# RESPONSE MODIFICATION FACTOR OF REINFORCED CONCRETE MOMENT RESISTING FRAMES IN DEVELOPING COUNTRIES

BY

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THESIS

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# Abstract

Vulnerability of buildings to seismic hazards is more drastic in developing countries with high seismicity, as compared to developed countries. This is primarily attributed to the lack of seismic design guidelines, which fit the type of structural systems and practices that are often applied in such parts of the world. Response reduction factors (R factors) are essential seismic design tools, which are typically used to describe the level of inelasticity expected in lateral structural systems during an earthquake. The R factors in many developing countries are often adopted from the well developed seismic design codes used in the United States or Europe. These R factors provide false representation for the structural practices applied in developing countries and thus considered unrealistic. So there is a dire need to come up with realistic R factors for various structural systems used in such countries. This study utilizes incremental dynamic analysis (IDA) and peak ground parameters to determine the R factor of reinforced concrete (RC) moment resisting frames (MRFs) in Pakistan. A suite of ground motion records from the region is used in the study. Two-dimensional building models are developed in OpenSees and subjected to nonlinear time history analysis. Fiber sections with different material constitutive behaviors are constructed for each of the building's critical elements. The reinforcement detailing of the analyzed buildings is determined from two prototype buildings in Pakistan. A parametric study involving RC MRFs with variation in dimensional and material properties was conducted to examine the effect of these properties on R factor. Results showed that R factor is affected by both geometric configuration and material strength; however, variation in geometric parameters tends to display more significant impact on the R factor value. The results also show that the R factors recommended by the United States seismic design provisions are unconservative and over estimate the R factor values for some selection of ground motion while it is conservative for others.

# DEDICATION

To my parents, wife - Mominah, children – Iman Zafar, Eshal Zafar and Aiza Zafar, the rest of the family and friends who have stood behind me.

# ACKNOWLEDGMENTS

I would like to express my appreciation and thanks to my advisor, Professor Bassem Andrawes, who provided me with opportunity to work with him. He has been of immense help, advising and guiding me during my studies. Discussion with him during the times of difficulties has put me back on my philosophical path of accomplishment.

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## TABLE OF CONTENT

# LIST OF FIGURES

# LIST OF TABLES

# CHAPTER 1

# INTRODUCTION

## 1.1 Motivation

There are many natural hazards in the world but earthquakes are one of the most destructive natural hazards that can result is severe social and economic impact. The devastating potential of an earthquake can have major consequences on infrastructures and lifelines. Roughly, 11,000 people die each year due to earthquakes (Kanamori, 1977), while annual economic losses run in billions of dollars, taking major toll on nation's economy.

Earthquake engineering has developed as a branch of engineering concerned with the estimation of earthquake impacts, since last few decades. It has become an interdisciplinary subject involving seismologists, structural engineer, geotechnical engineers, architects, urban planners, information technologists and social scientists. In the past few years, the earthquake engineering community has been reassessing its procedures, in the wake of devastating earthquakes which have caused extensive damage, loss of life and property. These procedures involve assessment of seismic force demands on the structure and then developing design procedures for the structure to withstand the applied actions.

Conventional seismic design in codes of practice is entirely force-based, with a final check on structural displacements. Force-based design is suited to design for actions that are permanently applied. Members are designed to resist the effects of these actions. Seismic design follows the same procedure, except for the fact that inelastic deformations may be utilized to absorb certain levels of energy leading to reduction in the forces for which structures are designed. This leads to the creation of the Response Modification Factor (R factor); the all important parameter that accounts

for over-strength, energy absorption and dissipation as well as structural capacity to redistribute forces from inelastic highly stressed regions to other less stressed locations in the structure. The concept of Response Modification Factor or also commonly known as Force Reduction Factor, has emerged as a single most important number, reflecting the capability of the structure to dissipate energy through inelastic behavior. This factor is unique and different for different type of structures and materials used. Hence classification of Response modification factor for various structural systems is extremely important in order to do evaluation based on demand (earthquake ground motion) and capacity of the structure.

The R factors in many developing countries are often adopted from the well developed seismic design codes used in the United States or Europe. These developing countries sometimes have more severe nature of seismic hazard, but lack technology to construct structures according to any seismic guidelines. One example of such developing country is Pakistan, which faces high seismic hazard because of its proximity to a major fault zone. In this study, Pakistan will be considered as an example of other developing countries that have similar vulnerability to seismic hazard. Pakistan has adopted its seismic design provisions based on code of practice used in the United States. Structures in Pakistan face different nature of vulnerability from those in the United States, because of different level of seismic hazard and building inventory. Therefore, it is reasonable to comment that the buildings in Pakistan are more prone to seismic hazard, as compared to developed countries. This is primarily due to lack of seismic design guidelines, which fit the type of buildings and practices that are applied in Pakistan. Therefore the use of R factors computed for the United States provides false representation for the structural practices applied in Pakistan and thus considered unrealistic. R factors recommended by the United States seismic design provisions are unconservative and can over estimate the R factor values. During a recent Kashmir 2005 earthquake, many modern reinforced concrete structures which were designed based on seismic provisions from codes in the United States, collapsed. One of the buildings was a residential apartment, Margalla towers (shown in Figure 2-5), which

suffered abrupt collapse and loss of life during the Kashmir 2005 earthquake. Many similar failures confirm the fact that Pakistan needs to have its own seismic provisions and its own seismic code to calculate R factor based on the seismic hazard and the type of structures which exists in the region.

## 1.2 Objective

The primary objective of this study is to evaluate R factor for typical reinforced concrete (RC) moment resisting frames (MRFs) which exist in Pakistan, using nonlinear analytical tools, and compare the calculated R factor with the values given in seismic code of practice. Incremental dynamic analysis (IDA) was utilized in this study to find the response modification factor for RC-MRF in Pakistan using a suite of ground motion records representative of the region. Another objective of this thesis is to conduct a parametric study to evaluate the effect of variation of material and geometric properties of RC-MRFs on R factor.

## 1.3 Framework of Thesis

This thesis is divided into the following chapters:

### 1.3.1 Chapter 2 : Seismicity in Pakistan and Surrounding Countries

This chapter will discuss the tectonic settings of Pakistan. Various seismic hazards and its effects on structures will be shown. Damages to structures during recent earthquakes and their causes will also been discussed.

### 1.3.2 Chapter 3 : Response Modification Factor

Basic concepts of seismic design and a conceptual framework of response modification or force reduction factor (R factor) will be introduced in this chapter. A brief

review of historical development of this factor along with its use in various countries codes is also presented.

### 1.3.3 Chapter 4: Type of Building Systems and Design Codes in Pakistan

The various type of construction in Pakistan is described along with shortfalls in construction techniques. A review of seismic codes in Pakistan and comparison with International codes is also discussed.

### 1.3.4 Chapter 5: R Factor Calculation Methods

This chapter provides an overview of various methods that are currently being used to calculate R factor for reinforced concrete buildings. An in depth discussion on the application of incremental dynamic analysis (IDA) and peak ground parameters to find R factor is presented.

### 1.3.5 Chapter 6 : Structural Systems Models and Selection of Ground Motions

A description of RC-MRF structures, specific to Pakistan is introduced in this chapter. Variation in material properties and building geometric parameters in Pakistan is also discussed along with the ground motions, which best represent the seismic hazard in the sub-continent region.

### 1.3.6 Chapter 7 : Analytical Modeling using OpenSees

This chapter presents a detailed description of the 2-D analytical OpenSees models that were developed and used to compute the R factor.

### 1.3.7 Chapter 8 : Evaluation of R Factors of Prototype Buildings

Results from nonlinear time history analysis of two prototype buildings in Pakistan are presented in this chapter and the computed R factors for these buildings are discussed and compared with those recommended by existing building codes.

### 1.3.8 Chapter 9 : Parametric Study on R Factor

The influence of various building and material parameters on the R factor is investigated in this chapter through a parametric study.

### 1.3.9 Chapter 10 : Conclusion

### 1.3.10 Chapter 11: References

## CHAPTER 2

## SEISMICITY IN PAKISTAN AND NEIGHBORING COUNTRIES

### 2.1 Tectonic Settings of Pakistan and Surrounding Countries

Seismic activity in South Asia is a direct result of the collision of the Indian and the Asian plates, which is because of the northwestern motion of the Indian Plate at the rate of 4-5 cm per year (Figure 2-1).

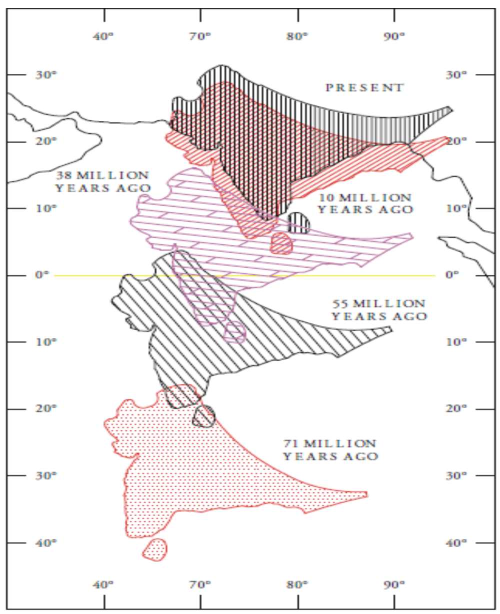



Figure 2-1. Continued drift of the Indian plate towards Asian plate causing major Himalayan earthquakes (Molnar, 1977)

The resulting collision has fractured the Indian plate into several slices beneath the Kashmir Basin. The collision of India with Asia has resulted in giving rise to stresses that are responsible for many of the earthquakes in central India (Bilham and Ambraseys, 2005).

The recent Kashmir earthquake in 2005, which affected Kashmir, Jammu and the North-West Frontier Province of Pakistan, is associated with the great plate boundary region as shown in Figure 2-2, where the Indian Plate is subducting under the Asian Plate. The same tectonic movement in the region is responsible for the creation of the Himalayan mountain ranges through compressive and bending stresses. As a result of this subduction mechanism, many major earthquakes have been triggered. The recent seismic event of 2005 lies at the western tip of the active subduction Himalayan belt.

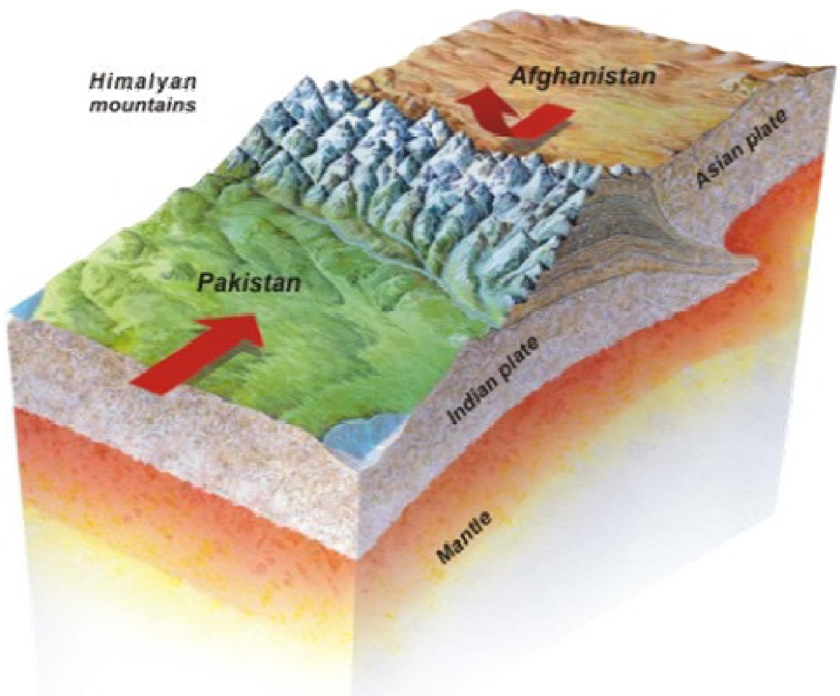


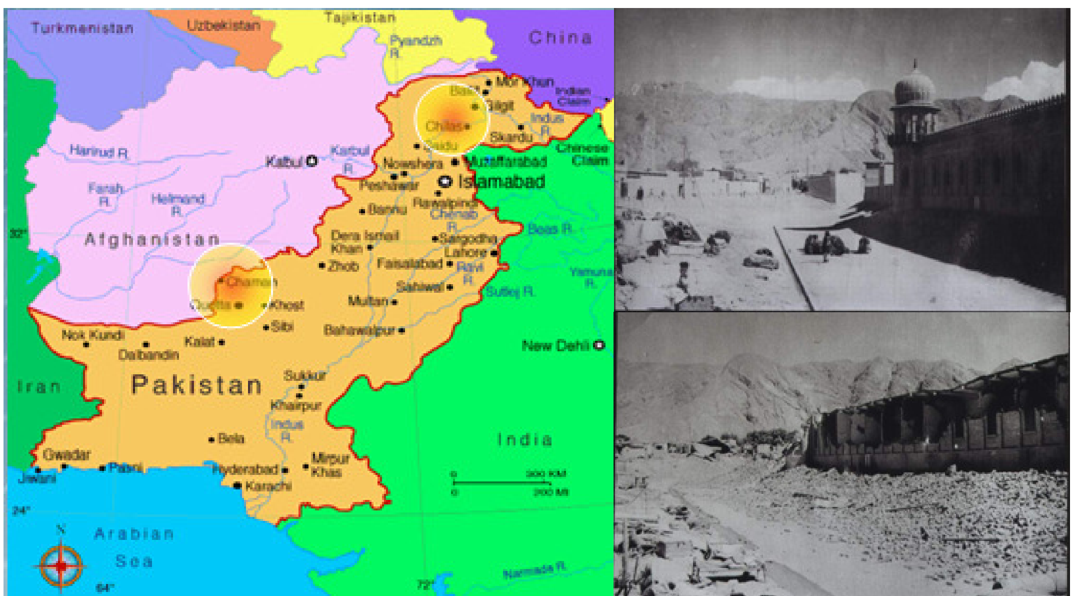

Figure 2-2. Global tectonic setting of the Kashmir earthquake within the Indian-Asian plates subduction region (MAE center report on Kashmir 2005 earthquake)

## 2.2 Seismic Hazard in the Region and its Effects

Pakistan has a long history of earthquakes mainly due to intersection of tectonic plates in Karakoram Range. Many major earthquakes like Quetta-1935, Pattan-1974 and Kashmir-2005 have struck Pakistan causing huge devastation. These series of ground shaking is associated with complex fault mechanism which surrounds Pakistan.

Many researchers and geologists have tried to assess the slip between the plates in the region and to predict the next major earthquake event. In 2000 Ambraseys and Bilham stated that it is possible, that earthquakes in the past two centuries have not been representative of infrequent great plate boundary events (Ms > 8) that could occur. Their conclusion is based on the annual slip rate observed in the fault zone.

A major earthquake of magnitude 7.5 on Richter scale, hit Quetta city in South west province of Pakistan in 1935. Another major earthquake struck Pattan in Northern Pakistan in 1974 with magnitude of 6.2 on Richter scale. (See Figure 2-3).




(a) (b)

Figure 2-3. (a) Epicenter of 1935 Quetta and 1974 Pattan earthquakes. (b) Fruit market in Quetta before and after the 1935 Quetta earthquake

Recently in 2005 an earthquake of 7.6 magnitude hit Pakistan's Northern areas causing wide spread damage. (See Figure 2-4). The earthquake caused extensive land slides and damage to infrastructure including buildings, bridges, roads, tunnels, rail roads etc. Most building damage resulted from ground shaking, though a large number of buildings located were mostly on or near the slopes, resulting in failure due to land sliding.

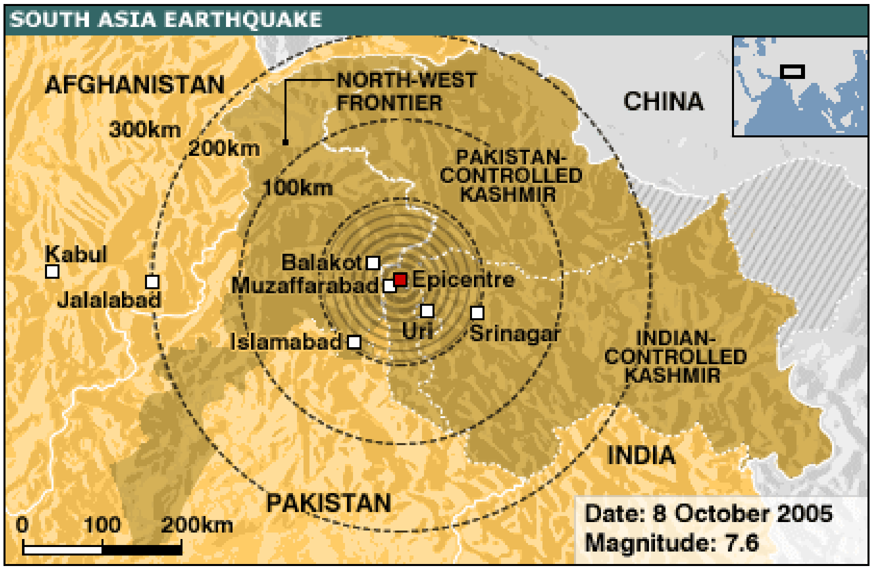



Figure 2-4. Epicenter of the October 2005 Kashmir earthquake (USGS)

The 2005 Kashmir earthquake, in a regional setting, is still considered to be a moderate earthquake. The region is susceptible to great earthquakes of magnitudes > 8.0. Estimates of slip rates suggest an average slip of ~ 18 mm/year (Bilham and Ambraseys, 2005), averaged over the entire India-Tibet collision zone. The average slip observed in earthquakes in the past 5 centuries amounts to less than 3 mm/year. The most likely outcome of the above would be a massive earthquake. With the Kashmir earthquake releasing less than 10% of the energy stored in the collision region (Bilham and Ambraseys, 2005), many large population centers throughout northern Pakistan and India are exposed to serious seismic risk.

The above finding is of great concern and leads to a very important conclusion about future seismic activity in subcontinent. A major earthquake in the region is inevitable and it is only a matter of when. The Northern Pakistan region is subjected to significant earthquake hazard, which is translated to exceptionally high risk when taking into account the level of vulnerability.

## 2.3 Structural Damages during Recent Earthquakes and their Causes

The largest concentration of destroyed or damaged buildings during Kashmir 2005 earthquake were in Muzaffarabad and Balakot, where buildings were either destroyed or badly damaged in the main event due to extensive shaking. Damage in Balakot was directly related to fault rupture. (See Figure 2-4). Structures located on ridges and along steep slopes were subjected to a greater degree of damage in comparison to those located in valleys, due to amplification of ground shaking. Majority of the buildings in the affected region used poor construction material without much seismic considerations, leading to collapse of the structure.

Collapse of the high-rise Margala Towers in Islamabad (See Figure 2-5), located over 80 km from the epicenter, was an example of collapse of a structure due to poor construction quality.

Figure 2-5: Collapsed RC building during Kashmir 2005 earthquake

Numbers of the common deficiencies found in structural systems were evident, such as frame members not properly detailed for ductile behavior, masonry infill walls not reinforced for out-of-plane loading, infill walls not isolated from the adjoining concrete frames, roof / floor diaphragms not properly connected to the lateral system, inadequate layout of footings etc. Furthermore, the design was based on inappropriate

seismic design criteria. Some of the structural damages to reinforced concrete and masonry structures have been shown in Figure 2-6 and 2-7.

Figure 2-6. Collapse of reinforced concrete moment resisting frames in Muzaffarabad, Pakistan. (Ahsan and Saif, 2008)

(a) (b)

Figure 2-7. (a) Development of shear cracks in a reinforced concrete structure. (b) Storey failure due to development of soft storey. (MAE center report, 2005).

Quality control was one of the most disregarded issues in the earthquake affected area, both in non-engineered and engineered building construction (Ahsan and Saif, 2008). The construction materials and skills were extremely deficient in the area. The steel observed in the area was of an extremely low grade, with all possible

deficiencies such as high brittleness and flakiness. Hand mixing is the most common method of concrete preparation, and concrete vibrators are rarely used for compaction, resulting in low-grade, honeycombed concrete. Curing of concrete is still not practiced as an integral part of the concreting process. The concrete blocks were of poor quality because of the poor quality of the concrete, a lack of compaction and very little or no curing.

## 2.4 Seismic Zoning of Pakistan

Unavailability of strong ground motion records, points towards the need for a well developed seismic monitoring network and seismic design procedures of not only the area affected by the recent earthquake, but for all of Pakistan. The region has been infested with numerous major earthquakes namely Quetta 1935, Pattan 1974 and Kashmir 2005. These earthquakes have had implication on not only infrastructure including buildings, bridges, roads, dams etc. but also on the social sector. The social and economic impact of any earthquake is always difficult to predict but can be minimized by reducing the number of casualties and minimizing damage sustained by the built infrastructure.

After the October 8, 2005 Pakistan earthquake, the National Engineering Services Pakistan (NESPAK) was made responsible by the Government of Pakistan, for preparing a new seismic code (still under development) for Pakistan to better protect buildings against future damages caused by high intensity earthquakes. Although a preliminary version of this code came out in 2006, referred as NESPAK-2006, the work on its development and release is still in progress. Due to delay in publication of NESPAK code, Earthquake Reconstruction and Rehabilitation Authority (ERRA), responsible to reconstruct in the aftermath decided to adopt the Uniform Building Code (UBC) 1997. This adopted version is named as Building Code of Pakistan (BCP 2007) which takes into account the seismic characteristics of the Pakistan region has officially been published and released. Five different seismic zones (1, 2A, 2B, 3, 4) were

identified in this code, each one with a range of peak horizontal ground acceleration (see Table 2-1 and Figure 2-8). All the equations to compute the base shear as well as all the tables for each coefficient used in the equations are the same as those in UBC 97.

| Seismic Zone | Peak Ground Acceleration |
|---|---|
| 1 | 0.05 to 0.08g |
| 2A | 0.08 to 0.16g |
| 2B | 0.16 to 0.24g |
| 3 | 0.24 to 0.32g |
| 4 | >0.32g |

Table 2-1. Seismic zone and related peak ground acceleration (BCP 2007)

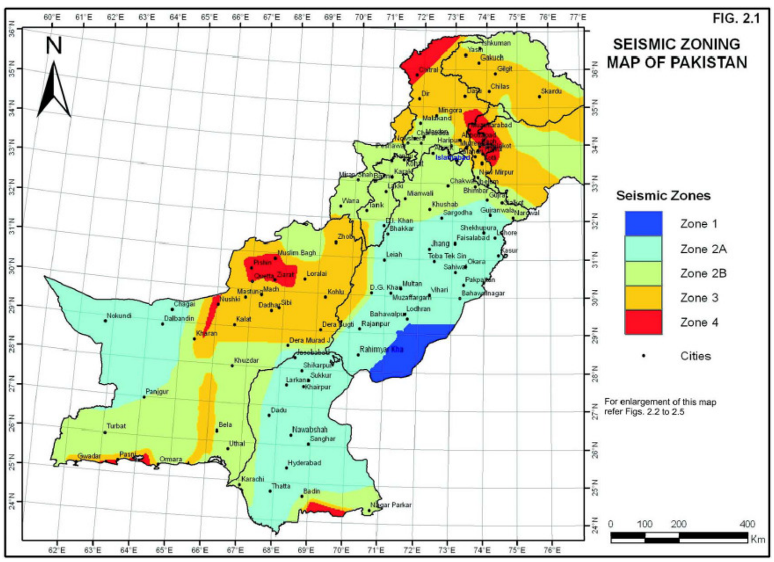



Figure 2-8. Seismic zoning map of Pakistan (BCP 2007)

Despite having progress in the development of some kind of seismic provisions for Pakistan, there is still a need for finding a more accurate and detailed procedure to calculate R factor for MRFs in Pakistan. This research aims at determining the method to evaluate R factor for MRF buildings in Pakistan, by incorporating ground motions from the region and building models which best represent the building stock in Pakistan.

# CHAPTER 3

# RESPONSE MODIFICATION FACTOR

## 3.1 Introduction to Basic Concept of Seismic Design

Design requirements for lateral loads, such as winds or earthquakes, are inherently different from those for gravity (dead and live) loads. Due to frequency of loading scenario, design for wind loads is a primary requirement. But in areas of high seismicity, structures are designed to withstand lateral actions also. Since the seismic design deals with events with lower probability of occurrence, it may therefore be highly uneconomical to design structures to withstand earthquakes for the performance levels used for wind design. For example, building structures would typically be designed for lateral wind loads in the range of 1% to 3% of their weight. Earthquake loads may reach 30%-40% of the weight of the structure, applied horizontally. If concepts of elastic design normally employed for primary loads are used for earthquake loads, the result will be in the form of extremely heavy and expensive structures. Therefore, seismic design uses the concepts of controlled damage and collapse prevention.

