An-Najah National University

Faculty of Graduate Studies

Effect of Column Orientation on Response Modification Factor (R-Factor) of Reinforced Concrete Frames

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Dedication

This thesis is dedicated to my dear parents who have encouraged and supported me. Also, to my great family and friends.

Acknowledgment

First of all, I am thankful to the almighty God for granting me good health, strength and peace throughout the research period.

I would like to thank everyone who has contributed to accomplishing this thesis.

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الإقرار

انا الموقع أدناه مقدم الرسالة التي تحمل العنوان:

Effect of Column Orientation on Response Modification Factor (R) of Reinforced Concrete Frames

اقر بأن ما اشتملت عليه عليه هذه الرسالة إنما في نتاج جهدي الخاص، باستثناء ما تمت الإشارة إليه حيثما ورد، وأن هذه الرسالة ككل، أو أي جزء منها لم يقدم من قبل لنيل أية درجة علمية أو بحث علمي أو بحثي لدى أي مؤسسة تعليمية أو بحثيه اخرى.

Declaration

The work provided in this thesis, unless otherwise referenced, is the Researcher's own work, and has not been submitted elsewhere for any other degree or qualification.

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Date: 20 / 9 / 2018 **التاريخ: 20/ 9 /2018**

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Abstract

Response Modification factor (R-Factor) is an essential seismic design parameter, which is typically used to describe the level of inelasticity expected in structural systems during an earthquake and is used to reduce the anticipated earthquake load due to the inherent inelasticity of the structure. International building codes provide fixed values for this factor for each category of building system despite the fact that its value depends on the details of the structural system and thus should differ for each building. One of the aspects of buildings in Palestine is their irregularity and this includes the disorientation of columns strong axes in the building plan to suit architectural needs. In Palestine, the international codes are generally applied with little to no guidelines on the validity of these codes to the buildings being designed. To-date, there are no guidelines as to how this R-Factor would change due to the disorientation of main axes of the load-bearing columns in the building. This study comes as a step towards investigating the validity of the code-specified values of the R-factor for framed buildings with disoriented columns.

To achieve the above-mentioned goal, pushover analysis is considered as a nonlinear procedure to predict the inelastic behavior of framed buildings,

by exposing the structure to increasing lateral loads, until failure occurs. The finite element software SAP2000 is used to generate the nonlinear behavior curve through incremental elastic-plastic analysis with concentrated plasticity in the plastic hinges within the structural members. Two building layouts were used in the study, one square and the other is rectangular, with variable number of storey's and variable column orientation. The results show that the R-Factor increases as the number of storey's increase, and it attains a maximum value when the loading direction coincides with the strong axes of the columns. The R-factor is minimum when the main quake load coincides with the weak axes of the columns. These results were invariable for both building layouts. Also, it is found that the R-Factor recommended by the seismic design provisions (IBC 2012 for example) may not be conservative for use in buildings with disoriented columns. In fact, it is found that for buildings of 4 floors, the value of R-factor from IBC 2012 is higher than that obtained from the push over analysis. This means that using IBC2012 value of R-Factor would give lower induced seismic forces for design, which may lead to detailing level that does not warrant the realistic R-Factor for the building being designed.

The study is only a first step towards scrutinizing the validity of the international building codes for use in Palestine and further research is needed to advance this study. As a future research topic, it is recommended to conduct nonlinear time-history analysis using actual earthquake records in order to compare the inelastic behavior of these buildings to the actual earthquake loads in these buildings.

Chapter One

Introduction

1.1. Motivation

The force-based design for earthquake resistant structures is still the most common approach despite all the studies that encourage the more realistic displacement-based approaches. In the force-based design, the seismic action is represented as a set of forces that are applied on the structure, thus simplifying the analysis and design of such structures for earthquakes. Building codes prescribe parameters to estimate the value of the seismic induced forces, and these parameters depend on the structural system and level of detailing in the buildings to be designed. Generally, these parameters are established for uniform buildings with regular framing systems that exhibit similar behavior regardless of direction of anticipated load.

Engineers in Palestine, suffering from lack of resources, Israeli occupation and still in the early stages of development, take these parameters from international building codes and apply them on their designed buildings, sometimes without due consideration to their validity. Buildings in Palestine are known for their irregularities and lack of uniformity, in particular the disorientation of columns and frames in the buildings. It is quite common to find buildings in Palestine that are rectangular in the outer shape, but the grid of columns and main frames being irregular and disoriented in plan. Therefore, it is of prime importance to investigate the validity of building codes parameters for use in Palestine.

One of the most important factors used in the earthquake design is the R-factor, known as "response modification factor" or "load reduction factor". This factor is used to reduce earthquake forces due to the inherent ductility and inelasticity in the structure. Building codes provide values for this factor for each category of building system despite the fact that its value depends on the details of the structural system and thus should differ for each building.

While buildings in Palestine are mainly irregular and columns are generally disoriented in plan to suit the architectural needs, the values of R-factor are still quoted from international building codes without any modifications to reflect the irregularities found in local buildings. To-date, there are no guidelines as to how this R-factor would change due to the disorientation of load-bearing columns in the building. This study comes as a step towards investigating the validity of the R-factor for framed buildings with disoriented columns.

The response modification factor (R-Factor) calculates the flexibility of the building and adjusts the design lateral loads accordingly. Studies on existing buildings during earthquakes have shown that flexible buildings act much better in seismic events than rigid buildings because of the ability of flexible systems to dissipate the energy of the ground motion.

1.2. Palestine Seismicity

Palestine is highly exposed to the risks posed by nature: the most important of these are earthquakes, landslides, drought and desertification. The region often faces disasters that may be small to medium in size and sometimes have high potential for large-scale (urban) disasters. The geodynamic processes on the seismic activities occurring in Palestine are largely influenced and controlled by the Dead-Transform DST. Daylight saving time is a side error between the Arabian Peninsula and the tectonic plates in Sinai, which carry the opening in the Red Sea to the Taurus-Zagros collision zone. The left lateral shear along the Dead Sea explains the methodological approach of up to 105 km from many of the previous features of Myosinology. (Quennell 1959 and 1983, Fruend 1968).

The seismic activity of the region shows that the concentration of seismic activity occurs along the main pathways of the fault and the associated areas. Based on the location and seismic nature of the area, an earthquake with a magnitude of more than 6 degrees is expected. Considering the devastating earthquake of 1927 (6.25 degrees and 15 kilometers north of the Dead Sea), a large earthquake was expected to occur at any time in the near future in the northern Dead Sea, which in turn would cause significant damage and losses due to the severe weakness of common buildings. On the other hand, according to other studies in the region can be expected epic earthquake entered into the southern part of the Dead Sea.

In view of the Earthquake Acceleration (PGA) map of Palestine, we can see that Palestine is divided into the following areas: 1, 2A, 2B and 3 (see Figure 1). Based on international and local codes such as the UBC97, IBC, the Jordanian Building Code 2008 and the Arab Common Code 2006, the Z-zone factor is presented on the rock of the above-mentioned areas in

Figure 1. Palestine is a moderate seismic zone to a relatively strong seismic zone.

For all these conditions mentioned above, the Palestinian Engineers Association has made it mandatory that buildings be designed to resist earthquake forces and that structural design be consistent with design codes such as UBC97 or IBC2012. Therefore, it has become a common practice for engineers to assume values from such international codes and apply them to local buildings without any guide that can help them to verify their choices. For this reason, this thesis comes in this regard to be a practical guideline for the selection of an important parameter that is used in earthquake resistant design.

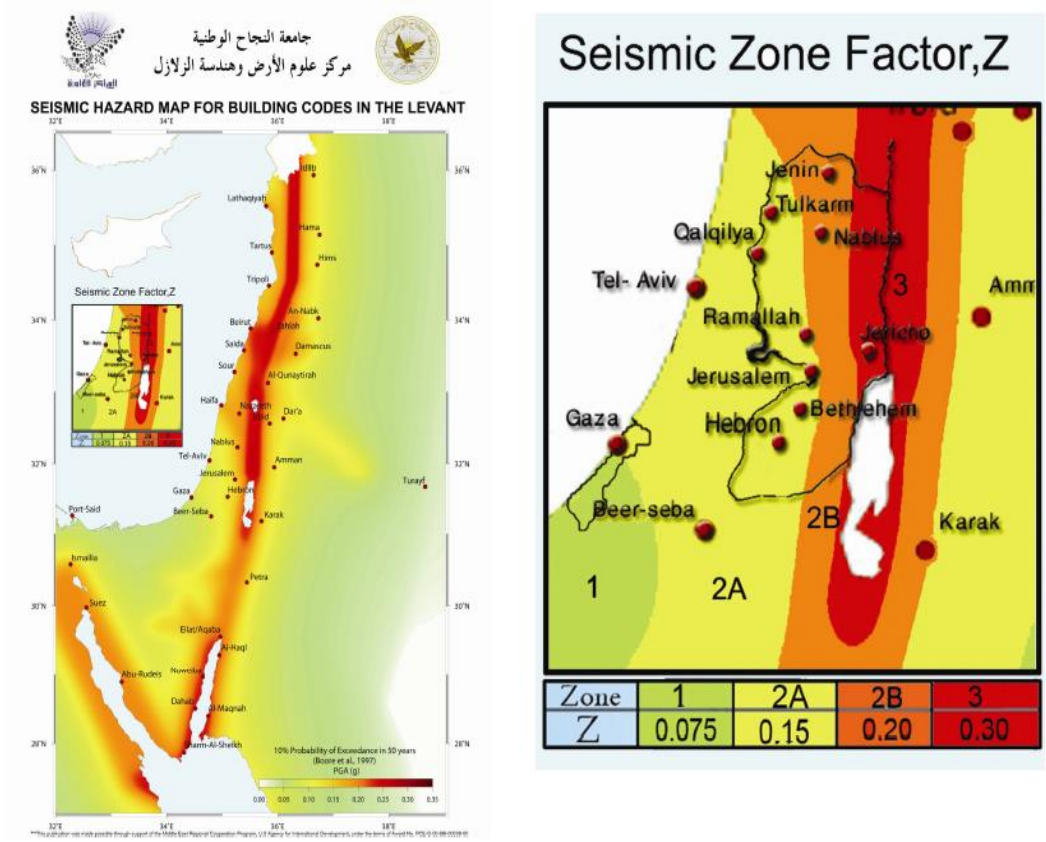


Figure 1: Seismic Hazard Map & Seismic Zone Factor,Z(Filippou, 2013)

1.3. Research Significance

Palestine is an example of the developing countries that have buildings mostly irregular. Palestine relies on foreign seismic design codes which are used in the United States. Structures in Palestine are facing different nature of the vulnerability of those in the United States, due to the different levels of risk and the risk of seismic construction. One of the key aspects of the buildings in Palestine is the irregular and random orientation of columns. In Palestine it is very common to see columns directed randomly even for some ordinary buildings.

To date no study had focused on assessing how this columns disorientation can affect the value of R-Factor. There are no guidelines that can tell the variation in R-Factor due to variations in column orientations.

Therefore, the use of R-Factors considered in the United States may provide a false representation of the structural practices applicable in Palestine. In fact, buildings in Palestine need a seismic provisions and special seismic code to calculate the R-Factor depending on the seismic hazards and its own type of structures used in the region. This thesis comes as part of raising the level of awareness among local engineers in this regard.

1.4. objective

The primary objective of this study is to re-evaluate code-based R-Factor for typical reinforced concrete (RC) moment resisting frames (MRFs) used in Palestine, through nonlinear analysis tools, and study the effect of columns disorientation on calculated R-Factor.

