and capacity frames for the first and second applied earthquakes in series.

**Table 3-1**: Earthquake sequences for gravity, direct, and capacity frames under two earthquake motions.

<table>
<thead>
<tr>
<th>Gravity</th>
<th>1st Record</th>
<th>2nd Record</th>
<th>1st Record</th>
<th>2nd Record</th>
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</table>

3.5.3 **Real earthquake sequence**

Earthquake sequences measured from the recent March 11 Tohoku earthquakes in Japan as well as the Christchurch earthquake in New Zealand are used in this study. In these two earthquake sequences, thousands of earthquake motions are monitored at each station, therefore records of PGA less than 0.15g were not considered in the earthquake sequences selected in this study to save the analysis time.

- **Tohoku earthquake sequence**

The Tohoku earthquake sequence, in Japan, generated thousands of aftershocks after the 9.0 magnitude main shock that occurred on March 11. It is reported that more than 1,000 aftershocks of magnitude 4+ were measured one month after the earthquake. The largest aftershock is reported on April 7, this aftershock is of magnitude 7.4 (Figure 3-34).
The April 7 aftershock epicenter was close to the Japanese shore, much closer than the epicenter of the main shock on March 11. That’s why this aftershock caused heavy damage and collapse of buildings especially in big cities located on the shore such as Sendai city. Figure 3-35 shows damaged reinforced concrete buildings in Sendai city after April 7, it is worth noting that these buildings were slightly damaged after the March 11 main shock. The heavy damage associated with the aftershock highlights the significance of this research and emphasizes on the effects of multiple earthquakes on the behavior of reinforced concrete buildings.

**Figure 3-34:** Fore-, main-, and aftershocks of the Tohoku earthquake sequence.

**Figure 3-35:** Damaged reinforced concrete buildings due to the M7.4 April 7 aftershock, the buildings were slightly damaged after the M9.0 March 11 main-shock.
Tens of monitoring stations in Japan experienced multiple earthquakes, each of which of magnitude more than 5.5. Station IBR013 reported the maximum number of significant earthquakes, where 27 successive records of strong accelerations are experienced. Table 3-2 shows station codes and the corresponding number of significant earthquakes measured each station. Additional information about stations and their records that are monitored two month after the March 11 earthquakes is provided in APPENDIX C.

Table 3-2: Number of significant earthquakes measured at some stations from March 11 to May 1, 2011 at different locations in Japan.

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</table>

Three stations are considered in this study, namely stations FKS013, IWT007, and IWT010. The locations of these stations as well as the epicenter of the March 11 main shock are shown in Figure 3-36, additional information with regards to the stations and
the earthquake sequence can be found at http://www.k-net.bosai.go.jp/. The stations are selected to have the same distance from the main shock epicenter. Records measured on each station are put in series with 20 seconds time buffer between successive records, as indicated in the previous section to ensure that the structures are brought to rest. Since thousands of records are monitored in each station, insignificant records defined by having PGAs less than 0.15g are omitted to save analysis time.

**Figure 3-36:** Stations location and the March 11 main shock epicenter (denoted by the star).

Station FKS013: Significant records (PGA>0.15g) measured at station FKS013 are considered in this study. The epicenter locations of the main-shock and aftershocks are provided in Figure 3-37 (a). Measured earthquake sequence consists six significant records that contain the main shock on March 11, 14:46 (record 1), and aftershocks on April 7, 23:32 (record 2), April 11, 17:26 (record 3), April 11, 17:58 (record 4), April 11,
20:42 (record 5), and April 12, 14:07 (record 6). The six records are plotted in series in Figure 3-38 (a) and the response spectra of each individual record as well as the whole sequence are plotted in Figure 3-38 (b), the first mode fundamental periods of the frame systems are also shown on the same figure.
Figure 3-37: Epicenter location of significant earthquake measured at stations (a) FKS013, (b) IWT007, and (c) IWT010; square size denotes the magnitude.
Figure 3-38: Records breakdown at station FKS013 (a); and response spectra of individual records (b).
Station IWT007: acceleration time histories of significant records are similarly selected and insignificant ones are omitted (Figure 3-39). Two records are only considered in the analysis for this station to save computational time since these two records are of much higher PGA than the other measured records at this station. The first record is the March 11 main shock and the second is the April 7 (M7.4) aftershock. The response spectra of the two individual records and earthquake sequence are plotted in Figure 3-39, the first mode fundamental periods of the frames are also shown on the same figure.

Station IWT010: similar to the selection criteria followed for records measured at station IWT007, two records are only considered for station IWT010. The records represent the March 11 main shock and April 7 aftershock, acceleration time history of records and response spectra are shown in Figure 3-40.
Figure 3-39: Records breakdown at station IWT007 (a); and response spectra of individual records (b).
Figure 3-40: Records breakdown at station IWT010 (a); and response spectra of individual records (b).
The ground motion characteristics of all records measured at stations FKS013, IWT007, and IWT010 are shown in Table 3-3. Ground motion characteristics of each individual records are provided in terms of Peak Ground Acceleration (PGA, $g$), Peak Ground Velocity (PGV, cm/sec), Peak Ground Displacement (PGD, cm), maximum Velocity to maximum Acceleration ratio (V/A, sec), Arias Intensity (AI), Characteristic Intensity ($I_c$), Housner Intensity, Predominant period ($T_p$), and Mean Period ($T_m$).

**Table 3-3**: Ground motion characteristics of individual records at stations FKS013, IWT007, and IWT010.

<table>
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<th>Parameter</th>
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<th>IWT010</th>
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<td>0.71</td>
<td>0.87</td>
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<tr>
<td>PGV</td>
<td>26.10</td>
<td>40.87</td>
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<td>PGD</td>
<td>153.93</td>
<td>27.33</td>
<td>43.98</td>
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<tr>
<td>V/A</td>
<td>0.07</td>
<td>0.11</td>
<td>0.04</td>
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<tr>
<td>AI</td>
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<td>22.20</td>
<td>11.92</td>
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<tr>
<td>$I_c$</td>
<td>0.10</td>
<td>0.07</td>
<td>0.04</td>
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<tr>
<td>HI</td>
<td>82.70</td>
<td>127.13</td>
<td>84.30</td>
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<tr>
<td>$T_p$</td>
<td>0.50</td>
<td>0.60</td>
<td>0.12</td>
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<tr>
<td>$T_m$</td>
<td>0.38</td>
<td>0.49</td>
<td>0.21</td>
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</tbody>
</table>

**Christchurch earthquake sequence**

The ongoing Christchurch earthquake described in section 1.1, started with the 7.1 magnitude Canterbury earthquake on September 4 2010. This earthquake is followed by many other earthquakes of high magnitude, the most damaging one amongst all is the 6.3 magnitude Christchurch earthquake that occurred on February 22 2011. The latter earthquake caused significant damaged when compared to the former, however it is of smaller magnitude, but the Christchurch earthquake epicenter was located right underneath the Christchurch city. The magnitudes, epicentral location, depth, and time of individual earthquakes in the sequence are shown in Table 3-4.

Similar to the Tohoku earthquake sequence discussed in the previous section, in this earthquake sequence, thousands of records are monitored at many stations in New Zealand.
Zealand. Only significant records of high PGAs that correspond to large magnitude earthquakes are considered in this study to save analysis time. Table 3-5 shows a few number of stations, which their measured records are selected to be used in this study.

Table 3-4: Earthquakes (M>4.0) following the September 4 (M7.1) Canterbury earthquake.

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Mw</th>
<th>Earthquake Epicenter</th>
<th>Depth (kms)</th>
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</thead>
<tbody>
<tr>
<td>22-Feb-11</td>
<td>12:51 pm</td>
<td>6.2</td>
<td>10 km south of Christchurch</td>
<td>5.0 km</td>
</tr>
<tr>
<td>22-Feb-11</td>
<td>1:04 pm</td>
<td>5.5</td>
<td>10 km south of Christchurch</td>
<td>5.9 km</td>
</tr>
<tr>
<td>22-Feb-11</td>
<td>2:50 pm</td>
<td>5.6</td>
<td>Within 5 km of Lyttelton</td>
<td>6.72 km</td>
</tr>
<tr>
<td>22-Feb-11</td>
<td>2:51 pm</td>
<td>4.5</td>
<td>Within 5 km of Lyttelton</td>
<td>7.3 km</td>
</tr>
<tr>
<td>22-Feb-11</td>
<td>4:04 pm</td>
<td>4.5</td>
<td>Within 5 km of Christchurch</td>
<td>12.0 km</td>
</tr>
<tr>
<td>22-Feb-11</td>
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<td>4.4</td>
<td>20 km south-east of Christchurch</td>
<td>12.0 km</td>
</tr>
<tr>
<td>5-Mar-11</td>
<td>7:34 pm</td>
<td>4.6</td>
<td>10 km south-east of Christchurch</td>
<td>9.5 km</td>
</tr>
<tr>
<td>20-Mar-11</td>
<td>9:47 pm</td>
<td>4.5</td>
<td>10 km east of Christchurch</td>
<td>11.83 km</td>
</tr>
<tr>
<td>16-Apr-11</td>
<td>5:49 pm</td>
<td>5</td>
<td>20 km south-east of Christchurch</td>
<td>10.6 km</td>
</tr>
<tr>
<td>30-Apr-11</td>
<td>7:04 am</td>
<td>4.9</td>
<td>60 km north-east of Christchurch</td>
<td>8.7 km</td>
</tr>
<tr>
<td>10-May-11</td>
<td>3:04 am</td>
<td>4.9</td>
<td>20 km west of Christchurch</td>
<td>14.4 km</td>
</tr>
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<td>6-Jun-11</td>
<td>9:09 am</td>
<td>5.1</td>
<td>20 km west-south of Christchurch</td>
<td>8.1 km</td>
</tr>
<tr>
<td>13-Jun-11</td>
<td>1:00 pm</td>
<td>5.3</td>
<td>10 km south-east of Christchurch</td>
<td>8.9 km</td>
</tr>
<tr>
<td>13-Jun-11</td>
<td>2:20 pm</td>
<td>5.9</td>
<td>10 km south-east of Christchurch</td>
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</tr>
<tr>
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Table 3-5: Significant earthquakes monitored at different stations in New Zealand.

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<th>Station</th>
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<th>Station</th>
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<td>19:49:57</td>
<td>5.02</td>
<td>LPCC Lyttelton Port Company</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Mw</th>
<th>Station</th>
</tr>
</thead>
<tbody>
<tr>
<td>2010-09-03</td>
<td>16:35:41</td>
<td>7.10</td>
<td>REHS Christchurch Resthaven</td>
</tr>
<tr>
<td>2010-09-07</td>
<td>19:49:57</td>
<td>5.13</td>
<td>REHS Christchurch Resthaven</td>
</tr>
</tbody>
</table>

At station CBGS shown in the above table, nine significant earthquakes of PGA>0.2g are monitored. The acceleration time history of the earthquake sequence is shown in Figure 3-41 along with the response spectra of each individual record. The records are defined by their local time of generation, see Table 3-5; for example the third record occurred at 21:30:15 and its ID code is 213015 as shown in Figure 3-41.
Figure 3-41: Earthquake sequence at station CBGS in New Zealand, and the response spectrum of each individual record.

Summary of ground motion characteristics of individual records of Christchurch earthquake records measured at station CBGS is shown in Table 3-6. The input motion characteristics of each record are provided in terms of Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), Peak Ground Displacement (PGD), maximum Velocity to maximum Acceleration ratio (V/A), Arias Intensity (AI), Characteristic Intensity ($I_c$), Housner Intensity, Predominant period ($T_p$), Mean Period ($T_m$), spectral accelerations of 5% damping response spectrum that correspond to the first fundamental periods of vibration of gravity, direct, and capacity designed frames $S_{a_g}$, $S_{a_s}$, $S_{a_c}$, and the supply to demand ratio ($F/C$) of three frames. The $F/C$ ratio is the ratio of imposed
demand shear to the capacity of the frame system. The demand base shear \((F)\) is estimated based on the input motion spectral acceleration derived by a 5\% damping elastic response spectrum multiplied by the mass of the frame. In this approach, the spectral acceleration is the one corresponding to the first mode fundamental period of the structure (i.e. this case neglects higher modes contribution for simplicity). The capacity of the frame \((C)\) is obtained from a pushover analysis and defined as the ultimate strength of the frame system.