In earthquake engineering, the aim is to have a control on the type, location and extent of the damage along with detailing process. This is illustrated in Figure 3-1, where the elastic and inelastic responses are depicted, and the concept of equal energy (discussed further in subsequent sections) is employed to reduce the design force from $V_e$ to $V_d$ (denoting elastic and design force levels).

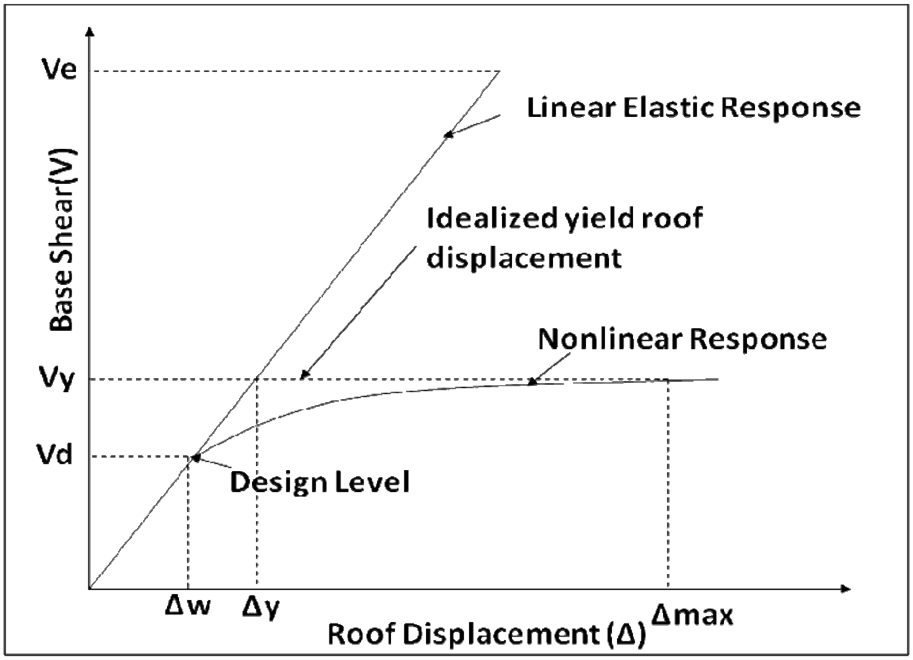



Figure 3-1. Force displacement response of elastic and inelastic systems

## 3.2 Definition of R Factor and its Components

As already discussed, R factors are essential seismic design tools, which defines the level of inelasticity expected in structural systems during an earthquake event. The commentary to the 1988 NEHRP provisions defines R factor as "…factor intended to account for both damping and ductility inherent in structural systems at the displacements great enough to approach the maximum displacement of the systems." This definition provides some insight into the understanding of the seismic response of buildings and the expected behavior of a code-compliant building in the design earthquake. R factor reflects the capability of structure to dissipate energy through inelastic behavior. R factor is used to reduce the design forces in earthquake resistant design and accounts for damping, energy dissipation capacity and for over-strength of the structure.

Conventional seismic design procedures adopt force-based design criteria as opposed to displacement-based. The basic concept of the latter is to design the

structure for a target displacement rather than a strength level. Hence, the deformation, which is the major cause of damage and collapse of structures subjected to earthquakes, can be controlled during the design. Nevertheless, the traditional concept of reducing the seismic forces using a single reduction factor, to arrive at the design force level, is still widely used. This is because of the satisfactory performance of buildings designed to modern codes in full-scale tests and during recent earthquakes.

In order to justify this reduction, seismic codes rely on reserve strength and ductility, which improves the capability of the structure to absorb and dissipate energy. Hence, the role of the force reduction factor and the parameters influencing its evaluation and control are essential elements of seismic design according to codes. The values assigned to the response modification factor (R) of the US codes (FEMA, 1997; UBC, 1997) are intended to account for both reserve strength and ductility (ATC, 1995). Some literature also mentions redundancy in the structure as a separate parameter. But in this study, redundancy is considered as a parameter contributing to overstrength, contrary to the proposal of ATC-19 (ATC, 1995), splitting R into three factors: strength, ductility and redundancy.

The philosophy of earthquake resistant design is that a structure should resist earthquake ground motion without collapse, but with some damage. Consistent with this philosophy, the structure is designed for much less base shear forces than would be required if the building is to remain elastic during severe shaking at a site**.** Such large reductions are mainly due to two factors: (1) the ductility reduction factor ($R\mu$ ), which reduces the elastic demand force to the level of the maximum yield strength of the structure, and (2) the overstrength factor, ($\Omega$), which accounts for the overstrength introduced in code-designed structures. Thus, the response reduction factor (R) is simply $\Omega$ times $R\mu$. See Figure 3-2.

$$R = R\mu \times \Omega \qquad (1)$$

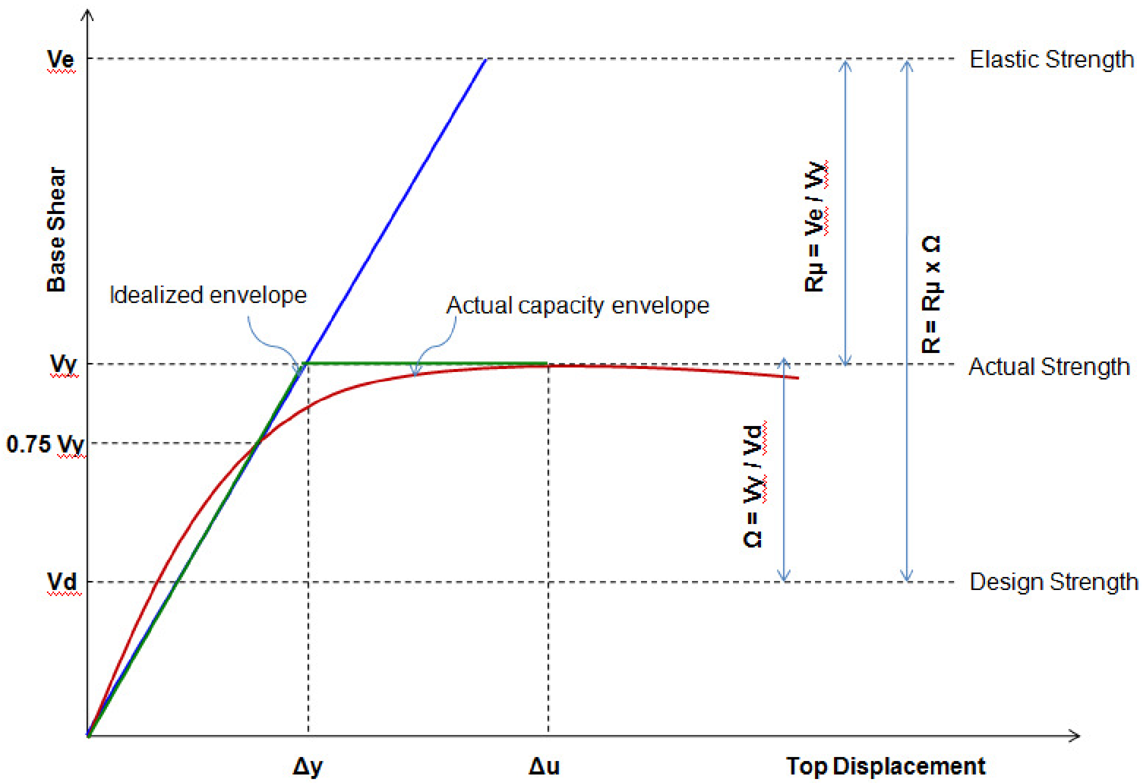



Figure 3-2: Relationship between force reduction factor (R), structural overstrength (Ω), and ductility reduction factor (Rμ)

### 3.2.1 Ductility Reduction Factor (Rμ)

The ductility reduction factor (Rμ) is a factor which reduces the elastic force demand to the level of idealized yield strength of the structure and, hence, it may be represented as the following equation:

$$R\mu = Ve / Vy \quad (2)$$

Ve is the max base shear coefficient if the structure remains elastic. The ductility reduction factor (Rμ) takes advantage of the energy dissipating capacity of properly designed and well-detailed structures and, hence, primarily depends on the global ductility demand, μ, of the structure (μ is the ratio between the maximum roof

displacement and yield roof displacement . Newmark and Hall (1973, 1982) made the first attempt to relate Rμ with μ for a single-degree-of-freedom (SDOF) system with elastic-perfectly plastic (EPP) resistance curve. They concluded that for a structure of a natural period less than 0.2 second (short period structures), the ductility does not help in reducing the response of the structure. Hence, for such structures, no ductility reduction factor should be used. For moderate period structures, corresponding to the acceleration region of elastic response spectrum T = 0.2 to 0.5 sec the energy that can be stored by the elastic system at maximum displacement is the same as that stored by an inelastic system. For relatively long-period structures of the elastic response spectrum, Newmark and Hall (1973, 1982) concluded that inertia force obtained from an elastic system and the reduced inertia force obtained from an inelastic system cause the same maximum displacement. This gives the value of ductility reduction factor in a mathematical representation as:-

$$R\mu = \mu \qquad (3)$$

### 3.2.2 Structural Overstrength (Ω)

Structural overstrength plays an important role in collapse prevention of the buildings. The overstrength factor (Ω) may be defined as the ratio of actual to the design lateral strength:

$$\Omega = Vy / Vd \qquad (4)$$

Where Vy is the base shear coefficient corresponding to the actual yielding of the structure; Vd is the code-prescribed unfactored design base shear coefficient.

The inertia force due to earthquake motion, at which the first significant yield in a reinforced concrete structure starts, may be much higher than the prescribed unfactored base shear force because of many factors such as (1) the load factor applied to the code-prescribed design seismic force; (2) the lower gravity load applied at the time of the seismic event than the factored gravity loads used in design; (3) the strength

reduction factors on material properties used in design; (4) a higher actual strength of materials than the specified strength; (5) a greater member sizes than required from strength considerations; (6) more reinforcement than required for the strength; and (7) special ductility requirements, such as the strong column-weak beam provision. Even following the first significant yield in the structure, after which the stiffness of the structure decreases, the structure can take further loads. This is the structural overstrength which results from internal forces distribution, higher material strength, strain hardening, member oversize, reinforcement detailing, effect of nonstructural elements, strain rate effects.

## 3.3 Historical Perspective and Overview of R Factor

### 3.3.1 Historical Perspective

Efforts to construct buildings which can safely resist seismic events in the modern era has just passed 100 years which can be divided into three periods: the first utilizing a prescribed percent of the building weight as an applied load; the second using forms of the equation V = ZKCW relating the seismic base shear (V) to a seismic zone factor (Z), the building's period (C), the building's weight (W) and the building system type (K); and most recently the use of site specific ground motion maps, building period, importance factors, site (soil) factors and 'Response Modification Factors (R)' to compute equivalent lateral forces on the structure.

The 1961 UBC Code introduced the use of four K factors to categorize building system type. Following research and recommendations included in ATC-3-06 (1978), the 1988 UBC introduced the use of Rw factors with twenty-nine structural system types. By 1993 the BOCA Code included the R factor for the same twenty-nine systems plus three additional for inverted pendulum systems. 1993 BOCA also included the Cd factor for deflection amplification whereas previously deflection amplification was computed based on a multiplier (0.7) of the Rw factor. Cd factor addresses the

likelihood that the deformations of the structure in an earthquake will be greater than those indicated by the linear deformation equations. The 1994 Northridge earthquake was followed by widespread application of seismic design throughout the United States for the first time. The combining of Codes and the almost uniform adoption of International Building Code (IBC) has helped insure a uniform design approach. However, IBC standards have been changing quickly. The latest edition of IBC 2006 has eighty-three building Response Modification Factors. These include R=3 for reinforced concrete systems not specifically detailed for seismic resistance. Uniform Building Code has an R factor of 3.5 for the same lateral resisting system. Values of R factor recommended in UBC 97 for moment resisting frame systems are shown in Table 3-1.

| Basic Structural System | Lateral-Force-Resisting System Description | R |
|---|---|---|
| Moment-resisting frame system | 1. Special moment-resisting frame (SMRF) | |
| | a. Steel | 8.5 |
| | b. Concrete[4] | 8.5 |
| | 2. Masonry moment-resisting wall frame (MMRWF) | 6.5 |
| | 3. Concrete intermediate moment-resisting frame (IMRF)[5] | 5.5 |
| | 4. Ordinary moment-resisting frame (OMRF) | |
| | a. Steel[6] | 4.5 |
| | b. Concrete[7] | 3.5 |
| | 5. Special truss moment frames of steel (STMF) | 6.5 |

Table 3-1. R factor values in UBC-97 for moment resisting frames (MRF)

### 3.3.2 Overview of R Factor

. The seismic force values used in the design of buildings are calculated by dividing forces that would be associated with elastic response by a response modification factor. Concept of R factor was proposed based on the fact that well detailed framing systems could sustain large inelastic deformation without collapse (ductile behavior) and develop lateral strength in excess of their design strength (often termed as reserve strength or overstrength). Level of this reduction normally specified in code is based on the observation of the performance of different structural systems in

previous earthquakes or during tests in laboratories. The R factor is assumed to represent the ratio of the forces that would develop under the specified ground motion if the framing system was to behave entirely elastically to the prescribed design forces at the strength level. (See Figure 3-2)

R factors are used in current building codes to estimate strength demands for structural systems designed using linear methods but responding in nonlinear manner. Their values are vital in the specification of design seismic loading. R factors were originally based on judgment and qualitative comparisons with known response of some of the framing systems. Now it has come a long way by actually quantifying it using nonlinear analysis tools and peak ground and spectral parameters.

Response modification factor (also termed as force reduction factor) plays a key role in seismic process. No other parameter in the design base shear equation (equation 5) given in UBC-97, impacts the design actions in a seismic framing system as does the value assigned to R.

$$V = 2.5 * (pga) * \frac{W}{R} \tag{5}$$

where,

V = Design base shear

W = Weight of the building

R = Response modification factor.

pga = Peak ground acceleration

As mentioned previously, structures are not designed to resist earthquake forces in their elastic range, instead, concepts of energy absorption in the inelastic range are used to reduce the elastic forces. Lower force levels are generated in the inelastic system, due to energy absorption by hysteresis (inelastic force-displacement response). By having extensive analysis and study of elastic and inelastic spectra, three regions of

response, dependent on the period of the structure, have been identified by Newmark and Hall (1982), which are as follows:

| | | | |
|---|---|---|---|
| Short period | $T < 0.2$ seconds | $P_d = P_e$ | (6) |
| Intermediate period | $0.2 < T < 0.5$ seconds | $P_d = \frac{P_e}{\sqrt{2\mu - 1}}$ | (7) |
| Long period | $T > 0.5$ seconds | $P_d = \frac{P_e}{\mu}$ | (8) |

where Pd is the design force and Pe is the elastic force. μ is the ductility and T is the fundamental period of vibration of the structure.

The first region is referred to as equal acceleration, the second as equal energy and the third as equal displacement. The limits of these regions are not fixed for all earthquakes and the above period ranges are only indicative. All three are represented graphically in Figure 3-3 upon which equations 6-8 are based on.

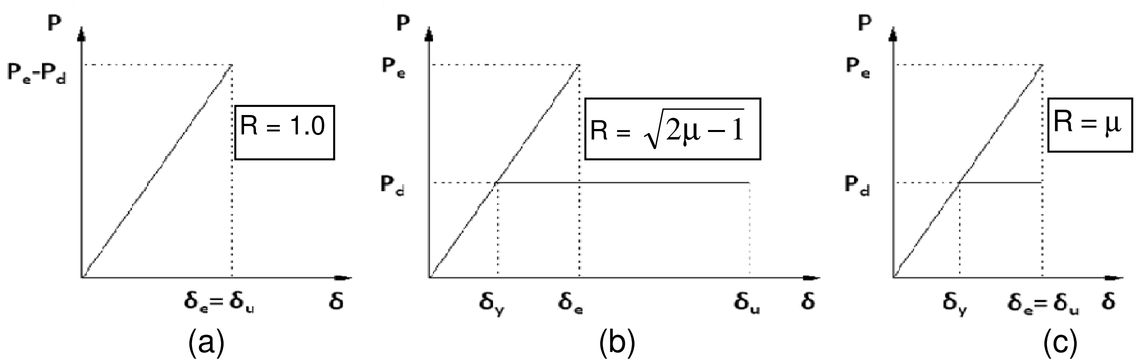



Figure 3-3: Relationship between elastic and inelastic forces for (a) short period (b) intermediate period (c) long period structures. (Newmark and Hall, 1982).

The above relationship shows that R is a function of μ (keeping overstrength Ω as unity or constant) within the three period zones suggested by Newmark and Hall (1982). Increase in value of R factor with increase in period is due to the tendency of the system to exhibit inelastic behavior, meaning R factor is period-dependent.

## 3.4 R Factor in Seismic Code of Various Countries

The use of response modification factor or its equivalent has been introduced in the seismic codes in many parts of the world. Use of R factor in the seismic design code, in the countries which are affected by earthquakes, reiterates its importance. A brief overview of response modification factor in the seismic codes of Europe, Japan, Mexico, and Egypt is presented in the following paragraphs.

### 3.4.1 Europe (Euro code 8)

The seismic design procedure in Euro code is a single level design procedure that reduces elastic spectral demands to the strength design level through the use of a period-dependent response factor, known as 'behavior factor q.' This behavior factor varies as a function of ductility, building strength, structural system and stiffness regularity. Following equation is used in Euro code to determine the q factor:

$$q = q_0 . k_D . k_R . k_W \tag{9}$$

where, $q_0$ is the basic value for response factor, $k_D$ represent ductility class, $k_R$ is a factor reflecting structural irregularity in elevation and $k_W$ reflects prevailing failure mode (for MRF frames $k_W$ is taken to be 1). Values of above factors are shown in tables 3-2 to 3-4.

| Structural type | | $q_0$ |
|---|---|---|
| Frame system | | 5.0 |
| Dual system | Frame equivalent | 5.0 |
| | Wall equivalent, with coupled walls | 5.0 |
| | Wall equivalent, with uncoupled walls | 4.5 |
| Wall system | with coupled walls | 5.0 |
| | with uncoupled walls | 4.0 |
| Core system | | 3.5 |
| Inverted pendulum system | | 2.0 |

Table 3-2. Basic value of response factor $q_0$ in Eurocode

| Ductility class | $k_D$ |
|---|---|
| DC"H" | 1.0 |
| DC"M" | 0.75 |
| DC"L" | 0.5 |

Table 3-3. Values of $k_D$ represent ductility class in Eurocode

| Regularity in elevation | $k_R$ |
|---|---|
| Regular structures | 1.0 |
| Non-regular structures | 0.8 |

Table 3-4. Values of $k_R$ reflecting structural irregularity in elevation in Eurocode

Values of $q_o$ factor range between 2 and 5 for reinforced concrete framing system (See Table 3-2). A more complex formulation of this q factor has also been devised by incorporating period of the structure. Presentation of this formulation is out of the scope of this thesis.

### 3.4.2 Japan

The Japanese building standard law (BSL) includes a two-phase or two-level procedure for the seismic design of buildings (ATC-19). The first phase design follows an approach in which strength design is used for reinforced concrete structures. Seismic actions are computed using unreduced seismic forces. The second phase design is a direct evaluation of strength and ductility and may be regarded as check of whether these are sufficient for severe ground shaking. BSL uses R in a different format as is done in codes in USA. A ductility factor (1/Ds) which is equivalent to R factor is used for all building systems which range from 1.8 to 4. The BSL requires that, in addition to sizing the members for the serviceability limit state, the building's strength is checked for the ultimate limit state. In BSL, the reduction factor due to ductility (1/Ds), for special ductile moment frames is equal to 4. (See Table 3-5). The equivalent structure with high ductility as specified in UBC has a R factor of 8 which is significantly more as compared to BSL

### 3.4.3 Mexico

The Mexico City Building Code uses a period-dependent reduction factor (Q') to reduce elastic spectral demands to a strength design level. Values of Q' range between 2.5 and 4. (See Table 3-5).

| Response Modification Factor Comparison | | | | |
|---|---|---|---|---|
| Structural System | Period | Europe | Japan | Mexico |
| RC Moment resisting frame | T=0.1 sec | 2.3 | 3.3 | 2.5 |
| | T=1.0 sec | 5 | 4 | 4 |

Table 3-5. Comparison of R factor for EC-8, Japan and Mexico seismic codes

### 3.4.4 Egypt

In chapter 8 of the Egyptian code, "Loads and forces on structural and nonstructural systems", the R factor defined for reinforced concrete structure is either 5 or 7 for RC moment resisting frames, based on level of ductility. This level of ductility is either sufficient or non-sufficient, which in turn is based on detailing, number and location of plastic hinges and failure mode.

| R factor in Egyptian code | | |
|---|---|---|
| Structural System | Ductility | R |
| RC Moment resisting frame | Sufficient | 7 |
| | Not Sufficient | 5 |

Table 3-6. R factor in Egyptian seismic code

In this new era of seismic design and provisions, all countries which are affected by seismic forces should consider modern seismic design practices by making structure exhibit more ductile behavior. The R factor is unique for every kind of structure, ground motion and site condition. It is therefore a pressing need for all developing countries to

formulate their own seismic provisions regarding seismic design and R factor based on their local conditions and building parameters.

Based on vulnerability towards seismic events, Pakistan, as one of the most seismically vulnerable developing countries, needs to have its own seismic design code based on these response reduction factors. These reduction factors are affected by many distinguished variables, such as type of seismic zones, types and configurations of buildings, characteristics of construction materials, etc. Most of these variables which are unique for different regions will have to be studied independently to come up with seismic design code for Pakistan.

# CHAPTER 4

# TYPE OF BUILDING SYSTEMS AND DESIGN CODES IN PAKISTAN

## 4.1 Introduction to Type of Construction in the Region

The majority of the buildup areas in Pakistan consist of residential buildings. Roughly 70-80% of all buildings are residential complexes with first floor being shops and rest of the building being residential apartments. October 2005 Kashmir earthquake had most of its impact in northern areas of Pakistan where medium high rise structures exist in the population centers.

In rural areas the residential buildings are divided into two distinguished categories, namely katcha (temporary) and pucca (permanent) houses. A katcha house has mud or stone rubble walls with a flat mud roof supported on timber beams to support heavy mud insulation and snow load. A pucca house typically has stone rubble or fired brick masonry walls with cement-sand mortar and a corrugated metal sheet or precast reinforced concrete (RC) flat slab roof. The main cause of collapse of both types is the heavy weight of the roof which attracts large inertia forces. The unreinforced walls experience out of plane failure due to inertial forces and collapse under the weight of the roof.

In relatively urban areas, the use of masonry blocks with a reinforced concrete slab is very common. There also exist many reinforced concrete frames with infill walls in mid to large size towns. Many of such semi-engineered buildings either completely collapsed or suffered serious damage in recent Kashmir 2005 earthquake. The collapse or damage has generally been linked to poor quality construction, deficient detailing, and lack of seismic consideration.

In bigger cities like Islamabad, Lahore and Karachi, one can see many high rise and modern construction which are primarily reinforced concrete moment resisting frames, but exception of steel frames is also there. During Kashmir 2005 earthquake development of soft story and column sway mechanism was very common. This is mainly attributed to deficient detailing, lack of seismic design, stiffness discontinuities and out of plain failure of unreinforced masonry infill.

## 4.2 Description of RC Structures in Pakistan

Based on verbal discussions with researchers from the University of Engineering and Technology at Peshawar (Dr. Qaisar Ali and his research team), the following information was obtained on the type of RC structures in the region,

(i) Reinforced concrete structures in Pakistan are mostly in the form of commercial buildings and constitute approximately 10% to 15% of the building stock in the urban areas. In some instances, multi storey residential complexes in the urban areas, especially in the metropolitan cities of Pakistan like Karachi, Islamabad, Lahore and Peshawar etc. are constructed of reinforced concrete. In rural areas, the percentage of RC structures is less than 2%.

(ii) For reinforced concrete (RC) structures, the number of stories range from 3 to 4 while the number of bays is from 3 to 5. The bay width is 12 to 20 feet and storey height is 10 to 12 feet.

(iii) The type of lateral system for the RC structures is generally moment resisting frames, however such buildings are seldom designed for lateral forces.

(iv) Usually the buildings are rectangular in both horizontal and vertical planes but in some instances geometric irregularities are present in the buildings.