To achieve this primary goal, first a literature review is conducted where recent studies about R-Factor are explored to give a knowledge about this subject. Then non-linear static pushover analysis is utilized in this study to find the response modification factor for RC-MRF with disoriented columns. For doing so, a parametric study will be conducted to evaluate the effect of variation of geometric properties such as column orientation, storey numbers of RC-MRFs and building layout on R-Factor. Finally, conclusions will be summarized based on the findings of this research.

1.5. Framework of Thesis:

This thesis is divided into the following chapters:

Chapter one, which provides an introduction that briefly explains the problem statement and the significance of the research.

Chapter two, R-Factor & nonlinear Static Procedure (NSPs) used to calculate R-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 will also be presented. This chapter provides an overview of various methods that are currently used to calculate R-factor for reinforced concrete buildings. An in-depth discussion on the application of pushover analysis to find R-factor will be presented.

Chapter Three presents the case study building and the challenges in its modeling, particularly the modeling of plastic hinges.

Chapter Four present the results of parameter analysis and evaluation of R-Factors for selected mode buildings

Results from nonlinear pushover analysis of prototype buildings will be presented in this chapter and the computed R-Factors for these buildings will be discussed and compared to those recommended by existing building codes, such as, IBC2012.

Chapter Five: Conclusions and Recommendations

Chapter Two

R-Factor& Nonlinear Static Procedure (NSPs)

2.1. Introduction

Design requirements for lateral loads are different from those for gravity loads. In areas of high seismicity, it is compulsory to design structures to resist lateral loads. If flexible design concepts, typically used for gravitational loads, are used for earthquake loads, the result will be in the form of very heavy and expensive structures. Therefore, seismic design uses the concepts of damage control and prevent collapse and allows the design forces to be reduced based on the fact that damage in buildings reduces the side rigidity of the building.

The purpose of seismic engineering is to control the type, location and size of the damage through the detailing process. This is illustrated in Figure (2), where elastic and inelastic responses are drawn. In Figure (2) we can see that if the building continues to behave elastically up to the point of failure, the induced force resistance in the building due to earthquake will be quite large (V_e in Fig. 2), and thus the designer has to proportion the members in such a way that the design becomes uneconomical. This is logical if the purpose of the designer is to maintain the building in the elastic stage up to the end of the earthquake. However, for typical buildings, the design codes of practice allow for reduction in the earthquake force due to the induced damage in the building that happens by the earthquake. As it seen in Fig. (2), the plastic deformations in structure cause the stiffness of structure to reduce and thus the nonlinear behavior emerges and the building fails at a larger displacement under a lower lateral force (V_y in Fig.2). This can only happen if the building is sufficiently detailed to ensure that it has sufficient ductility at the joints. Common building codes prescribe a certain factor, called response modification factor, R-Factor, that is used to reduce the seismic (earthquake) lateral load from that resulting in an elastic analysis to the realistic inelastic value. In this chapter we will focus on the evaluation of the R-Factor either in codes of practice or as studied by researchers, and then we will present the nonlinear static procedure that is considered one of the methods used to reproduce the value of R-Factor for buildings.

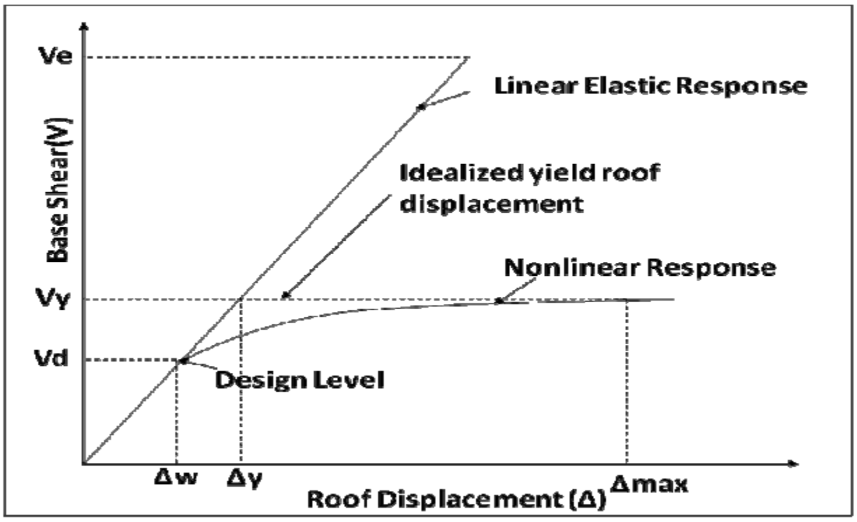


Figure 2: Force displacement response of the elastic and non-elastic systems

2.2. Response Modification Factor (R) and its Components

R-Factors are essential in seismic design tools, because they determine the scale of inelasticity that will be expected in structures during an earthquake. The provisions define R-Factor as “factor purposed to account for damping and ductility inherent in structural systems at the displacements great enough to approach the maximum displacement of the systems”(Raheem, 2013).

This definition gives some understanding of the seismic response of buildings and the expected behavior in the design. R-Factor reflects the ability of the building or structure to disperse the energy through plastic deformation.

R-Factor is used to:

- reduce the design forces in earthquake resistant design to account for inelasticity.
- consider energy dispersion and hysteretic damping.
- utilize the over-strength of the structure as a reserved capacity.

In earthquake resistant design the structure must resist ground movement of earthquakes without collapse, but with some damage. To achieve that, the structure is designed for less base shear forces than required if the building is to remain elastic during heavy shaking at a site. These large reductions are due to two factors:

1. The ductility reduction factor (R_μ), which reduces the elastic demand force to the level of the maximum yield strength of the Structure.
2. The over strength factor (Ω), which accounts for the over strength introduced in code-designed structures.

Thus, the response reduction factor (R) is simply defined as Ω times R_μ.

$$R = R_\mu * \Omega \qquad 2.1$$

2.2.1. Ductility Reduction Coefficient (Rμ)

R_μ is a factor that reduces the strength of elastic demand or the elastic force (V_e) to the ideal level of productive force or yield strength (V_y) of the structure, and it can be calculated as follows:

$$R_\mu = \frac{V_e}{V_y} \qquad 2.2$$

R_μ factor takes advantage of the energy dispersing ability of fairly designed structures. It depends on the global ductility demand (μ) of the structure which is defined as the ratio between the maximum roof displacement and yield roof displacement.

The first study to relate R_μ with μ was made by Newmark and Hall for a single degree of freedom (SDOF) system with elastic perfectly plastic (EPP) resistance curve(Christiana, 2013). They concluded that:

- For short period structures (**T < 0.2 second**) the ductility is useless in reducing the response of the structure. Thus, a ductility reduction factor should not be used for this type of structure.
- For moderate period structures (**0.2 sec < T < 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. Thus, using this principle, called an equal energy principle, the reduction factor R becomes: $R = \sqrt{2\mu - 1}$.
- For relatively long-period structures, the inertia and induced inertia forces from (an elastic and an inelastic system respectively) cause the same maximum displacement. This gives the value of ductility

reduction factor computed based on this “equal displacement” principle as:

$$R_\mu = \mu \qquad 2.3$$

2.2.2. Structural Over Strength Factor (Ω)

Structural over strength factor (Ω) have an important role to prevent the collapse of the buildings. It can be defined as the ratio of actual lateral strength to the design lateral strength:

$$\Omega = \frac{V_y}{V_d} \qquad 2.4$$

Where:

V_y: is the base shear coefficient corresponding to the actual yielding of the structure.

V_d: is the code-prescribed un-factored design base shear coefficient.

The strength of the earthquake (E.Q. force) with the first large return in a reinforced concrete structure may be much higher than the standard (un-factored) shear strength determined by several factors such as:

1. Load ultimate factor applied to the code-determined design seismic force.
2. Lower gravity load applied at the time of the seismic event compared to the factored gravity loads used in design.
3. Variation in the material properties that used in design, where higher strength of materials can happen.
4. Member sizes are greater than desired from strength considerations

5. Further reinforcement of the required strength.
6. Special ductility requirements that may improve the member behavior, such as confinement due to stirrups.

2.3 Literature Review of R-Factor

2.3.1. Previous Studies

The R-Factors are originally built on judgment and qualitative comparisons with the well-known response of some of the framing systems. Now it has come a long way through the measurement actually using non-linear analysis tools and (peak ground and spectral) parameters.

The evaluation of R-Factor for different building systems has been the subject of many studies, these studies were mainly numerical in nature.

The first systematic and analytical treatment of R-Factor was done by Newmark and Hall (1982) who identified three reaction areas, based on the structure period, as follow:

- The short period, $T < 0.2$ seconds

$$V_d = V_e \tag{2.5}$$

- The intermediate period, $0.2 < T < 0.5$ seconds

$$V_d = \frac{V_e}{\sqrt{2\mu - 1}} \tag{2.6}$$

- The long period, $T > 0.5$ seconds

$$V_d = \frac{V_e}{\mu} \tag{2.7}$$

where;

Vd: design force.

Ve: elastic force.

μ: ductility ratio or ductility degree.

T: is the fundamental period of vibration of the building or structure.

The first zone is determined by acceleration, the second is characterized by energy dissipation and the third is dominated by displacement. The boundaries of these zones are not constant for all earthquakes and the time ranges mentioned above are only indicative ranges. See Figure (3).

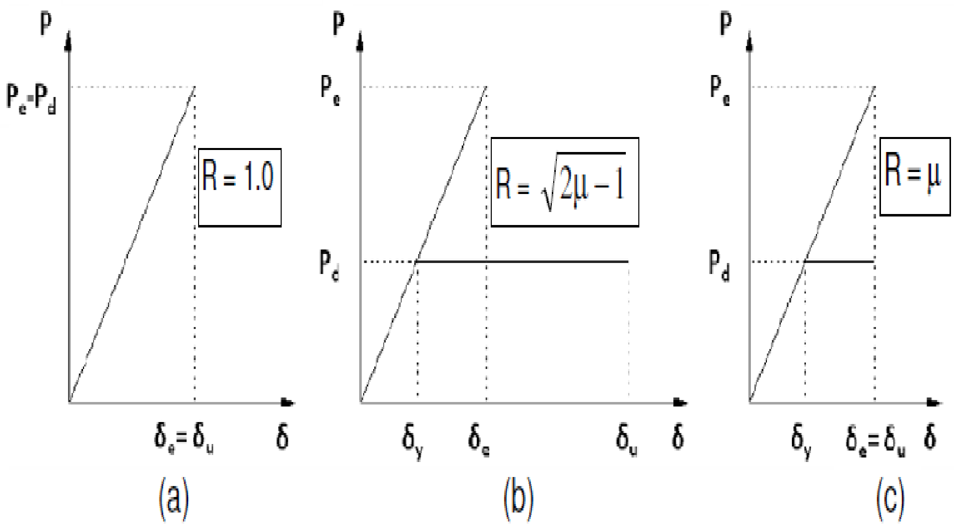


Figure 3 : the Relation between elastic and non -elastic (inelastic) forces for((a) the short (b) the intermediate and (c) the long periods) Structures(Newmark & Hall, 1982).

The previous relationship shows, the Factor (R) as a function of ductility (μ) in the three regions that were proposed by Newmark and Hall (1982). The increasing in R-Factor value with the increasing in period due to the system tends to display inelastic behavior, which means that factor R depends on the period(Newmark & Hall, 1982).