Table 3-6: Input motion characteristics of Christchurch individual earthquake records measured at station CBGS.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA</td>
<td>1466.00</td>
<td>1527.69</td>
<td>3453.90</td>
<td>5190.80</td>
<td>4377.70</td>
<td>1936.80</td>
<td>1423.80</td>
<td>1585.90</td>
<td>2294.00</td>
</tr>
<tr>
<td>PGV</td>
<td>21.83</td>
<td>8.77</td>
<td>18.14</td>
<td>62.95</td>
<td>33.18</td>
<td>9.40</td>
<td>20.36</td>
<td>16.04</td>
<td>29.53</td>
</tr>
<tr>
<td>PGA</td>
<td>10.73</td>
<td>3.58</td>
<td>26.57</td>
<td>23.48</td>
<td>5.52</td>
<td>1.55</td>
<td>5.83</td>
<td>6.38</td>
<td>5.13</td>
</tr>
<tr>
<td>V/A</td>
<td>0.15</td>
<td>0.06</td>
<td>0.05</td>
<td>0.12</td>
<td>0.08</td>
<td>0.05</td>
<td>0.14</td>
<td>0.10</td>
<td>0.13</td>
</tr>
<tr>
<td>AI</td>
<td>0.55</td>
<td>0.14</td>
<td>0.46</td>
<td>2.83</td>
<td>0.80</td>
<td>0.34</td>
<td>0.43</td>
<td>0.42</td>
<td>0.46</td>
</tr>
<tr>
<td>Ic</td>
<td>26927.78</td>
<td>7041.55</td>
<td>17673.59</td>
<td>87090.24</td>
<td>29613.64</td>
<td>13881.63</td>
<td>16748.44</td>
<td>1646.02</td>
<td>17641.47</td>
</tr>
<tr>
<td>HI</td>
<td>76.02</td>
<td>20.05</td>
<td>39.32</td>
<td>257.99</td>
<td>116.22</td>
<td>35.59</td>
<td>75.25</td>
<td>61.63</td>
<td>91.28</td>
</tr>
<tr>
<td>Vp</td>
<td>0.42</td>
<td>0.30</td>
<td>0.14</td>
<td>0.48</td>
<td>0.20</td>
<td>0.32</td>
<td>0.44</td>
<td>0.38</td>
<td>0.34</td>
</tr>
<tr>
<td>Tm</td>
<td>0.81</td>
<td>0.32</td>
<td>0.29</td>
<td>0.95</td>
<td>0.67</td>
<td>0.39</td>
<td>0.79</td>
<td>0.69</td>
<td>0.74</td>
</tr>
<tr>
<td>Sa_g</td>
<td>2350.74</td>
<td>455.31</td>
<td>796.48</td>
<td>6293.69</td>
<td>5746.74</td>
<td>953.31</td>
<td>2150.25</td>
<td>1440.34</td>
<td>3445.40</td>
</tr>
<tr>
<td>Sa_s</td>
<td>3341.48</td>
<td>3084.24</td>
<td>3269.19</td>
<td>14302.32</td>
<td>5889.86</td>
<td>3454.80</td>
<td>3978.78</td>
<td>4200.36</td>
<td>4503.60</td>
</tr>
<tr>
<td>Sa_c</td>
<td>3856.97</td>
<td>4327.28</td>
<td>9577.47</td>
<td>11463.65</td>
<td>5564.04</td>
<td>7406.88</td>
<td>3852.74</td>
<td>5777.07</td>
<td>4356.64</td>
</tr>
<tr>
<td>F/C_g</td>
<td>2.60</td>
<td>0.50</td>
<td>0.88</td>
<td>6.97</td>
<td>6.36</td>
<td>1.06</td>
<td>2.38</td>
<td>1.59</td>
<td>3.81</td>
</tr>
<tr>
<td>F/C_s</td>
<td>0.97</td>
<td>0.90</td>
<td>0.95</td>
<td>4.17</td>
<td>1.72</td>
<td>1.01</td>
<td>1.16</td>
<td>1.22</td>
<td>1.31</td>
</tr>
<tr>
<td>F/C_c</td>
<td>0.56</td>
<td>0.63</td>
<td>1.39</td>
<td>1.66</td>
<td>0.80</td>
<td>1.07</td>
<td>0.56</td>
<td>0.84</td>
<td>0.63</td>
</tr>
</tbody>
</table>

3.5.4 Individual records scaling

For replicate and random earthquake sequences, individual records are scaled to ensure constant demand to capacity \((F/C)\) ratio for all three frames. The demand is chosen to be higher than or equal to the capacity, where the \(F/C\) ratio ranges between 1.0 and 3.0. The capacity is defined as the maximum base shear value on the base shear versus displacement curve (i.e. ultimate base shear). The resulting equation used in calculating the scaling factor used to scale the records is:
\[ n = \frac{F \cdot V_{\text{capacity}}}{W \cdot S_a} \]  \hspace{1cm} (14)

where; \( n \) is the scaling factor, \( V_{\text{capacity}} \) is the capacity of the structure (defined from pushover analysis), \( W \) is the weight of the structure, and \( S_a \) is the spectral acceleration in terms of \( g \). The scaling factors used in the analyses are calculated based on \( F/C \) ratios equal to 1.00, 1.25, 1.50, 1.75, 2.00, 2.25, 2.5, 2.75, and 3.00 for the gravity, direct, and capacity designed frames. Figure 3-42, Figure 3-43, and Figure 3-44 show the scaling approach of replicate motions for the gravity, direct and capacity frames based on \( S_a \) ratio equals to 1.00. It is worth noting that real earthquake sequences obtained from Tohoku and Christchurch earthquakes are not scaled.

**Figure 3-42:** Scaling approach of replicate records for the gravity frame (\( S_a = 1.00 \)).
Figure 3-43: Scaling approach of replicate records for the direct frame ($S_u = 1.00$).

Figure 3-44: Scaling approach of replicate records for the capacity frame ($S_u = 1.00$).
3.6 Summary and Conclusion

In this chapter, degrading concrete and steel material models which are used in this study are introduced. The degrading features of both models are highlighted. The concrete model depicts the stiffness and strength degrading response based on a plastic-damage energy model; and the steel model utilizes the modified Menegotto Pinto model that considers Bauschinger effects, while adding other degrading features to it, including buckling and fracture of reinforcing bars. Moreover, the development of both material models is briefly discussed and optimization methods used to speed their analysis time, when these materials are utilized in large finite element fiber-based models under long duration non-linear dynamic analysis, are also provided. The implementation process of the models in the open source, Zeus-NL software is described in detail in this chapter. Uniaxial and flexural response of elements using the existing non-degrading and the newly implemented degrading models are compared with emphasis on the drawbacks of the non-degrading models and giving reasons why they cannot be used in this study given their simplicity in implementation and their cheapness in modeling and analysis.

The analytical models established using Zeus-NL platform is discussed. First, a simple pier model is built using the non-degrading and degrading materials. This model is used to provide an overall insight on the effects of degradation and damage accumulation on the response while saving modeling and analysis time. Hence it outlines a plan for future more advanced analyses, while highlighting the demand and supply effective parameters that need to be carefully studied.

Three complicated reinforced concrete fiber-based models are established to study accurately the behavior of typical reinforced concrete buildings when subjected to
multiple earthquakes shaking. The frame systems comprise three different design approaches, namely, gravity, direct, and capacity design concepts. These frames are the lateral resisting systems of a typical structure that is selected for this study. First, an overview of the structure in terms of location, dimensions, and properties are introduced. Then the three design approaches are summarized and a brief discussion on the frames properties is outlined.

Prior to conducting dynamic response history analysis of the structures described above, ground motions are selected based on how they are applied on the simplified pier models and frame systems. Replicate and random ground motion sequences are considered in order to simplify the parameterization of the problem with regards to input motion parameters only and also to perform an extreme point parametric analysis with more control on the input ground motions. For these sequences (replicate and random ground motion sequences), the input motion parameters were dependent on the structures characteristics; for example, the motions selection criteria is based on the ratio of the motion predominant period and the first mode fundamental period of vibration of the structure. Real ground motion scenarios of earthquake sequences are also included for this study. The March 11 and Christchurch earthquake records are used. Acceleration records measured at different stations for successive earthquakes are obtained. Motion characteristics are described. The records are applied in series to the structures with time buffer ranging between 10 to 20 seconds between the preceding and succeeding earthquake motion.
CHAPTER 4 RESULTS AND OBSERVATIONS OF DEGRADING SYSTEMS

4.1 Introduction

Dynamic response history analyses are performed in order to investigate the effect of multiple earthquake loadings on the performance of degrading reinforced concrete frames that are design using different approaches. Prior to conducting the dynamic analysis, an Eigen value of analysis of the frames is carried out to determine the dynamic characteristics of the frame systems, in terms of their mode shapes and fundamental periods of vibration for the first three modes. Pushover analysis is also overtaken to determine the properties of the frame systems in terms of stiffness, strength, and ductility, since they are the three main structural seismic parameters. Moreover, from the pushover analysis one can get an idea of design weakness and possible failure behavior of frames, in addition to determining the structure components that are prone to localized failure.

First, a comparison between the response captured from the non-degrading systems and the degrading ones is provided. Based on this comparison, the importance of using degrading material models while studying multiple earthquake effects is emphasized. Second, the behavior of the frame systems under different earthquake sequences, of different ground motion parameters, is discussed in detail.

4.2 Eigen Value Analysis and Fundamental Periods

Eigen value analyses are conducted to investigate the modes of vibrations and the fundamental periods of the structures. In the Eigen value analysis, lumped masses are assigned at the nodes of beam elements. The masses are calculated based on the dead and live load combination indicated in equation (15).
The first two natural periods of the structures are listed in Table 4-1. As listed in the table, the periods of vibration of the gravity frame are longer than their direct and capacity counterparts. This is because the gravity frame has lower initial stiffness than the direct and capacity frames respectively as listed in Table 4-2. It is worth noting that the mode participation factor of first mode exceeded 90% for the three frame systems.

**Table 4-1**: First two natural periods of the structures.

<table>
<thead>
<tr>
<th>Frame ID</th>
<th>T1 (sec)</th>
<th>T2 (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity</td>
<td>1.10</td>
<td>0.46</td>
</tr>
<tr>
<td>Direct</td>
<td>0.48</td>
<td>0.17</td>
</tr>
<tr>
<td>Capacity</td>
<td>0.28</td>
<td>0.09</td>
</tr>
</tbody>
</table>

### 4.3 Pushover Analysis

Conventional Pushover analyses are conducted on the frame systems to determine their characteristics in the lateral direction in terms of stiffness, strength and ductility. Moreover, localized failures in the frame systems, such as plastic hinging in beam/column elements and soft story behavior, are monitored. The pushover analysis is undertaken to shed light on design weaknesses of gravity and direct designed frames and to verify that the design concept of strong column weak beam approach is imposed on the capacity frame. Description of the pushover incremental-iterative solution is discussed in Elnashai and Di Sarno (2008).

Prior to starting the pushover analysis, the structure is subjected to constant gravity loads that are applied to the system using the seismic loading combination vertical loads which is:
where, the DL indicates dead load which includes structure self-weight, flooring and partitions, and LL indicates live load. Dead and live loads are calculated in APPENDIX A.

After applying constant gravity loads, the structure is subjected to monotonically increasing lateral force pattern. The force pattern using in this study is the inverted triangle load pattern since it is assumed that the frame systems dynamic response is governed by their first mode of vibration. Response control loading phase in Zeus-NL software, is used for the pushover analysis in order to capture the post-peak (softening) response of the frame systems.