(v) The foundation type for the RC structures is usually isolated column footings. However structures that have more bays and storey may have raft or stripped foundations too.

The main reasons for the common use of MRFs in Pakistan and rest of the neighboring countries are:

(i) Ease in construction and availability of labor, as not much skill and technicality is required during construction process.
(ii) Availability of cheap material to construct MRFs even in remote parts of Pakistan.

Furthermore, a common construction practice in Pakistan is the use of rigid masonry infill in external moment resisting frames which provides a good solution for providing thermal and acoustic insulation and weather-proofing. The masonry infill causes a large increase in strength and stiffness, at the expense of a large reduction in ductility. If the infill is not uniform across the building, unsafe conditions such as creation of soft storey can result. Experience from recent Kashmir earthquake is that frames which are rigidly infilled with brittle brickwork suffer very serious problems, particularly formation of weak storey, creation of short column phenomena and danger from falling blocks/bricks.

### 4.3 Shortcomings in Construction Practices

During Kashmir earthquake the structural damages which occurred were expected, due to the poor quality of construction of traditional housing and modern RC structures not designed to resist earthquake action. Although the shaking was comparable to previous major damaging earthquakes like ones in Turkey, USA and Japan, still poor construction quality and lack of seismic detailing played an important role in the collapse of many structures. Since wind design is relatively mild in the region, even engineered structures are not expected to resist significant lateral loads since they were not designed to resist significant wind loads. The engineered structures were fairly well constructed, and cases of failure were due mainly to layout defects, such as soft ground storey, short columns, irregular plans and elevations, as well as lack of maintenance on a few cases. (MAE center report, 2005)

## 4.4 Review of Seismic Codes in Pakistan

After the Kashmir 2005 earthquake, a dire need was felt to design the buildings in Pakistan with some resistance to seismic forces. At the time, Pakistan did not have any seismic code. The National Engineering Services Pakistan (NESPAK) was made responsible by the Government of Pakistan, for preparing a new seismic code (still under development). Although an initial version of this code came out in 2006, referred as NESPAK-2006, but the work on its development and release is still in progress. Due to delay in publication of NESPAK code, Earthquake Reconstruction and Rehabilitation Authority (ERRA), responsible to reconstruct in the aftermath of the earthquake, decided to adopt Uniform Building Code (UBC) 1997. This adopted version is named as Building Code of Pakistan (BCP 2007) which takes into account the Peak Ground Acceleration (PGA) maps and seismic zoning of Pakistan. Although, BCP-2007 which was published in 2007, was adopted to fulfill the immediate need of reconstruction and serve as a guideline in rehabilitation, the future code of Pakistan still remains the NESPAK code. Presently the NESPAK 2006 code has a similar basic outline as of BCP, but different values for various parameters are recommended. These differences lead to significant variation in spectral response and consequently in base shear calculations. The present code of practice in use in Pakistan, namely BCP 2007 gives out values of R factors based on ductility and overstrength of the structures designed in USA. The level of ductility for the structures being built in Pakistan is nowhere close to the high level of ductility in USA which is achieved by following better seismic reinforcement detailing guidelines. Thus using R factors from the USA or Europe codes will be a false representation of building's seismic performance, which may lead to failure or even collapse of the structure during an event of a major earthquake.

### 4.4.1 Building Code of Pakistan 2007

The officially published building code of Pakistan was recently released and is a modified version of the Uniform Building Code (UBC) 1997 that takes into account the

seismic characteristics of the Pakistan region. There are five different seismic zones (1, 2A, 2B, 3, 4), each one with a range of peak horizontal ground acceleration (see Figure 2-8). All the equations to compute the base shear as well as the tables for each coefficient used in the equations are the same as those in UBC 97. The total design base shear, $V$, as described in BCP-07 and UBC-97, is determined using the following equation:

$$V = 0.11C_a IW \leq V = \frac{C_v I}{RT} W \leq V = \frac{2.5C_a I}{R} W \quad (10)$$

where, $C_a$ and $C_v$ are seismic coefficients, which depend on the type of soil and the seismic zone factor $Z$. $R$ is the numerical coefficient representative of the inherent overstrength and global ductility capacity of the global resisting system. $T$ is the period of the structure and $I$ is the importance factor. The design base shear, $V$ should be distributed in each floor of the building. The shear force can then be presented in the following form:

$$V = F_t + \sum_{i=1}^{n} F_i \quad (11)$$

where $F_i$, is the force at each storey level, and $F_t$ is a concentrated force at the top is computed as:

$$F_t = 0.07TV \geq 0.25\text{V} \quad (12)$$

$F_t$ can be taken as zero when $T$ is 0.7 or less.

The forces at each storey, $F_i$, are computed according to the following equation:

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \quad (13)$$

The period of the structure can be approximated by the following equation:

$$T = C_t H_n^{0.75} \tag{14}$$

where $C_t$ is a constant and its value depends on type of structural system, for example, for reinforced concrete frames $C_t$ =0.07 while $C_t$ is 0.08 for steel frames, while $H_n$ is the storey dimension in meters. When using response spectrum analysis, the criteria used to determine the minimum amount of modes to be considered is to make sure that the sum of the modal masses is no less than 90% of the total building mass. Also, all modes having a mass exceeding 5% of the building mass should be considered.

For time history analysis, at least three appropriate recorded ground-motions or simulated ground motion time history record can be used. The parameter of interest should be calculated for each time history analysis. If three analyses are performed, take the maximum response parameter for design; if seven or more time history analyses are performed, use the average value.

### 4.4.2 NESPAK Draft Code 2006

After the October 8, 2005 Pakistan earthquake, the National Engineering Services Pakistan, NESPAK was responsible for preparing a new seismic code in Pakistan to better protect buildings against future damages caused by high intensity earthquakes. Previously, the seismic code mostly used by engineers in the region was the UBC 1997 code. In the NESPAK code the seismic maps were changed from what was provided in BCP 2007 code. The base shear ($V_t$) using equivalent load method is given by,

$$V_t = \frac{WA(T_1)}{R_a(T_1)} \geq 0.10 A_o I W \tag{15}$$

where $W$ is the weight of the structure and $T_1$ is the first natural period of vibration of the structure. The spectral acceleration coefficient, $A$ is computed as:

$$A(T) = A_0 I S(T) \tag{16}$$

where $A_o$ is the peak ground acceleration (PGA) found on the seismic map for a 10% probability of exceedance in 50 years. *I*, is the building importance factor and *S* is the spectrum coefficient which is a function of the period of the structure and the local site conditions. In order to obtain the *S(T)* value, the spectrum characteristic periods, $T_A$ and $T_B$, need to be determine. $T_A$ and $T_B$ are a function of the local site class. Also required to obtain the spectrum coefficient is the period of the structure which can be approximated by the following equation when the height of the building is less than 15 m:

$$T_1 = T_{1A} = C_t (H_N)^{0.75} \tag{17}$$

where the $C_t$ value will depend on the type of structural system used and $H_N$ is the storey height in meters. For example, for reinforced concrete frames $C_t$ is 0.07 while $C_t$ is 0.08 for steel frames. For buildings 15 m or higher, the first natural period of vibration is calculated as:

$$T_1 = 2\pi \left[ \frac{\sum_{i=1}^{N} (m_i d_{fi}{}^2)}{\sum_{i=1}^{N} (F_{f_i} d_{fi})} \right]^{0.5} \tag{18}$$

where $m_i$ is the mass of the ith storey and $F_{fi}$ is:

$$F_{f_i} = \frac{w_i H_i}{\sum_{j=i}^{N} (w_i H_i)} * \Delta \tag{19}$$

where Δ is a fictitious load acting on the $i^{th}$ storey, $w_i$ is the weight of each storey and $H_i$ is the height of each floor. In order to determine the seismic loads on the structure, the acceleration coefficient needs to be divided by the Response factor, $R_a$. The value $R_a$ is obtained as follows:

$$R_a(T) = 1.5 + (R - 1.5)\frac{T}{T_A} \qquad (0 \le T \ge T_A) \tag{20}$$

$$R_a(T) = R \qquad (T > T_A) \tag{21}$$

where $R$ is a function of the structural system of the building. The $R$ factors vary depending on whether the structure has a nominal or high ductility. $R$ which is ductility dependent factor increases with higher ductility. The definition of nominal and high ductility is defined for concrete and steel in their specific chapters in the code.

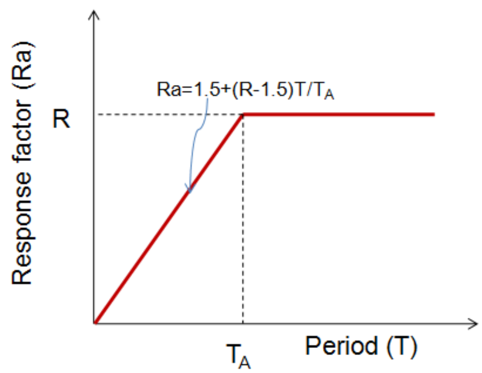



Figure 4-1: Graphical representation of equation 20 and 21

### 4.4.3 IBC 2006

The International Building Code (IBC), which was released in 2000, superseded the UBC-97 code and is meant only for USA, as all its equations are based on PGA maps for USA. Later a 2006 version of IBC was also released, which has been referred to in this study for comparison. Although IBC cannot be applied for buildings in Pakistan, it is still worth to discuss the code in order to draw comparison of R factors and base shear equations.

The equivalent lateral force or simplified analysis can be used for Seismic use group I only if it is a light frame building less than three stories high or any other type of building no less than two stories high. For this type of analysis, the seismic base shear equation is:

$$V = \frac{1.2 S_{DS}}{R} W \tag{22}$$

where $W$ is the total weight of the structure and $S_{DS}$ is the design spectral response acceleration at short period. The International Building Code (IBC) requires two period values to compute the spectral response acceleration: the short period $Ss$ and the 1-second period $S_1$. The values for $Ss$ and $S_1$ can be obtained from figures in IBC code (Reference 1613.5(1) through (14)).

These values are then adjusted to account for the type of site class by site coefficient factors, $F_a$ and $F_v$. The site factor values depend on the site class and values of $S_s$ and $S_1$. The modified spectral response acceleration parameters are then computed as:

$$\begin{aligned} S_{MS} &= F_a S_S \\ S_{M1} &= F_v S_1 \end{aligned} \tag{23}$$

According to the IBC06 code, the five percent design spectral response acceleration at short period $S_{DS}$ and at 1-second period $S_{D1}$ shall be determined as follows:

$$\begin{aligned} S_{DS} &= \frac{2}{3} S_{MS} \\ S_{D1} &= \frac{2}{3} S_{M1} \end{aligned} \tag{24}$$

The other value needed to compute the design response spectra $S_a$ as shown in figure 4-2, obtained from IBC06 code. $T_o$ and $T_s$ are two periods used to define the spectral acceleration plot vs period. $T_o$ and $T_s$ are defined as:

$$T_o = 0.2\, S_{D1} / S_{DS} \qquad T_s = S_{D1} / S_{DS}. \tag{25}$$

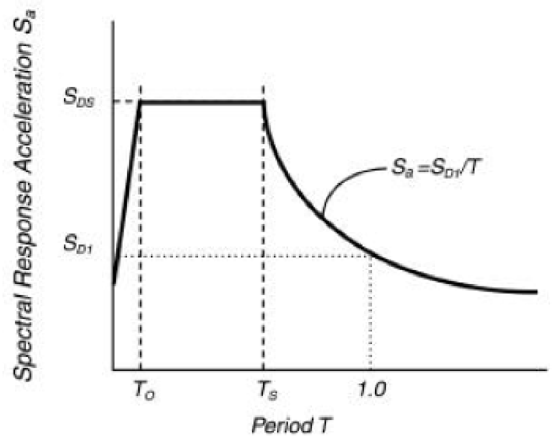



Figure 4-2: Spectral acceleration plot vs period (T) (MAE center report, 2005)

For periods less than or equal to $T_o$, the design spectral response acceleration, $S_a$, shall be determined using the following equation:

$$S_a = 0.6\frac{S_{DS}}{T_0}T + 0.4S_{DS} \tag{26}$$

For periods greater or equal to $T_o$ and less or equal to $T_s$, $S_a$ is equal to $S_{DS}$ and for periods greater than $T_s$, $S_a$ is equal to:

$$S_a = \frac{S_{D1}}{T} \tag{27}$$

Each structure shall be assigned an importance factor that will depend on the occupancy type of the building. Also, each structure shall be assigned a seismic design group based on their seismic use group and its design spectral coefficient values, $S_{DS}$ and $S_{D1}$.

### 4.4.4 Comparison between Codes

The main differences in the seismic codes are the calculation of base shear forces and the computation of spectral acceleration. The spectral acceleration values are the same for UBC97/BCP07 and IBC06, while NESPAK Draft Code 06 has higher values. The base shear equations in all 3 codes are compared in Table 4-1. In second

row of table, if all the codes have same value of spectral acceleration, then the equations look similar.

| Building reference | NESPAK Code | UBC97 / BCP07 | IBC06 |
| --- | --- | --- | --- |
| Base Shear Equation | $V = \frac{A_o S(T)}{R} IW$ | $V = \frac{2.5C_a}{R} IW$ | $V = \frac{S_{DS}}{R} IW$ |

Table 4-1: Comparison of base shear equation in different seismic codes.

By having same spectral acceleration values, they arrive at the same equation for the shear, but then there is a difference in their importance factor *I* and response modification factor *R*.

Also there is a difference in suggested values of importance factor (*I*) for each building. For example, for schools, NESPAK Draft Code 06 gives an *I* value of 1.4, UBC97/PK07 gives 1.0 while IBC06 gives it a 1.2. (MAE center report, 2005)

There is also a difference in the values of R factor. Variation of R factor in the codes for various moment resisting frames is presented in Table 4-2.

| R factor for Moment Resisting Frame Systems | | | |
| --- | --- | --- | --- |
| **Structural System** | **IBC 06** | **UBC 97 / BCP 07** | **NESPAK** |
| Special RC Moment frame | 8 | 8.5 | 7 |
| Intermediate RC Moment frame | 5 | 5.5 | 6 |
| Ordinary RC Moment frame | 3 | 3.5 | 4 |

Table 4-2: R factor in IBC 06, UBC 97 / BCP07 and NESPAK code

The R factor for an ordinary concrete frame is given as **4** in NESPAK Draft Code 06, **3.5** in UBC97/PK07 and **3** for IBC06. Also there is difference in the values of spectral acceleration ($S_a$). The reason that the spectral acceleration value in NESPAK code is not the same as UBC97/PK07 and IBC06 is because $S_a$ is a function of the importance factor ($S_a = A_0 I S_{(T)}$ ) in the NESPAK code while it is not for the other two cases.

# CHAPTER 5

# R FACTOR CALCULATION METHODS

## 5.1 Introduction

In chapter 3, Response modification or Force reduction factor (R) was discussed in detail from the capacity point of view. In this chapter reduction factor from demand perspective will be considered. The reduction factor 'demand' is defined as the ratio between the elastic ($Sa_{elastic}$) and the inelastic ($Sa_{inelastic}$) response spectral ordinates corresponding to a specific period T (ATC-19). (See Figure 5-1).

$$\text{Force Reduction Factor} = \frac{Sa_{elastic}\,(T)}{Sa_{inelastic}\,(T)} \tag{28}$$

Thus it expresses the ratio of the elastic strength demand to the inelastic strength demand for a specified constant ductility μ.

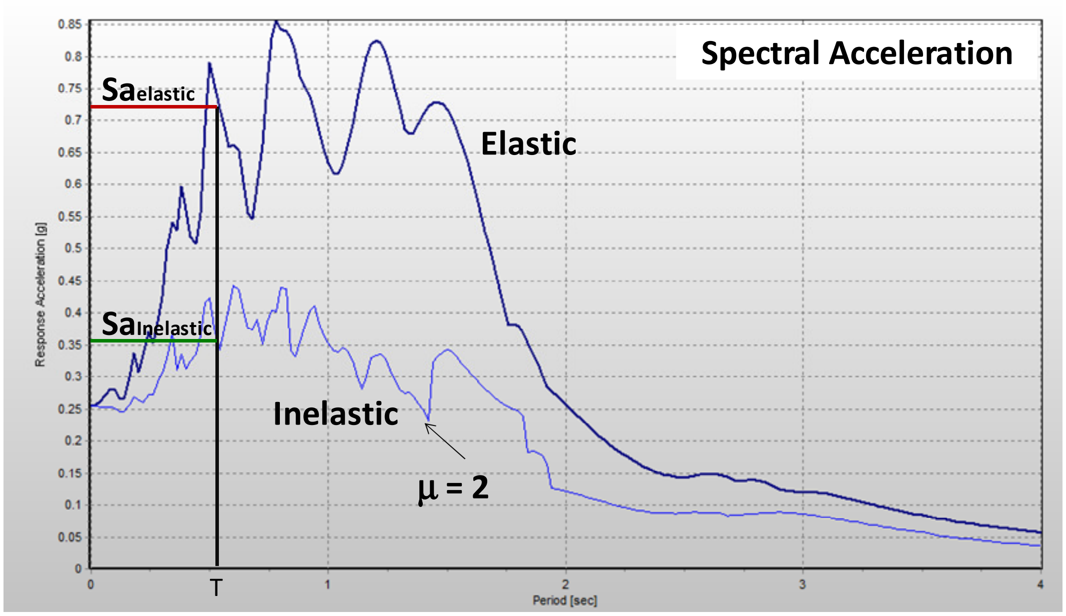



Figure 5-1: Elastic and inelastic acceleration response spectra of Abbotabad, Kashmir 2005 ground motion

The response factor ‘demand’ represents the minimum reduction coefficient corresponding to a specific level of ductility obtained from inelastic constant ductility spectra and elastic spectra (See Figure 5-1). For a given period, the elastic spectral ordinate should be divided by the inelastic counterpart for a value of ductility expected for the structural system under consideration. Some of observations in this regards are as following:

- R factor is not constant but rather varies considerably with period.
- At very short periods, the R factor is almost unity.
- For low levels of ductility, the statically derived relationships of R=1, R=μ, and R= $\sqrt{2\mu - 1}$ hold quite well (Newmark and Hall,1982), but are distinct from the actual R factors for higher ductility levels

## 5.2 Various Methods used for Calculating R Factor

The relationship between displacement ductility and ductility-dependent R factor has been the subject of considerable research. A few of most frequently used relationships reported in the technical literature are discussed below:

### 5.2.1 Newmark and Hall (1982)

In this early study, Rμ was determined to be a function of μ. It was observed that in the long period range, elastic and ductile systems with the same initial stiffness reached almost the same displacement. As a result, the response factor can be considered equal to the displacement ductility. This is referred to as ‘equal displacement’ region. For intermediate period structures, the ductility is higher than the response factor and the ‘equal energy’ approach may be adopted to calculate force reduction. The relationship derived for Rμ as a function of μ, for short, intermediate and long period structures is presented below:

| | | |
|---|---|---|
| Short period | T < 0.2 seconds | $R\mu=1$ |
| Intermediate period | 0.2 < T < 0.5 seconds | $R\mu=\sqrt{2\mu-1}$ |
| Long period | T > 0.5 seconds | $R\mu=\mu$ |

### 5.2.2 Krawinkler and Nassar (1992)

A relationship was developed for the force reduction factor derived from the statistical analysis of 15 western USA ground motions with magnitude between 5.7 and 7.7 (Krawinkler and Nassar, 1992). The influence of response parameters, such as yield level and hardening coefficient α, were taken into account. A 5% damping value was assumed. The equation derived is given as:

$$R_\mu = [c(\mu - 1) + 1]^{1/c} \tag{29}$$

$$c(T, \alpha) = \frac{T^a}{1+T^a} + \frac{b}{T} \tag{30}$$

where c is a constant which is dependent on period (T) and α which is the strain hardening parameter of the hysteretic model and a and b are regression constants. Values of the constants in above equations were recommended for three values of hardening α as in Table 5-1below:

| HARDENING VALUE | MODEL PARAMETERS | |
|---|---|---|
| α | a | b |
| 0 % | 1.00 | 0.42 |
| 2 % | 1.01 | 0.37 |
| 10 % | 0.80 | 0.29 |

Table 5-1: Model parameter constants for Krawinkler and Nassar

### 5.2.3 Miranda and Bertero (1994)

The equation for reduction factor introduced by Miranda and Bertero (1994) was obtained from a study of 124 ground motions recorded on a wide range of soil conditions. The soil conditions were classified as rock, alluvium and very soft sites

characterized by low shear wave velocity. A 5% of critical damping was assumed. The expressions for the period-dependent force reduction factors Rμ are given by:

$$R_{\mu} = \frac{\mu - 1}{\Phi} + 1 \tag{31}$$

where Φ is calculated from different equations for rock, alluvium and soft sites as shown below:

$$\begin{aligned}
\Phi &= 1 + \frac{1}{10\,T - \mu\,T} - \frac{1}{2\,T}\exp\left[-1.5\left(\ln T - 0.6\right)^2\right] && \text{for rock site} \\
\Phi &= 1 + \frac{1}{12\,T - \mu\,T} - \frac{2}{5\,T}\exp\left[-2\left(\ln T - 0.2\right)^2\right] && \text{for alluvium site} \\
\Phi &= 1 + \frac{T_1}{3\,T} - \frac{3\,T_1}{4\,T}\exp\left\{-3\left[\ln\left(T/T_1\right) - 0.25\right]^2\right\} && \text{for soft site}
\end{aligned} \tag{32}$$

where $T_1$ is the predominant period of the ground motion. The latter corresponds to the period at which the relative velocity of a linear system with 5% damping is maximum within the entire period range.

#### 5.2.4 Vidic et al. (1994)

The reduction coefficients Rμ calculated by Vidic *et al*. (1994) were approximated by a bilinear curve. In the short period range, the reduction factor increases linearly with the period from 1.0 to a value that is almost equal to the ductility factor. In the remaining part of the period range the reduction factor is constant. To calculate the reduction factor, a bilinear response model and a stiffness degrading 'Q-model' were employed. In this work, the standard records from California (USA) and Montenegro-1979 (Yugoslavia) were chosen as being representative for 'standard' ground motion, i.e. severe ground motion at moderate epicentral distance, with a duration ranging between 10 and 30 seconds and predominant period between 0.3 and 0.8 seconds. The proposed formulation of reduction factor, for special strong motion features, is:

$$R_\mu = c_1 (\mu - 1)^{c_R} \frac{T}{T_0} + 1 \qquad \text{when } T < T_0$$
$$R_\mu = c_1 (\mu - 1)^{c_R} + 1 \qquad \text{when } T \geq T_0 \tag{33}$$

where $T_0$ is the period dividing the period range into two portions. It is related to the predominant period of the ground motion $T_1$ by means of:

$$T_0 = c_2 \, \mu^{c_T} \, T_1 \tag{34}$$

The coefficients $c_1$, $c_2$, $c_R$ and $c_T$ in the above equations depend on the hysteretic behavior, either bilinear or with degrading stiffness, and damping, e.g. time dependent or independent. The values of the model parameters are outlined in Table 5-2 below:

| Model | Damping | $c_1$ | $c_2$ | $c_R$ | $c_T$ |
|---|---|---|---|---|---|
| Bilinear | Mass-proportional | 1.35 | 0.75 | 0.95 | 0.20 |
| Bilinear | Istantaneous stiffness proportional | 1.10 | 0.75 | 0.95 | 0.20 |
| Q-model | Mass-proportional | 1.00 | 0.65 | 1.00 | 0.30 |
| Q-model | Istantaneous stiffness proportional | 0.75 | 0.65 | 1.00 | 0.30 |

Table 5-2: Model parameter constants for Vidic et al.