In 2010 Zafar” evaluated R-Factor for typical reinforced concrete (RC) moment resisting frames (MRFs) which exist in Pakistan, he used non-linear analytical tools, and compared the calculated R-Factor with the values given in seismic code of practice. He used the Incremental dynamic analysis (IDA) to find the response modification factor for RC-MRF in Pakistan using a suite of ground motion records representative of the region. A parametric study to evaluate the effect of variation of material and geometric properties of RC-MRFs on R-Factor was also conducted.(Zafar, 2010)

Zafar found that the R-Factor suggested in seismic codes gives false representation of the building response during a seismic event. He also found that a single value of R-Factor as suggested in Pakistan code BCP 2007 (UBC 97) or the NESPAK 2006 can become un-conservative.(Zafar, 2010)

In 2001 Collier has published an article and got a conclusion of the established firm relationship between the force reduction and the overstrength factors. He suggested applying a gradual increase in R code factors and rigorous assessment of the performance of buildings designed accordingly. It has been suggested to increase factors initially by 10-20% for hybrid structures and by 30-40%for regular frame systems designed to medium and high ductility levels. Whereas significant increase in R factors is recommended, the suggested margins remain adequately conservative.

Adoption of the proposals would render EC8 a more economic code, without jeopardising the reliability and safety of the buildings.(Collier & Elnashai, 2001)

In 2013 Filippou studied the assessment of R-Factor for an existing multistory building and examined the effect of different reinforcing scenarios of its concrete frames on the R-Value. (Filippou, 2013)

Linear and nonlinear pushover analyses were used for the capacity assessment of the construction. The results from each technique were compared in order to acquire the differences among the analysis methods.(Filippou, 2013)

2.3.2. Code Provisions of R-Factor

2.3.2.1 American Practice

Earthquake resistant design in most common American codes Universal Building Code, (UBC), has passed through main three stages:

- In the first stage, a specified percentage of the building weight was applied as lateral load. In this stage all design was elastic and no mention of "R" at all.
- In the second stage, the equation ($V = ZKCW$) was adopted for 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). Again here, no use of "R-Factor" in design.
- In third stage, the most recent, the site-specific ground motion maps, the building period, the importance factors, the site (soil) factors and

the Response Modification Factors (R) were considered in computing equivalent lateral forces on the structure.

In 1961, the UBC-Code introduced the use of four K factors to classify the building system type(ASCE, 2000). Following research and testaments included in ATC-3-06 (Jong-Waha-Bai, 2004), the 1988 UBC introduced the use of R_w 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. The same year BOCA also included the C_d factor for deflection amplification whereas previously deflection amplification was computed based on a multiplier (0.7) of the R_w factor. C_d factor addresses the probability of the deformations of structure in an earthquake greater than those indicated by the linear deformation equations(FEMA356, 2000).

In 1994 Northridge earthquake was followed by widespread application of seismic design throughout the U.S.A. for the first time. The combining of Codes and the almost uniform adoption of International Building Code (IBC) has helped to ensure a uniform design approach. However, IBC standards have been changing quickly. The latest edition of IBC 2012 has eighty-three building Response Modification Factors for each framing system type listed in IBC 2012/ASCE 7-10 Table 12.2-1. These include R=5 for reinforced concrete Intermediate Moment Resisting Frames. UBC 97 has an R-Factor of 5.5 for the same lateral resisting system. Values of R-Factor recommended in UBC 97 for moment resisting frame systems are shown in Table (1).

Table1: R factor values in UBC-97 for moment resisting frames (MRF)

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

2.3.2.2. Europe (Euro Code 8)

In Euro code, the general seismic design procedure is to reduce the elastic spectral demands to the strength design level by the use of a period-dependent response factor, called the behavior factor $q_\circ$. This behavior is a function of ductility, building strength, structural system and stiffness regularity. To determine the q factor according to EC8, the following equation is used (Eurocode8):

$$q = q_\circ * k_D * k_R * k_W \qquad 2.8$$

where;

$q_\circ$: is the basic value for response factor.

k_D: represent ductility class.

k_R: is a factor reflecting structural irregularity in elevation.

k_W: reflects prevailing failure mode (for MRF K_W is taken to be 1).

Values of above factors are shown in tables (2) to (4).

Table2: Basic Value of Response Factor $q_\circ$ in Eurocode

Structural type		*q0*
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

Table3: Values of k_DRepresent Ductility Class in Eurocode

Ductility class	k_D
DC"H"	1.0
DC"M"	0.75
DC"L"	0.5

Table4: Values of k_RReflecting Structural Irregularity in Elevation Eurocode

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

Values of $q_\circ$ factor range between 2 and 5 for reinforced concrete framing system as mentioned in Table (2).

2.3.2.3. Japan

The Japanese Standard Building Act (BSL) includes a two-phases procedure for seismic design of buildings (ATS-19). The design of the first phase follows an approach in which force design is used in reinforced concrete structures. Seismic effects are calculated using unreduced seismic forces.

The second phase design is a direct estimation of strength and ductility. BSL uses R in a different format. A ductility factor ($1/D_s$) which is equivalent to R-Factor is used for all building systems and ranges 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. (Uang, 1991)

2.3.2.4. Egypt

In chapter 8 of the Egyptian code, "Loads and forces on structural and nonstructural systems", the R-Factor defined for reinforced concrete structure can be taken 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. (Zafar, 2010)

Table 5 : R-Factor in Egyptian seismic code

Structural System	Ductility	R
RC Moment resisting frame	Sufficient	7
	Not Sufficient	5

2.3.2.5. summary

As can be seen from the review and studies, all countries do consider modern seismic design practices by making the structure exhibit more ductile behavior. This ductile behavior is ensured through detailing process and is reflected through the relevant R-Factor based on their detailing requirements. The R-Factor is thus unique for every kind of structure, ground motion, site condition and local practices in construction. 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(ECP2012, 2012).

Based on vulnerability towards seismic events, Palestine, as one of the 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 Palestine.

2.4. Nonlinear Static Procedures

Generally, most building codes admit four levels of analysis and evaluation, they are: (LSP, NSP, LDP, NDP). There is a hierarchy of four levels of structural analysis appropriate for the evaluation of existing buildings. Each higher-level procedure provides a more accurate model of the actual performance of a building subjected to earthquake loads, but requires greater effort in terms of data preparation time and computational effort. The two most basic procedures, the Linear Static Procedure (LSP) and the Linear Dynamic Procedure (LDP), are mainly suitable for buildings which respond primarily in the elastic range. The Nonlinear Static Procedure (NSP) can evaluate buildings loaded beyond the elastic range but does not fully capture the dynamics of response, especially higher mode effects. The Nonlinear Dynamic Procedure (NDP) is the most complete form of analysis, modeling both dynamic effects and inelastic response. However, it is sensitive to modeling and ground motion assumptions.(Kelly & Chambers, 2000)

The pushover analysis is considered a nonlinear procedure used to predict the nonlinear behavior of structures, by exposing the structure to increasing lateral loads, until failure occurs. The finite element software SAP2000 generates the nonlinear behavior curve through incremental analysis assuming elastic behavior between each increment where plastic hinges form at each increment.

The resulting curve is called “capacity curve” whose shape and values depend on the stiffness, strength, sequence of plastic hinges formation, and ductility of the components of the structure. Typical capacity curve is shown in Figure 5 where each point on the curve represent a state of damage in the building. Performance levels are then defined by building codes (see FEMA356) as a certain point on this curve.

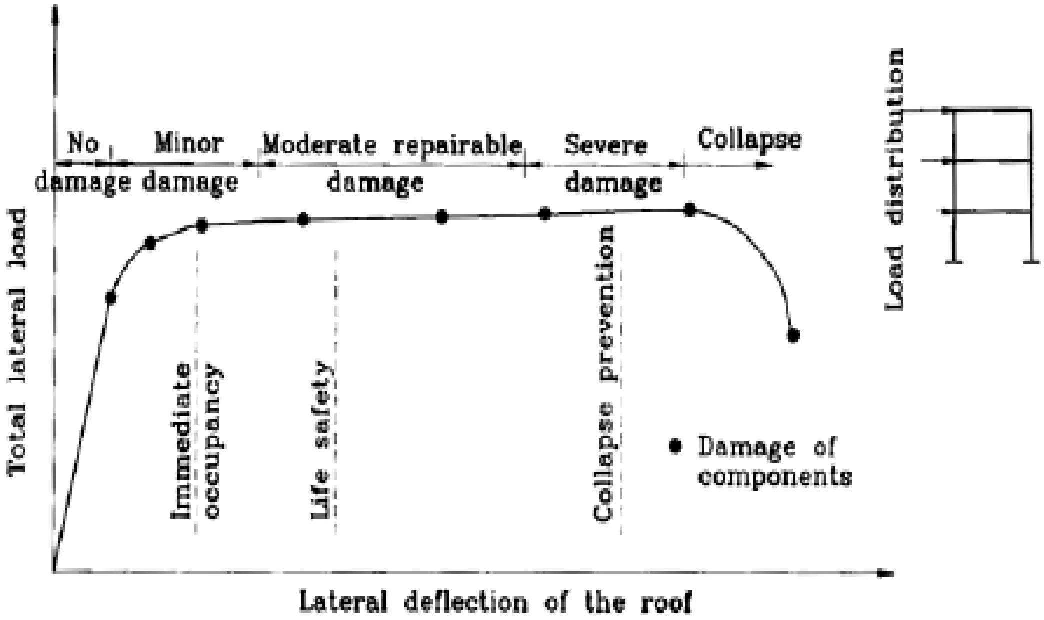


Figure 4: Typical Pushover Curve with Performance Levels

To be useful for earthquake design, the distribution of the pushover lateral loads must be related to the expected loads from the earthquake ground acceleration. Such distribution is typically selected based on modal shape from modal analysis of the structure. Usually the first modal shape, which is considered a fundamental mode, is used as a pattern for the distribution of the lateral pushover loads.

The pushover curve generally relates to the Roof displacement of the building (structure) versus base shear force. Top (Roof) displacement is taken at the middle of the roof mass in order to be relevant to the mass of the floor as an SDF equivalence.

The pushover analysis can be performed using either the force-controlled or the displacement-controlled procedure. The force-controlled is the best use for certain situations or cases where the capacity curve stays monotonic, like, for gravity loads the displacement controlled is the best use for pushover analysis with lateral loads where behavior is followed until failure. In this case, the curve may decline after a certain ultimate value (Shehadah, 2017).

The advantage of using the displacement-control can be observed with lateral loads in Figure (5). Due to the nature of the overall structural behavior, displacement is always increasing, but the load starts declining at ultimate point.

Therefore, in the displacement-control procedure, the structure is exposed to equal displacement increments and these increments can show the curve bending down. On the other hand, the load increments cannot show turning point of the curve in load-controlled analysis.