The response of the frames is monitored in terms of plots of inter-story drift versus base shear and figures indicating the location of developed plastic hinges of the three types of frame systems. From the plots of drift versus base shear, stiffness, strength and ductility of the systems are estimated. The figures showing location of plastic hinges formation pattern throughout the analysis provide an overview of the failure behavior of these frame systems.

The gravity frame exhibit plastic hinges at columns only (Figure 4-1). This is because the columns are designed mainly under vertical loads, i.e. high axial forces and small bending moments, while the beams are designed for high moments due to transverse distributed dead and live loads. This design approach yield large section sizes of beams compared to column and deploys the weak column strong beam theory.
Figure 4-1: Plastic hinges in frame systems, i.e. the numbers indicate the sequence of plastic hinges formation with respect to load steps; (a) gravity, (b) direct, and (c) capacity designed frames.

In the direct designed frame, plastic hinges are initiated at the lower story where high lateral forces are imposed. This resulted in an observed soft story behavior at the first story level. High localized strains at the first story mid-column lead to global instability of the system while load redistribution was not observed. On the other hand, the capacity designed frame behaved as designed. Plastic hinges are developed at the beams first; actions redistribution that took place lead to localized failure in columns followed by collapse. The capacity frame exhibited the highest global stiffness, strength and displacement ductility values as shown in Table 4-2. The direct frame has the lowest ductility and the gravity frame has the lowest stiffness and strength.
Table 4-2: Stiffness, strength and ductility of frame systems.

<table>
<thead>
<tr>
<th>Frame ID</th>
<th>Stiffness (KN/m)</th>
<th>Strength KN</th>
<th>Ductility m/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity</td>
<td>93</td>
<td>179</td>
<td>4.80</td>
</tr>
<tr>
<td>Direct</td>
<td>670</td>
<td>680</td>
<td>3.93</td>
</tr>
<tr>
<td>Capacity</td>
<td>1670</td>
<td>1370</td>
<td>6.90</td>
</tr>
</tbody>
</table>

4.4 Comparison with Non-Degrading Models Predictions

The comparison between the degrading and non-degrading models predictions is provided in this section. Simple pier models discussed in section 3.3 as well as gravity frame systems discussed in section 3.4 are used in this comparison. The pier models are subjected to replicate motions, the response of the damaged piers under the second motion is reported using the degrading and non-degrading material models discussed in 3.2. A comparison between the predictions from both models is provided. In addition, the gravity frame systems are subjected to the March 11 Tohoku earthquake sequence, the degrading and non-degrading models predictions are also compared.

4.4.1 Simple pier model

For the simple pier models, a typical response of the undamaged (under one earthquake only) and damaged (under the second earthquake considering prior damage induced from the first earthquake) systems is plotted in Figure 4-2. The plot provides the displacement time histories of the non-degrading and degrading models respectively. It is shown that, for the non-degrading system, the displacement response of the damaged and undamaged piers match very well after the models reach their peak displacements. This observation was indicated by Aschheim (1999) when non-degrading simple degree of freedom
models were used to study the effects of repeated earthquakes on non-linear response of structures (Aschheim et al., 1999).

Prior to the peak displacement, the non-degrading displacement response shows longer period for the damaged system. This is explained as follows: (1) stiffness of the undamaged system at peak displacement reaches its lowest value and stays constant throughout the whole replicate motion analysis, since stiffness reduction in non-degrading models is influenced solely by the maximum displacement the system experiences; (2) P-Δ effects play a minimal role on stiffness reduction and that explains why the response after the peak displacement matched very well however the damaged systems experience residual displacements in some cases.

On the other hand, the response of degrading systems for the damaged and undamaged cases is quite different in terms of displacement amplitudes, and predominant periods of vibration. The response of damaged and undamaged systems is discrepant pre- or post-the peak displacement, unlike the non-degrading case. In addition, longer periods are captured for the damaged systems as shown in the figure.
Further analyses are conducted on pier models with varying natural periods of vibration under different PGA scaling levels for the replicate motion. The ratios between maximum displacements of the damaged and undamaged systems are calculated. The ratios variation under increasing PGA scaling levels is plotted in Figure 4-3. The figure shows that, for the non-degrading models, displacement ratios are slightly above unity; this is due to the P-∆ effects on the stiffness reduction of the non-degrading damaged systems, where residual displacements are introduced at the end of the first earthquake response. For the degrading systems, ratios are close to unity at low PGA levels where high inelasticity of materials is not introduced. Under higher values of PGAs, high
displacement ratios are observed in the degrading piers. Moreover, in some cases, the maximum displacement of the damaged model exceeds twice its undamaged counterpart. The relationship between the PGA and displacement ratio has not shown a specific trend.

![Graph showing displacement ratios vs PGA for non-degrading and degrading models.](image)

**Figure 4-3:** Maximum displacement ratios of damaged and undamaged pier models (0.12 seconds period).

### 4.4.2 Frame models

The top displacements of the gravity frame are monitored when subjected to the March 11 Tohoku earthquake sequence. Significant records monitored at station IWT010 discussed previously in section 3.5 are selected and used in this analysis. This station measured records of high PGAs at the March 11 main shock and the April 7 aftershock, the acceleration time histories records of both earthquakes are plotted in series in Figure 3-40. Top displacements response for the non-degrading and degrading gravity frame
models are reported in Figure 4-4 and Figure 4-5 for the damaged and undamaged cases. A zoom in the response of the damaged and undamaged systems under the April 7 aftershock (from 315 to 330 seconds and plotted on top of each other) is provided to better compare the displacement amplitudes and periods for the damaged and undamaged systems using both models.

Similar observations, to the non-degrading pier mode results, are revealed for the non-degrading frame. The damaged response of the non-degrading frame under the April 7 aftershock matches very well with the undamaged frame response, post the peak displacement while the behavior prior to the peak displacement observes longer period and larger displacements. For the degrading model, the displacements under the April 7 aftershock for the damaged and undamaged cases are discrepant (Figure 4-5). Longer periods and larger displacement amplitudes are revealed for the damaged system; in addition higher residual displacements are monitored for the damaged case. In summary, a totally different response is predicted for the gravity designed frame under the April 7 aftershock if the damage effects of the main shock are considered in the analysis when the degrading material models are utilized. This highlights the importance of including damage dependent models in studying the effects of multiple earthquakes shaking on the response of structures. The results described confirm that non-degrading models cannot accurately predict the response of structures under more than one earthquake. Hence further analyses in this study are conducted using the degrading models only.
Figure 4-4: Top displacements monitored for the non-degrading gravity frame model under (a) two and (b) one earthquakes; and (c) a comparison of damaged and undamaged response.
Figure 4-5: Top displacements monitored for the degrading gravity frame model under (a) two and (b) one earthquakes; and (c) a comparison of damaged and undamaged response.
4.5 Response of Degrading Frame Systems

In this section, the behavior of the frame models namely, gravity, direct, and capacity designed frames described in section 3.4, is studied. The frame models discussed in this section and all succeeding sections utilize the degrading steel and concrete material models explained in sections 3.2.1 and 3.2.2 respectively.

The frame models are subjected to different combination of ground motion sequences namely, replicate, random and real earthquake sequences discussed in section 3.5. Dynamic response history analyses are conducted and motions are applied in series as discussed. Sample results from the analyses are provided in this section to give an insight on the performance of each type of frames separately.

4.5.1 Gravity frame

i Replicate motion

The gravity frame is subjected to two identical Loma Prieta earthquake ground motions, the characteristics of this motion is indicated in section 3.5.1 for replicate ground motions. Two scaling levels are considered using $F/C$ ratios of 1.00 and 2.75. The purpose of having these two distinct scaling levels is to investigate the effect of accelerations amplitude on the degrading response of the structure. Figure 4-6 shows inter-story drift response histories of the first, second and third stories.

For $F/C$ ratio equals to 1.00, the maximum inter-story drifts reported for the undamaged frame are 0.35%, 0.42%, and 0.84% for the first, second and third stories respectively. For the damaged frame the inter-story drifts are 0.38%, 0.44%, and 0.93%. The percentage increase of inter-story drifts in the damaged case is 8.57%, 4.79%, and 10.71% compared to the undamaged case. For $F/C$ ratio equals to 2.75, inter-story drifts
are 1.44%, 1.23%, and 2.16% for undamaged case and 2.00%, 1.67%, and 2.55% for the damaged case; with a percentage increase of inter-story drifts in the damage case of 38.89%, 35.77%, and 18.06% for the first, second and third stories. It is worth noting that no plastic hinges were developed in the gravity frame beam and column elements at $F/C = 1.00$ for the undamaged frame, while three plastic hinges were formed due to the second earthquake. In case of $F/C = 2.75$, twenty four and twenty seven plastic hinges are developed for the undamaged and damaged cases respectively. Plastic hinges definitions and formation are discussed in section 5.2.2.

**Figure 4-6:** Inter-story drifts under two identical Loma Prieta ground motions, $F/C = 1.00$ (up) and 2.75 (down).
The results indicate that the ground motion amplitude for the replicate motion case has a significant impact on the response of the damaged model when compared to its undamaged counterpart. This is due to that when applying higher acceleration amplitudes, larger forces are imposed on the systems, and consequently higher inelasticity is introduced at the material level resulting in higher degradation.

![Figure 4-7: Inter-story drifts under sequences 3-10 (up) and 10-3 (down).]

**i Random motion**

In addition to studying the response under replicate motion, the frame behavior is studied under random motions as well. Motions number 3 and 10 are used in this section. The sequences applied to the gravity frame are sequences 3-10 and 10-3, the sequences characteristics are discussed in section 3.5.2. For more clarification on the notations used
to define the random earthquake sequences, in sequence 3-10, record number 3 is applied first to the frame system followed by record 10; while the opposite is true for sequence 10-3. Therefore, sequences 3-10 and 10-3 could be defined as reverse motion sequences.

Figure 4-7 shows the inter-story drifts of the gravity frame under the two sequences. In addition, strain time histories monitored at the reinforcing bars of sections located at both ends of column C2 are shown in Figure 4-8. For sequence 3-10, it is noted that the tensile strains exceeded the yield strain during both individual earthquakes 3 and 10. Moreover, the strains did not reach twice the yield during the whole sequence. On the other hand, in sequence 10-3, the strains were below the yield strain during record 10 however during record 3, the strains exceeded twice the yield.
Figure 4-8: Location of reinforcing bars where strains are monitored at (up), strains due to earthquake sequences 3-10 (middle) and 10-3 (down).

Table 4-3 provides a comparison between the frames response in terms of maximum inter-story drifts and number of developed plastic hinges monitored during the first and
second records under sequences 3-10 and 10-3. The number of plastic hinges developed due to 3-10 and 10-3 sequences are 11 and 13 hinges; while for the first record only, the number of developed hinges is 11 and 6. The discrepancies reported in the inter-story drifts and strains of the frame systems under the reverse motion sequences indicate that the order of applied motions significantly affect the behavior of the frames.

**Table 4-3**: Inter-story drifts and number of developed plastic hinges during the 1st and 2nd records of earthquake sequences 3-10 and 10-3.

<table>
<thead>
<tr>
<th>ID (%)</th>
<th>3-10</th>
<th></th>
<th>10-3</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st story</td>
<td>2nd story</td>
<td>1st story</td>
<td>2nd story</td>
</tr>
<tr>
<td>1st story</td>
<td>1.03</td>
<td>0.73</td>
<td>0.59</td>
<td>1.12</td>
</tr>
<tr>
<td>2nd story</td>
<td>0.87</td>
<td>0.71</td>
<td>0.58</td>
<td>0.95</td>
</tr>
<tr>
<td>3rd story</td>
<td>0.83</td>
<td>0.85</td>
<td>0.95</td>
<td>1.10</td>
</tr>
<tr>
<td>Number of Plastic hinges</td>
<td>11</td>
<td>11</td>
<td>6</td>
<td>13</td>
</tr>
</tbody>
</table>

**Christchurch earthquake sequence**

The gravity frame is subjected to the Christchurch earthquake sequence using records measured at station CBGS. Two cases are considered in the analysis. The first case (case 1) considers all nine records measured at the station while the second case (case 2) excludes prior shaking of the first three records and considers only the last six records. The inter-story drifts are plotted in Figure 4-9. In case 1, the analysis was not completed due to convergence problems that occurred during the fourth record. This convergence was caused due to excessive drifts, which exceeded 10%, at the first story. In case 2, the analysis under the fourth to ninth records series converged and a maximum drift of 3.04% is reported at the first story. These results ensure that the effects of prior shaking of the first, second and third records caused the complete collapse of the first story of the
gravity frame system during the application of the forth record. However, when prior shaking effects are not considered, the frame withstood the forth earthquake as well as all the subsequent earthquakes in the sequence.