### 5.2.5 Borzi and Elnashai (2000)

A very elaborate ground motion dataset was used to derive response modification factors (demand). Regression analyses for the evaluation of the ratio between the elastic and inelastic acceleration spectra were undertaken. An elastic perfectly plastic hysteretic model was utilizing to study the influence of ductility and input motion parameters, especially magnitude, distance and soil conditions, on the response factor. It was observed that the influence of input motion parameters on elastic and inelastic acceleration spectra is similar and significant. However, the effect cancels out for their ratio. Ductility is the most significant parameter, influencing the response modification factor. Consequently, analyses to define period dependent response factor functions for all the ductility levels and all structural models were undertaken. The

average values and the standard deviations were calculated considering various combinations of input motion parameters.

$$
\begin{aligned}
q &= (q_1 - 1)\frac{T}{T_1} + 1 && \text{when } T \le T_1 \\
q &= q_1 + (q_2 - q_1)\frac{T - T_1}{T_2 - T_1} && \text{when } T_1 < T \le T_2 \\
q &= q_2 && \text{when } T > T_2
\end{aligned}
\tag{35}
$$

The values $q_1$, $q_2$, $T_1$ and $T_2$ that define approximate spectra for all ductility levels and hysteretic parameters are summarized in Table 5-3:

| | $\mu=2$ | | | | $\mu=3$ | | | | $\mu=4$ | | | | $\mu=6$ | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $T_1$ | $T_2$ | $q_1$ | $q_2$ | $T_1$ | $T_2$ | $q_1$ | $q_2$ | $T_1$ | $T_2$ | $q_1$ | $q_2$ | $T_1$ | $T_2$ | $q_1$ | $q_2$ |
| EPP | 0.20 | 0.79 | 2.06 | 2.20 | 0.21 | 0.78 | 2.89 | 3.31 | 0.22 | 0.87 | 3.59 | 4.34 | 0.25 | 0.99 | 4.81 | 6.13 |
| $K_3=0$ | 0.20 | 0.56 | 2.20 | 2.51 | 0.25 | 1.67 | 3.10 | 4.09 | 0.27 | 1.55 | 3.76 | 5.45 | 0.29 | 1.26 | 4.78 | 7.79 |
| $K_3=10\%\ K_y$ | 0.21 | 0.54 | 2.04 | 2.33 | 0.27 | 1.80 | 2.78 | 3.62 | 0.29 | 1.64 | 3.25 | 4.56 | 0.33 | 1.54 | 3.93 | 6.10 |
| $K_3=-20\%\ K_y$ | 0.26 | 0.26 | 2.43 | 2.43 | 0.24 | 1.76 | 2.83 | 3.93 | 0.25 | 1.69 | 3.25 | 5.12 | --- | --- | --- | --- |
| $K_3=-30\%\ K_y$ | 0.26 | 0.26 | 2.42 | 2.42 | 0.24 | 1.85 | 2.76 | 3.81 | --- | --- | --- | --- | --- | --- | --- | --- |

Table 5-3: Parameters defining approximate spectra for all ductility level and hysteretic parameters used by Borzi and Elnashai.

where $K_3$ is the post yield stiffness of the primary curve of hysteretic hardening-softening model.(See Figure 5-2)

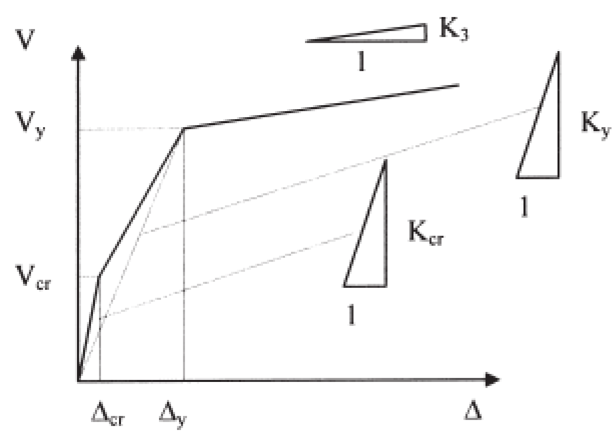



Figure 5-2: Primary tri-linear curve of force displacement relationship (Borzi and Elnashai, 2000)

Finally, the coordinates of the points that allow the definition of the approximate spectra were expressed as a function of ductility and given as:

$$\begin{aligned} T_1 &= b_{T1} \\ T_2 &= a_{T2}\,\mu + b_{T2} \\ q_1 &= a_{q1}\,\mu + b_{q1} \\ q_2 &= a_{q2}\,\mu + b_{q2} \end{aligned} \qquad (36)$$

where, $b_{T1}$, $a_{T2}$, $b_{T2}$, $a_{q1}$, $b_{q1}$, $a_{q2}$ and $b_{q2}$ are constants. Different values of $a_{q1}$, $b_{q1}$, $a_{q2}$ and $b_{q2}$ correspond to the different hysteretic behavior patterns, are given in Table 5-4 below:

| | $b_{T1}$ | $a_{T2}$ | $b_{T2}$ | $a_{q1}$ | $b_{q1}$ | $a_{q2}$ | $b_{q2}$ |
|---|---|---|---|---|---|---|---|
| EPP | 0.25 | 0.163 | 0.60 | 0.69 | 0.90 | 1.01 | 0.24 |
| $K_3$=0 | | | | 0.55 | 1.37 | 1.33 | 0 |
| $K_3$=10% $K_y$ | | | | 0.32 | 1.69 | 0.96 | 0.51 |
| $K_3$=-20% $K_y$ | | | | 0.38 | 1.67 | 1.24 | 0 |
| $K_3$=-30% $K_y$ | | | | 0.29 | 1.83 | 1.21 | 0 |

Table 5-4: Model parameter constants for Borzi and Elnashai

## 5.3 Analysis Methods

The two common methods of nonlinear analysis are nonlinear static analysis and nonlinear time history analysis. For both methods, framing systems are modeled and analyzed as an assembly of elements and components.

### 5.3.1 Nonlinear Static Analysis (Pushover analysis)

Nonlinear static analysis (Pushover analysis) was used in this study to evaluate the global limit states of the RC MRF in terms of drift and force level. In this analysis, the increasing forcing function, either in terms of horizontal forces (representation of inertial forces along the height of the structure) or displacements are imposed on a

mathematical model of a building. The analysis is terminated when the target displacement or ultimate limit state is reached. The target displacement or drift represents a maximum building displacement or drift during earthquake shaking. This kind of analysis can estimate the maximum strength and deformation capacity of the building. They also help in identifying potential weak and soft stories in the building.

Nonlinear static analysis is used to find the global limit states with loading profile of the first mode shape. The mode shapes, period of structure in each mode and modal participation factor are evaluated using ‘modal or eigen value analysis’. This simple analysis is useful as an initial validation tool of the analytical models.

Generally nonlinear static analysis is integrated into following steps, as follows:

(i) Develop 2D structural model of the building.

(ii) Impose gravity loads and apply static lateral loads or displacements in the pattern that approximately captures the relative inertial forces developed at locations of substantial mass or where the mass of each floor is lumped in the model.

(iii) Push the structure using the load pattern of step 2 to a target displacement level (i.e. the displacement of the target node reaches the target displacement).

(iv) Estimates the forces and deformations in each element at the level of displacement corresponding to the target displacement.

(v) Plot the base shear vs top storey displacement or storey shear vs storey displacement.

### 5.3.2 Incremental Dynamic Analysis (Nonlinear Time History Analysis)

Incremental dynamic analysis (IDA) method has been in use as early as 1977 but has become more popular recently due to advancement in computational capability. Vamvatsikos and Cornell (2001) showed the benefits of using IDA by changing the level of intensity of a specific ground motion. IDA method involves subjecting a structural

model to one or more ground motion records. Each record is then scaled to multiple levels of intensity, thus producing one or more load displacement curves. With growth in computer processing power, analysis methods have progressively moved from elastic static analysis to dynamic elastic, non-linear static non-linear dynamic analysis and finally incremental dynamic analysis.

Incremental dynamic analysis, also termed dynamic pushover, is a nonlinear time history analysis method that can be utilized to estimate structural capacity (or supply) under earthquake loading. It provides a continuous picture of the system response, from elasticity to yielding and finally to collapse. Several nonlinear time history analyses are undertaken and the response from these analyses is plotted. The resulting plots, termed IDA plots, give an indication of the system performance at all levels of excitation in a manner similar to the load displacement curve from static pushover.

The choice of a suitable intensity and damage depend on the purpose of the analysis and the system considered. For example, to assess structural damage of buildings, the maximum interstorey drift $(d/h)_{max}$ (where d is the storey displacement and h is the storey height) is a reasonable choice since it is directly related to joint rotations and global collapse of the structure. Some of the advantages offered by incremental dynamic analysis are as under:

(i) Thorough understanding of the range of response or ‘demands’ versus the range of potential levels of a ground motion record.

(ii) Better understanding of the structural implications of more severe ground motion levels.

(iii) Better understanding of the changes in the nature of the structural response as the intensity of ground motion increases (e.g. changes in peak deformation patterns with height, onset of stiffness and strength degradation and their patterns and magnitudes).

(iv) Given a multi-record IDA study, understanding how stable (or variable) all these items are from one ground motion record to another.

The procedure of developing an IDA plots involves the following steps:

(i) Conduct a nonlinear time history analysis of a structure for specific acceleration record by using the lowest intensity scaling factor and get maximum roof displacement and corresponding base shear. This gives one point on IDA plot.

(ii) Increase the intensity of the scaling factor for the ground motion and repeat this iteration process until creating enough points on the IDA plot to give a complete spectrum of structural response.

(iii) Stop the analysis when the base shear value drops below the ultimate limit state.

(iv) During the analysis, monitor both limit states i.e yield and ultimate at two levels: member and system level. Determine the PGA values that correspond to yield and ultimate limit states, i.e. $PGA_{yield}$ and $PGA_{ultimate}$, respectively.

(v) The ratio between $PGA_{ultimate}$ and $PGA_{yield}$, gives R factor:

$$R = \frac{PGA_{ultimate}}{PGA_{yield}} \tag{37}$$

## 5.4 Method Employed to find R factor based on Peak Ground Parameters

### 5.4.1 Conceptual Framework

A methodology using peak ground parameters and IDA has been used in this study to compute R factor for MRFs under various ground motions. This methodology is being explained in detail in this section, so as to build a basis for its use later in the study. As discussed in section 5.1, R factor (demand) is defined as the ratio between the elastic ($Sa_{elastic}$) and the inelastic ($Sa_{inelastic}$) response spectral ordinates corresponding to a specific period T. There is some inherent inaccuracy with use of equation 28, as this procedure involves inelastic spectra. Inelastic spectra are derived by assuming elastic-plastic models which exhibit higher energy absorption and dissipation than a degrading system. Since in reality, all material and structures do

exhibit strength degradation, so use of $S_{a\,elastic}$ and $S_{a\,inelastic}$ is not very popular amongst many researchers as it is not a refined way of calculating R factor.

A much more accurate and refined method was introduced by A.J Kappos (A.J.Kappos 1992) which takes into account peak ground parameters, either in terms of acceleration, velocity or displacement depending on type of structure and its period. It takes into account both, the actual structural response and actual ground motion, giving a coupled response by considering both demand and supply. The reduction factor is thus defined as the ratio between the peak ground acceleration of the record causing ultimate limit state and the peak ground acceleration of the record causing yield limit state.

$$\text{Force Reduction Factor} = \frac{\text{PGA of the record causing ultimate limit state}}{\text{PGA of the record causing yield limit state}}$$

Velocity and displacement based peak ground parameters can also be used in place of acceleration, but will depend on the period of the structure being assessed. For long period structure, using displacement based peak ground parameter is a better approach. However for the 3 storey RC MRF being evaluated in this study (will be presented in Chapter 6), which have short fundamental periods, it is appropriate to use peak ground acceleration as the governing parameter. Short period structures are sensitive to acceleration, intermediate period structures to velocity and long period structures are sensitive to displacement.

To allow for the variation of ground motion, incremental dynamic analysis (IDA) approach is used in the study (Vamvatsikos et al 2001). One of the advantages of IDA is that it provides a continuous picture of the system response, from elasticity to yielding and finally to collapse. The response parameters and the limit states used in this study are discussed in section 5.4.2.

### 5.4.2 Response Parameters

The evaluation of the deformation quantities $\Delta u$ and $\Delta y$ from action-deformation relationships is not always straightforward. Park (1988) made an effort to define yield and ultimate deformation based on force deformation relationship in order to quantify global ductility of structural systems. These definitions for yield and ultimate deformation are presented in following sections.

#### 5.4.2.1 Yield deformations

Yield points in reinforced concrete structures are often not well defined because of nonlinearities associated with cracking of concrete and formation of plastic hinges in beams, columns and joints. Various definitions for yield deformations have been proposed as summarized below (Park, 1988):

a. Deformation corresponding to first yield. See Figure 5-3.
b. Deformation corresponding to the yield point of an equivalent elasto-plastic system with the same elastic stiffness and ultimate load as the real system. See Figure 5-4.
c. Deformation corresponding to the yield point of an equivalent elasto-plastic system with the same energy absorption as the real system. See Figure 5-5.
d. Deformation corresponding to the yield point of an equivalent elasto-plastic system with reduced stiffness computed as the secant stiffness at 75% of the ultimate lateral load of the real system. See Figure 5-6.

The use of secant stiffness accounts for the reduction of structural stiffness due to cracking at the elastic limit; the latter is the most realistic definition for yield deformation in RC structures.

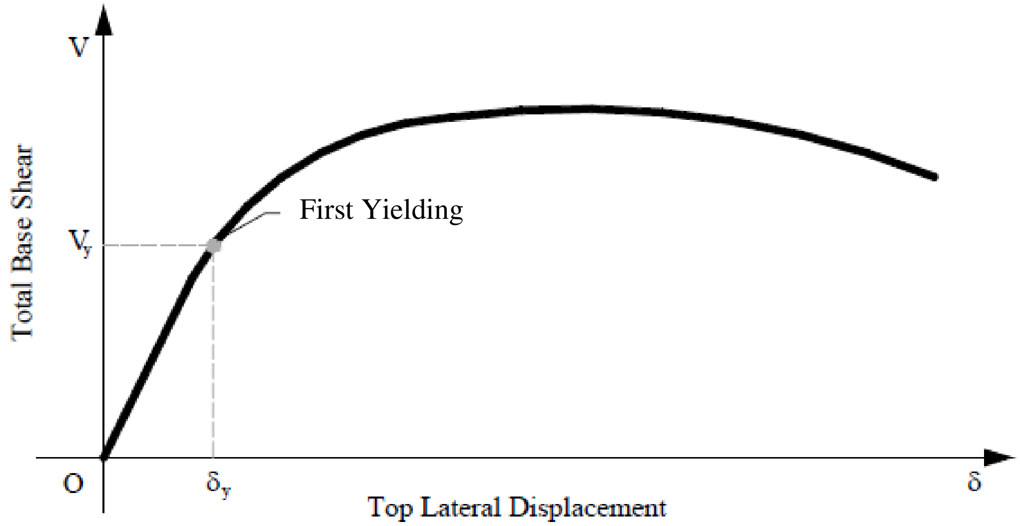


Figure 5-3: Definitions of yield deformation, corresponding to first yield (Park, 1988)

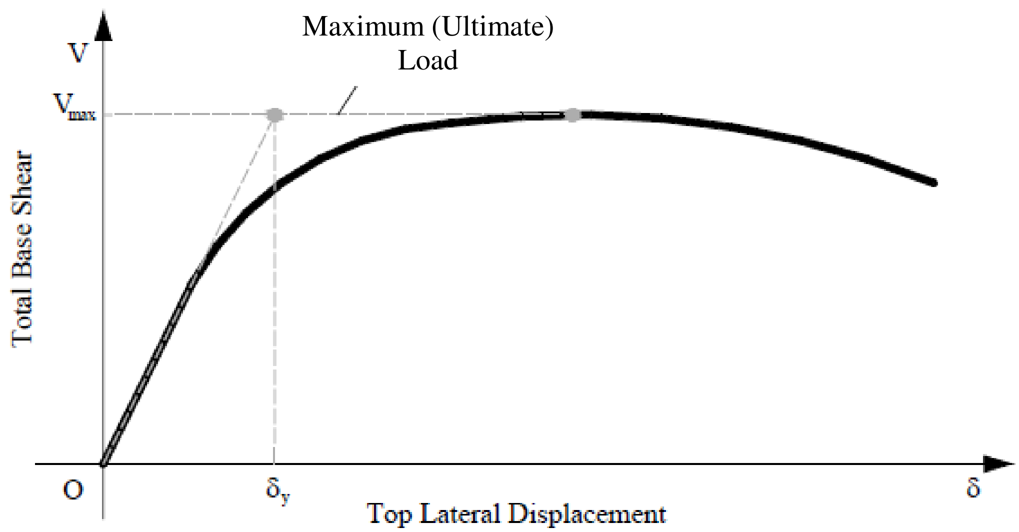



Figure 5-4: Definitions of yield deformation, based on equivalent elasto-plastic yield (Park, 1988)

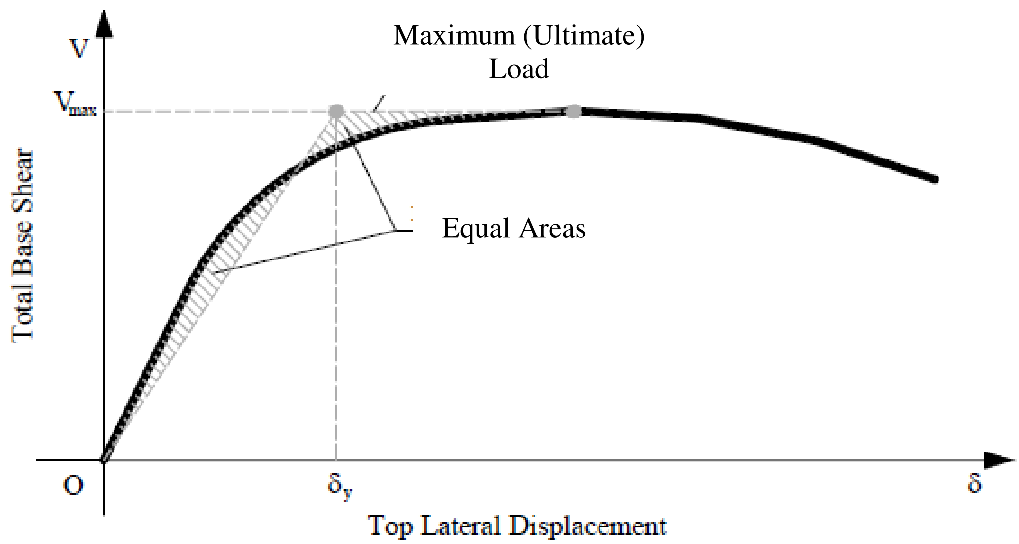



Figure 5-5: Definitions of yield deformation, based on equivalent elasto-plastic energy absorption (Park, 1988)

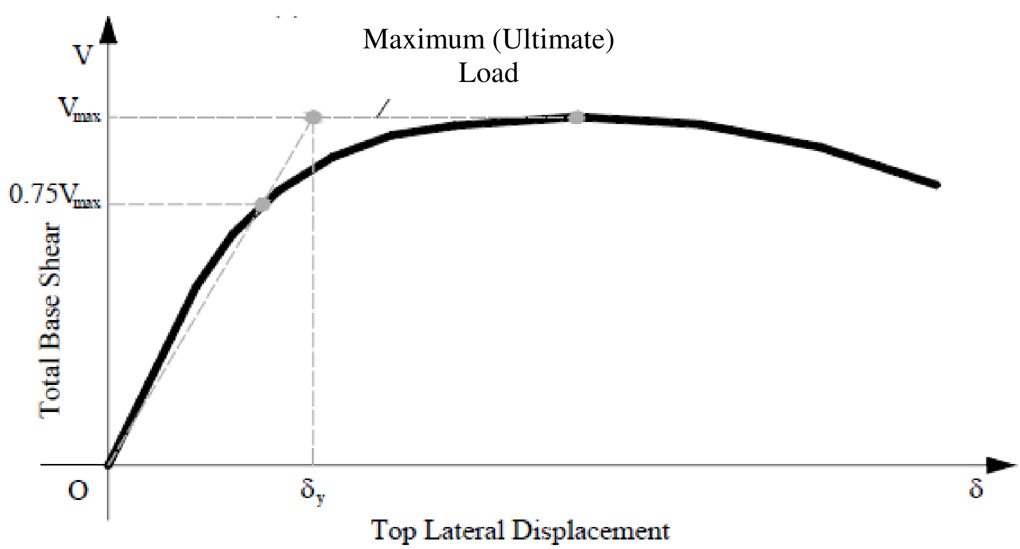



Figure 5-6: Definitions of yield deformation, based on reduced stiffness equivalent elasto-plastic yield (Park, 1988)

5.4.2.2 Ultimate deformations

Definitions for ultimate deformations are as follows (Park, 1988):

a. Deformation corresponding to a limiting value of strain. See Figure 5-7

b. Deformation corresponding to the apex of the load-displacement relationship. See Figure 5-8

c. Deformation corresponding to the post-peak displacement when the load carrying capacity has undergone a small reduction (often taken as 10%-15%). See Figure 5-9

d. Deformation corresponding to fracture or buckling. See Figure 5-10

Ductile structures usually have post-peak load strength and their load-deformation curves do not exhibit abrupt reduction in resistance, especially for MRFs. Definition of ultimate deformations given in Figures 5-7and 5-8 may underestimate the actual structural response. Hence, the most realistic definitions are those given in Figures 5-9 and 5-10, because they account for the post-peak deformation capacity.

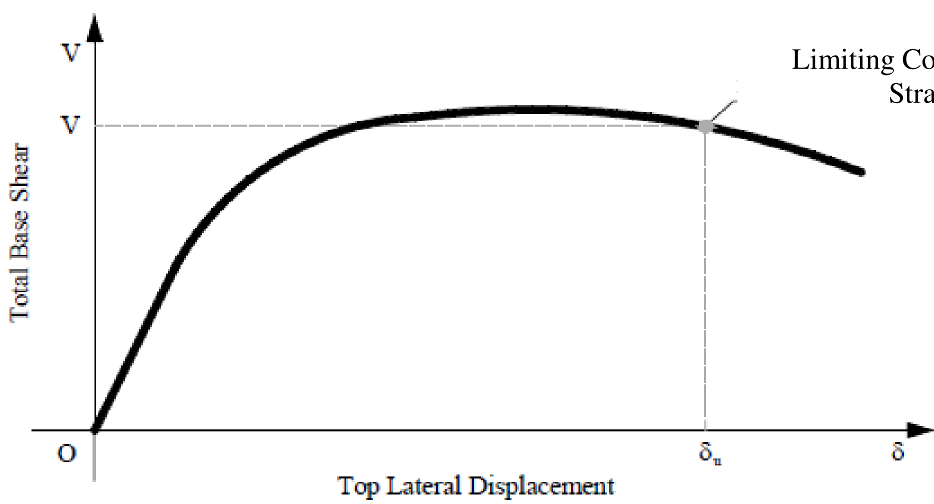



Figure 5-7: Definitions of ultimate deformation, based on limiting compressive strain (Park, 1988)

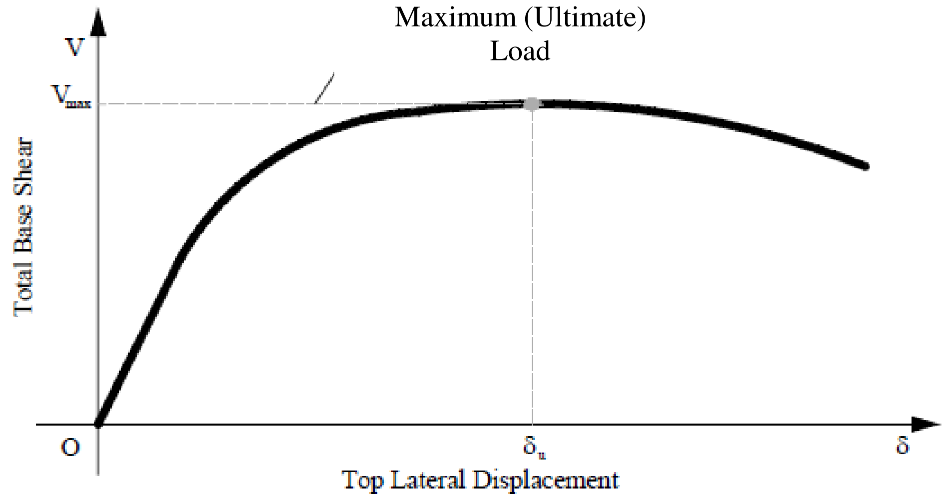



Figure 5-8: Definitions of ultimate deformation, based on peak load (Park, 1988)

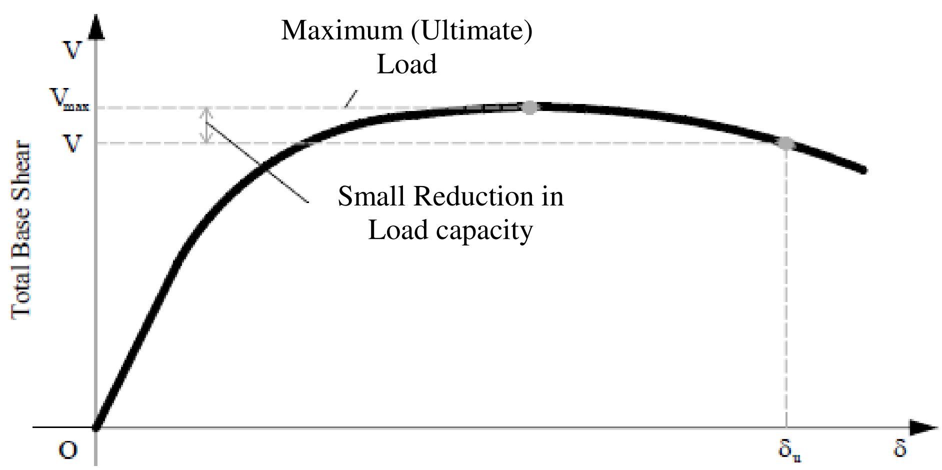



Figure 5-9: Definitions of ultimate deformation, based on significant Load capacity after peak load (Park, 1988)

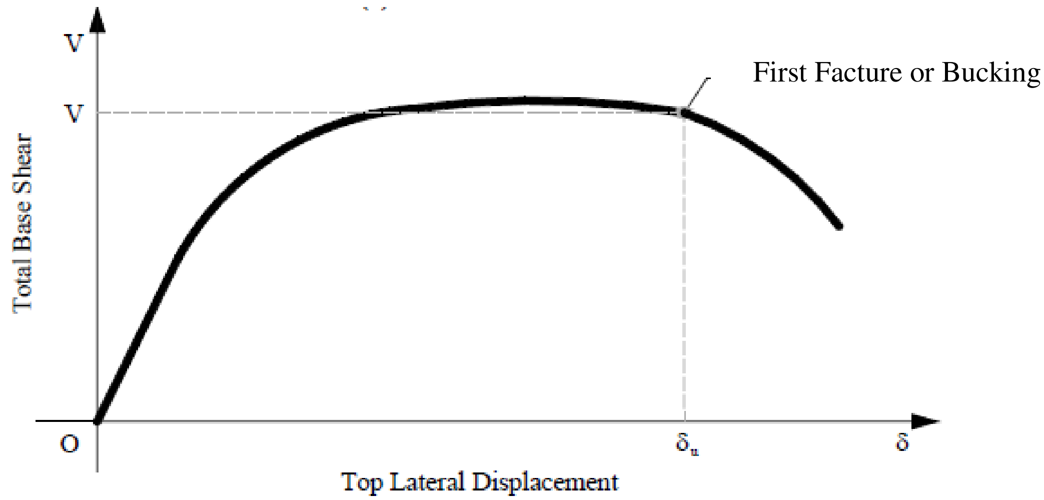



Figure 5-10: Definitions of ultimate deformation, based on fracture and/or buckling (Park, 1988)

### 5.4.3 Relationship between Limit states and IDA

The definition of yield and ultimate deformation as explained in section 5.4.2 can be employed on both pushover curve and IDA plot. Since in this study, IDA plot is being used to come up with structural response, so the definition of yield and ultimate deformation will be used on IDA plot. By doing so we can find the scaling factors (S.F) used to scale the ground motion or the PGA causing yield and ultimate limit states. For the limit state definition used in this study an elastic–perfectly-plastic idealization of the real system is employed. The initial stiffness is evaluated as the secant stiffness at 75% of the ultimate strength from incremental dynamic analysis (IDA plots). Projection of the intersection of this secant with elastic–perfectly-plastic idealization, on the IDA plot gives the yield point as illustrated in Figure 5-11. Similarly, from IDA plot, the ultimate limit state corresponding to development of plastic hinge in any 1$^{st}$ storey column in the frame was identified. In this study development of plastic hinge in RC member is defined as crushing of confined concrete after yielding of longitudinal reinforcement. IDA curves from analysis and application of limit states will further be discussed in chapter 8.