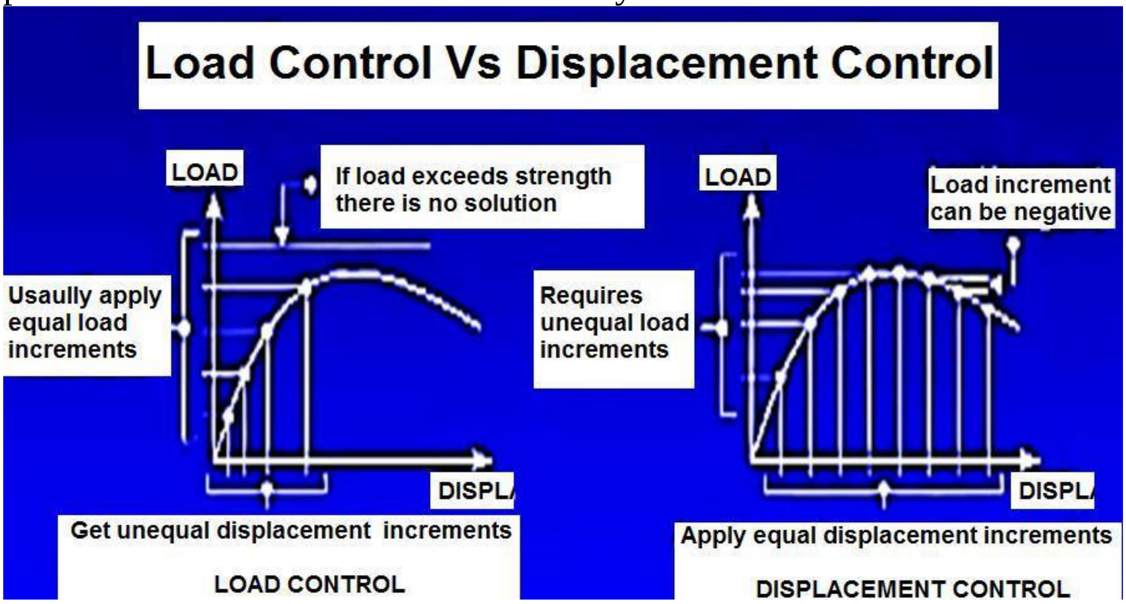


Figure 5: Load-Control versus Displacement-Control

2.4.1. Pushover Analysis Use

Pushover method is a very good procedure for tracking the realistic behavior of the existing structures under exposure of lateral loads. It is simple for calculation, straightforward, and clear in concept. Because of these advantages, it is favored than non-linear dynamic procedure (NDP). NDP is time consuming and complex to proceed.

Pushover analysis is preferred more than both linear static and linear dynamic procedures because these linear procedures do not show the real behavior of the structure and the effect of weak members in the structure. On the other hand, pushover analysis can show the following points for analyzed structure(Christiana, 2013):

- Regression of the general structure in each floor level.
- The demand forces on each member of the structure specially columns and beams.
- Location of weak elements of the structure. Collapse sequence through formation of plastic hinges.
- The effect of the failing members on the overall structure behavior.
- The irregularity of the strength and stiffness of the building in horizontal plane or vertically.
- The load path adequacy

2.4.2. Pushover Analysis Limitations

Pushover analysis is an effective and accurate procedure when compared with the elastic analysis, but the limitations of this method must be known and identified. These limitations are generally related to the selection of representative horizontal load patterns, and target displacement at the center of roof mass. Limitations of pushover analysis can be summarized in the following points:

- The top roof displacement is generally selected to represent (as SDOF) target displacement for a multi-DOF of the structure. However, if the structure is dominated by more than one mode shape, then the top roof displacement may not be a good indicator for the behavior of the overall structure. Therefore, this method is meaningful if the structure vibrates in a single dominant mode.
- Distribution of inertia forces are represented by the lateral loads, which are imposed on the structure during an earthquake. The distribution of inertial forces varies with the severity of earthquake and with time during earthquake due to varying acceleration and load reversal. However, usually in pushover analysis, an invariant lateral load pattern is used and the distribution of inertial forces is assumed to be constant during earthquake. Therefore, selection of load pattern distribution is as important as selection of target displacement.

The previous limitations made many researchers to try to adopt adaptive load patterns in order to consider changes in load pattern with the level of inelasticity. The improvements of these adaptive methods include the redistribution of the lateral load shape as a function of the current inelastic deformations. Also, other researchers tried to apply displacement loads instead of static force, in what is called (displacement-based pushover analysis). These new methods are not yet well developed for use.

However, even if these load patterns are invariant or adaptive, they are still static loads and cannot represent inelastic dynamic response with high accuracy. The above discussion about target displacement and lateral load pattern shows that pushover analysis supposes that the response of a building can be related to that of an equivalent SDOF system. This means that the building must be controlled by major fundamental mode even if with adaptive methods.

2.5. Modeling Nonlinear Behavior

The definition of the plastic hinges is the most important step in carrying out the push-over analysis. Plastic hinge is the zone inside the member where all strains become plastic due to stresses and hence the section start rotating under constant moment. Plastic hinges are defined through the load-deflection or moment-curvature curves that govern the behavior of the member and/or the cross-section under increasing member forces.

Generally, there are two approaches for modeling the inelastic behavior of the members. The continuous plasticity approach, where the plastic zone is allowed to have varied properties along the member wherever behavior becomes inelastic. The other approach is the discrete or concentrated approach, where the inelastic behavior is considered to occur at a single point on the member, and that location is called the "plastic hinge" where all plastic deformation is assumed to take place.

The continuous models are more accurate than the concentrated models in capturing the inelastic behavior of the members, however, such models are more complex and computationally demanding and this makes it not favorable to be used in large structures. Therefore, many researchers prefer the use of concentrated models where plastic deformations are assumed to occur at the location of local maximum internal force in the member.

The load-deflection curves for the plastic hinges are dependent on the type of loading on the member (whether axial, moment or shear) and the level of ductility available in the member, which in turn depends on the level of detailing and reinforcement of the member. Obtaining such curves is quite difficult for each member, however, some codes provide tools that quantify this behavior through idealized curves.

FEMA 356 provides parameterized curves for the behavior of the plastic hinges under different loading scenarios and detailing levels. These tables and curves (Table 6-8, and Fig. 6) are quite useful as they simplify the modelling process and also are specifically established for design checks wherever the performance of the member needs to be obtained.

FEMA 356 classifies the structural types by materials, such as steel, concrete, masonry, wood and light metal framing. For each structural type, FEMA 356 describes the procedure for evaluating seismic performance based on member-level limits.

Figure (6) shows the general capacity curve parameters and numerical acceptance criteria for RC beams, RC columns, and RC beam-column joints.

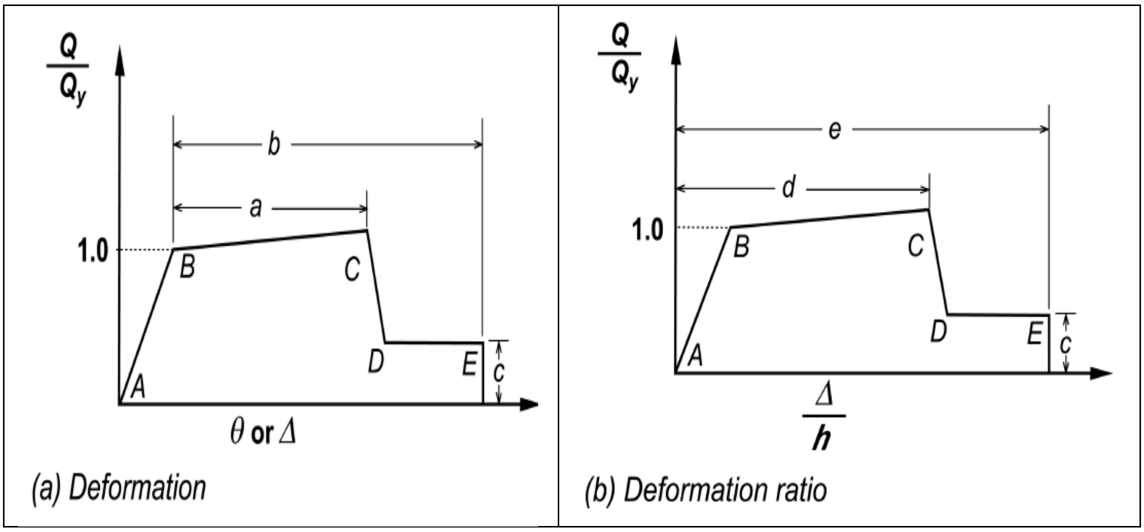


Figure 6 : The Force-Deformation Relations for Concrete Components (FEMA356, 2000)

Figure (6) (Force-Deformation) for plastic hinge shows that point A is the unloaded condition and point B represents yielding of the element. The ordinate at C is the nominal strength and abscissa at C is the deformation at which significant strength degradation begins. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained.

Table6: Numerical Acceptance Criteria and Modeling Parameters for Non-linear Procedures According to FEMA 356 for RC beams (ASCE, 2000)

Conditions			Modeling parameters[3]			Acceptance criteria[3]			
			Plastic rotation angle, radians		Residual strength ratio	Plastic rotation angle, radia			
						Performance level			
							Component type		
							Primary		Secon
			a	b	c	IO	LS	CP	LS
i. Beams controlled by flexure[1]									
$\frac{\rho-\rho'}{\rho_{bal}}$	Transverse Reinforce-ment[2]	$\frac{V}{b_w d\sqrt{f'_c}}$							
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.01	0.02	0.025	0.02
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005
ii. Beams controlled by shear[1]									
Stirrup spacing $\leq$ d/2			0.003	0.02	0.2	0.0015	0.002	0.003	0.1
Stirrup spacing $\geq$ d/2			0.003	0.01	0.2	0.0015	0.002	0.003	0.005
iii. Beams controlled by inadequate development or splicing along the span[1]									
Stirrup spacing $\leq$ d/2			0.003	0.02	0	0.0015	0.002	0.003	0.1
Stirrup spacing $\geq$ d/2			0.003	0.01	0	0.0015	0.002	0.003	0.005
iv. Beams controlled by inadequate embedment into beam-column joint[1]									
			0.015	0.03	0.2	0.01	0.01	0.015	0.02

Notes:

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the m appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcer component is conforming if, within the flexural plastic hinge region, hoops are spaced at $\leq$ d/3 for components of moderate and high ductility demand, the strength provided by the hoops (' least three-fourths of the design shear. Otherwise, the component is considered nonconforming
3. Linear interpolation between values listed in the table shall be permitted.

Table7: Numerical Acceptance Criteria and Modeling Parameters for Non-linear Procedures According to FEMA 356 for RC columns (ASCE, 2000)

Conditions			Modeling parameters[4]			Acceptance criteria[4]				
			Plastic rotation angle, radians		Residual strength ratio	Plastic rotation angle, radians				
						Performance level				
							Component type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Transverse Reinforcement[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1,3]										
All cases[5]			-	-	-	-	-	-	0.003	0.004
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing $\leq d/2$			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing $\geq d/2$			0	0.01	0.2	0	0	0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1,3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0	0.005	0.01	0.01	0.02
All other cases			0	0	0	0	0	0	0	0

Notes:

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at $\leq d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.
5. For columns controlled by shear, see Section 6.5.2.4.2 for acceptance criteria.

Table8 : Numerical Acceptance Criteria and Modeling Parameters for Non-linear Procedures According to FEMA 356 for RC beam-column joints(ASCE, 2000)

Conditions			Modeling parameters[4]			Acceptance criteria[4]				
			Plastic rotation angle, radians		Residual strength ratio	Plastic rotation angle, radians				
						Performance level				
							Component type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Interior joints[2,3]										
$\frac{P}{A_g f'_c}$	Transverse Reinforcement	$\frac{V}{V_n}$ [3]								
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2	0	0	0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2	0	0	0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0	0	0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.015	0.02	0.2	0	0	0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.005	0.02	0.2	0	0	0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2	0	0	0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2	0	0	0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2	0	0	0	0.01	0.015
ii. Other joints[2,3]										
$\frac{P}{A_g f'_c}$	Transverse Reinforce-ment[1]	$\frac{V}{V_n}$								
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2	0	0	0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2	0	0	0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2	0	0	0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2	0	0	0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2	0	0	0	0.0075	0.01
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2	0	0	0	0.0075	0.01
≥ 0.4	NC	≤ 1.2	0	0	-	0	0	0	0.005	0.007
≥ 0.4	NC	≥ 1.5	0	0	-	0	0	0	0.005	0.007

Notes:

1. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A jc is conforming if hoops are spaced at $\le h_c/3$ within the joint. Otherwise, the component is conside nonconforming.
2. P is the design axial force on the column above the joint and A_g is the gross cross-sectional area of joint.
3. V is the design shear force and V_n is the shear strength for the joint. The design shear force and sh strength shall be calculated according to Section 6.5.2.3.
4. Linear interpolation between values listed in the table shall be permitted.