Figure 4-9: Inter-story drift of the gravity frame under earthquake sequence measured at station CBGS in the Christchurch earthquake; response of the frame under the whole earthquake sequence (up); response of the frame under the last six records only (down).

4.5.2 Direct designed frame

i Replicate motion

Similar to the gravity frame, the direct frame is subjected to replicate motion of Chi-Chi earthquake record. The drifts of the first, second and third stories are presented in Figure 4-10. Different scaling levels are considered for $F/C$ equal to 1.00 and 2.75. The
maximum drifts reported for the undamaged frame at the first, second and third stories are 0.34%, 0.29%, and 0.15% for \( F/C \) ratio equal 1.00 and 1.51%, 1.00%, and 0.39% for \( F/C \) equal 2.75. The maximum drifts for the damaged case are 0.37%, 0.32%, and 0.16% for \( F/C \) equal 1.00 and 1.89%, 1.26%, and 0.36% for \( F/C \) equal 2.75. Higher drifts are reported for the damaged frames; however the third story drift, for \( F/C \) equal 2.75, monitored lower drift for the damaged frame (0.36%) when compared to the undamaged frame third story drift (0.39%). This is due to the development of first and second floor soft-stories that resulted in a low stiffness of the frame system in the lateral direction which lead the structure to attract less forces imposed on their above third story.

**Figure 4-10:** Inter-story drifts under replicate motion of Chi-Chi earthquake, \( F/C = 1.00 \) (up) and 2.75 (down).
Discrepancies of the inter-story drifts for the damaged and undamaged frames at $F/C$ equal 2.75 are revealed in terms of maximum values of drifts. Moreover, times corresponding to the peak drifts of the undamaged frame did not match with the times of peak drifts occurrence for the damaged case. For example, the maximum first floor inter-story drift, for the undamaged frame occurred at the second 18.155 measured with respect to the beginning of the record. The maximum first story drift for the damaged case occurred at the second 17.23 measured from the start of the second record. This indicates that peak displacements may not occur at the same time for damaged and undamaged cases.

**ii  Tohoku earthquake sequence**

The response of the direct frame under the Tohoku earthquake sequence is presented in this section. Significant records measured at station FKS013 are used (see section 3.5.3 for records characteristics). The sequence consists of six records, 1 to 6, of PGAs of 0.36g, 0.37g, 0.15g, 0.19g, 0.11g, and 0.31g. The first, second and sixth earthquakes are the most devastating ones within the sequence since they have the highest PGAs and correspond to earthquakes of high magnitudes (Figure 3-38). Figure 4-11 shows the inter-story drifts of the direct frame, the maximum first story drifts reported for records 1, 2, and 6 are 0.89%, 1.48%, and 1.15%. It is worth noting that the maximum first-story drift reported during record 6 is higher than the drift reported during record 1; however record 1 has a higher PGA almost 1.2 times the PGA of record 6.
The top displacements of the direct frame are compared for the undamaged and damaged cases under record 2. For the damaged case, effects of prior shaking due to record 1 is considered, while for the undamaged case, record 2 is applied to the frame in its initial state. The top displacements for the damaged and undamaged frames are plotted on top of each other as shown in Figure 4-12. The plot shows a similar response of both cases within time frames of 290-295 and 304-308 seconds, however discrepancies are revealed between the 295 and 304 seconds as well as 308 and 320 seconds. It is worth noting that the discrepancies in terms of displacement amplitude and period elongation is not as
significant when compared with the gravity frame response of damaged and undamaged cases.

![Graph showing top displacements monitored for the damaged and undamaged direct frames under record 2 of FKS station, Tohoku earthquake sequence.]

**Figure 4-12:** Top displacements monitored for the damaged and undamaged direct frames under record 2 of FKS station, Tohoku earthquake sequence.

### 4.5.3 Capacity designed frame

The capacity frame is subject to Tohoku earthquake sequence measured at station IWT007. Two significant records are used in the analysis; the first comprises the March 11 main shock and second comprises the April 7 aftershock. Inter-story drifts of the first, second and third floors are plotted in Figure 4-13. Higher drifts are observed during the April 7 aftershock due damage induced by the main shock.
Figure 4-13: Inter-story drifts of the capacity frame under Tohoku earthquake sequence, station IWT007.

A comparison between the initially damaged and undamaged frames is conducted. Figure 4-14 shows the top displacements of both frames. Fewer discrepancies are observed for the capacity frame response in the damaged and undamaged state in terms of overall frame displacement response. However, the maximum top displacement reported for the damaged frame under the April 7 aftershock is 37.63 mm at time 18.00 seconds measured from the beginning time of the aftershock record. For the undamaged frame, the maximum top displacement occurred at time 18.23 seconds and its value is 30.36 mm.
Figure 4-14: Top displacements of the damaged and undamaged frames under the April 7 aftershock, station IWT007.

4.6 Summary and Concluding Remarks

In this chapter, results obtained from the numerical analysis are discussed. First, Eigen value and pushover analyses of the frame systems are conducted. Second, a comparison between the non-degrading and degrading models under repeated earthquakes is provided. Third, the response of the frames under different combinations of earthquake sequences is studied.

Eigen value analyses are conducted for the three frame systems to determine their dynamic characteristics in terms of fundamental periods of vibration and mode shapes.
Pushover analyses are overtaken in this chapter to investigate the inelastic behavior of the frame systems when subjected to lateral loading. The results obtained from the pushover analysis indicated that the capacity frame has the highest stiffness, strength and ductility values, the gravity frame has the lowest strength and direct frame has the lowest ductility.

A comparison between the non-degrading and degrading models is provided. In this comparison, simple pier models and gravity frame systems utilizing both the non-degrading (stl1 and con2) and degrading (stl4 and con5) materials are subjected to same earthquake sequences. The results predicted using both models indicated that the non-degrading models are not appropriate to be used in this study since they do not accurately depict the effects of prior damage on the response of reinforced concrete structures. Drawbacks of using non-degrading models in studying the response of structures under repeated shaking are highlighted. Hence, further analyses in this chapter and till the end of the dissertation are conducted using degrading models only.

The degrading response of the gravity, direct, and capacity frames under different earthquake sequence combinations is presented. The results provided an insight on the behavior of each frame system under multiple earthquakes. Damaged and undamaged gravity designed frames showed high discrepancies in their response; discrepancies are less for the direct frame and even lesser for the capacity designed frame. The reason is, in the capacity frame high inelasticity is introduced in the beam elements when compared to columns. Hence columns in the capacity frame experience less degradation when compared to gravity and direct frames columns; when beams stiffness and strength deteriorate, the case for the capacity frame, load redistribution can take place and high ductility is maintained unlike column degradation (gravity and direct frames case).
In the next chapter, extensive analyses are conducted on the gravity, direct and capacity frames using wide range of earthquake sequence combinations. The response of the frames is compared. Guidelines for design and assessment are provided.
CHAPTER 5 ANALYTICAL INVESTIGATION AND DESIGN IMPLICATIONS

5.1 Introduction

In this chapter, non-linear dynamic response-history analyses are carried out using a suite of 70 earthquake sequences (section 3.5) to investigate the performance of reinforced concrete frames under multiple earthquake loadings. The goal of the parametric studies is to assess the effects of varying various demand and supply parameters on the global and local behavior of the frames. Demand parameters are divided into, parameters related to individual records such as PGA, frequency content, etc. and parameters related to earthquake sequence such as number of earthquakes, the order of individual earthquakes in the sequence, etc. For the supply, effects of design approaches, discussed in section 3.4.2 for gravity, direct and capacity frames along with their different design parameters, on the frames seismic performance are highlighted.

The response of the three frame systems is reported in terms of global and inter-story drifts monitored during each individual earthquake, period elongation caused from one earthquake to the other in the earthquake sequence, and plastic hinges development throughout the whole earthquake sequences. A comparison of the seismic behavior of each frame system is provided. Highlights on design guidelines are provided in order to increase the safety of frames located in regions prone to multiple earthquake shakings.

5.2 Comparison of Response History Analysis Results

In this section a comparison between the results of the gravity, direct and capacity frames is provided. The results are shown in terms of maximum global and inter-story
drifts, number of plastic hinges and period elongation. The results are provided for the three aforementioned earthquake sequence cases, replicate, random and real.

5.2.1 **Global and inter-story drifts**

*i Replicate earthquake sequence*

Figure 5-1 shows a comparison of the maximum total, first, second and third story drifts for all three frame systems due to the first and second records of the code compatible replicate earthquake sequence. As shown in the figure, different scaling levels of $F/C$ ratio are introduced. The drifts almost increase linearly with the linear increase of $F/C$ ratio for the three frame systems. The gravity frame exhibited the largest displacements and drifts in the first and second earthquakes while the capacity frame experienced the lowest.

Figure 5-2 and Figure 5-3 show the same results for the Chi-Chi and Loma Prieta replicate motion sequences respectively. In the Chi-Chi earthquake sequence, the capacity frame experienced the largest drifts while the gravity frame experienced the least. In the Loma Prieta case, the opposite is observed since the gravity frame had the highest drifts and capacity had the lowest. It is worth noting the Chi-Chi earthquake record is measured on soft soil and has a long predominant period (1.2 sec) while the Loma Prieta record is of short period (0.3 sec) and is measured on hard rock soil conditions. The fundamental periods of the gravity and capacity frames are 1.10 and 0.28 seconds respectively. Therefore it can be concluded that records of predominant period distinct to the structure period impose higher demands on the systems if the same $F/C$ ratio is maintained.
Figure 5-1: Effect of $F/C$ ratio on the drifts the maximum drifts during the first and second code compatible earthquake replicate sequence.
Figure 5-2: Effect of $F/C$ ratio on the maximum drifts during the first and second Chi-Chi earthquake replicate sequence.
Figure 5-3: Effect of $F/C$ ratio on the maximum drifts during the first and second Loma Prieta earthquake replicate sequence.

**ii Random earthquake sequence**

Random earthquake sequences discussed in section 3.5.2 are applied to the gravity frame under different scaling levels of $F/C$ ratio as shown in Figure 5-4. The random sequences for the gravity frame consists of 3 records that form 9 sequences combinations as shown in Table 3-1. The records used are records number 3, 10, and 12. Records 3 and 12 represent ground motions of predominant periods (0.40 and 1.40 sec respectively) distinct to the first mode fundamental period of the gravity frame (1.1 sec). Record 10 predominant period (1.12 sec) matches with the frame period.
Figure 5-4: Maximum top drifts of gravity frame, due to first (1) and second (2) motion, under random earthquake sequences; (a) $F/C = 1.00$; (b) 2.00; and (c) 3.00.

For $F/C$ equal to 1.00, the difference between maximum top drifts induced by the first and second records is negligible for all nine sequence combinations. This is due to that high inelasticity is not introduced to the frame system at $F/C$ equal 1.00 and hence less degradation is induced. For $F/C$ equal to 2.00 and 3.00 discrepancies between the maximum drifts are observed for each earthquake sequence, even reversed ground motion
sequences reported different values of drifts. Highest drift values are observed at motion sequences that contain records 3 and 12 (3&3, 3&12, 12&3, and 12&12). These two records are of predominant periods distinct to the frame fundamental period of vibration. Similar to the gravity frame, response history analyses are conducted on the capacity frame under random motion sequences. Figure 5-5 shows the maximum top drifts of the capacity frame for $F/C$ ratio equal to 1.00, 2.00 and 3.00. The records used in this analysis are number 12 and 26 of predominant periods 1.4 and 0.3 sec respectively. The capacity frame fundamental period is 0.28 sec which matches with record 26 predominant period.