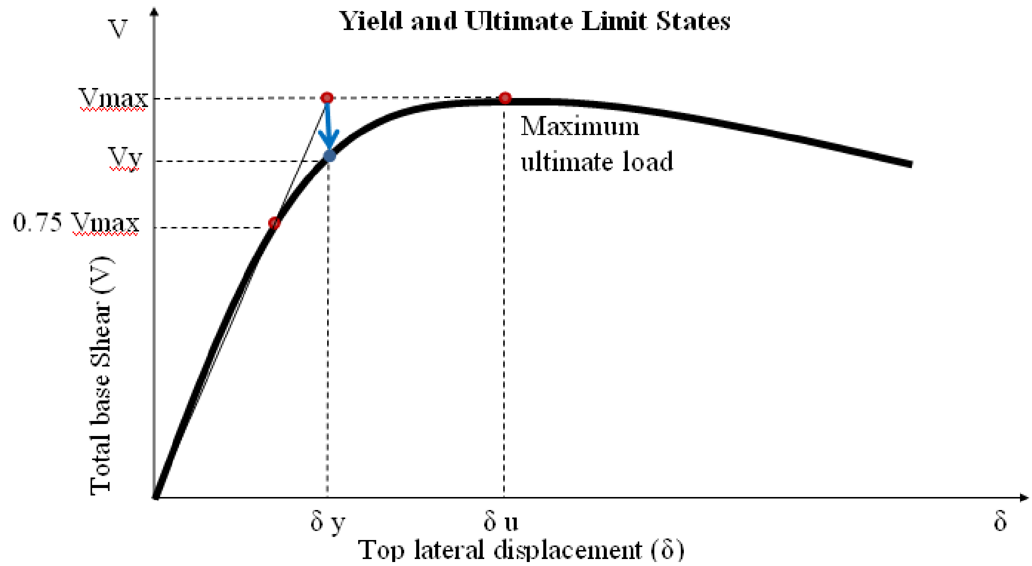


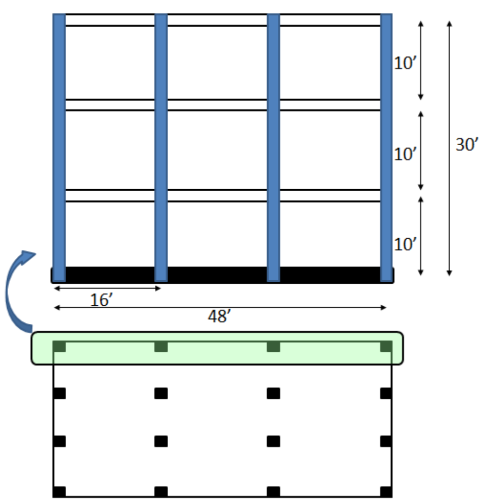

Figure 5-11: Limit states definition used in the study

**CHAPTER 6**

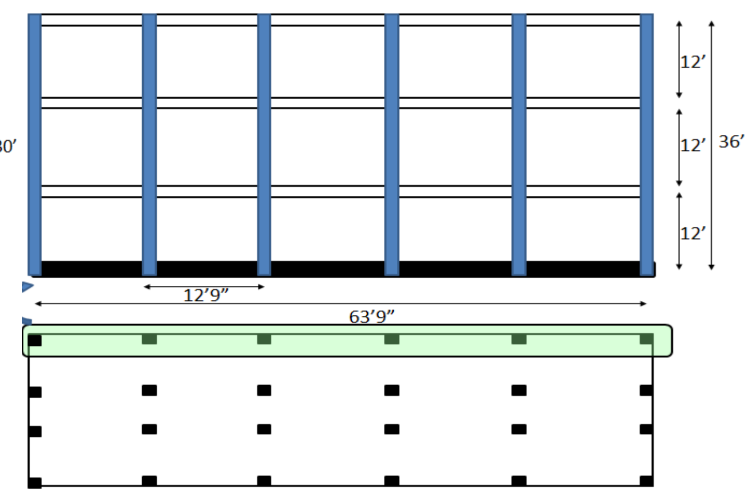
**STRUCTURAL SYSTEM MODELS AND SELECTION OF GROUND MOTIONS**

## 6.1 Structural Models

In order to represent the MRFs in Pakistan, two existing prototype RC MRFs (Three bay-three storey (3B3S) and Five bay-three storey (5B3S)) were used in this study to evaluate the effect of various ground motions on the R factor. The frame configuration of the two prototype MRFs are shown in Figure 6-1. These prototypes are existing buildings constructed in Northwestern area of Pakistan, which is in close proximity of the deadly 2005 Kashmir earthquake. Plans of the actual buildings along with cross section of the members used are shown in Figures 6-2 to Figure 6-4. The analyses results of these prototype buildings are presented in Chapter 8. Furthermore, in Chapter 9, an ordinary MRF with single bay-three storey (see Figure 6-5) was used in a parametric study to examine the effect of structural properties on the R factor of MRFs.




(a) 3B3S




(b)

Figure 6-1: (a) Plan and elevation of an administration block (b) Plan and elevation of an academic block at a University in NW Pakistan.

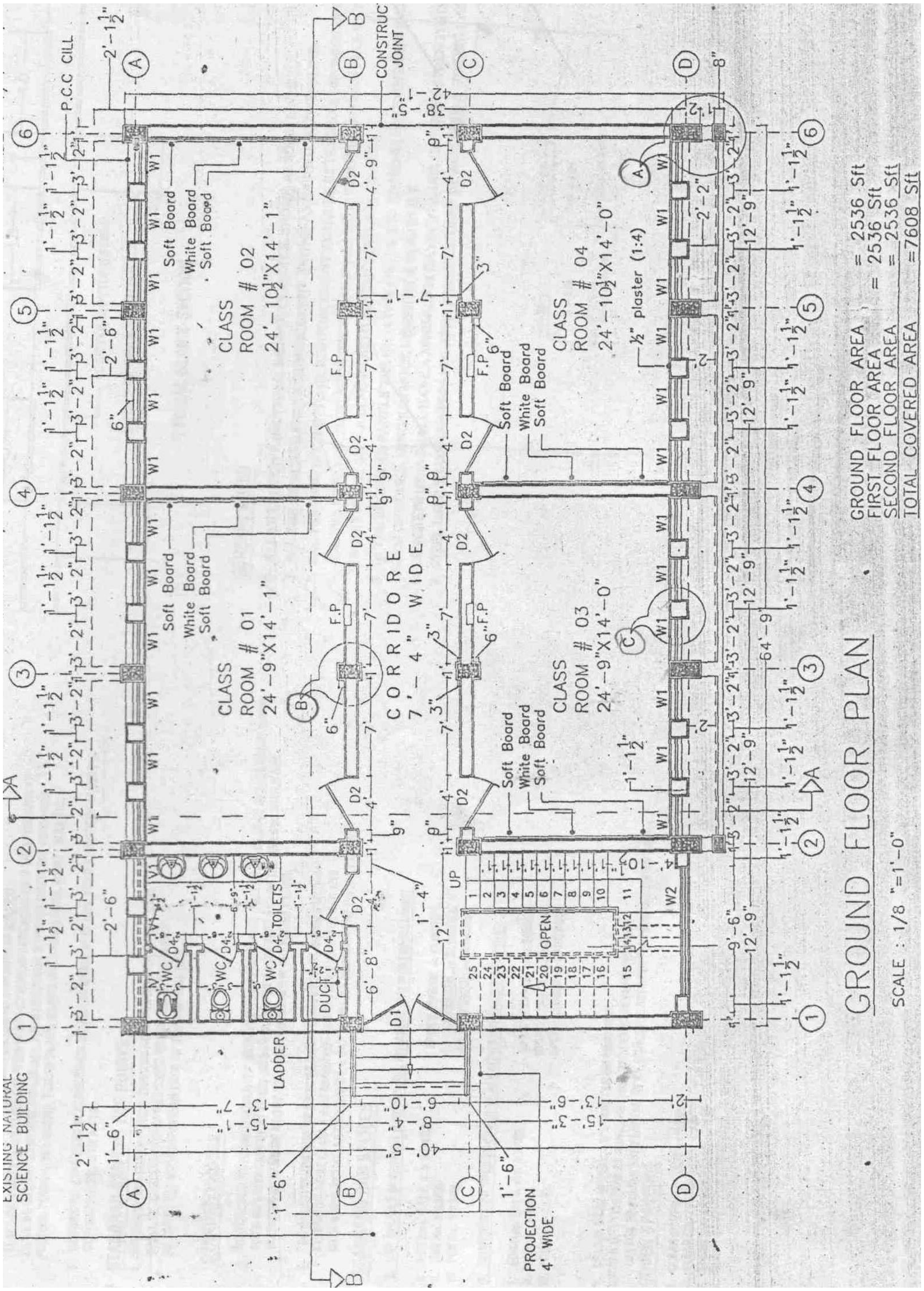



Figure 6-2: Ground floor plan of an existing prototype 5 bay-3 storey structure

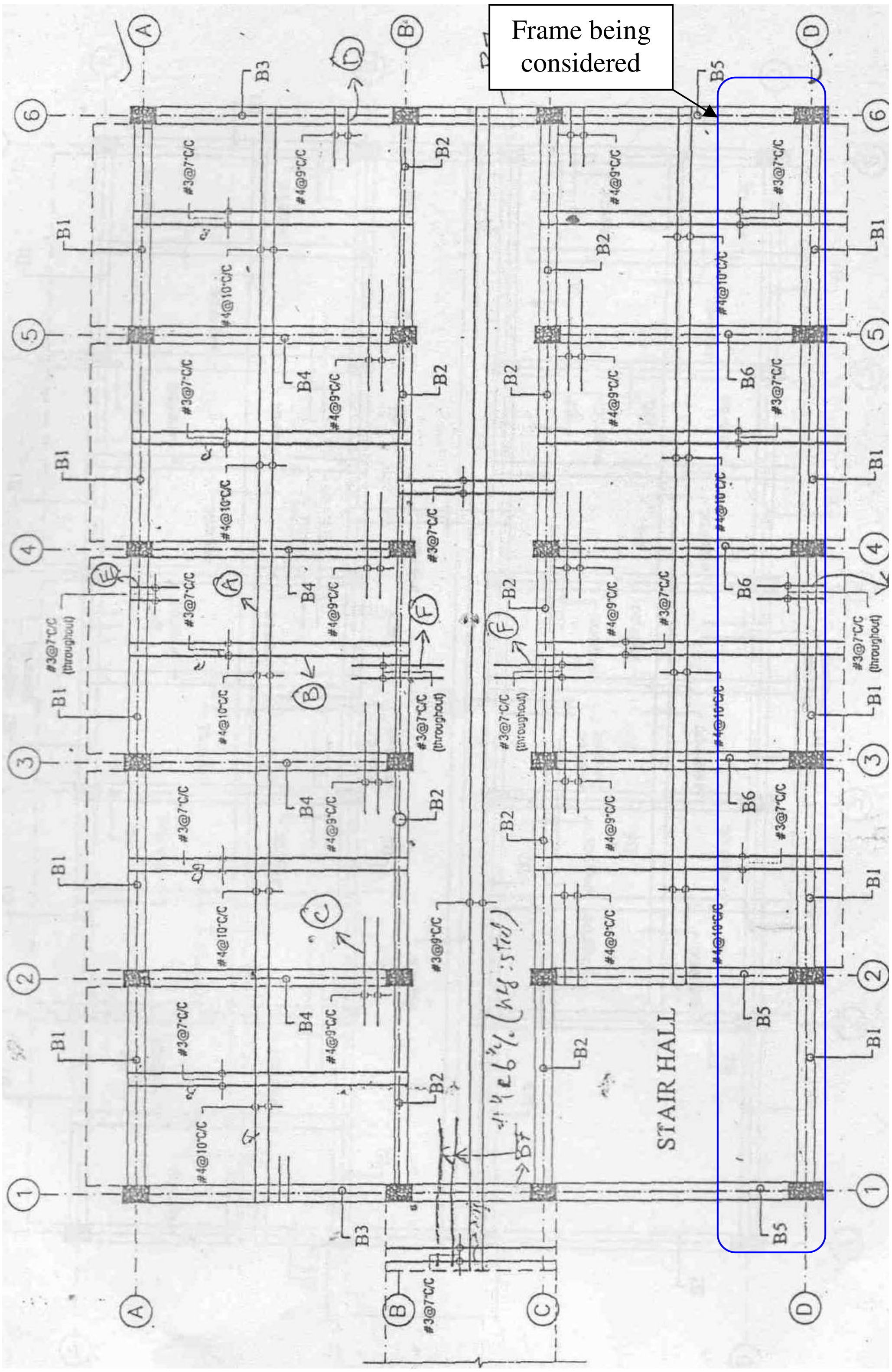
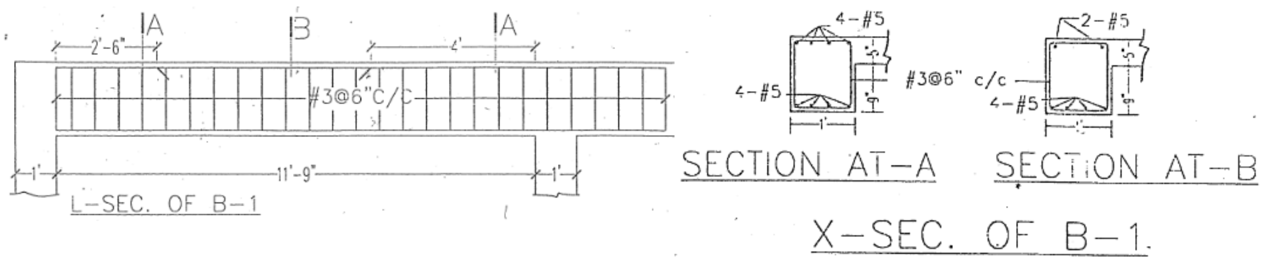
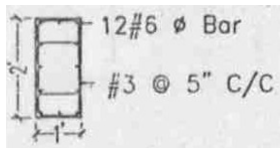



Figure 6-3: Top storey floor plan of a 5 bay-3 storey structure




Plan and cross-sec of Beam (B1)




Cross section of column (C1)

Figure 6-4: Cross section of beam (B1) and column(C1) for both 3 bay-3 storey and 5 bay-3 storey structures

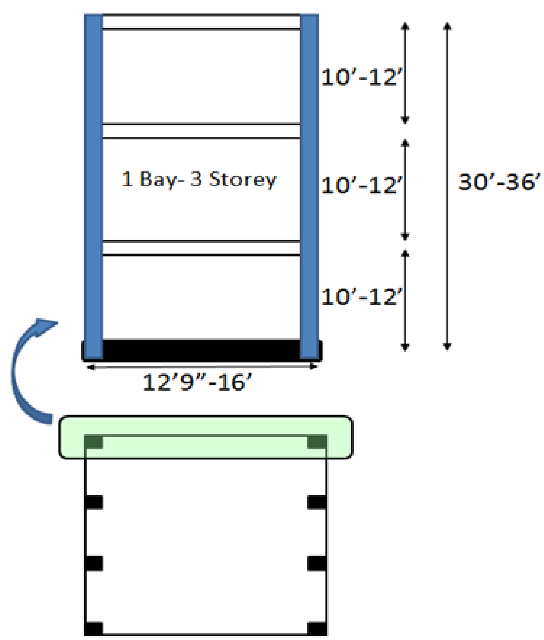



Figure 6-5: Building model used for the parametric study

## 6.2 Material Properties

### 6.2.1 Concrete

The mix design of the concrete used typically in Pakistan is aimed at design cylinder strength of 3000 psi - 5000 psi. A typical mix design for this strength has a slump range of 1 to 2 inch; the maximum size of coarse aggregate is ¾ inch. The mix proportions which provide this strength are 1:2.5:3.5 as cement, sand and coarse aggregate.

### 6.2.2 Steel

The steel reinforcement used in reinforced concrete structures is usually grade 40 and 60 (yield strength of 40 ksi and 60 ksi). Modulus of elasticity is approximately 29000 ksi. The typical sizes used in reinforced concrete works are: Beams and Columns: #4, #6 and #8 bars are used in longitudinal steel while #3 bars are used as lateral reinforcement. In slabs, #3 and #4 bars are used.

## 6.3 Selection of Ground Motions

Pakistan exists in a high seismicity area and has been prone to many deadly and damaging earthquakes in the past. 1935 Quetta and 2005 Kashmir earthquakes are among the worst natural disasters ever to hit the region. Despite the high seismic hazard in Pakistan, it is surprising that, not enough ground motions from previous earthquakes are available. To best represent the earthquake hazard for Pakistan, 2005 Kashmir earthquake of magnitude 7.6 on Richter scale was selected for analysis from which records from Abbotabad and Nilore stations were available. Abbotabad record had a PGA of 0.236g and was recorded for 154 seconds, while Nilore record had a PGA of 0.024g and was recorded for 86 seconds.

Because of the unavailability of other ground motion records from Pakistan, records from the region, which best represent the attenuation relationship in Pakistan were selected. The aim was to develop a suite of ground motions, which could best represent seismic hazard in the area and at the same time capture different seismic characteristics, like spectral acceleration, PGA, duration, epicentral distance, magnitude and predominant period. Earthquake records from neighboring countries of Pakistan, namely India and Iran were selected for this purpose. Two major earthquakes which had hit the region, namely Tabas 1976 (Iran) and Bhuj 2001 (India) were chosen to represent the seismic hazard in the region. Tabas 1976 earthquake struck Iran and was one of the worst earthquakes to hit Iran and was measured at 7.8 on Richter scale. This earthquake is also in close proximity to the Pakistan's SW province (Balochistan). Three ground motion records from Tabas, Boshrooyeh and Dayhook ground stations were available for Tabas 1978 earthquake. These records had a PGA of 0.83g, 0.81g and 0.32g, respectively. Bhuj 2001 earthquake struck in India which is in close proximity to Pakistan's southern regions including Pakistan's biggest city of Karachi. Again this earthquake was also a very destructive one with magnitude of 7.7, as reported by USGS. Two ground motion records from separate stations, namely IITR Ahmedabad and Katrol hills were available for Bhuj 2001 earthquake. These records had a PGA of 0.98g and 0.78g, respectively. So in totality, 7 different natural earthquakes records were used in the study. Table 6-1 shows a summary of the characteristics of the selected ground motion records.

| Earthquake | Year | Magnitude (Mw) | Recording Station | Record Name | Hypocentral Distance (km) | PGA(g) |
|---|---|---|---|---|---|---|
| Kahmir-Pakistan | 2005 | 7.6 | Abbotabad | Abbotabad | 39 | 0.236 |
| | | | Nilore | Nilore | 92 | 0.024 |
| Tabas - Iran | 1978 | 7.4 | 9101 Tabas | Tabas-1 | 55.54 | 0.83 |
| | | | 70 Boshrooyeh | Tabas-2 | 74.88 | 0.81 |
| | | | 9102 Dayhook | Tabas-Dayhook | 21.41 | 0.32 |
| Bhuj-India | 2001 | 7.6 | IITR Ahmedabad | Bhuj-1 | 198.3 | 0.98 |
| | | | Bhuj | Bhuj-2 | 239 | 0.78 |

Table 6-1: Selected ground motions

The acceleration time history and the elastic acceleration spectra of the selected earthquake records are also shown in Figures 6-6 and 6-7, respectively.