Chapter Three

Case Study Description and Modeling Features

To study the effect of column orientation distribution on the R-Factor a case study building is assumed, this building will serve as a realistic vehicle to compare the result of R-Factor to the provided in building codes.

The soil of the site is assumed as stiff soil according to the classification of ASCE 7-10.Stiff soil comes as "D" among soil classifications(ASCE7-10, 2010), see Table (9).`

Table 9: Soil Classification (ASCE7-10, 2010)

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the following characteristics: —Plasticity index $PI > 20$, —Moisture content $w \geq 40\%$, —Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft^2 = 0.0479 kN/m^2.

3.1. Architectural Description

The assumed building is a six-story RC-MRF building, where all floors consists of stores with 3.7m height, and a total height of 22.2m.

The building has a regular rectangular shape in the vertical projection, with 24m long and 14.5m wide. The Figures (7, 8, 9 and 10) show the top and front views of the building and the distribution of columns.

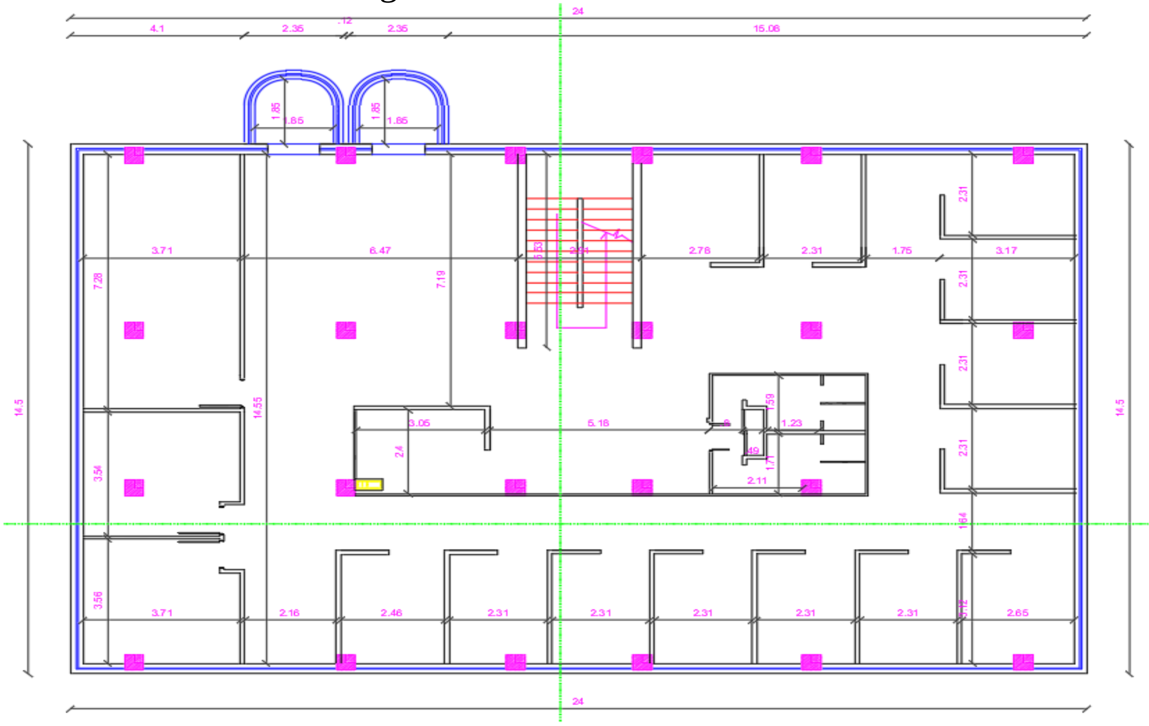


Figure 7: Ground Floor Plan and Repeated Floors 1, 2, 3, 4, and 5 Plan View

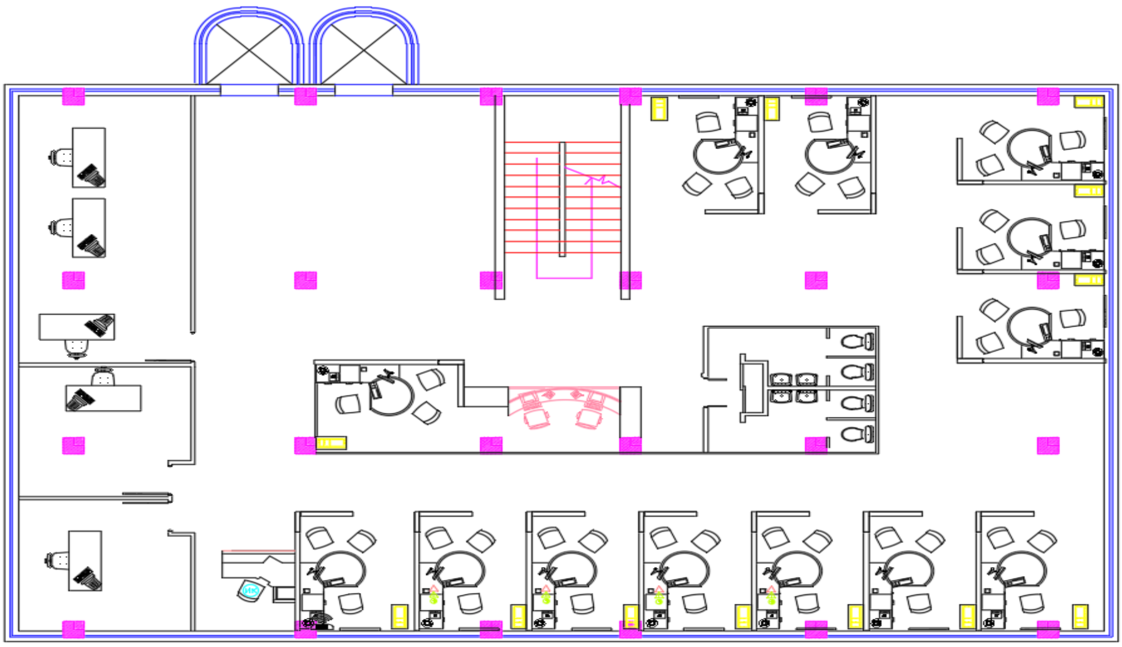


Figure 8: Typical Floor Furniture

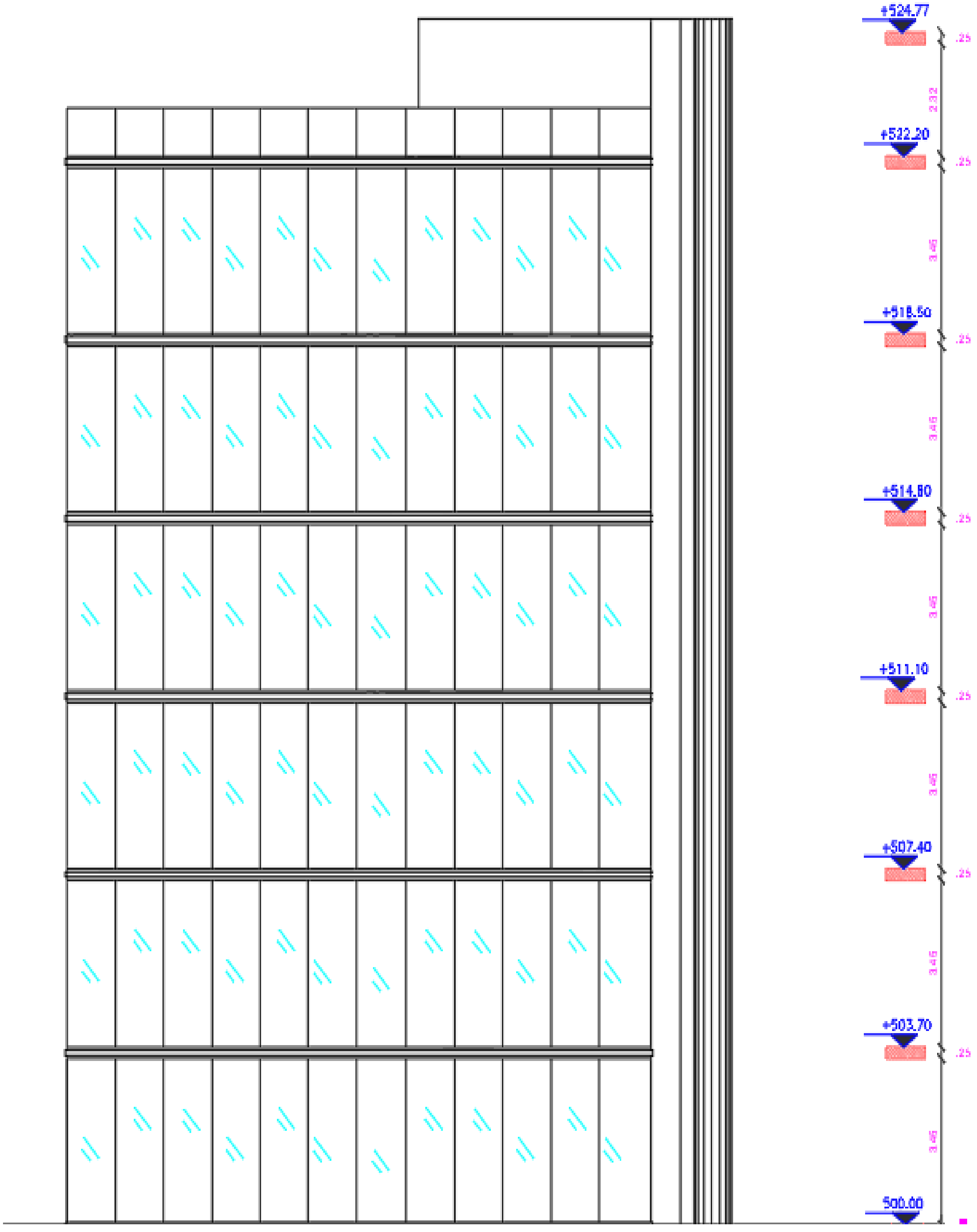


Figure 9: Elevation View of Case Study Building

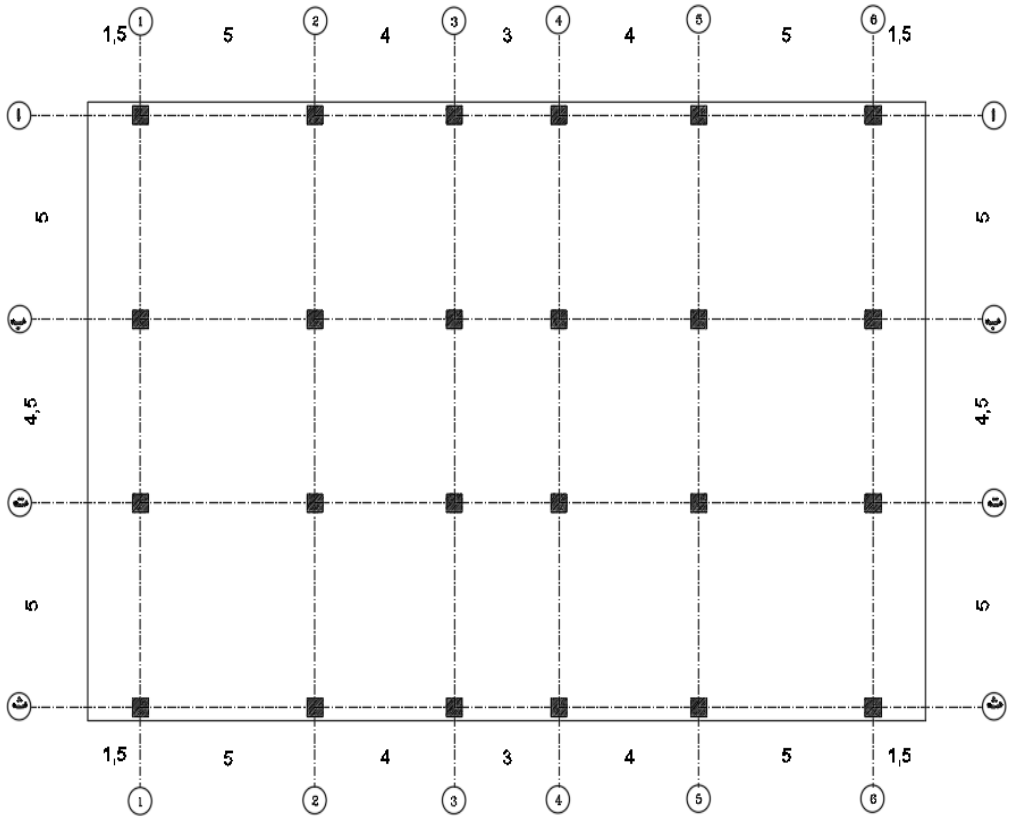


Figure 10: Columns Grid

3.2. Structural Details

The gravity loads are mainly transmitted by the main beams, then to columns then to foundations, which spread the loads into ground. The seismic loads are mainly taken by the frame systems consisting of beams and columns.

3.2.1. Structural systems

Both lateral and gravity loads are assumed to be transferred by frames. The selected building consists of one-way ribbed slab over of 25 cm depth. Beams are distributed in both principal directions. Main beams are designed in north-south direction, and the secondary beams are distributed in the other direction, see Figure (11).

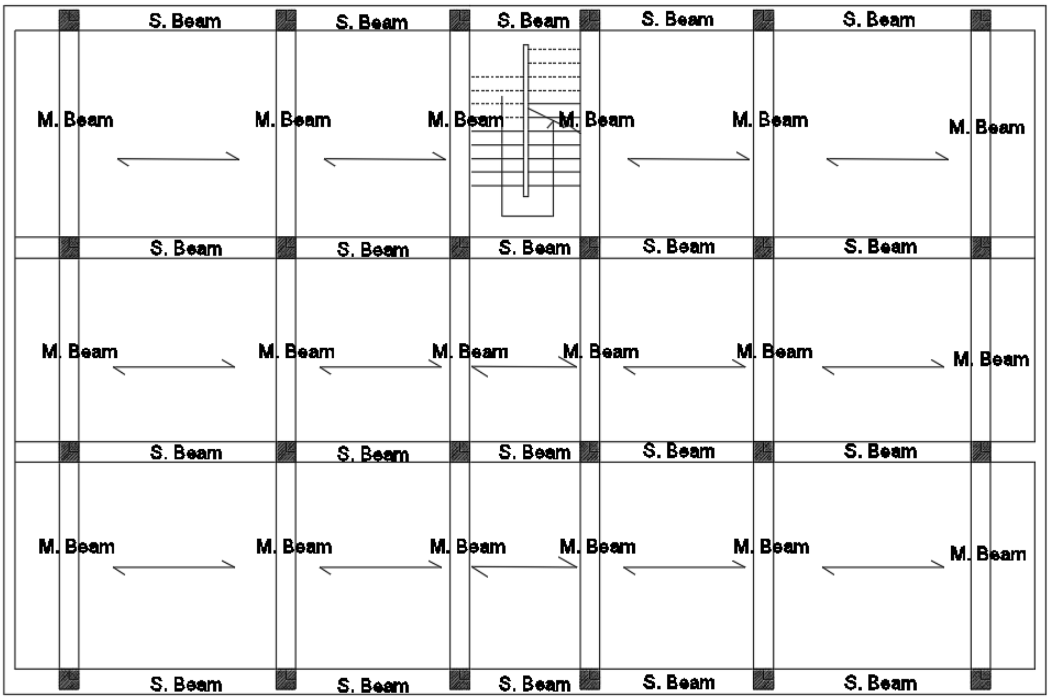


Figure 11: Beams Distribution

All columns are square columns of the same dimensions (0.45m*0.45m) and the same reinforcement. Stair case has four columns surrounding it, with no shear walls, but there are two block walls.

The foundation system shown in figure (12) consists of two strip footings and single footings connected to each other by tie beams 80cm deep and50cm wide.