Unlike the gravity frame results, maximum drift values of the capacity frame for the scaling level of F/C equal 1.00 are not close for the first and second records. For F/C equal 3.00, displacements induced due to 26-12 sequence showed largest value followed by 12-26. Reverse sequences (12-26 and 26-12) showed different results. The 12-26 sequence induced maximum displacements of 83.72 mm (undamaged case, record 12) and 64.72 mm (damaged case, record 26) during records 12 and 26 respectively. The 26-12 sequence induced maximum displacements of 27.87 mm (damaged case, record 26) and 86.87 mm (damaged case, record 12). The displacement of the capacity frame due to record 26 in the damage case (taking into account prior damage effects of record 26) is 232% higher than the maximum displacement for the undamaged case. On the other hand the damaged and undamaged displacements due to record 12 relatively are about 3.6% different.
iii Real earthquake sequence

Christchurch records measured at station CBGS are subjected to gravity, direct and capacity frames. At this station, nine significant records are selected to be used in the analysis as discussed in section 3.5.3. Individual records characteristics are shown in Table 3-6. In order to compare the damaged and undamaged systems response, nine
sequences are considered in the analysis. The first sequence (indicated as sequence 1 at
legend of Figure 5-6) comprise record number 9 only (in this case prior damage to
records 1 to 8 are not considered), the third sequence (3) consists of records 7, 8 and 9 in
series, while the ninth sequence (9) considers the whole sequence starting with records 1
and ending with 9, and so on and so forth. The top displacements of the frame systems
under the nine earthquake sequences are shown in Figure 5-6.

![Graphs showing top displacements](image)

**Figure 5-6:** Maximum top displacements of (a) gravity, (b) direct, and (c) capacity frames under selected
sequences of CBGS records.

The results show that highest displacements are reported at the gravity frame and lowest
by the capacity frame. The gravity frame under sequence 1 showed similar maximum
displacements due record 9, when prior shaking effects due to records 5, 6, 7 and 8 are
considered (earthquake sequences 1, 2, 3, 4, and 5). However, earthquake sequences 6, 7,
8 and 9 showed higher maximum displacements at record 9. It is worth noting that the maximum displacement at record 9 due to sequence 5 is equal to 70.83 mm while its equal to 201.29 for sequence 9. Similar observations are revealed for the direct frame. Unlike the gravity and direct frames, the capacity designed frame maximum top displacements at record 9 due to sequences 1 and 9 are equal to 12.53 mm and 17.00 mm respectively. The maximum displacement for the damaged case is almost 1.36 times their damaged counterparts, while for the gravity frame damaged displacement is 3.40 the undamaged one.

The same analysis approach used in the analysis of frames under the Christchurch sequences is followed but using records obtained from the Tohoku earthquake sequence. Records measured at station FKS013 are used. Figure 5-7 shows maximum top drifts of the frame systems in their damaged and undamaged cases. In this case six earthquake sequences are used. Similar to the Christchurch sequences, sequence 1 contains the sixth record only while sequence 6 contains all records from 1 to 6 applied in series. The results revealed that the capacity and direct frames performed well since the displacements at record 6 due to sequences 1 to 6 did not significantly vary. The ratios between the completely damaged (due to sequence 6) frame maximum top displacements at record 6 their undamaged (due to sequence 1) counterparts are 1.65 (73.77 mm / 44.72 mm), 1.49 (73.64 mm / 49.46 mm), and 1.34 20.52 mm / 15.28 mm, for the gravity, direct and capacity frames respectively.
5.2.2 Plastic hinges

Similar to what is done in the previous section to capture the displacements and drifts response; replicate, random and real earthquake sequences are subjected to the gravity, direct and capacity frames to capture the development of plastic hinges. In this section, plastic hinges formation at the ends of beam and column elements, during the individual records in the selected earthquake sequences are monitored. A plastic hinge is formed when the strains of the reinforcing bars at the sections located at a distance d/2 from the perpendicular element centerline exceed the yield strain of steel; where d is the depth of the perpendicular element as shown in Figure 5-8.

Figure 5-7: Maximum top displacements at the (a) gravity, (b) direct, and (c) capacity frames under selected sequences of FKS013 records.
Figure 5-8: Beam-column joint, figure shows reinforcing bars where strains are monitored at (marked in red).

i Replicate earthquake sequence

Plastic hinges developed under replicate motions during the first and second earthquakes are shown in Figure 5-9, Figure 5-10, and Figure 5-11 for the code compatible, Chi-Chi and Loma Prieta replicate sequences respectively. The figures report the number of plastic hinges formed with varying the $F/C$ ratio of the gravity, direct and capacity frames. The maximum number of plastic hinges that could be developed in the frame systems is 60. As shown in figures, the capacity design frame developed the maximum number of hinges. This is due to the ability of the capacity frame to redistribute the strains from local regions of high inelasticity to regions of lower inelasticity during the
earthquake sequences. Moreover, this indicates that capacity frames are better energy dissipative frames than their gravity and direct designed counterparts.

Figure 5-9: Effect of force/capacity ratio on the number of plastic hinges developed in frames after applying the 1\textsuperscript{st} and 2\textsuperscript{nd} compatible ground motions. Number of plastic hinges calculated based on criterion 1 (up) and 2 (down).
Figure 5-10: Effect of force/capacity ratio on the number of plastic hinges developed in frames after applying the 1st and 2nd identical ChiChi earthquake accelerations. Number of plastic hinges calculated based on criterion 1 (up) and 2 (down).
Figure 5-11: Effect of force/capacity ratio on the number of plastic hinges developed in frames after applying the 1st and 2nd identical Loma Prieta earthquake accelerations. Number of plastic hinges calculated based on criterion 1 (up) and 2 (down).

ii Random earthquake sequence

Figure 5-12 shows the number of plastic hinges developed for the gravity and capacity frames under the same random earthquake motions discussed in previous section at F/C equal 3.00. The capacity frame produced more plastic hinges during the earthquake sequences. In addition, plastic hinges recovery during the second earthquake is also observed for the capacity frame system since most of the capacity frame hinges are developed in beams and hence can be recovered easily in smaller subsequent shaking.
Recovery of plastic hinges developed in the first earthquake is reported in sequence 12-26 for the capacity frame, no recoveries are observed for the gravity frame.

![Chart](image)

**Figure 5-12**: Plastic hinges developed in the (a) gravity and (b) capacity frame systems.

### iii Christchurch earthquake sequence

The number of plastic hinges during the Christchurch earthquake sequence is reported in Figure 5-13 for the gravity, direct and capacity frames. The maximum numbers of plastic hinges developed in three frame systems are, 24, 26, and 21 for gravity, direct and capacity frames. However the maximum displacements showed discrepancies in the three frame systems. This is due to that plastic hinges are mainly developed in beam elements.
of the capacity frame unlike the other frame systems where plastic hinges are formed at columns. Development of Plastic hinges at beams limits the degradation effects on the global behavior of the frame system and prevents the occurrence of soft stories.

**Figure 5-13:** Plastic hinges formation under the Christchurch earthquake sequence (station CBGS).

### 5.2.3 Period Elongation

The periods of frame systems under the first, second, third, etc. earthquakes are predicted. Period elongation of the three frame systems due to repeated earthquakes is calculated based on the Fourier transformation. Fourier transformation is a mathematical representation of the amplitudes of the signal by decomposing a function into oscillatory function. The discrete Fourier transformation (DFT) is a periodic sequence of sampled values \( \{x_n\}_{n=0}^{N-1} \) of period \( N \) (or number of sample \( N \)) transformed into \( X_p \) values of using the following equation:
\[ X_p = \sum_{n=0}^{N-1} x_n e^{-j \frac{2\pi n p}{N}}, \quad p \in \{0, 1, ..., N - 1\} \quad (16) \]

Where \( e \) denotes the natural exponent and \( j = -1 \), and \( x_n \) is a complex number equal to \( x_{\text{real}} + j x_{\text{imag}} \). Similar to the DFT, the Fast Fourier Transform (FFT) of a periodic function is an extraction of the series of the sines and cosines for which the function is made up of (i.e., the superposition of the sines and cosines reproduces the function). In fact, FFT is nothing but an efficient algorithm used to compute the DFT and its inverse. A real periodic function \( x(t) \) can be expressed as sum of trigonometric series \((-L < t < L)\) as:

\[ x(t) = \frac{1}{2} a_o + \sum_{n=1}^{\infty} \left( a_n \cos \frac{\pi n}{L} t + b_n \sin \frac{\pi n}{L} t \right) \quad (17) \]

For which the coefficients can be computed by:

\[ a_n = \frac{1}{L} \int_{-L}^{L} x(t) \cos \frac{\pi n}{L} t dt \quad (18) \]
\[ b_n = \frac{1}{L} \int_{-L}^{L} x(t) \sin \frac{\pi n}{L} t dt \quad (19) \]

The generalization of the continuous Fourier series for infinite domains can be expressed by:

\[ x(t) = \int_{-\infty}^{\infty} F(f) e^{-2\pi if t} df \quad (20) \]

When the FFT is carried out on the function above, the result and imaginary terms for \( F(f) \) defined at all frequencies that indicates how big the amplitude of the sin wave has to be to make the function \( x(t) \) for all frequencies. The resulting \( F(f) \) is defined as:
The FFT algorithm within MATLAB, which is a high level technical computing language, is used to conduct the FFT on relative roof acceleration with respect to the ground acceleration. This is done to provide an insight on the predominant frequency response of the structure. This proves to be helpful when the response is predominantly mode one and the natural frequency of the structure are well spaced. Fourier transform is considered the primary tool for signal processing and interpretation of system response. It provides an insight on the inelastic period of the structure corresponding to period elongation when the response of the system is governed by the first mode.

\[ F(f) = \int_{-\infty}^{\infty} x(t)e^{-2\pi ift} dt \]  

(Figure 5-14: FFT of the roof acceleration of the gravity frame under the first and second earthquakes of 1989 Loma Prieta replicate motion.)
Figure 5-14 shows the FFT of the damaged and undamaged direct frame under different record scaling, F/C equal 1.25, 1.75, 2.25, and 2.75. The predominant period of the structure is estimated as the period corresponding to the peak FFT power. For F/C equal to 2.75, the predominant period under the first record (undamaged frame) is 0.48 sec, while the elongated period for the damaged structure is 0.57 sec. This indicates that the damage to the structure induced in the first earthquake elongated the period of vibration of the frame and hence the frame response is altered. This figure provides an example of period elongation of frames due to prior shaking. Similar observations are reported for the gravity and capacity frames not only under replicate motion but also under random and real earthquake sequences.

5.3 Case Study

In this section a case study that demonstrates the differences in the behavior of frame systems that are designed based on different design approaches. The three aforementioned frame systems namely gravity-, direct-, and capacity-designed frames are subjected to the Tohoku earthquake sequences measured at stations FKS013, IWT007, and IWT010. The response of the three frame systems in their damaged and undamaged conditions is studied and a comparison is provided to highlight the significance of design parameters on the response.

This section only provides a case study using only the Tohoku earthquake sequence. It also includes three frame systems that comprise three different design approaches only. The aim of this section to provide some insight on significant parameters that affects the response of RC frames under repeated shaking. These parameters can helpful to future researchers in parameterizing the problem. Problem parameterization is complex due to
the existence of many parameters related to the structures and input motions. Also, the interaction between the structural parameters and input motion play an important role in identifying the response of the reinforced concrete frames under repeated shaking. Figure 5-15 provides a flowchart that shows the parameters of significance and their interaction.

Figure 5-15: Flowchart showing significant parameters affecting the response of RC frames subjected to multiple earthquakes.