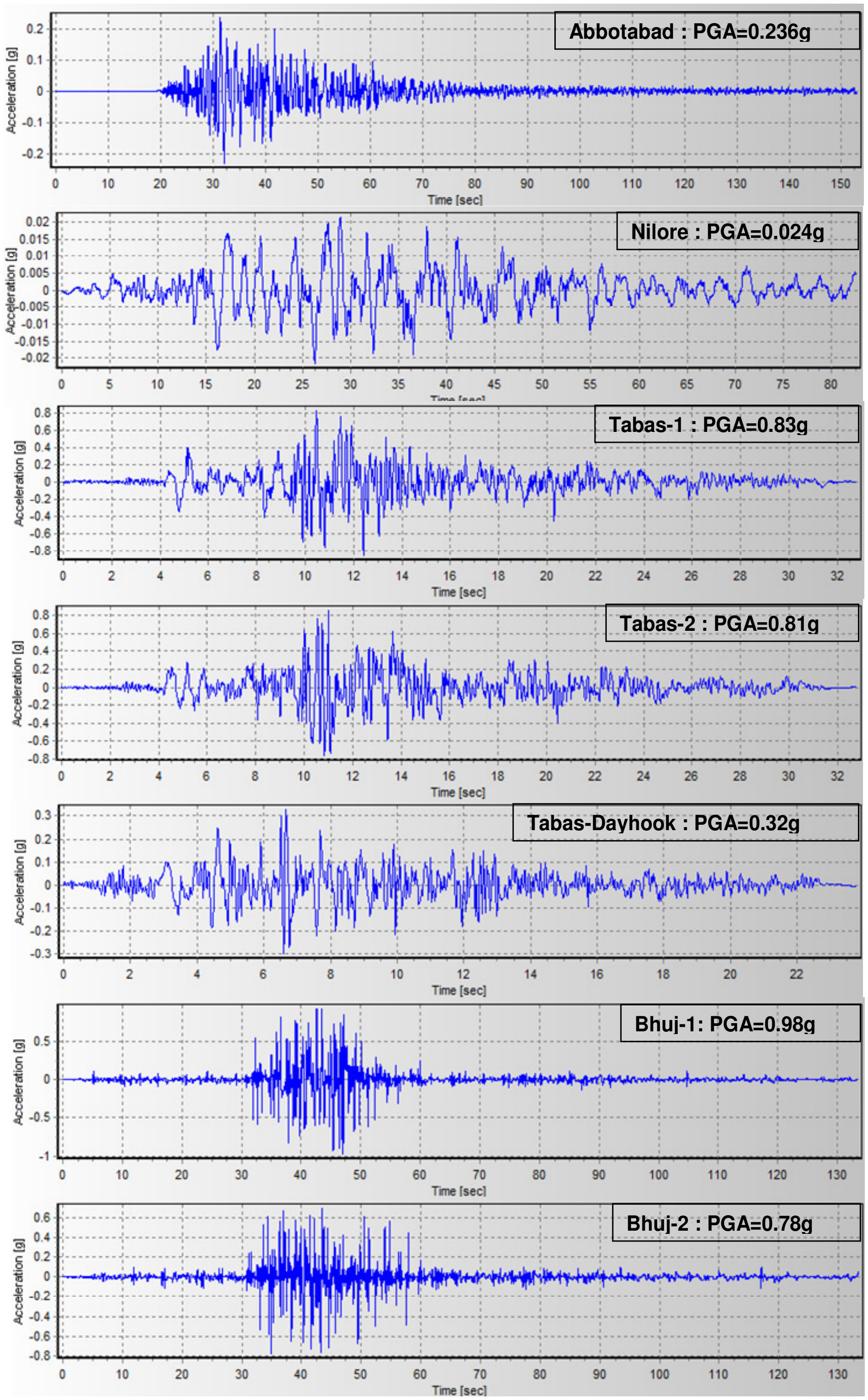



Figure 6-6: Strong motions from Kashmir 2005 (Pakistan), Tabas-1978 (Iran) and Bhuj 2001 (India) earthquake.

It is worth noting that the RC MRFs considered in this study had a period in the range of 0.25 to 0.4 seconds, which are considered to be relatively short to intermediate period structures. As mentioned earlier, the ground motion records selected for the analysis had different seismic characteristics. For example Abbotabad and Nilore ground motion records had lower PGA and maximum spectral acceleration values, but both were for a relatively longer duration. Similarly, both records from Bhuj earthquake displayed long duration of shaking, (134 seconds) but had higher PGA and maximum spectral acceleration values. Acceleration response spectra in figure 6-7 show that both Tabas-1 and Tabas-2 records had relatively shorter duration, but had higher values of PGA. Except for Bhuj-1 and Bhuj-2 records, all remaining five records were recorded at stations with hypocentral distance less than 100 km. Except for the Abbotabad and Nilore earthquakes, all other earthquakes showed high intensity level of spectral acceleration within the fundamental period range of the structures being studied. Hence, all the records selected had high intensity of spectral acceleration for a different range of period, thus effecting structures with different natural fundamental periods. This may prove to be crucial based on how much the period of the structure increases as damage starts to accumulate.

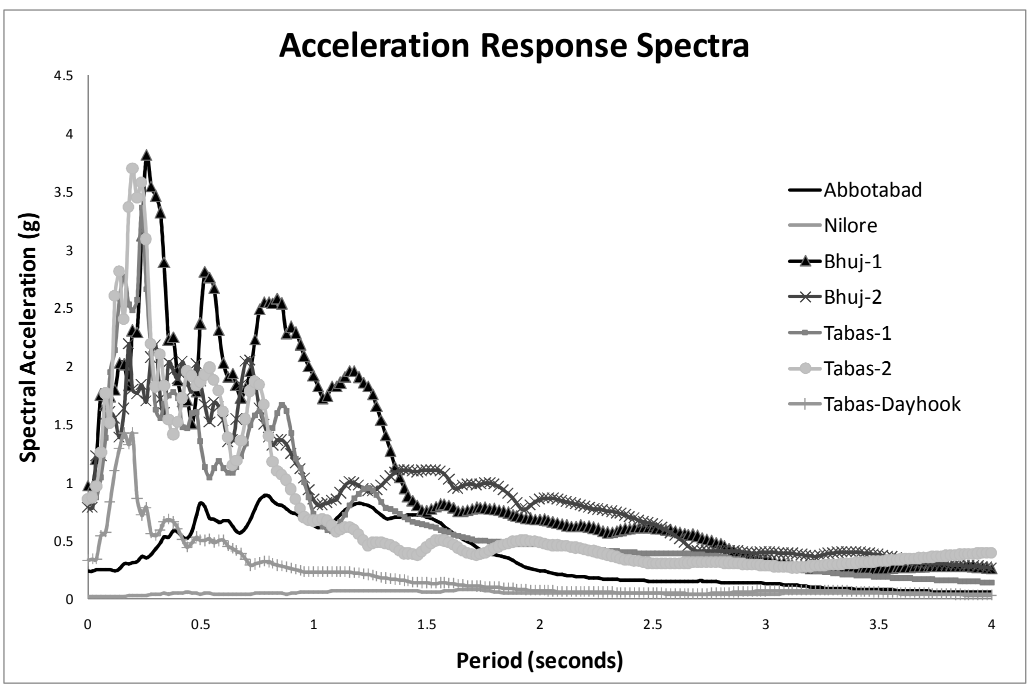



Figure 6-7: Acceleration response spectra of the earthquake records used in the study

# CHAPTER 7

# ANALYTICAL MODELING USING OPENSEES

## 7.1 Adopted Modeling Approach

Finite element program, OpenSees (Mazzoni et al. 2009), which has been specifically designed for seismic analysis and earthquake simulations, was utilized to perform the inelastic analyses in this study. Since the software is an open source, so many researchers use it and upgrade it based on their experience and work. The software has a very rich material and element library suitable for structural analysis under seismic loadings. Because of its ease of availability and extensive cataloging for tackling almost any kind of structure and material in either 2D or 3D, it has attracted many researchers to use it for seismic analysis. OpenSees program runs the analytical model for specific force or target displacement by incrementing time and monitors the user defined parameters for the duration of the analysis. The software has the capability to describe a structural component at element, section and fiber level. This analysis tool is widely being used by researchers, mainly because of its ability to accommodate user defined parameters and models. The user has the ability to define new constitutive material models or modify existing ones to match the experimental observations. Parameters defining these models can be adjusted, giving flexibility to incorporate wide range of geometry, material data, element type and different analyses procedures.

Nonlinear beam-column elements with fiber sections were used to model the frame elements with distributed plasticity. Uniaxial material models which have been predefined in OpenSees have been utilized in defining the constitutive behavior of concrete and steel materials. Concrete02 model, which considers concrete's tensile strength, was selected to represent the concrete behavior. The steel behavior was described using Steel02 model, which is based on the Giuffre-Menegotto-Pinto model

with isotropic strain hardening (Menegotto and Pinto 1973). The effect of concrete confinement was considered by using the model developed by Mander et al. (1988).

Figure 7-1 shows the geometric modeling of the 5 bay-3 storey frame used in the study. The frame columns were fixed at the base, and same cross section of 12”x24” has been used throughout the entire column length. However, the beam elements had 2 different cross sections, namely section AA and BB. Both of which are 12”x14” but have different reinforcement ratio, as shown in figure 7-1. Section AA has more top reinforcement to resist the negative moments developing near the beam-column joint. All the beams in the structure were divided into three equal lengths and the mass of the floor between columns was lumped at the beam joints and beam-column joints. The beam column intersection was modeled as rigid link elements to incorporate the effect of high stiffness within the joint. Also by using rigid links, the deformation and inelasticity in the joint was ignored.

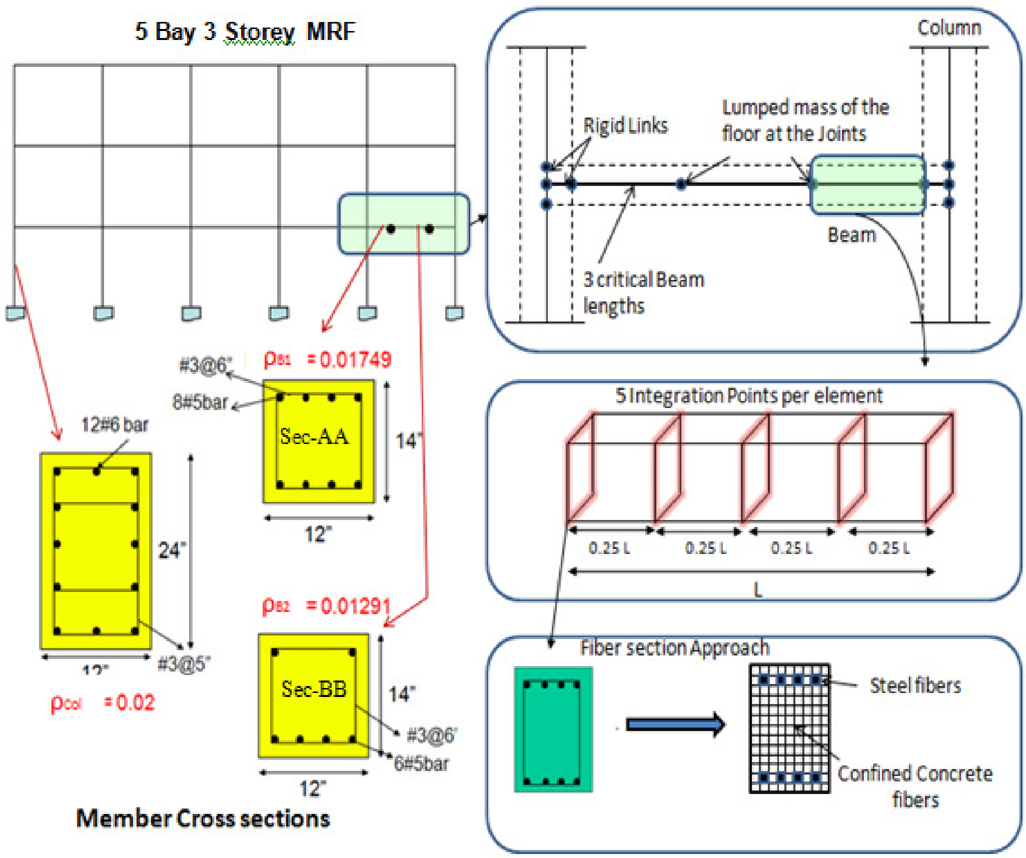



Figure 7-1: Adopted modeling approach in OpenSees

## 7.2 Element and Section Formulation

Both beams and columns were modeled using ‘forced-based nonlinear beam-column element’ defined in OpenSees library. This element is based on iterative force formulation, and considers the spread of plasticity along the element length. The integration along the element is based on Gauss-Lobatto quadrature rule (two integration points at the element ends). In OpenSees, 2D beam-column elements have three degrees of freedom (2 translation and 1 rotation) at each end node. In this study, fiber section approach has been used to model the sections. Cross-sections are modeled by defining geometric and material properties. Fibers form the basis of distributed plasticity models. Fiber based approach is the most reliable formulations to predict the earthquake response of structural systems because of its ability to localize the spread of plasticity to specific portion of the section and not the whole section.

For this study 5 integration points were used for each beam and column. The cross section at each integration point is then further discretized into fibers, which allows the section to be further divided into small areas with different constitutive models. In fiber-based formulations, the area A of the section is divided into finite regions (or fibers), e.g. a rectangular grid of lines parallel to cross-sectional principal axes. Each fiber is characterized by two geometric quantities: its location in the local reference system of the section and the fiber area dA. Typical subdivision in fibers for RC sections is shown in Figure 7-2. Localizing nonlinearities of concrete due to cracking, crushing and post-peak softening in a cross section can be easily accommodated by fiber models. Fiber section also allows the use of different constitutive models for different parts of the section such as unconfined concrete, confined concrete and steel reinforcement, as shown in figure 7-2. The number of fibers is dependent on the type of section, the target of the analysis and the degree of accuracy sought. More refinement or subdivisions will increase exponentially the computations required in the analysis.

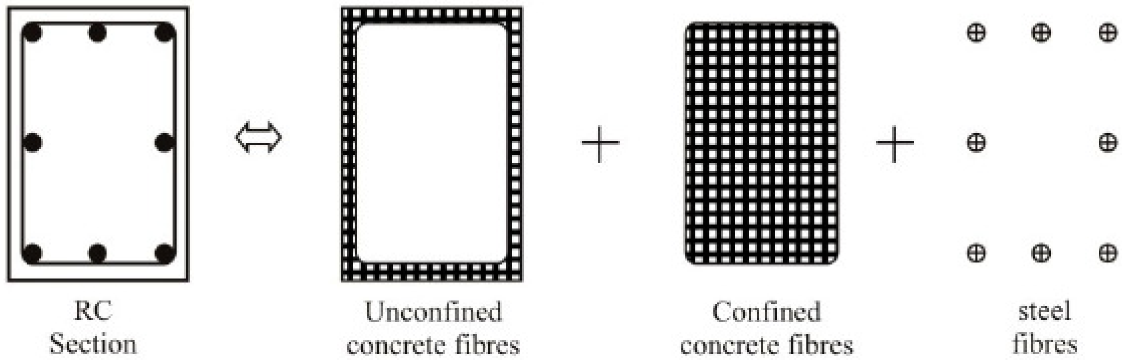



Figure 7-2: Fiber modeling of RC section

## 7.3 Defining Constitutive Material Models

### 7.3.1 Concrete

Uniaxial material models which are predefined in OpenSees have been utilized to describe the concrete behavior. For this study, Concrete02 model was selected for both unconfined and confined concrete, which considers linear tension softening of concrete. The input parameters needed to define Concrete02 model in OpenSees (see Figure 7-3) are compressive strength (fpc), strain at compressive strength (epsc0), ultimate (crushing) strength (fpcU), strain at ultimate strength (epsU), tensile strength (ft), tension softening stiffness (Ets), and ratio between unloading slope at epscU and the initial slope (Eo). The initial slope (Eo) in this model is automatically computed which is equal to 2*fpc/epsc0.

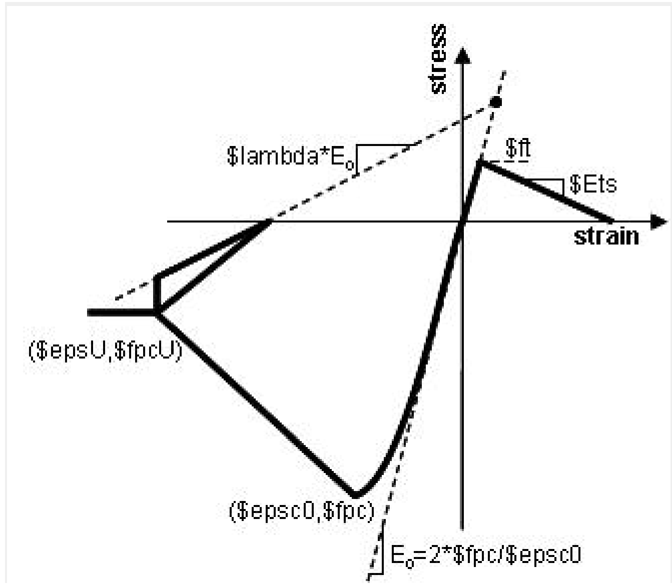



Figure 7-3: Concrete02 model parameters (OpenSees)

The parameters required to define the concrete model in OpenSees were computed based on study by Mander et al (1988), which is capable of predicting the effect of confinement in circular or rectangular columns due to steel transverse reinforcement. The equations derived in the study using the energy balance approach were used to come up with the parameters required to define the confined concrete constitutive relationship at the onset of fracture of transverse reinforcement. The lateral confining pressure is evaluated assuming that the transverse reinforcement has yielded. To account for the fact that the entire core area ($A_c$) is not effectively confined, the lateral confining pressure is reduced as described in Equations 39 and 40.

$$f'_l = k_e \times f_l \qquad (39)$$

$$k_e = A_e/A_c \qquad (40)$$

where $f'_l$ is the effective lateral confining pressure, $k_e$ is the confinement effectiveness ratio and $A_e$ is the effectively confined concrete area. The approach defines the effective core regions as a function of the configuration and spacing of the longitudinal and transverse reinforcement. It is assumed that the unconfined region extends inwards of

the centerline of the transverse and longitudinal reinforcement in the form of a second degree parabola with an initial tangent slope of 45° (see Figure 7-4).

For the unconfined concrete model, which was used to describe the behavior of cover concrete, strain at maximum strength of unconfined concrete ($\varepsilon'_c$) was assumed to be 0.002 and strain at ultimate stress was assumed to be $2\varepsilon'_c$. Residual stress of 20% of compressive strength ($f'_{co}$) was assumed for value of stress corresponding to the peak ultimate strain.

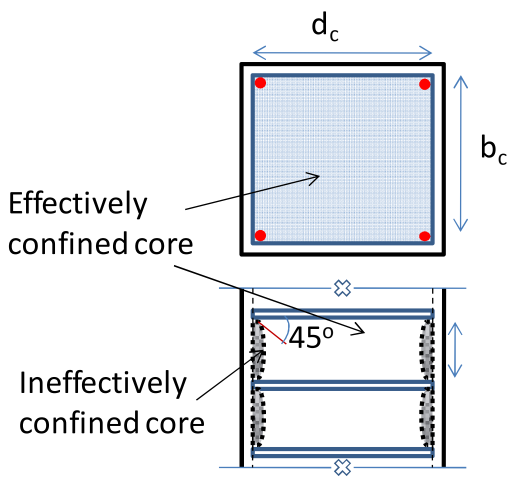


Figure 7-4: Effectively confined core for rectangular hoop reinforcement.

The stress strain behavior shown in figure 7-5 is true for both circular and rectangular sections and is based on Equation 41, which was initially suggested by Popovics (1973).

$$\frac{f_c}{f'_{cc}} = \frac{\varepsilon_c / \varepsilon'_{cc} \times n}{n - 1 + (\varepsilon_c / \varepsilon'_{cc})^n} \tag{41}$$

where, $$n = \frac{E_c}{(E_c - f'_{cc}/\varepsilon'_{cc})} \tag{42}$$

and $f'_{cc}$ is the compressive strength and $\varepsilon'_{cc}$ is the corresponding strain of the confined concrete. $E_c$ is the modulus of elasticity of concrete. $f_c$ is the stress and $\varepsilon_c$ is the strain in concrete at any point on the stress-strain curve.

The strain at this peak stress is predicted by the relationship suggested in equation 43, where $f'_{co}$ is the unconfined concrete compressive strength:

$$\varepsilon'_{cc} = \varepsilon'_{c} \left( 5 \times f'_{cc} / f'_{co} - 4 \right) \tag{43}$$

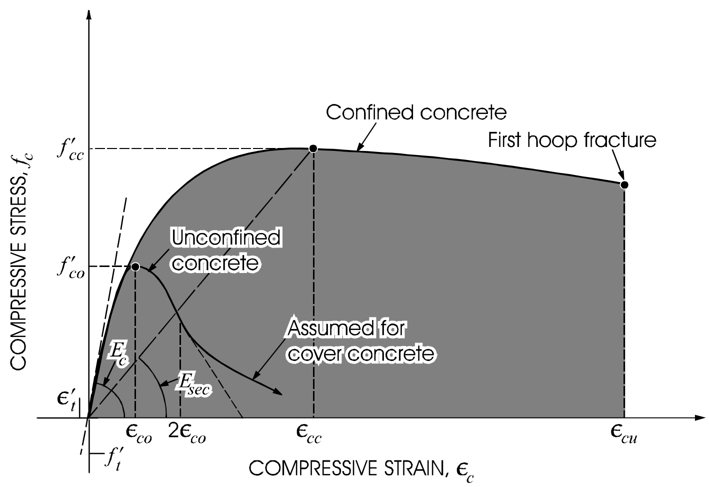



Figure 7-5: Schematic of stress-strain behavior of concrete (Mander 1988)

Lateral confining pressure on both sides of a rectangular section are used to calculate confined strength ratio based on a chart defined in the study by Mander et al. (1988) which requires largest confining stress ratio and smallest confining stress ratio (Figure 7-6).

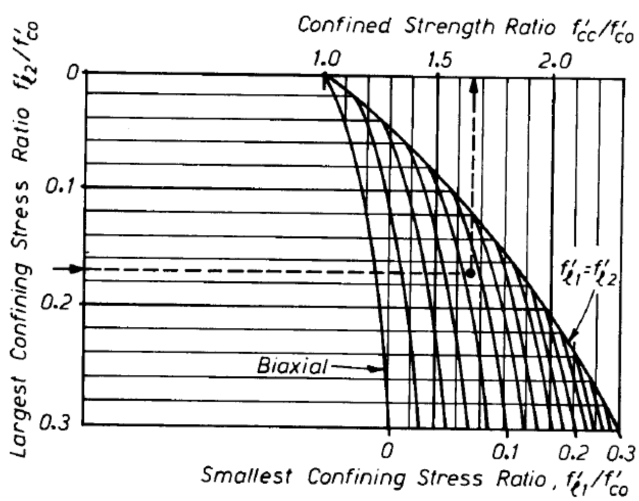



Figure 7-6: Confined strength determination for rectangular columns (Mander et al, 1988)

Using the above defined procedure, the confinement factor ($K_{fc}$), which is the ratio of confined to unconfined concrete compressive strength, also known as confined strength ratio, was calculated for the column and the beam cross sections. Because of the change in longitudinal reinforcement along the length of the beam, two different confinement factors were evaluated for the beam at sections AA and BB.

After calculation of confinement factor ($K_{fc}$), the compressive strength of confined concrete ($f'_{cc}$) can be computed using equation 44. Strain corresponding to peak stress ($\varepsilon'_{cc}$) can be computed using equation 43.

$$f'_{cc} = K_{fc} \times f'_{co} \tag{44}$$

In order to compute ultimate strain as described in Figure 7-5, Mander et al (1988) assumed failure to be reached at first hoop fracture. This event is evaluated by equating the energy at rupture in the hoop reinforcement and energy required to maintain yield in longitudinal steel to the compressive strain energy stored in the compressed concrete. This is mathematically expressed in equation 45, which can be used to compute strain at ultimate point.

$$110\rho_s = \int_0^{\varepsilon_{cu}} f_c d\varepsilon_c + \int_0^{\varepsilon_{cu}} f_{sl} d\varepsilon_c - 0.017\sqrt{f'_{co}} \tag{45}$$

where, $\rho_s$ is the ratio of volume of transverse reinforcement to volume of concrete core , $f_c$ can be computed from equation 41, and $f_{sl}$ which is stress in longitudinal bar can be plugged in as a function of longitudinal strain.