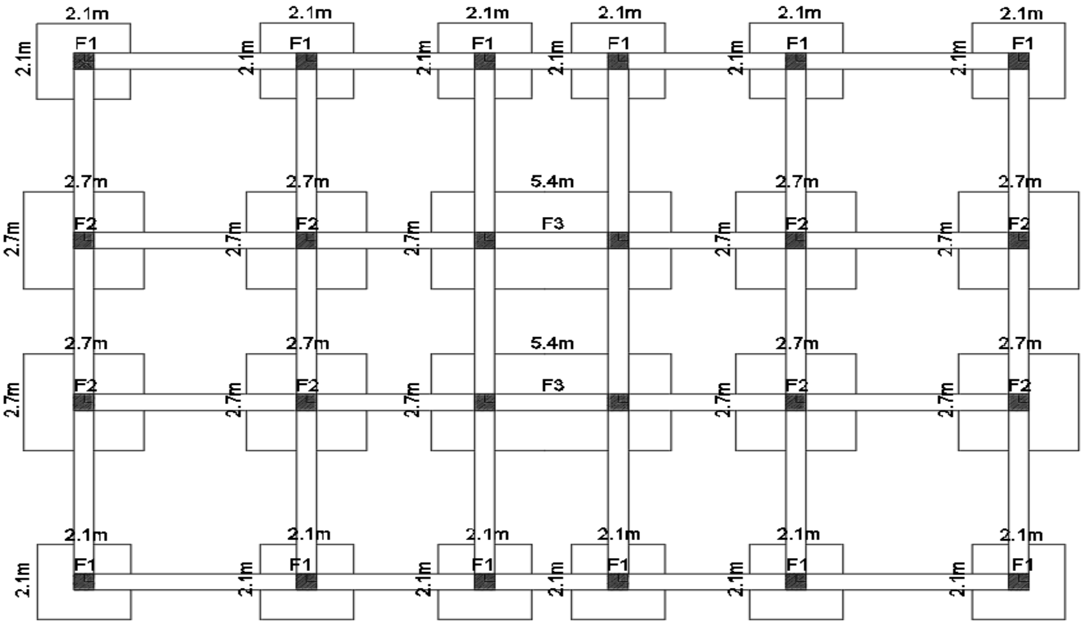


Figure 12: Foundation System

The characteristics of Structural elements are shown in details in Table (10).

Table 10: Characteristics of Structural Elements

Name	Width (cm)	Depth (cm)	Longitudinal reinforcement		Ties
Columns	45	45	12 ϕ16mm		6ϕ8mm/m
Beams			Top reinforcement	Bottom reinforcement	
-Main beams	45	60	5 ϕ16mm	4ϕ16mm	6ϕ8mm/m
-Secondary Beams	45	60	5 ϕ16mm	4 ϕ16mm	6ϕ8mm/m

The cover for beams reinforcement is considered 4cm and for columns reinforcement is 4cm too.

3.3. Materials

The selected building is made of reinforced concrete. The concrete compressive strength f_c' differs for slabs, columns, footings and other structural elements. Concrete unit weight is 25 KN/m^3. The steel type used is ASTM A615 Grade 60. More characteristics of used materials is shown in Table (11).

Table 11: The characteristic of the used materials

Slabs & Beams Concrete		Columns & Footings Concrete	Steel ASTM A615 Gr60	
f_c' (cylinder) (MPa)	21	24	E (GPa)	200
E (GPa)	21.54	23.03	Poisson's ratio (v)	0.3
Poisson's ratio (v)	0.2	0.2	Min. yield strength Fy (MPa)	414
			Min. tensile strength Fu (MPa)	620
Unit weight (kN/m^3)	25	25	Specific weight (kN/m3)	77

The relationship between modulus of elasticity and compressive strength for concrete is taken according to the following empirical formula(ACI318-14, 2014).

$$E = 4700\sqrt{f_c'} \text{(in MPa)} \qquad 3.1$$

Where,

E= Elastic modulus Mpa.

f_c'= 28-day compressive strength Mpa.

3.4. Vertical Loads

The live loads were taken according to "ASCE standard ASCE/SEI 7-10". The super imposed dead load is calculated based on typical finishes in Palestine.(ASCE7-10, 2010).

The considered vertical loads are summarized in Table (12). Figure (13) shows slab cross sections used to calculate the own weight of slab.

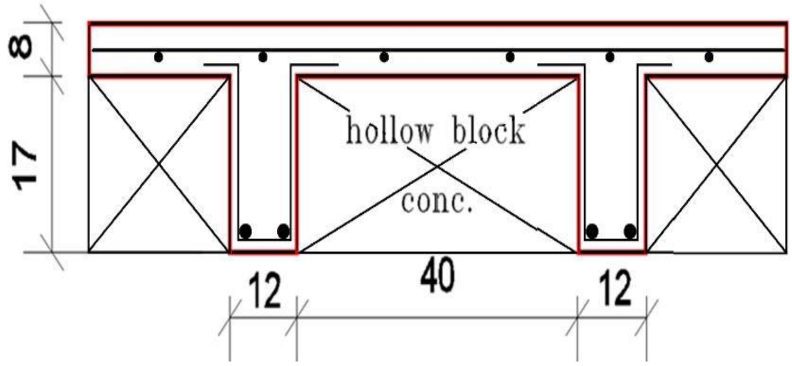


Figure 13: Slab Cross Section

Table 12: Summary of Adopted Vertical Loads

	Design load	Value	Unit
Dead Load	Slab own-weight	4.22	kN/m^2
	Super Imposed load	3.5	kN/m^2
Live	Commercial building	4.8	kN/m^2

3.5. Elastic Analysis and Checks

3.5.1. Gravity Loads Analysis

Gravity loads studied in this thesis include dead and live loads that were assigned Equilibrium check was made for the structural model that can be seen below

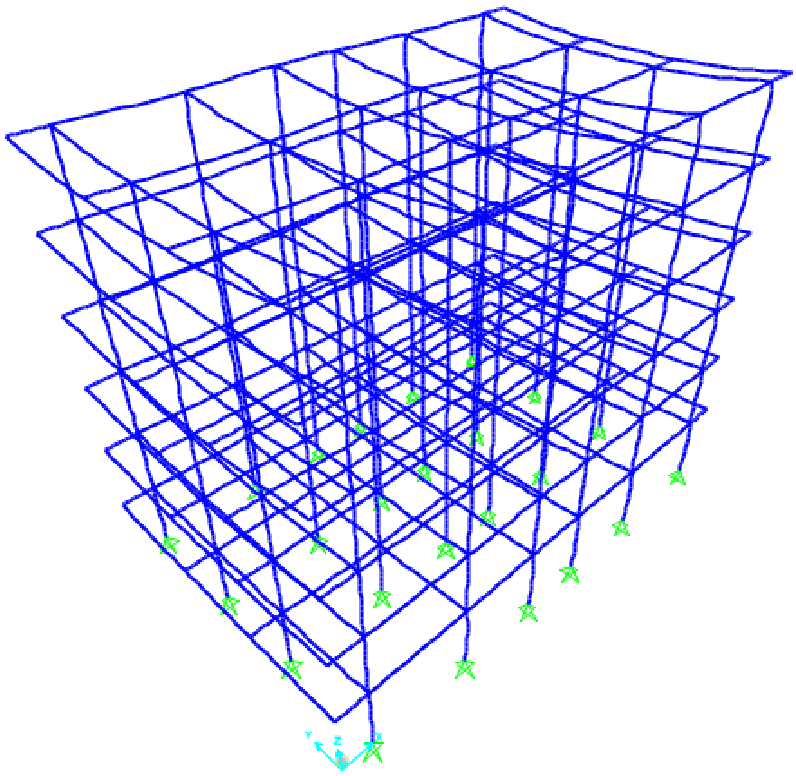


Figure 14 : Deformed 3D-model from Gravity Loads on SAP2000 Software(SAP, 2017).