Figure 5-16 shows a comparison of the largest absolute top drifts monitored for the three frame systems during the simulations under the Tohoku earthquake sequences, stations FKS013, IWT007, and IWT010. The simulations are conducted under the main-shock only (case 1), aftershocks only (case 2) and the complete earthquake sequence of main-shock and aftershocks (case 3). The aim of this analysis is to compare the largest absolute
top drifts experienced at each frame system while including (case 3) and excluding (cases 1 and 2) multiple earthquake effects.

Figure 5-16: Top drifts of frame systems; (a) gravity; (b) direct; and (c) capacity.

For station IWT010, the maximum absolute drifts for the capacity frame in cases 1, 2 and 3 are 0.42%, 0.42%, and 0.48%; similarly, for the direct frame 0.68%, 1.04%, and 0.98%; while for the gravity frame, the drifts are 0.78%, 1.32%, and 1.72%. It is noted that the maximum drifts monitored at the capacity frame did not differ much in case of including and excluding the multiple earthquake effects, this means that damage accumulation does not significantly influence the response of capacity designed frames unlike the direct and gravity designed frame systems.

Similarly, Figure 5-17 shows the number of developed plastic hinges for all three frame systems (cases 1, 2 and 3). The capacity designed frame generated the maximum number
of plastic hinges as shown in the figure and also maintained the lowest drift values (Figure 5-16). In contrary the gravity designed frame developed the lowest number of plastic hinges and experienced the largest values of drifts.

![Figure 5-17: Number of developed plastic hinges; (a) gravity; (b) direct; and (c) capacity.](image)

5.4 Qualitative Observations

Current design codes do not consider multiple earthquake effects, however structures are designed to withstand seismic loads imposed due to the most damaging earthquake the structure experiences throughout its lifetime. For ordinary structures, the design lifetime of the structure is usually defined as 50 years. The structure is designed to withstand an earthquake of magnitude of 10% probability of exceedance during the structure 50 years life time (earthquake of return period equals to 475 years). The design earthquake is magnitude based on the mentioned return period is estimated from seismic hazard maps.
developed based on the region seismicity. Damage accumulated due to smaller earthquakes of high frequency (high probability of exceedance) is not accounted for.

In this study different design approaches are introduced and their seismic performance under repeated earthquakes is investigated. Moreover, the degradation effects on the response of frames are studied by comparing the previously damaged and initially undamaged frames performances. The damaged and undamaged capacity designed frames response presented in this study showed the least discrepancies when compared the damaged and undamaged gravity and direct designed frames however high inelasticity was observed in the three frame systems. The inelasticity level was measured by the number of plastic hinges developed throughout the earthquake sequence. This indicates that the capacity design approach limits the degradation effects on the global frame behavior unlike the direct design approach. This is due to that the degradation in capacity frames are introduced only in beam elements (energy dissipative zone) and not in columns (load carrying elements), this scenario allows for load redistribution and prevents soft-story behavior which imposes high residual displacements that yield unfavorable behavior of structures under repeated earthquakes.

In design practice, local failure of materials, sections and elements is acceptable under some conditions. Local failure usually occurs due to high inelasticity introduced to structural components. In reinforced concrete structures high inelasticity is introduced mainly due to (1) yielding of reinforcing bars in tension/compression; (2) cracking of concrete in tension; (3) crushing of concrete in compression; (4) buckling of reinforcing bars; and (5) bars fracture. This study categorizes the failure behavior into two categories: (1) favorable failure; and (2) unfavorable failure. The favorable failure behavior is
defined as the failure of reinforced concrete components that does not significantly affect the level of deterioration in the stiffness and strength of structure; while the unfavorable behavior is the failure that causes dramatic loss of global stiffness and strength of the system.

The favorable failure behavior includes cracking of concrete and yielding of reinforcing bars. Cracking of concrete is unavoidable especially in seismic design of reinforced concrete structures; same applies to reinforcing bars yielding. These two damage features cause a dramatic loss of material stiffness however the stiffness recovers when smaller actions are imposed with no significant deterioration. The unfavorable behavior of materials such as concrete crushing, steel bucking and fracture cause a dramatic loss of stiffness and strength of the structural components that are not recoverable.

As for design guidelines, based on this study the following is recommended to increase the safety of reinforced concrete frame systems under multiple earthquakes by following the guidelines listed below for designing frame columns and beams.

a) For columns:
   - Capacity design approach should be imposed (Elnashai and Di Sarno 2008).
   - At columns ends, yielding of reinforcing bars and cracking of concrete is allowed only if the sections of the connecting beams ends yield and crack first.
   - Strains of concrete should not exceed the crushing strain; this can be achieved by designing appropriate level of reinforcement that enforces yielding of these bars with no crushing of concrete.
   - Buckling of reinforcing bars is prohibited; this is achieved by having the bars confined with uncrushed concrete under compression. In addition, stirrups
spacing should maintain a least of 100 mm or \( s = \frac{d^2}{8} \sqrt{\frac{\pi^3 E}{F_y}} \) in order to prevent buckling of bars, assuming unconfined reinforcement (crushing of concrete cover will take place), where, \( s \), \( d \), \( E \), and \( F_y \) are stirrups spacing, longitudinal bar diameter, steel elastic modulus, yield strength of steel.

- Fracture of bars should be avoided by providing sufficient reinforcement.

b) For beams:

- Yielding steel and cracking of concrete are acceptable.

- Crushing of concrete is acceptable but under-reinforced flexural sections should be designed in a manner that yielding of steel is prior to concrete crushing.

- Fracture of reinforcing bars is prohibited.

5.5 Conclusion

In this chapter, the response of gravity, direct and capacity frames is studied under various ground motion sequences. The sequences contained records of different scaling levels, frequency contents, soil conditions and durations. Replicate, random and real earthquake sequences are considered.

The results of the analyses indicated that the damage accumulation effects on the capacity designed frame is minimal compared to the direct and gravity designed ones. The performance of the damaged capacity frame is not much different than its undamaged counterpart in terms of peak displacements. When the capacity frame experiences an earthquake sequence, high inelasticity mainly occurs at beam ends. High inelasticity causes stiffness and strength deterioration. Deterioration of stiffness and strength at these energy dissipative zones allows for force redistribution among the frame components.
This explains why the capacity frame developed more plastic hinges than the gravity and direct frames; however smaller displacements were reported from the response of the capacity frame.

Period elongation of the frame systems is estimated by using the FFT approach. The inelastic period of the frame system during each individual record in the earthquake sequence is computed. The results showed longer periods for the damaged frames compared to the initially undamaged ones, a correlation between the damaged frame period and their undamaged counterparts could not be estimated given the complexity of problem in quantifying the parameters affecting period elongation.

The implication of the behavior of the frame systems on design is assessed through evaluating the three design concepts. Local inelastic behavior of frame components that significantly affect their global degradation level is reported. Guidelines are implemented to limit these unfavorable localized failures, which include crushing of concrete, buckling and fracture of reinforcing bars. Capacity design approach is recommended for frames located in regions prone to multiple earthquakes since it is proven that the capacity frames are less degradable when subjected to repeated earthquakes.
CHAPTER 6 CONCLUSIONS AND FUTURE RESEARCH REQUIREMENTS

6.1 Summary of Current Work

In this thesis, a new methodology for seismic evaluation of reinforced concrete frame systems prone to multiple earthquakes is proposed. The methodology includes establishing numerical models that depict the accurate degrading behavior of concrete elements on the material-level. Replicate, random and real ground motion sequences are selected to represent a wide range of input motion parameters. Non-linear response history analyses are performed to evaluate the behavior of three frame systems designed using different approaches.

Degrading concrete and steel material models used in this study included important features that consider accumulated damage under large amplitude plastic excursions. These features depict the accurate stiffness and strength degradation behavior, alongside pinching of load-displacement loops. A comparison between the response of models utilizing the degrading materials introduced in this study and non-degrading commonly used concrete and steel material models indicated that dismissing the salient damage features leads to misleading response of reinforced concrete structures subjected to more than one earthquake.

The seismic performance of frame systems utilizing gravity, direct and capacity design approaches is assessed. The results of the response history analyses are used to observe the design parameters that resulted in a favorable behavior of reinforced concrete frames subjected to repeated shaking. In addition, local failures in frame components which lead to dramatic deterioration of the global stiffness and strength of the system are
highlighted. Design guidelines are provided to limit structures degradation under more than one earthquake.

6.2 Conclusions

In this study, selected earthquake sequences are applied to reinforced concrete frame systems. The earthquake sequences comprised a wide range of input motion parameters. The response of the reinforced concrete structures is monitored under earthquake sequences (damaged case) and individual records within the corresponding applied sequence (undamaged case). The response of damaged and undamaged systems is discrepant in case of using models of accurate degrading features. The level of discrepancies is measured in terms of drifts, plastic hinges development and period elongation for three different frame systems. The following conclusions are drawn based on the revealed results:

− Multiple earthquake effects have significant impact on the behavior of reinforced concrete structures in a manner that cannot be predicted from simple analysis conducted using current numerical tools. Moreover, the degrading response is not accurately captured based on simplified system level or component level models, that include damage features, presented in previous studies. Therefore complicated degrading material models have to be implemented to depict the precise deteriorating behavior of reinforced concrete systems.

− Damage induced to frame systems due to prior shaking affects significantly their performance under subsequent shaking. The damaged structure might attract less
seismic forces that lead to a better performance compared with initially undamaged systems.

- Crushing of concrete, buckling and fracture of reinforcing bars are the key contributions to significant deterioration of reinforced concrete structures under repeated earthquakes. On the other hand, yielding of steel and cracking of concrete do not have a significant impact on the system degradation. Crushing could be avoided by using under reinforced concrete sections which ensures excessive yielding of reinforcing bars without introducing large strains on concrete that may exceed its crushing strain. Buckling of reinforcing bars is avoided when cover concrete remain uncrushed, uncrushed concrete confines the bars and prevents buckling. However in this study it is recommended to use a proposed equation for determining the maximum stirrups spacing between longitudinal bars that ensures complete yielding of bars under compression without buckling assuming crushed concrete. Fracture of reinforcing bars is limited by using sufficient reinforcement in section design.

- Capacity designed frames are proven to perform better than gravity and direct frames. Capacity frame response revealed formation of larger number of plastic hinges, compared to gravity and direct frames, and at the same time limited inter-story and global drifts was observed. In addition, most the analysis results showed unfavorable soft-story behavior of gravity and direct frames which was not the case for the capacity ones. This is due to that localized degradation was only introduced at capacity frame beam elements only which allowed force distribution in a ductile manner.
6.3 Future Research Requirements

The results in this study and the conclusions drawn from them point towards the necessity of conducting detailed and comprehensive analyses of different structural systems using realistic models to parametrically quantify the effect of multiple earthquakes on seismic response metric. The outcome from such parameterization would then be used to formulate design procedures that result in levels of structural safety for systems subjected to more than one earthquake that are consistent with current levels of safety in earthquake engineering design codes. This research only focuses on multiple earthquake effects on three specific reinforced concrete systems; therefore future research directions can include the following:

- Studying the response of structures of different lateral supporting systems under multiple earthquakes, this includes reinforced concrete shear wall systems, braced frames, steel structures and bridges.

- Developing an experimental program, that verifies the analytical degrading models predictions under repeated earthquakes. In this experimental program the test specimen is subject to multiple earthquake loadings and the response of the damaged specimen is compared with their analytical predictions and model refinements are executed based on experimental results.

- Introducing new material models less degrading features than reinforced concrete to be used at regions where high inelasticity is expected at. The new materials should have high ductility and strength and their behavior should not be significantly alter by previous damage accumulation.
- Deriving fragility curves of typical structures and lifeline systems in regions prone to multiple earthquakes. This could be done based on rich databases of input motion characteristics their sequence prediction in the seismic region. This will help in socio-economic loss estimation of the region when multiple earthquakes are experienced.

- Deriving inelastic response spectra, this can be used in design of structures or families of typical structures, in regions prone to multiple earthquakes. This helps designers to simply design their structures based on the equivalent static force approach which they are familiar with. In this case there is no reason to establish complicated degrading models and running inelastic dynamic analyses that are time consuming and computationally expensive for design purposes.