Using equation 41, constitutive relationship was obtained for column and beam cross-sections, as shown in Figure 7-7

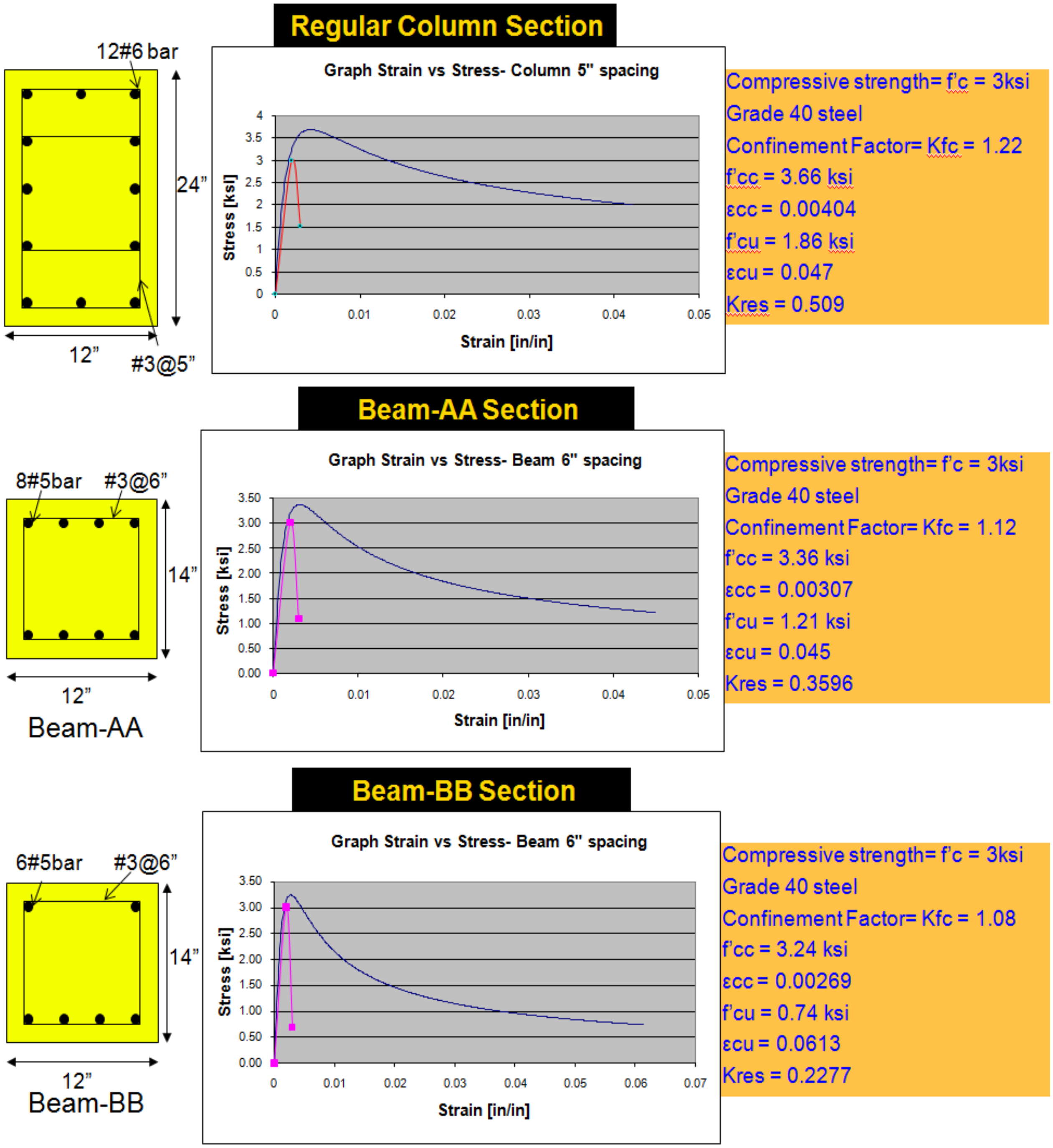



Figure 7-7: Constitutive relationship for column and beam reinforced concrete cross sections using Mander et al (1988) model.

7.3.2 Steel

Steel02 material model in OpenSees was used for modeling the steel reinforcement in this study. Steel02 follows Giuffre-Menegotto-Pinto model (Menegotto and Pinto 1973). The input parameters needed to define Steel02 model in OpenSees are yield strength (Fy), initial elastic tangent (E), strain hardening ratio (b) which is defined as ratio between post-yield tangent and initial elastic tangent and three constants (R0, cR1, cR2) in order to control transition from elastic to plastic branch. Recommended values for these constants are 18 for R0, 0.925 for cR1 and 0.15 for cR2. Details are shown in Figure 7-8 below:

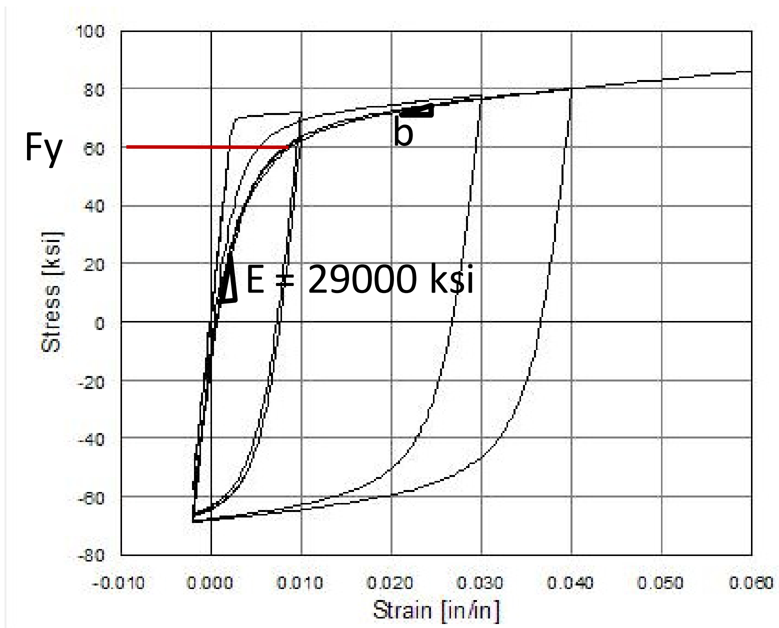



Figure 7-8: Stress-strain behavior of Steel02 model in OpenSees

## 7.4 Application of Gravity Loads

The majority of the moment resisting frames in Pakistan are designed only for dead and live loads as part of gravity load take down, according to ASCE-07 load specifications. In the prototype models, it was assumed that the density of concrete is 150 pcf and the thickness of slab is 5 in. Self weight of beam, columns, and slab was calculated based on size of the sections used. Superimposed dead load of 10 psf, live

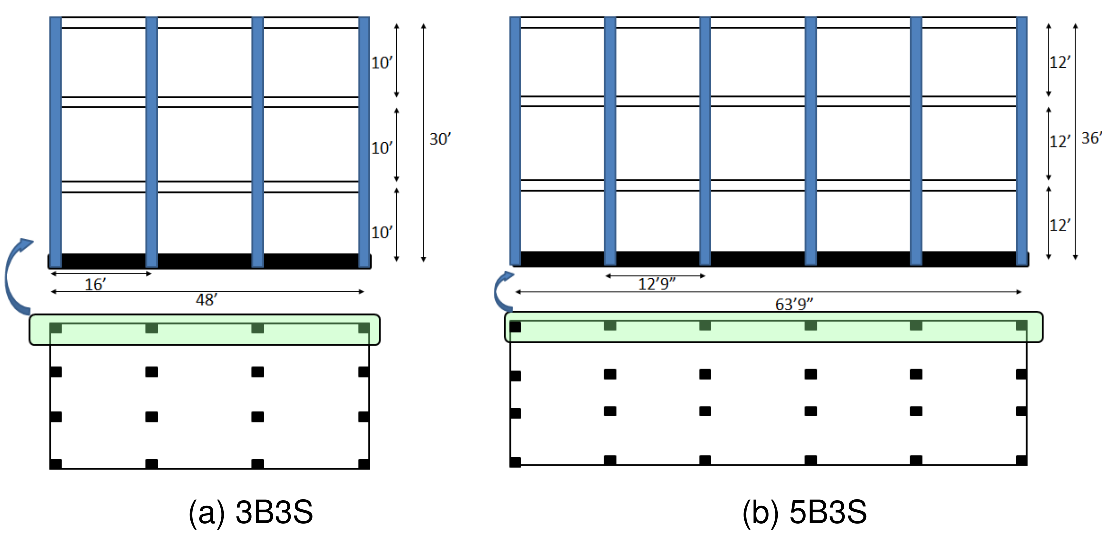
load of 55 psf and cladding of 10 psf was considered in load calculations according to ASCE-07. Based on tributary area, these loads were converted in concentrated loads and applied at the location where mass in lumped for each floor, i.e beam column joints and along the length of the beam. Figure 7-1 shows the location of joints where gravity loads are applied.

# CHAPTER 8

# EVALUATION OF R FACTORS OF PROTOTYPE BUILDINGS

## 8.1 Prototype RC MRF and their Analysis

Two prototype RC MRFs, namely a 3 bay-3 storey (3B3S) model of an administration building and a 5 bay-3 storey (5B3S) model of an academic building were selected for the analyses (see Figure 8-1). Both of these buildings physically exist on ground in Northwestern (NW) region of Pakistan, where the deadly 2005 Kashmir earthquake struck. Important dimensional parameters, like height of storey and length of bays were different for both prototype buildings, giving a range of most common type of RC buildings existent in Pakistan. Steel yield strength was reported to be 60 ksi, while concrete compressive strength of 3.5 ksi was considered in this study, which is the general range of strength of concrete used in the building types being considered. The storey height and bay length for 3B3S building was10’ and 16’, respectively, while for 5B3S the storey height and bay length were 12’ and 12’-9”, respectively.




(a) 3B3S (b) 5B3S

Figure 8-1. Plan and elevation of: (a) administration building (b) academic building at a University in NW Pakistan.

The member cross sections along with longitudinal and lateral reinforcement for columns and beams were shown in Figure 7-1. Building configuration of the two prototype buildings along with dimensions and material properties are also shown in Table 8-1.

| Building reference | Steel Grade | Concrete Strength | Height of Storey | Length of bay |
|---|---|---|---|---|
| 3 Bay-3 Storey configuration (3B3S) | 60 | 3.5 ksi | 10’ | 16’ |
| 5 Bay-3 Storey configuration (5B3S) | 60 | 3.5 ksi | 12’ | 12’9" |

Table 8-1: Material and building parameters for the two prototype MRFs

Modal analyses were conducted first to determine the uncracked/elastic fundamental periods of vibration. This simple analysis is also useful as an initial validation tool of the analytical models. Results from the modal analysis of the 5 Bay-3 Storey building are shown in Figure 8-2.

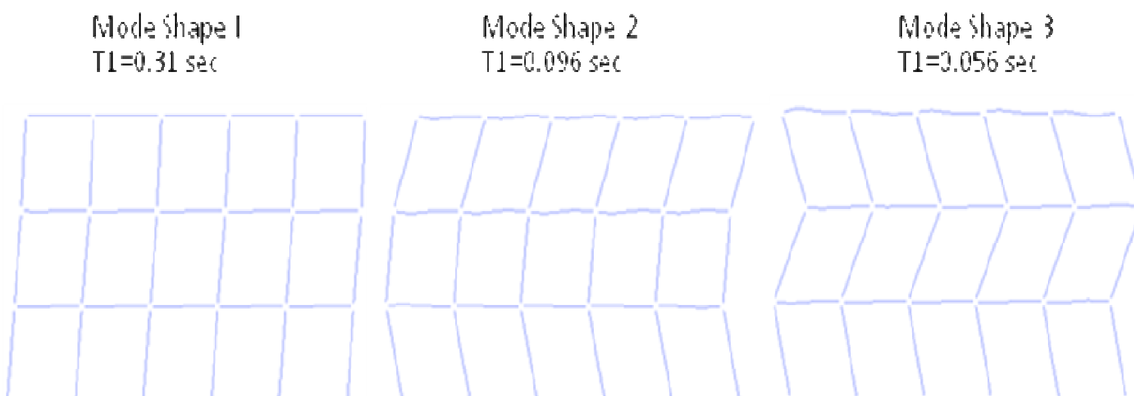



Figure 8-2: Results from modal analysis of a 5 bay-3 storey configuration

Inelastic static pushover analysis (using 5% target drift ratio) was also performed for the buildings by using an inverted triangular load pattern. This analysis procedure is employed to evaluate the structural capacity and compare it with that from IDA curve. Finally, extensive nonlinear time-history analysis was performed using the previously discussed IDA technique, under the selected input excitations. After meeting the required limit states criteria for both yielding and ultimate stages as defined earlier, R

factors were calculated based on peak ground parameters. A total of 14 IDA curves were plotted by subjecting these two prototype buildings to 7 different ground motions. On average, 30 nonlinear time-history analysis were conducted for each ground motion to come with each IDA curve. So in total, 452 time-history analysis runs were performed for the two buildings to come up with R factors. Figure 8-3 shows the seven IDA curves for the 3 bay-3 storey building. A comparison with pushover curve (5% drift ratio) has also been shown in the Figure 8-3.

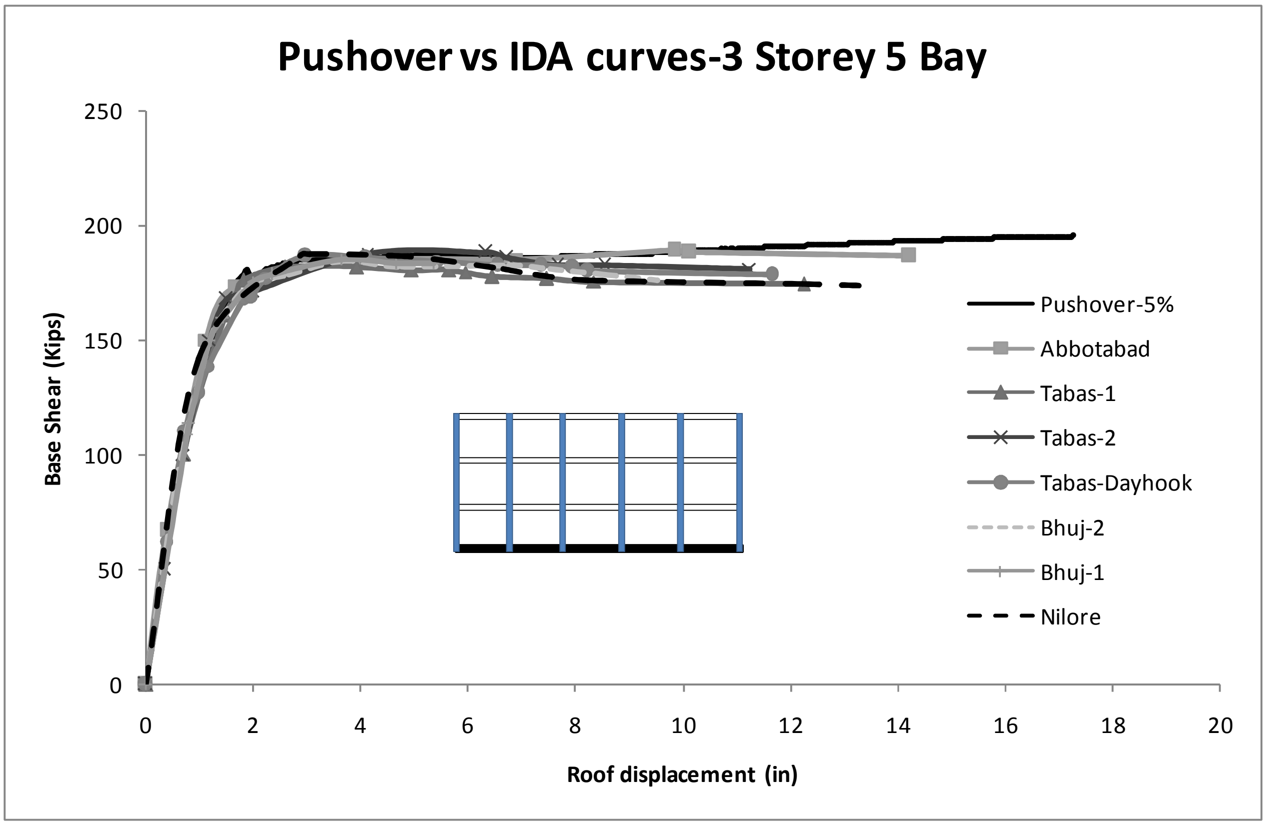



Figure 8-3. Comparison of pushover curve and IDA curves of a 3 bay-3 storey configuration when subjected to the various ground motions

Figure 8-4 shows the same comparison of IDA curves with pushover curves for the 5 bay-3 storey configuration.

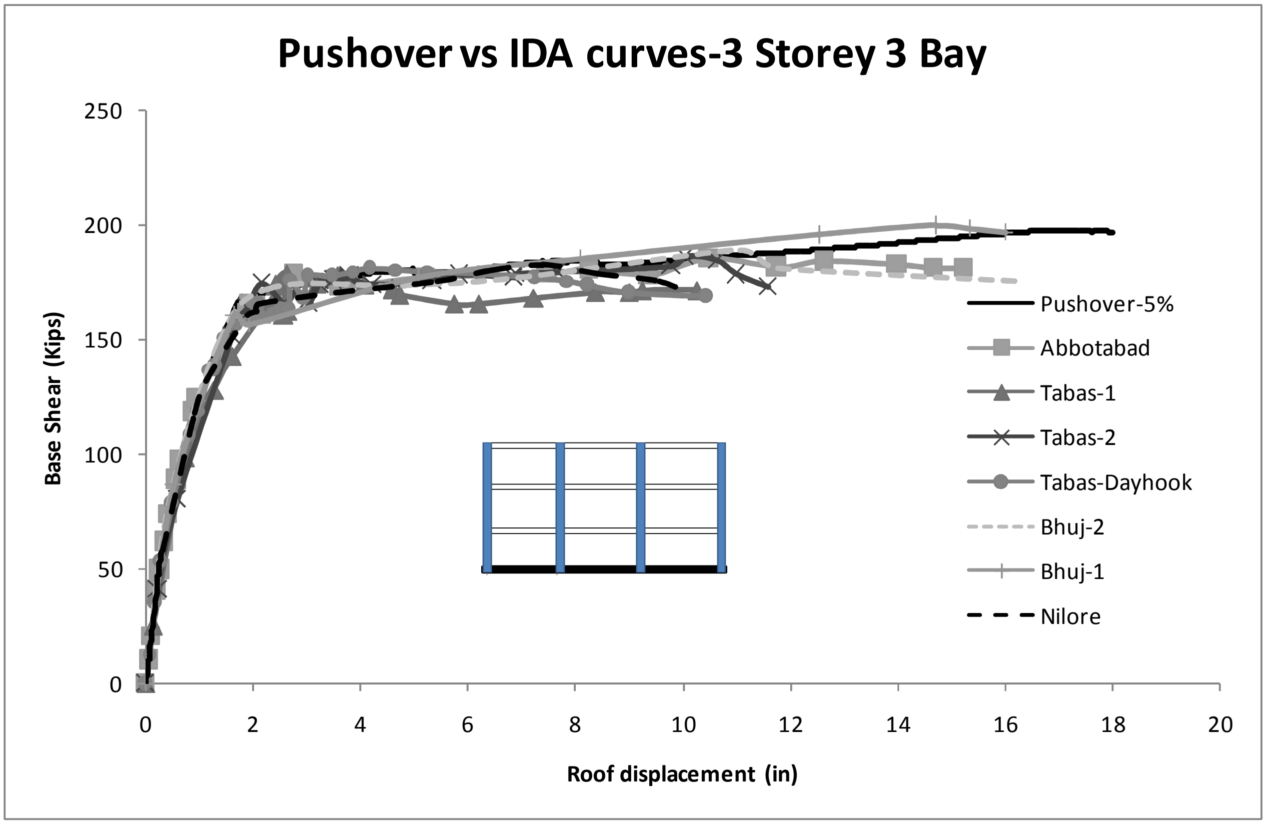



Figure 8-4. Comparison of pushover curve and an IDA curve of a 5 bay-3 storey configuration when subjected to the various ground motions

## 8.2 Computation of R Factor

As described in section 5.4, R factors are computed based on peak ground acceleration parameter. Variation of the R factor for the 3 Bay-3 Storey configuration for all 7 ground motions is tabulated in Table 8-2.

| S/No | Ground motion | PGA (g) | 3 Bay-3 Storey configuration PGA yield (g) | PGA ult (g) | R factor | R factor in BCP/UBC | R factor in NESPAK |
|---|---|---|---|---|---|---|---|
| 1 | Abbottabad | 0.236 | 0.296 | 0.96 | **3.24** | **3.5 ( RC OMRF)** | **4 ( RC OMRF)** |
| 2 | Nilore | 0.024 | 0.333 | 1.05 | **3.13** | | |
| 3 | Tabas-1 | 0.83 | 0.279 | 0.982 | **3.51** | | |
| 4 | Tabas-2 | 0.81 | 0.284 | 0.895 | **3.15** | | |
| 5 | Tabas-Dayhook | 0.32 | 0.315 | 1.14 | **3.62** | | |
| 6 | Bhuj-1 | 0.98 | 0.255 | 0.817 | **3.27** | | |
| 7 | Bhuj-2 | 0.78 | 0.26 | 0.92 | **3.53** | | |

Table 8-2. Variation of R factor for 3 bay-3 storey structure subjected to 7 ground motions

Similarly, variation of the R factor for the 3 Storey-5 Bay configuration for all 7 ground motions is tabulated in Table 8-3.

| S/No | Ground motion | PGA (g) | 5 Bay-3 Storey configuration | | | R factor in BCP/UBC | R factor in NESPAK |
|---|---|---|---|---|---|---|---|
| | | | PGA yield (g) | PGA ult (g) | R factor | | |
| 1 | Abbottabad | 0.236 | 0.276 | 0.96 | **3.48** | **3.5 ( RC OMRF)** | **4 ( RC OMRF)** |
| 2 | Nilore | 0.024 | 0.314 | 1.055 | **3.36** | | |
| 3 | Tabas-1 | 0.83 | 0.265 | 0.98 | **3.7** | | |
| 4 | Tabas-2 | 0.81 | 0.275 | 0.902 | **3.28** | | |
| 5 | Tabas-Dayhook | 0.32 | 0.312 | 1.17 | **3.76** | | |
| 6 | Bhuj-1 | 0.98 | 0.246 | 0.816 | **3.32** | | |
| 7 | Bhuj-2 | 0.78 | 0.278 | 0.977 | **3.51** | | |

Table 8-3. Variation of R factor for 5 bay-3 storey structure subjected to 7 ground motions

Table 8-2 and 8-3 show that, R factors computed are highly dependent on the type of building and the input ground motion. It is not just a single figure as suggested by seismic codes but depends mainly on the ductility of the structural system and type of excitation. It is also found that, the R factor suggested by both BCP-2007 and NESPAK 2006 for RC-OMRF is unconservative for some ground motions. For Nilore ground motion and for 3B3S configuration, R factor was calculated to be 3.13 as opposed to 3.5 and 4.0 as suggested in BCP-2007 and NESPAK 2006. So, the R factor for this case was unconservative by approximately 11% and 22% for the BCP-2007 and the NESPAK 2006 codes, respectively. Similar pattern of fluctuation of R factor was observed for the 5B3S configuration. It is worth noting that the R factors predicted from the analysis may still require further reduction in order for it to be valid for Pakistan. This additional reduction is primarily attributed to the human factor which in this case is represented by the lack of construction worker's experience and skills required for reinforcement detailing.

# CHAPTER 9

## PARAMETRIC STUDY ON R FACTOR

Response modification factor is affected by a number of factors related to the structure and the input excitation. By just changing earthquake ground motion, the response modification factor for the same structures changes as seen from results in chapter 8. In order to explore the effects of variation in structural geometric and material properties on the R factor, a parametric study was conducted.

### 9.1 Parameters Description

Several building parameters including: storey height, length of each bay, material strengths were used in this study. Table 9-1 shows all four parameters which were varied along with their range of values.

| Parameter | Steel Grade (Fy) | Concrete Strength (f'c) | Height of Storey | Length of bay |
|---|---|---|---|---|
| Range | 40-60 ksi | 3-5 ksi | 10'-12' | 12'-16' |

Table 9-1: Parameters and their range

A matrix of single bay three storey building configurations, value of geometric and material parameters and their elastic period ($T_{elastic}$) is shown in Table 9-2.

| S/No | Configuration | Building Reference | Single Bay - Three Storey | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | f'c (Ksi) | Fy (Ksi) | Bay length | Storey Ht | Variation | $T_{elastic}$ |
| 1 | Configuration#1 | B1 | 3 | 40 | 16' | 12' | Reference | 0.351 |
| 2 | Configuration#2 | B2 | 3 | 60 | 16' | 12' | Fy | 0.317 |
| 3 | Configuration#3 | B3 | 5 | 40 | 16' | 12' | f'c | 0.288 |
| 4 | Configuration#4 | B4 | 5 | 60 | 16' | 12' | Fy and f'c | 0.289 |
| 5 | Configuration#5 | B5 | 3 | 60 | 12'-9" | 12' | Bay Length | 0.301 |
| 6 | Configuration#6 | B6 | 3 | 60 | 16' | 10' | Storey Height | 0.274 |

Table 9-2: Building configurations used for parametric study

In total, six different building configurations were subjected to nonlinear analysis to examine the effects of variation of different parameters on R factor and to draw comparison. In Table 9-2, Configuration#1 (designated nomenclature B1) considered the concrete compressive strength of 3 ksi, grade 40 steel, bay length of 16 feet and storey height as 10 feet. Similarly other configurations were developed to account for variation in these parameters, in order to reflect any change in the response of the structure. The single bay three storey building configurations used for the parametric analysis is shown in Figure 9-1.