In order to verify the model elastically it is required to determine and calculate manually the followings:

Compatibility: which means that all the structural members are connected together as assumed. It can be shown through the deformed shape and starting animation in the program.

Equilibrium: it can be approved by calculating the weight of structural elements and assigned loads then compare it with the base shear reaction.

Stress-strain relationship: It can be approved by calculating the moments and deformations manually and compare them to the program results for certain selected members.

Elastic period of the structure: this can be achieved by calculating effective mass and flexural stiffness for each floor. Then converting the MDOF system into Equivalent SDOF system to form an equation of motion through Rayleigh's method.

Table 13: Dimensions of Structural Elements

Name	Width (cm)	Depth (cm)
Columns of ground floor	45	45

The beams weights are assumed to be part of the slab area self-load, which is mentioned in table (14), which is 4.22 KN/m^2

Table 14: Loads Ratios and their Positions

<table>
<tr><td>Concrete</td><td colspan="3">ɣ (kN/m^3)</td><td>25</td></tr>
<tr><td rowspan="3">Slab</td><td colspan="3">Self-Load (kN/m^2)</td><td>4.22</td></tr>
<tr><td colspan="3">SIDL (kN/m^2)</td><td>3.50</td></tr>
<tr><td colspan="3">Slab Area (m^2) (14.5m*24m)</td><td>348</td></tr>
<tr><td rowspan="2">Columns</td><td colspan="3">No.</td><td>24</td></tr>
<tr><td colspan="3">Volume (m^3) (0.45*0.45*3.70)</td><td>1.07</td></tr>
<tr><td rowspan="2">Beams</td><td>Main beams</td><td>0.6*.45</td><td>6*14.5</td><td rowspan="2"></td></tr>
<tr><td>Secondary beams</td><td>0.6*.45</td><td>4*24</td></tr>
<tr><td>Live load</td><td colspan="3">Roof wt. (KN/m^2)</td><td>4.8</td></tr>
</table>

Table 15: Manual Calculations for Weights and their Positions in (kN) Unit

	Self-weights and added loads				Service Dead Loads	Live load	Ult. Loads
	Slab self	Slab SID	Columns	Beams			
GF	1426.8	1218	376.65	720.56	3742	1670.4	7163.04
F1	1426.8	1218	376.65	720.56	3742	1670.4	7163.04
F2	1426.8	1218	376.65	720.56	3742	1670.4	7163.04
F3	1426.8	1218	376.65	720.56	3742	1670.4	7163.04
F4	1426.8	1218	376.65	720.56	3742	1670.4	7163.04
F5	1426.8	1218	376.65	720.56	3742	1670.4	7163.04
SUM	8560.8	7308	2259.9	4323.36	22452	10022.4	42978.24

Table 16: Summary of Loads in Model

OutputCase	CaseType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m
DEAD	LinStatic	-5.185E-14	-2.043E-14	15102.407	109492.4537	-181228.889	7.816E-14
Live	LinStatic	-8.507E-15	-7.105E-15	10022.4	72662.4	-120268.8	8.527E-14
SIDL	LinStatic	-9.298E-15	-2.665E-14	7308	52983	-87696	-2.593E-13
Push X	LinStatic	-18.84	-5.64	2.33E-14	81.474	-271.876	68.91
Total Dead	Combination	-6.115E-14	-4.707E-14	22410.407	162475.4537	-268924.889	-1.812E-13
1.2/1.6	Combination	-8.699E-14	-6.786E-14	42928.329	311230.3844	-515139.95	-8.1E-14
Service Loa...	Combination	-6.965E-14	-5.418E-14	32432.807	235137.8537	-389193.69	-9.592E-14

Percentage of error in loads of model with respect to manual calculations:

% error of service dead loads = (22452-22410.407)/22452= 0.18%

% error of live loads = (10022.4-10022.4)/10022.4= 0%

Therefore, the calculated loads of the model don’t differ from manual calculated loads

Elastic period: in order to calculate the period of the structure, the mass matrix of the structure floors is calculated in equilibrium step. But the

Stiffness of each floor needs to be calculated and verified. The manual calculations of stiffness for each floor will be compared to program SAP2000 results by these steps:

Table 17: Calculating Stiffness for the Floors of Case Study Building, X-Direction

X-Direction								
Name	Width	Length	Height	I	I/L^3	Factor	Item No.	∑3*(EI/ L^3)
GF 1	0.45	0.45	3.1	0.0034	1E-04	1	24	1.94E+05
1	0.45	0.45	3.1	0.0034	1E-04	1	24	1.94E+05
2	0.45	0.45	3.1	0.0034	1E-04	1	24	1.93E+05
3	0.45	0.45	3.1	0.0034	1E-04	1	24	1.93E+05
4	0.45	0.45	3.1	0.0034	1E-04	1	24	1.94E+05
5	0.45	0.45	3.1	0.0034	1E-04	1	24	1.94E+05
Ecol.	2.35E+07							
							SUM	1162536.5

Table 18: Calculating Stiffness for the Floors of Case Study Building, Y-Direction

Y-Direction								
Name	Width	Length	Height	I	I/L^3	Factor	Item No.	∑3*(EI/ L^3)
GF 1	0.45	0.45	3.1	0.0034	1E-04	1	24	1.94E+05
1	0.45	0.45	3.1	0.0034	1E-04	1	24	1.94E+05
2	0.45	0.45	3.1	0.0034	1E-04	1	24	1.93E+05
3	0.45	0.45	3.1	0.0034	1E-04	1	24	1.93E+05
4	0.45	0.45	3.1	0.0034	1E-04	1	24	1.94E+05
5	0.45	0.45	3.1	0.0034	1E-04	1	24	1.94E+05
Ecol.	2.35E+07							
							SUM	1162536.5

Period verification using Rayleigh’s method:

Table 19: Rayleigh's Method (A)

Floor No.	F. Disp. Shape	F. mass	F. k-X-dir	F. k-Y-dir	L
6	1.00	356.89	1.94E+05	1.94E+05	1
5	0.95	356.89	1.94E+05	1.94E+05	1
4	0.88	356.89	1.94E+05	1.94E+05	1
3	0.77	356.89	1.94E+05	1.94E+05	1
2	0.64	356.89	1.94E+05	1.94E+05	1
1	0.46	356.89	1.94E+05	1.94E+05	1
0	0.00				

Table 20: Rayleigh's Method (B)

m	1388.66		
k-X-dir	54397.60	k-Y-dir	54397.6
L	1677.38		
ω-X-dir	6.26	ω-Y-dir	6.259
Tn-X-dir	1.00	Tn-Y-dir	1.00
Γ(L/m)	1.21		

Table 21: Comparing the Results of the Period.

	Rayleigh's method	SAP	Diff. %
Tn-X-dir	1	1.06	5.66
Tn-Y-dir	1	1.06	5.66

As seen, the differences between the base reactions in the model do not exceed 0.18% of the manual calculated values. Therefore, the model can be considered equivalent in gravity loads.

The design checks of the structural elements satisfy the gravity loads combination proposed by ASCE-10 to sustain the ultimate loads predicted in this equation:

1.4D

1.2D + 1.6L

3.5.2. Modal analysis

The goal of modal analysis is to determine the natural modal shapes and their frequencies for the structure during free vibration. The modal shapes are computed by SAP2000 using Eigen vectors method, which uses distribution of the mass and the stiffness of the structure. The output of the system is Eigenvectors and Eigen values that represent frequencies of the modal shapes of the structure. The lowest frequency indicates the fundamental mode.

The results of modal analysis and modal mass participation ratios for the model are shown in table (22).

Table 22: Modal Analysis and Modal Mass Participation Ratios Results

Modal Participating Mass Ratios

File View Edit Format-Filter-Sort Select Options

Units: As Noted Modal Participating Mass Ratios

Filter:

OutputCase	StepType Text	StepNum Unitless	Period Sec	UX Unitless	UY Unitless	SumUX Unitless	SumUY Unitless	RZ Unitless
MODAL	Mode	1	1.063145	1.828E-20	0.94234	1.828E-20	0.94234	0
MODAL	Mode	2	1.019691	0.94686	2.934E-20	0.94686	0.94234	0
MODAL	Mode	3	0.992497	3.705E-20	0	0.94686	0.94234	0.94873
MODAL	Mode	4	0.324497	5.438E-19	0.0476	0.94686	0.98994	1.732E-20
MODAL	Mode	5	0.311841	0.04377	7.412E-18	0.99064	0.98994	1.154E-19
MODAL	Mode	6	0.301805	1.697E-17	1.666E-17	0.99064	0.98994	0.04154
MODAL	Mode	7	0.249981	1.513E-16	2.353E-16	0.99064	0.98994	0.00016
MODAL	Mode	8	0.246713	1.237E-18	4.743E-07	0.99064	0.98994	1.53E-20
MODAL	Mode	9	0.175623	6.073E-18	0.00706	0.99064	0.997	4.829E-18
MODAL	Mode	10	0.174321	7.028E-16	6.752E-15	0.99064	0.997	0.00463
MODAL	Mode	11	0.173869	0.00634	7.716E-17	0.99698	0.997	2.012E-18
MODAL	Mode	12	0.169075	2.132E-17	0.00018	0.99698	0.99718	1.183E-17

3.6. Modeling Pushover Analysis

In order to do the pushover analysis, the inelastic behavior of the elements must be defined first. The software "SAP2000" is a common Finite

Element (F.E) program that is easy and simple to be used and captures the plastic behavior through concentrated plasticity approach. In this approach, a single point called plastic hinge wherein all plastic deformation is concentrated represents the inelastic zone in the element. Therefore, the behavior of such plastic hinge must be identified.

3.6.1. Definition of plastic hinges

There are two methods to define the properties of plastic hinges by SAP2000: the manual method and the automatic method. The automatic characteristics are defined according to FEMA 356 tables and Caltrans standards.