- Adjusting the response modification (R) factor for design of structures, this is done while including the damage accumulation effects on the response of these structures, under long duration earthquakes or multiple ones.

- Providing more specific design guidelines based on a parametric analysis. In this parametric analysis the effect of each design parameter is assessed by studying the variation of each parameter on the global behavior of structures.

- Providing guidelines for damaged structures rehabilitation procedures, this can be done by studying the response of damaged systems under subsequent earthquakes, finding the damaged system weakness, and hence providing retrofitting solutions that limits the unfavorable response captured from predicted damaged response.

- Studying the effects of damage accumulation on 3D structures, this includes plane-regular and irregular ones. Stiffness and strength degradation of lateral
supporting systems of buildings might dislocate the center of stiffness of these structures when subject to future shaking. This issue is not taken care of in current codes.

- Including the vertical ground motion component in the analysis, could significantly affect the degrading response of vertical load carrying elements such as columns. This might have a significant effect on the response.

- Studying the soil structure effects on the response of structures under multiple earthquakes could have a significant impact on the results. Deterioration of the soil underneath the structure due to multiple earthquakes could significantly affect the input ground motion characteristics of subsequent earthquakes.

- Revising attenuation relationships used in regions hit by large magnitude earthquakes; attenuation relationships developed for rock could be significantly altered if the region experiences large earthquake(s). Repeated shaking of large magnitude earthquakes cause rock cracking and hence deteriorates the rock stiffness, this significantly affects the wave passage predictions based on un-cracked rock.

- Introducing the life-cycle analysis approach to structures under multi-hazards, such as fire, earthquakes, wind, and blast; considering structural degrading effects. Structures are usually design based on their initially undamaged conditions under the aforementioned loads; this introduces the idea that structures are not subject only to one load but to multiple subsequent loads and the response under each load type is dependent on the structure load history.
In general, the analytical evidence presented in this thesis highlight the potential of damage accumulation effects of structures in regions prone to multiple earthquake hazards.
LIST OF REFERENCES


APPENDIX A.

Frames Design

A.1 Building Configuration

Figure A-1: Plan view of the building.
Figure A-2: Side view of the studied frame.

A.2 Frame Base Shear ASCE 7-02

The maximum considered earthquake (MCE) spectral response acceleration (9.4.1) & (9.4.1.1)

\[ S_s = 1.5g \text{ @ } T = 0.2 \text{ sec} \quad \text{(for short period)} \quad \text{(Eq. 9.4.1.1(a))} \]

\[ S_I = 0.6g \text{ @ } T = 1.0 \text{ sec} \quad \text{(for long period)} \quad \text{(Eq. 9.4.1.1(b))} \]

Note: using the USGS website, the \( S_s \) and \( S_I \) values can be obtained for Aliso Viejo, CA region:

For 0.2 sec horizontal ground motion and 2% probability of exceedance in 50 years

\[ S_s = 2.35g \]

For 1.0 sec horizontal ground motion and 2% probability of exceedance in 50 years
$S_l = 0.1g$

The values of $S_s$ and $S_l$ are used to produce the coefficient $S_{DS}$ and $S_{DI}$, which are then used to construct the response spectra. The strength is determined the $S_{DS}$ value since it is higher than the $S_{DI}$. It is important to note that one could use the $S_{DS}$ and $S_{DI}$ that are based on actual values of $S_s$ and $S_l$ as oppose to the values of 1.5g and 0.6g.

**Site Coefficients to adjust the MCE spectral response (Table 9.4.1.2a and b)**

Soil class B

- $F_a = 1.0$ (Table 9.4.1.2.4a)
- $F_s = 1.5$ (Table 9.4.1.2.4b)

**Site Coefficient and Adjusted MCE Spectral Response Acceleration Parameters (9.4.1.2.4)**

The site coefficient is the MCE spectral response acceleration, adjusted for site class effects:

- $S_{MS} = F_a * S_s = 1.0 * 1.5g = 1.5g$ (Eq. 9.4.1.2-1)
- $S_{M1} = F_v * S_1 = 1.0 * 0.6g = 0.9g$ (Eq. 9.4.1.2-1)

**Design Spectral Response Acceleration**

- $SDS = (2/3) * S_{MS} = 2/3 * 1.5g = 1.0g$ (Eq. 9.4.1.2.5-1)
- $SDS = (2/3) * S_{M1} = 2/3 * 0.9g = 0.6g$ (Eq. 9.4.1.2.5-2)
Seismic Use Group (Table 9.1.3)

Is based on the occupancy category (Table 1.1)

Occupancy category is II

Therefore, Seismic Use Group = I

Importance factor = I (Table 9.1.4)

Response modification factor \( R = 6.0 \) (Table 9.5.2.2)

System overstrength factor \( \Omega = 2.5 \) (Table 9.5.2.2)

Deflection amplification factor \( C_d = 5.0 \) (Table 9.5.2.2)

Approximate fundamental period (9.5.5.3.2)

\[
T_a = C_1 \cdot h^x
\]  

(Eq. 9.5.5.3.2-1)

\( C_1 = 0.0466 \) for reinforced concrete moment resisting frames

\( h = \text{height above the base} \)

\( x = 0.9 \)

\[
T_a = 0.0446 \cdot (10)^{0.9} = 0.37 \text{ sec}
\]
Calculate Seismic Base Shear

\[ C_s = \frac{S_D S}{(R^*I)} = 0.125g \]

\[ V = C_s W \]

\[ W = \text{total dead load plus 0.25 live load} = 2808 \text{ KN} \]

\[ V = 351 \text{ KN} \]

Load Distribution over the Height

Lateral load distribution is based on the inverted triangle distribution

*Figure A-4: Distribution of base shear along the height.*
APPENDIX B.

Reinforcement Detailing of Frame Systems

B.1 Gravity Designed Frame

Figure B-1: Gravity frame flexure and shear reinforcement.
Figure B-2: Cross section detailing, all longitudinal bars are #7 and all stirrups are #3 bars.
B.2 Direct Designed Frame

Figure B-3: Direct frame flexure and shear reinforcement.
Figure B-4: Cross section detailing, all longitudinal bars are #7 and all stirrups are #3 bars.
B.3 Capacity Designed Frame

Figure B-5: Direct frame flexure and shear reinforcement.
Figure B-6: Cross section detailing, all longitudinal bars are #7 and all stirrups are #3 bars.
APPENDIX C

C.1 Random Records from 1 to 27

P0806 : EARTHQUAKE AND STATION DETAILS

<table>
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<tr>
<th>Location</th>
<th>Date/Time</th>
<th>Magnitude: M (7.1) ML ( ) MS (7.1)</th>
<th>Station:</th>
<th>Data Source:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cape Mendocino</td>
<td>1992/04/25 18:06</td>
<td></td>
<td>89005 Cape Mendocino</td>
<td>CDMG</td>
</tr>
</tbody>
</table>

Distance (km):
- Closest to fault rupture (8.5)
- Hypocentral ( )
- Closest to surface projection of rupture ( )

Site conditions:
- Geomatrix or CWB (A)
- USGS (A)

---

Record # 1

![Graph 1](image1)

![Graph 2](image2)
### P0873 : EARTHQUAKE AND STATION DETAILS

<table>
<thead>
<tr>
<th>Landers</th>
<th>1992/06/28 11:58</th>
<th>Magnitude: M (7.3) Ml ( ) Ms (7.4)</th>
<th>Station: 24 Lucerne</th>
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</thead>
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<td></td>
<td></td>
<td>Data Source: SCE</td>
<td>Site conditions:</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Geomatrix or CWB (A)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>USGS (A)</td>
</tr>
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</table>

**Distance (km):**
- Closest to fault rupture (1.1)
- Hypocentral ( )
- Closest to surface projection of rupture ( )

![Graph of Record # 2](image)

![Graph of S_v](image)
Loma Prieta 1989/10/18 00:05
Magnitude: M (6.9) Ml () Ms (7.1)

Station: 47379 Gilroy Array #1
Data Source: CDMG

Distance (km):
Closest to fault rupture (11.2)
Hypocentral ()
Closest to surface projection of rupture (10.5)

Site conditions:
Geomatrix or CWB (A)
USGS (A)
P0996 : EARTHQUAKE AND STATION DETAILS

Northridge 1994/01/17 12:31
Magnitude: M (6.7) ML (6.6) MS (6.7)

Station: 24207 Pacoima Dam (upper left)
Data Source: CDMG

Distance (km):
Closest to fault rupture (8.0)
Hypocentral
Closest to surface projection of rupture (8.1)

Site conditions:
Geomatrix or CWB (A)
USGS (A)

Record # 4

Time (sec)

\[ a_g (g) \]

Period (sec)

\[ S_v (cm/sec) \]
P0541 : EARTHQUAKE AND STATION DETAILS

<table>
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<th>N. Palm Springs 1986/07/08 09:20</th>
<th>Station: 5072 Whitewater Trout Farm</th>
</tr>
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<tr>
<td>Magnitude: M (6.0) Ml (5.9) Ms (6.0)</td>
<td>Data Source: USGS</td>
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Distance (km):
- Closest to fault rupture (7.3)
- Hypocentral ( )
- Closest to surface projection of rupture ( )

Site conditions:
- Geomatrix or CWB (C)
- USGS (A)

Record # 5

**Time (sec)**

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<th>Time (sec)</th>
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<th>5</th>
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<th>15</th>
<th>20</th>
<th>25</th>
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</thead>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<td>0</td>
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<th>0.8</th>
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<th>1.2</th>
<th>1.4</th>
<th>1.6</th>
<th>1.8</th>
<th>2</th>
</tr>
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<tbody>
<tr>
<td><strong>a_g (g)</strong></td>
<td>0</td>
<td>0.2</td>
<td>0.4</td>
<td>0.6</td>
<td>0.8</td>
<td>1</td>
<td>1.2</td>
<td>1.4</td>
<td>1.6</td>
<td>1.8</td>
<td>2</td>
</tr>
<tr>
<td><strong>S_v (cm/sec)</strong></td>
<td>0</td>
<td>200</td>
<td>400</td>
<td>600</td>
<td>800</td>
<td>1000</td>
<td>1200</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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**P1099 : EARTHQUAKE AND STATION DETAILS**

Kocaeli, Turkey 1999/08/17
Magnitude: M (7.4) Ml () Ms (7.8)

Station: Gebze
Data Source: ERD

Distance (km):
Closest to fault rupture (17.0)
Hypocentral ()
Closest to surface projection of rupture (17.0)

Site conditions:
Geomatrix or CWB (A)
USGS (A)

---

**Record # 24**

- **Time (sec)**: 0 to 15
- **$a_g$ (g)**: -0.2 to 0.3

**$S_v$ (cm/sec)**: 0 to 500

- **Period (sec)**: 0 to 2
KOCAELI, TURKEY 1999/08/17

MAGNITUDE: M (7.4) ML ( ) MS (7.8) STATION: IZMIT

DATA SOURCE: ERD

DISTANCE (KM):

CLOSEST TO FAULT RUPTURE (4.8)

HYPOCENTRAL ( )

CLOSEST TO SURFACE PROJECTION OF RUPTURE (4.8)

SITE CONDITIONS:

GEOMATRIX OR CWB (A)

USGS ( )

Record # 26

\[ a_g (g) \]

Time (sec)

S\_v (cm/sec)

Period (sec)
P0691 : EARTHQUAKE AND STATION DETAILS

WHITTIER NARROWS 1987/10/01 14:42

MAGNITUDE: M (6.0) ML (5.9) MS (5.7) STATION: 90019 SAN GABRIEL - E GRAND AV

DATA SOURCE: USC

DISTANCE (KM):

CLOSEST TO FAULT RUPTURE (9.0)

HYPOCENTRAL ( )

CLOSEST TO SURFACE PROJECTION OF RUPTURE ( ) SITE CONDITIONS:

GEOMATRIX OR CWB (A)

USGS (A)

Record # 27

\( a_g (g) \)

Time (sec)