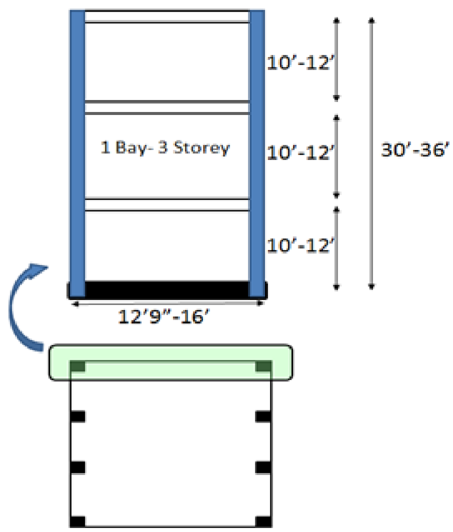



Figure 9-1: 1 bay-3 storey RC MRF building configuration used for the parametric analysis

## 9.2 Analyses and Results

Modal analyses were conducted to determine the uncracked / elastic fundamental periods of vibration. The elastic fundamental periods of vibration for all 6 configurations are shown in Table 9-2. For example, the $T_{elastic}$ of building B1 came out to be 0.351 seconds. As the compressive strength of concrete was increased from 3 ksi to 5 ksi, the fundamental period of the structure decreased. Similarly once the height of the storey was reduced, fundamental period of structure reduced. After this, inelastic static pushover analyses were performed for all the building configurations by using an

inverted triangular load pattern. This analysis procedure was employed to evaluate the global yield limit state and structural capacity. For all configurations, the pushover analysis was conducted with 4% target drift ratio, as this is roughly the range for the structure to reach its ultimate capacity. The results from pushover analysis for all 6 configurations are shown in Figure 9-2.

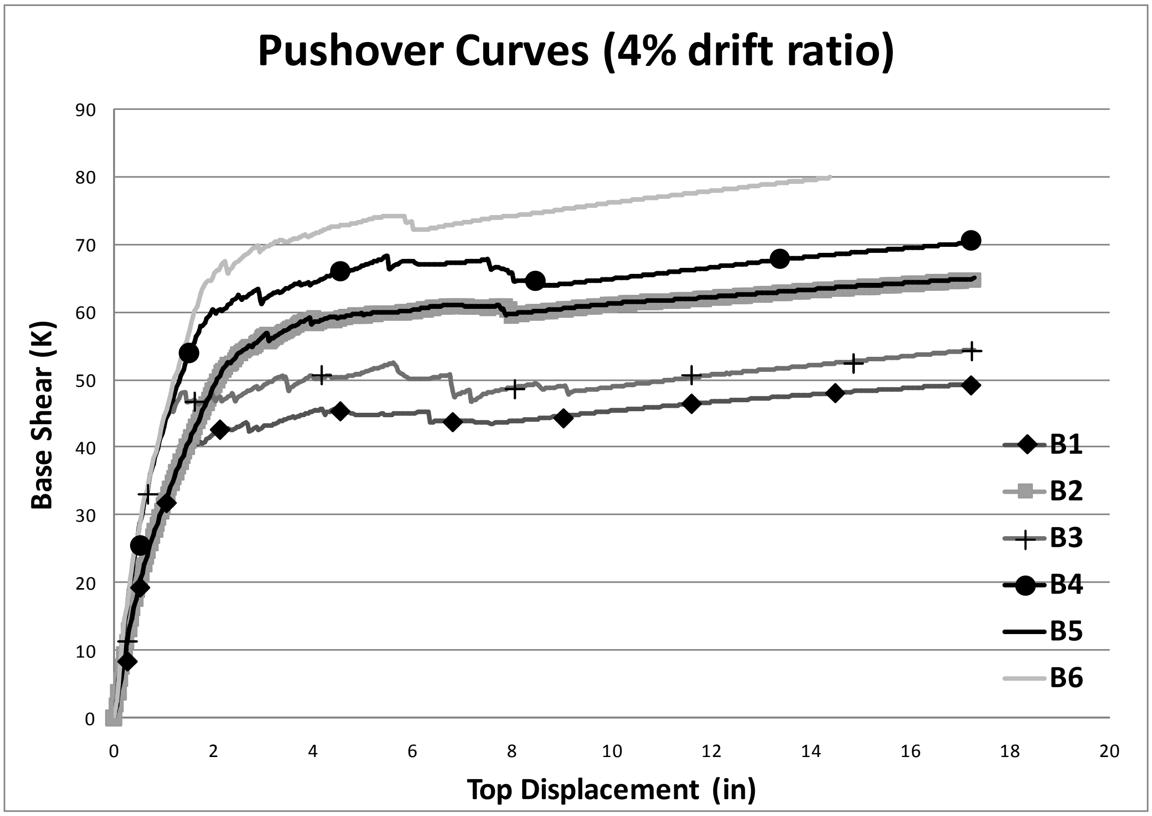
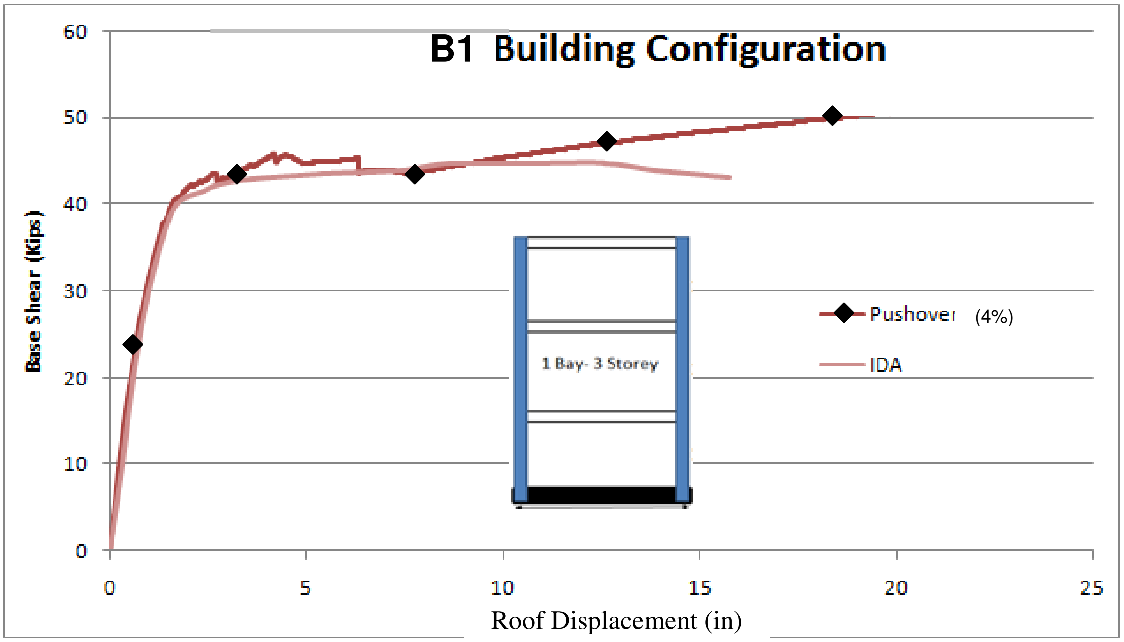



Figure 9-2: Pushover curves with 4% target drift ratio for all 6 configurations

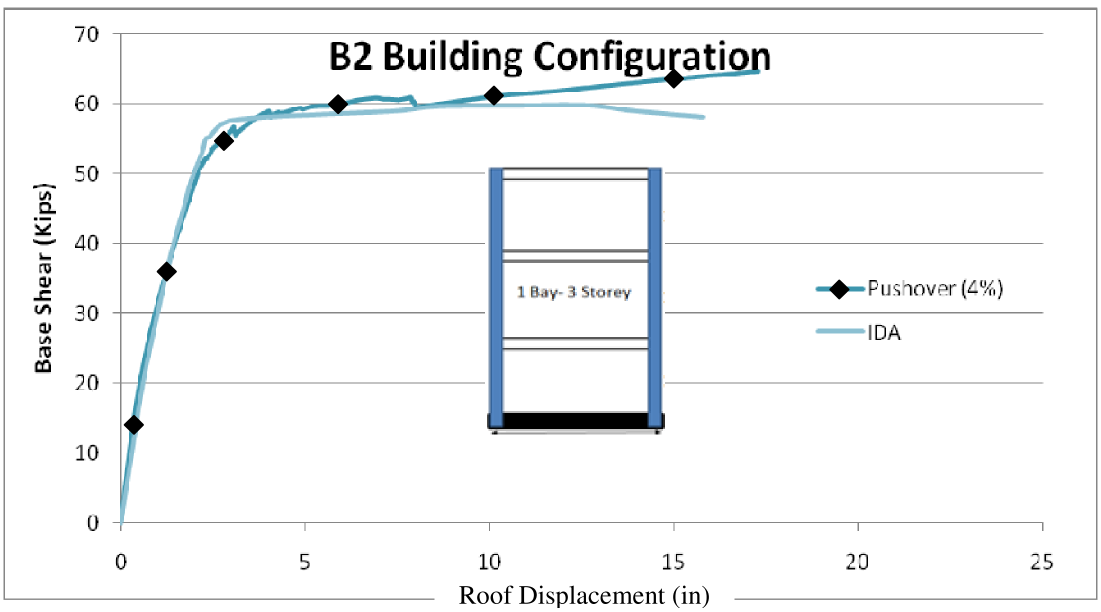
### 9.3 Calculation and Comparison of R Factor

In order to compute R factor, incremental dynamic analysis was performed using one natural ground motion. For parametric analysis, Abbotabad ground motion record from Kashmir 2005 earthquake was considered and several nonlinear time history analyses were conducted to find IDA curve for each of the 6 building configurations. The procedure for doing incremental dynamic analysis and computation of R factor has

already been described in detail in Chapter 5. The results from dynamic analysis along with pushover (PO) curve for each configuration are shown in Figure 9-3(a)-(f).

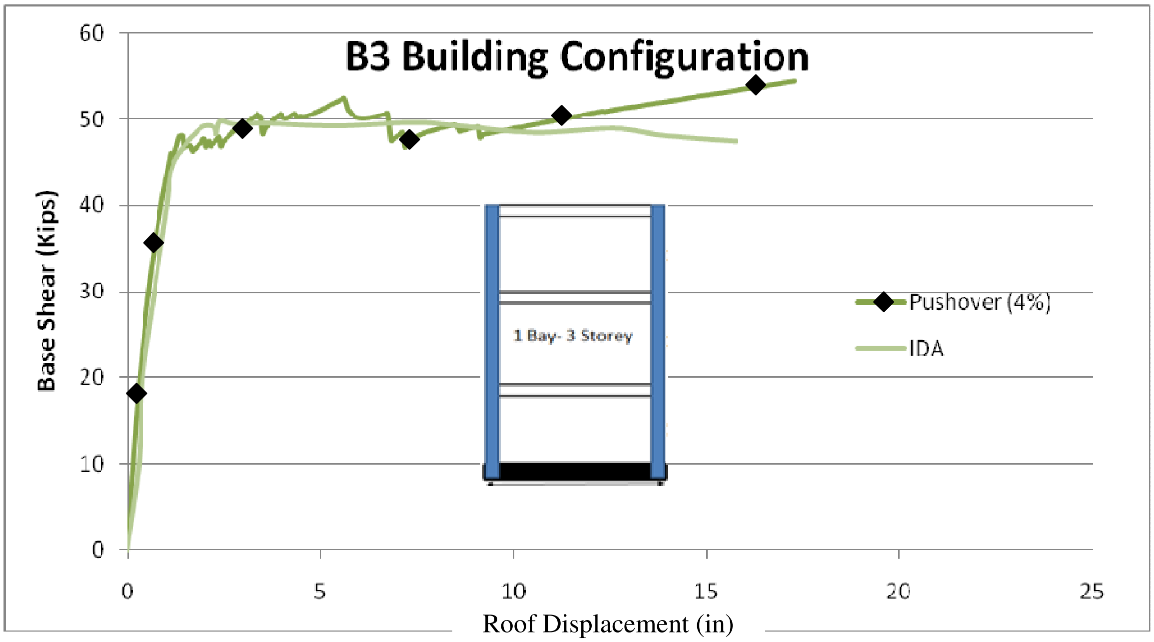



(a)

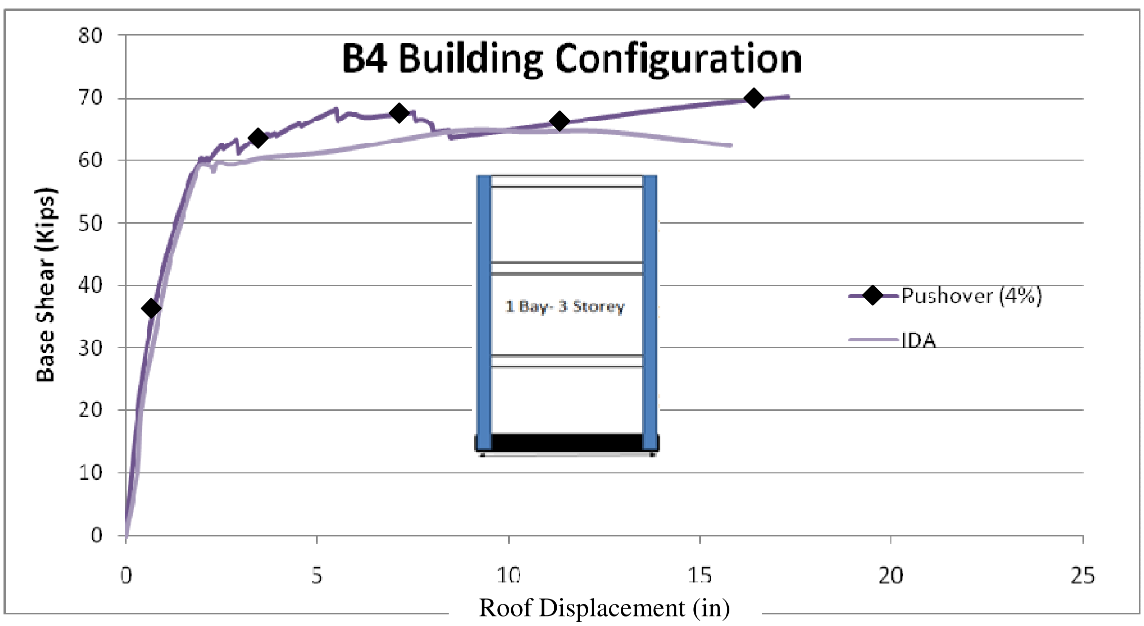



(b)

Figure 9-3: PO curve vs IDA curve under Abbotabad ground motion. (a) B1 configuration (b) B2 configuration.

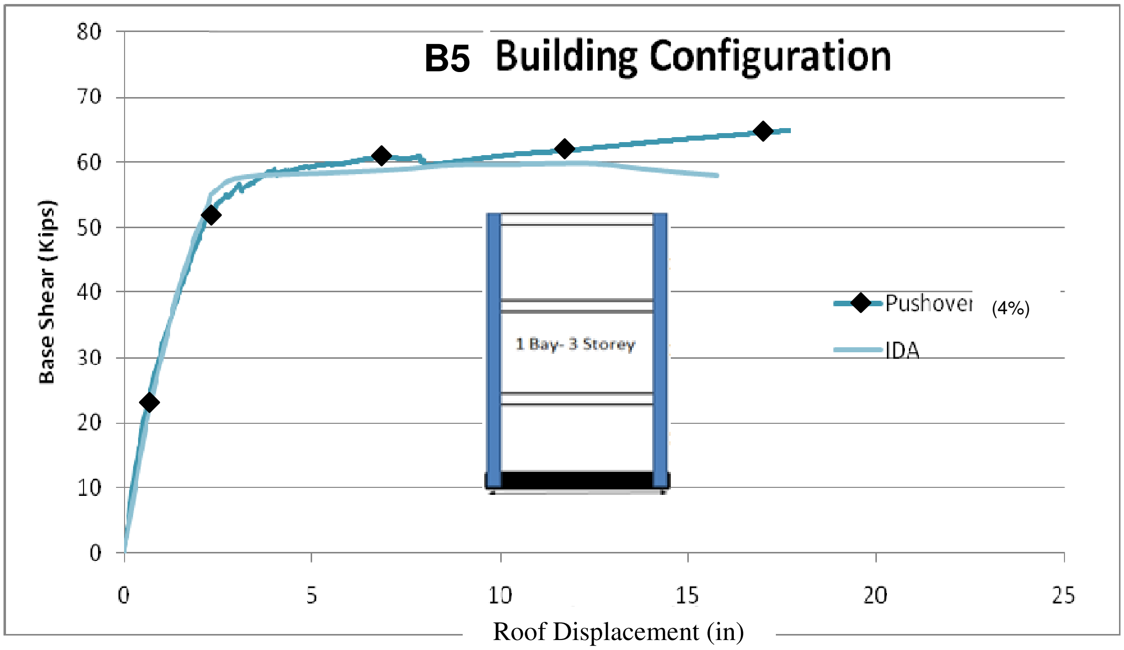



(c)

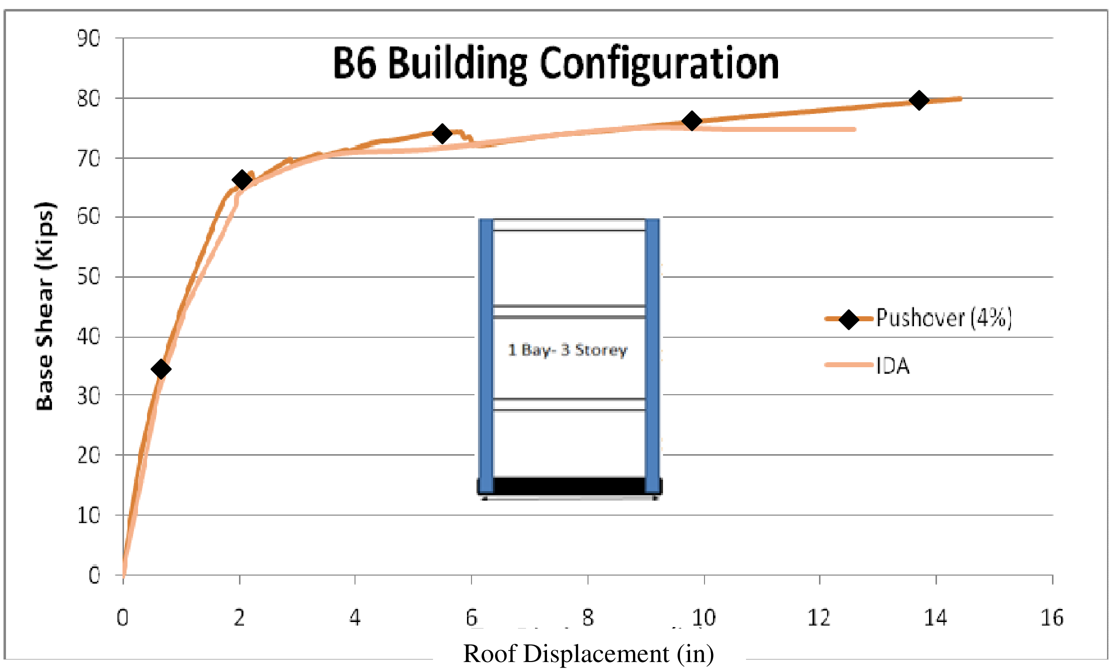



(d)

Figure 9-3 (cont'd) : PO curve vs IDA curve under Abbotabad ground motion. (c) B3 configuration (d) B4 configuration.




(e)




(f)

Figure 9-3 (cont'd) : PO curve vs IDA curve under Abbotabad ground motion. (e) B5 configuration (f) B6 configuration.

Based on response from incremental dynamic analysis coupled with IDA curves and the limit states defined, R factor was computed according to the procedure described in section 5.4. The computed values of R are shown in Table 9-3.

| S/No | Configuration | Building Reference | Abbotabad Ground Motion | | | R factor in BCP/UBC | R factor in NESPAK |
|---|---|---|---|---|---|---|---|
| | | | PGA yield (g) | PGA ult (g) | R factor | | |
| 1 | Configuration#1 | B1 | 0.249 | 0.767 | **3.08** | **3.5 ( RC OMRF)** | **4 ( RC OMRF)** |
| 2 | Configuration#2 | B2 | 0.283 | 0.91 | **3.21** | | |
| 3 | Configuration#3 | B3 | 0.25 | 0.775 | **3.10** | | |
| 4 | Configuration#4 | B4 | 0.291 | 0.977 | **3.36** | | |
| 5 | Configuration#5 | B5 | 0.32 | 1.11 | **3.48** | | |
| 6 | Configuration#6 | B6 | 0.29 | 0.983 | **3.39** | | |

Table 9-3: Computed values of R from parametric study

The computed values of R by employing limit states on the IDA response curve for all the building configurations is less than those suggested in the BCP 2007 and NESPAK 2006 codes. This fact suggests that the values given in code are sometimes unconservative, because higher R value will give lower design base shear. So structures designed for higher R value will have lesser capacity, in the wake of extreme seismic event. For instance, the R value computed for B3 is 3.1 as opposed to 3.5 from the BCP code and 4 from the NESPAK code; a difference of about 12% and 23%, respectively. Similar comparison of computed values of R factor for other configurations and the code values, reveal similar pattern of being unconservative. This again entails use of precaution while calculation of design base shear from the values provided in codes. For this parametric study only Abbotabad ground motion was used while using more ground motion records to determine R factor is useful in order to assess the pattern of fluctuation of the R factor from the variation of geometric and material parameters.

From the parametric study, it is evident that for higher grade steel, the R factor is higher and the frame is able to experience more inelastic behavior, before reaching

ultimate point. For example the R factor value was computed to be 3.21 for B2, in which Grade 60 is used, as compared to 3.08 for B1 in which Grade 40 steel was used. This might be attributed to the increase in the confinement pressure provided by Grade 60 stirrups compared to Grade 40 stirrups. On the other hand, the R factor showed less tendency of increase once the compressive strength of the concrete increased. For the B3 configuration, in which only compressive strength of concrete was varied, R factor only varied by 1% compared to the reference B1 configuration. However, for B4 configuration in which both, grade of steel and compressive strength on concrete were increased, the variation in the R factor was about 9%. R factor also showed tendency to vary, by the change in geometric parameters of the structure. Values of R factor increased as the bay length of the frame was decreased. R factor computed for configuration B5 was 3.48, as compared to 3.08 for the reference B1 configuration. This 13% increase in the value of R factor is a substantial variation and will have significant effect on the design base shear values. Similarly, value of R factor increased once the storey height was reduced from 12' to 10'. R factor value computed for B6 configuration was 3.39 which is a variation of about 10% from the reference B1 configuration.

From the results shown in Table 9-3, it is evident that the R factor is sensitive to both geometric configuration and material strength. However, variation in geometric parameters tends to display more significant impact on R factor value. It is also evident from the results that the stiffer the frame in general whether due to changes in material or geometric properties, the greater the value of R factor. The stiffer the structure, the lesser would be the deformation demand on the frame which requires more PGA or more seismic force to reach the ultimate point.

# CHAPTER 10

# CONCLUSIONS

This thesis explored analytically the response modification factor (R factor) of RC MRFs in developing countries and how it varies from the R factor values recommended by the seismic design codes in developed countries. The study utilized Pakistan as an example of an earthquake-prone developing country.

Two prototype RC buildings with varying geometric characteristics were studied to evaluate the R factor for RC MRFs in Pakistan. The study involved various analysis including Modal, inelastic static pushover and incremental dynamic analysis. Seven natural earthquake records were selected from the region and employed in the analysis. The input accelerograms were scaled up starting from lowest intensity, until the ultimate limit state was reached. It was found from analysis that the R factor suggested in seismic codes, which are being adopted in Pakistan gives false representation of building response during a seismic event. It was also found that a single value of R factor as suggested in BCP 2007 (UBC 97) or the NESPAK 2006 may become unconservative by as much as 11% and 22%, respectively for the ground motion records used in this study.

A parametric study was also conducted using a single bay-three storey building to explore the effect of geometry and material characteristics on the R factor value. The following are the conclusions of the study:

(i) R factor is sensitive to both geometric configuration and material strength; however, variation in geometric parameters tends to display more significant impact on the R factor value.

(ii) By decreasing the bay width and storey height, the structure exhibited delayed collapse mechanism, thus resulting in increased value of R factor.

(iii) The highest variation in R factor was observed from changing the bay length. A decrease in bay length of about 19% increased the value of R factor by 13%.

(iv) Decrease in storey height by 16% increased the R factor value by 10%.

(v) Increasing the stiffness of the frame in general, whether due to changes in material or geometric properties, leads to an increase in the R factor value.

Finally, it is important to note that another important factor that was not included in this study but yet need to be considered in the future is the human factor. A comprehensive study based on construction practices and workers skills in Pakistan, has to be conducted in order to achieve more precise predictions for the R factor values that should be adopted in Pakistan.

# CHAPTER 11

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