Loss

\( S_v (cm/sec) \)

Period (sec)
## P1461 : EARTHQUAKE AND STATION DETAILS

<table>
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<th>Location</th>
<th>Date</th>
<th>Magnitude: M (7.6) ML (7.3) Ms (7.6)</th>
<th>Station: TCU095</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Chi-Chi, Taiwan</td>
<td>1999/09/20</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Distance (km):
- Closest to fault rupture: 43.4
- Hypocentral:  
- Closest to surface projection of rupture: 43.44

### Site conditions:
- Geomatrix or CWB (1)
- USGS (B)

![Record # 6](image_url)

![Sv vs Period](image_url)
Landers 1992/06/28 11:58
Magnitude: M (7.3) Ml () Ms (7.4)

Station: 23 Coolwater
Data Source: SCE

Distance (km):
Closest to fault rupture (21.2)
Hypocentral ()
Closest to surface projection of rupture (22.8)

Site conditions:
Geomatrix or CWB (D)
USGS (B)

Record # 7

Sv (cm/sec)

Period (sec)
**P0912 : EARTHQUAKE AND STATION DETAILS**

<table>
<thead>
<tr>
<th>Northridge</th>
<th>1994/01/17 12:31</th>
</tr>
</thead>
<tbody>
<tr>
<td>Magnitude:</td>
<td>M ( 6.7 ) Ml ( 6.6 ) Ms ( 6.7 )</td>
</tr>
</tbody>
</table>

**Station:** 24400 LA - Obregon Park  
**Data Source:** CDMG

**Distance (km):**  
Closest to fault rupture ( 37.9 )  
Hypocentral ( )  
Closest to surface projection of rupture ( 35.9 )

**Site conditions:**  
Geomatrix or CWB ( D )  
USGS ( B )

**Record # 8**

**Time (sec):**
-0.6
-0.4
-0.2
0
0.2
0.4

**$a_g$ (g):**
-0.6
-0.4
-0.2
0
0.2
0.4

**Period (sec):**
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2

**$S_v$ (cm/sec):**
0
100
200
300
400
500
600
700

**Period (sec):**
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
**P0056 : EARTHQUAKE AND STATION DETAILS**

<table>
<thead>
<tr>
<th>San Fernando 1971/02/09 14:00</th>
<th>Station: 24278 Castaic - Old Ridge Route</th>
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<tbody>
<tr>
<td>Magnitude: M (6.6) Ml ( ) Ms (6.6)</td>
<td>Data Source: CDMG</td>
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<tr>
<td>Distance (km): Closest to fault rupture (24.9) Hypocentral ( ) Closest to surface projection of rupture (24.2)</td>
<td>Site conditions: Geomatrix or CWB (B) USGS (B)</td>
</tr>
</tbody>
</table>

**Record # 9**

![Record # 9 graph](image1)

![Record # 9 graph](image2)
**P0266 : EARTHQUAKE AND STATION DETAILS**

Victoria, Mexico 1980/06/09 03:28

Magnitude: M ( ) MI (6.1) Ms (6.4)

Station: 6604 Cerro Prieto

Data Source: UNAM/UCSD

<table>
<thead>
<tr>
<th>Distance (km):</th>
<th>Site conditions:</th>
</tr>
</thead>
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<td>Closest to fault rupture ( )</td>
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<tr>
<td>Hypocentral (34.8)</td>
<td>USGS (B)</td>
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<tr>
<td>Closest to surface projection of rupture ( )</td>
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---

**Record # 10**

![Seismic waveforms](image)

- **Time (sec):** 0 to 25
- **$a_g (g):$** -0.8 to 0.6
- **$S_v (cm/sec):$** 0 to 1400

---

**Period (sec):** 0 to 2

- **$S_v (cm/sec):$** 0 to 1400
Chi-Chi, Taiwan 1999/09/20
Magnitude: M (7.6) Ml (7.3) Ms (7.6)

<table>
<thead>
<tr>
<th>Distance (km):</th>
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<th>Hypocentral</th>
<th>Closest to surface projection of rupture (6.79)</th>
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<tr>
<td>Station:</td>
<td>CHY080</td>
<td>Data Source: CWB</td>
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<td>Geomatrix or CWB (-) USGS (B)</td>
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**Record # 22**

![Time vs. Acceleration Plot](image)

![Period vs. Sv Plot](image)
### P1340: EARTHQUAKE AND STATION DETAILS

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<td>Chi-Chi, Taiwan</td>
</tr>
<tr>
<td>Date</td>
<td>1999/09/20</td>
</tr>
<tr>
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<td>M (7.6) Ml (7.3) Ms (7.6)</td>
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<td>Data Source</td>
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<tr>
<td>Hypocentral</td>
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<td>Closest to surface projection of rupture</td>
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<tr>
<td>USGS</td>
<td>C</td>
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</table>

#### Graphs

**Record # 11**

- **Time (sec)**: 0 to 20
- **Acceleration (g)**: -0.4 to 0.3

**Sv (cm/sec)**

- **Period (sec)**: 0 to 2
## P0170 : EARTHQUAKE AND STATION DETAILS

<table>
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<tr>
<th>Imperial Valley 1979/10/15 23:16</th>
<th>Station: 6605 Delta</th>
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</thead>
<tbody>
<tr>
<td>Magnitude: M (6.5) Ml (6.6) Ms (6.9)</td>
<td>Data Source: UNAM/UCSD</td>
</tr>
</tbody>
</table>

**Distance (km):**
- Closest to fault rupture (43.6)
- Hypocentral ( )
- Closest to surface projection of rupture (32.7)

**Site conditions:**
- Geomatrix or CWB (D)
- USGS (C)

![Graph 1](image1.png)

![Graph 2](image2.png)
# Earthquake and Station Details

**Loma Prieta** 1989/10/18 00:05  
**Magnitude:** M (6.9) ML ( ) Ms (7.1)  
**Station:** 57425 Gilroy Array #7  
**Data Source:** CDMG  
**Distance (km):**  
- Closest to fault rupture (24.2)  
- Hypocentral ( )  
- Closest to surface projection of rupture (24.3)  
**Site conditions:**  
- Geomatrix or CWB (B)  
- USGS (C)

---

**Graph 1:**  
- Graph title: Record #13  
- **Axes:**  
  - Y-axis: $a_g (g)$  
  - X-axis: Time (sec)  
  - Grid lines  
  - Data points: S-wave acceleration  

**Graph 2:**  
- Graph title:  
- **Axes:**  
  - Y-axis: $S_v$ (cm/sec)  
  - X-axis: Period (sec)  
  - Grid lines  
  - Data points: S-wave displacement
P0914 : EARTHQUAKE AND STATION DETAILS

Northridge 1994/01/17 12:31
Magnitude: M (6.7) ML (6.6) Ms (6.7)

Station: 90091 LA - Saturn St
Data Source: USC

Distance (km):
Closest to fault rupture (30.0)
Hypocentral ( )
Closest to surface projection of rupture (23.2)

Site conditions:
Geomatrix or CWB (D)
USGS (C)
Whittier Narrows 1987/10/01 14:42
Magnitude: M (6.0) Ml (5.9) Ms (5.7)

Station: 90079 Downey - Birchdale

Data Source: USC

Distance (km):
Closest to fault rupture (56.8)
Hypocentral ()
Closest to surface projection of rupture ()

Site conditions:
Geomatrix or CWB (D)
USGS (C)
Cape Mendocino 1992/04/25 18:06
Magnitude: M (7.1) Ms (7.1)
Station: 89324 Rio Dell Overpass - FF
Data Source: CDMG

Distance (km):
Closest to fault rupture (18.5)
Hypocentral ( )
Closest to surface projection of rupture (12.3)

Site conditions:
Geomatrix or CWB (C)
USGS (B)
### P1143 : EARTHQUAKE AND STATION DETAILS

<table>
<thead>
<tr>
<th>Location</th>
<th>Date</th>
<th>Magnitude: M (7.6) Ml (7.3) Ms (7.6)</th>
<th>Station: CHY041</th>
<th>Data Source: CWB</th>
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<tr>
<td>Chi-Chi, Taiwan</td>
<td>1999/09/20</td>
<td>Closest to fault rupture (25.96)</td>
<td>Hypocentral</td>
<td>Closest to surface projection of rupture (25.96)</td>
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#### Graphs

- **Record # 16**
  - $a_g (g)$ vs. Time (sec)
  - $S_v (cm/sec)$ vs. Period (sec)

---

186
Kobe 1995/01/16 20:46  
**Magnitude:** $M (6.9) Ml () Ms ()$  
**Station:** 0 Kakogawa  
**Data Source:** CUE  
**Distance (km):**  
- Closest to fault rupture: (26.4)  
- Hypocentral: ()  
- Closest to surface projection of rupture: ()  
**Site conditions:**  
- Geomatrix or CWB (E)  
- USGS (D)
### P0753 : EARTHQUAKE AND STATION DETAILS

<table>
<thead>
<tr>
<th>Loma Prieta</th>
<th>1989/10/18 00:05</th>
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<td>Magnitude:</td>
<td>M (6.9) MI () Ms (7.1)</td>
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<tr>
<td>Station:</td>
<td>1002 APEEL 2 - Redwood City</td>
</tr>
<tr>
<td>Data Source:</td>
<td>USGS</td>
</tr>
</tbody>
</table>

**Distance (km):**
- Closest to fault rupture: 47.9
- Hypocentral: ()
- Closest to surface projection of rupture: ()

**Site conditions:**
- Geomatrix or CWB (D)
- USGS (D)

---

### Record # 18

**Data**

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<tr>
<th>Time (sec)</th>
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<tr>
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<td>0.5</td>
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<tr>
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<tr>
<td>17.5</td>
<td>0.5</td>
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<td>20</td>
<td>0</td>
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**Sv (cm/sec)**

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<th>$S_v$ (cm/sec)</th>
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<td>500</td>
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<td>2500</td>
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<td>2000</td>
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<tr>
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**P0984 : EARTHQUAKE AND STATION DETAILS**

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<td>M (6.7) MI (6.6) Ms (6.7)</td>
</tr>
<tr>
<td>Distance:</td>
<td>Closest to fault rupture (12.3)</td>
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<tr>
<td>Hypocentral:</td>
<td>( )</td>
</tr>
<tr>
<td>Closest to surface projection of rupture (86.8)</td>
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</table>

| Station: 90011 Montebello - Bluff Rd. |
| Data Source: USC |

Site conditions:
- Geomatrix or CWB (D)
- USGS (D)
Superstition Hills (B) 1987/11/24 13:16
Magnitude: M (6.7) ML ( ) Ms (6.6)

Station: 5062 Salton Sea Wildlife Refuge
Data Source: USGS

Distance (km):
Closest to fault rupture (27.1)
Hypocentral ( )
Closest to surface projection of rupture ( )

Site conditions:
Geomatrix or CWB (D)
USGS (D)

Record # 20

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<table>
<thead>
<tr>
<th>Period (sec)</th>
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</thead>
<tbody>
<tr>
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P0736 : EARTHQUAKE AND STATION DETAILS

Loma Prieta 1989/10/18 00:05
Magnitude: M (6.9) Ml () Ms (7.1)

Station: 47381 Gilroy Array #3
Data Source: CDMG

Distances (km):
- Closest to fault rupture: 14.4
- Hypocentral: 14.0
- Closest to surface projection of rupture: 14.0

Site conditions:
- Geomatrix or CWB (D)
- USGS (C)
C.2 Tohoku Earthquake Sequences at Stations

Figure C-1: Earthquake sequences at different stations during the Tohoku earthquake sequence.
CHB010

EW

0 100 200 300 400 500 600 700 800 900 1000
-0.2
-0.1
0
0.1
0.2

EW

NS

0 100 200 300 400 500 600 700 800 900 1000
-0.2
-0.1
0
0.1
0.2

NS

FKS004

EW

0 500 1000 1500 2000 2500
-1
-0.5
0
0.5
1

EW

NS

0 500 1000 1500 2000 2500
-1
-0.5
0
0.5
1

NS

196