Plastic hinges appear usually at the ends of the beams and columns, where is moment is maximum, or under concentrated loads, or at fixed supports because of exceeding the yielding point. The main cause of the appearance of plastic hinges in the beams is the bending moment. As for the columns, the main cause that leads to plastic hinge is the interaction of the axial force (P) with the moments (M). Therefore, two types of elements are needed for plastic hinges (P.H) definition:

- Beam elements: Flexure is the main cause of plastic hinge appearance as mentioned before; therefore, automatic flexural hinges will be assigned at the end of the beams as shown in Figure (15).

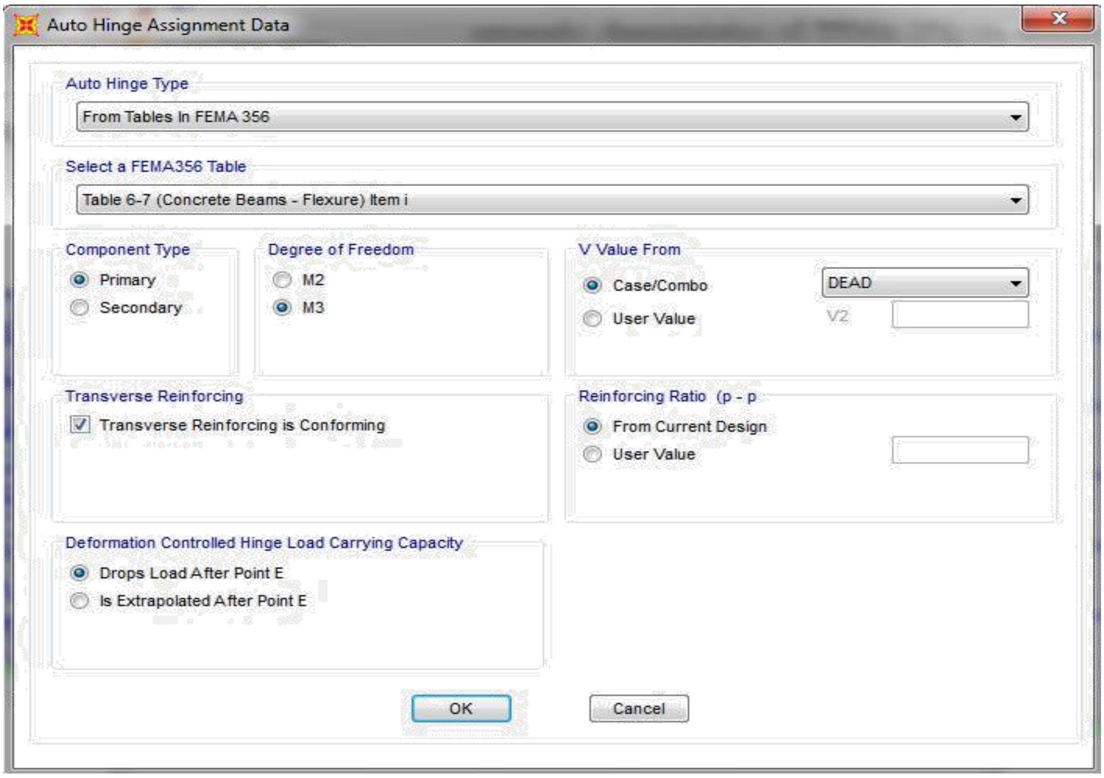


Figure 15: Assign Plastic Hinges for Beams

- Columns: columns are exposed to axial loads and moments, and these loads are the cause of plastic hinges occurrence. Therefore, automatic (P-M) hinges will be assigned at the columns ends as shown in Figure (16).

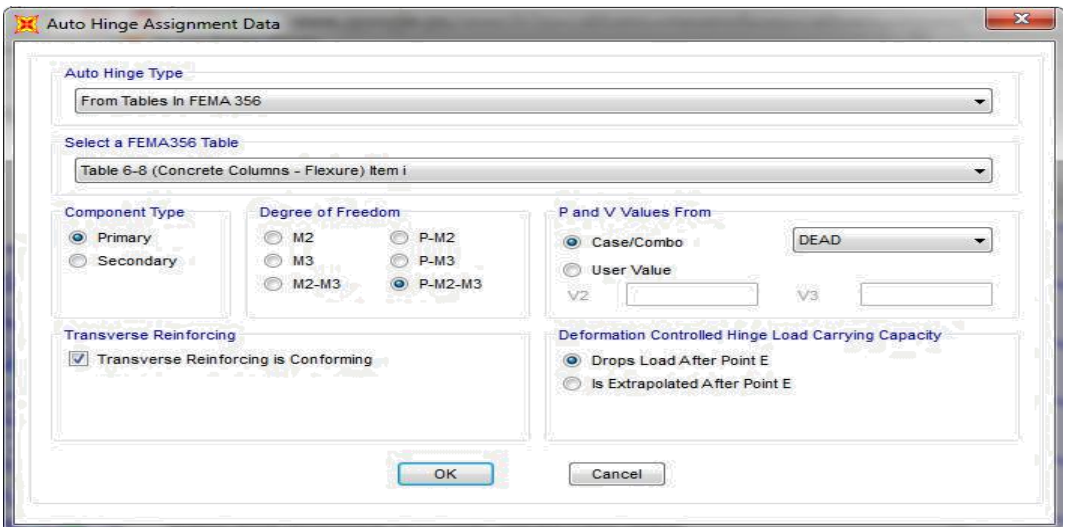


Figure 16: Assign plastic hinges for columns.

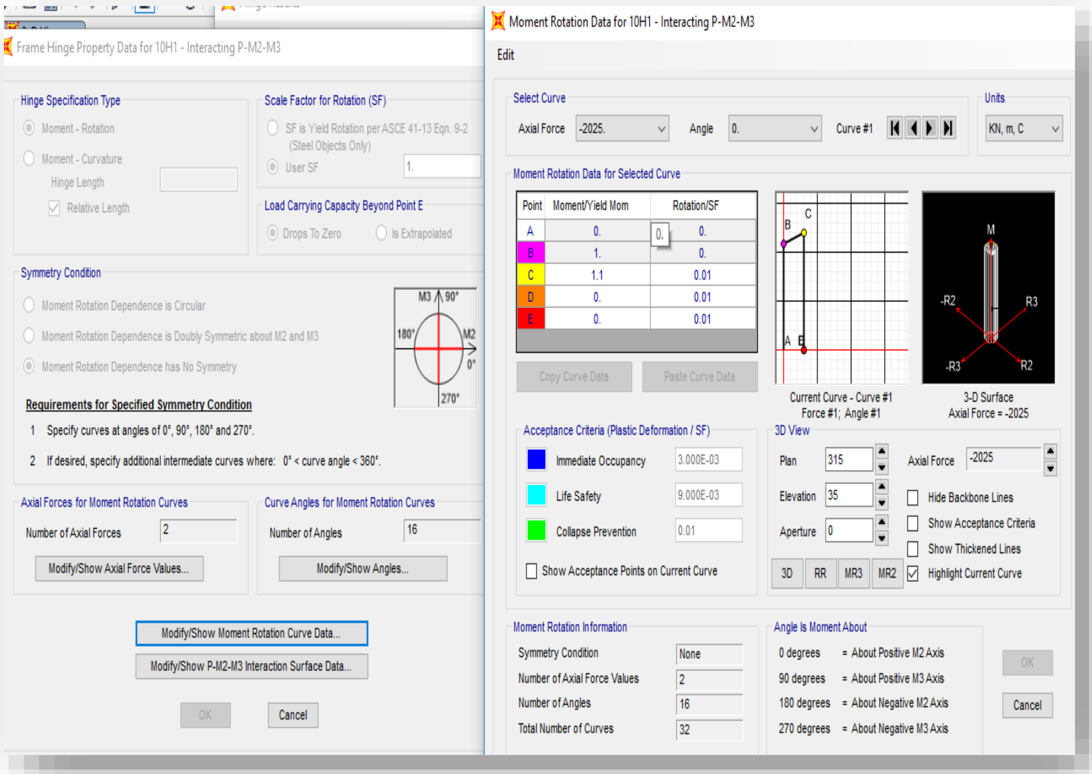


Figure 17: Generated properties by FEMA356 criteria of column section.

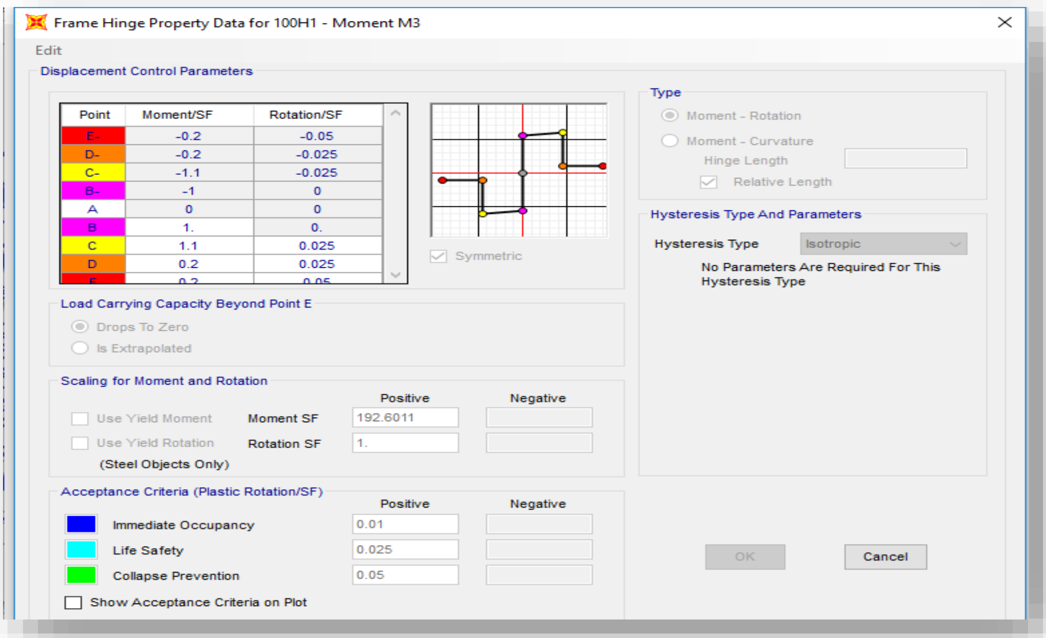


Figure 18: Generated Properties by FEMA356 Criteria of Beam Section

3.6.2. Loads

The loads affecting the structure are divided into gravity loads and lateral loads, and these loads are transmitted through structural elements to the ground. Gravity loads are mainly transmitted by the slabs, to beams, then through columns and shear walls to footings and finally to ground. Beams are exposed to flexure, shear, and torsion while transmitting loads to vertical components (columns and shear walls).

2.11.2.1 Defining Initial Load Conditions for Pushover Analysis

The gravity loads must be accounted for as initial loading condition before the seismic loads start to take effect on the structure. The gravity loads include the dead loads (structure weight and super imposed loads) and live loads (IBC, 2012).

The gravity loads are defined as nonlinear load case and the full gravity loads are applied during this stage, except live load, where only 25% participation ratio is assumed.

3.6.2.2. Lateral load patterns

FEMA356 method proposes that pushover analyses are performed by using lateral load patterns related to fundamental mode shapes, in order to be as close as possible to the expected earthquake load distribution. Table (23) below displays the modal shape vectors for the two translational fundamental modes, and normalized by the top floor drift.

Table 23: Normalization of fundamental mode shape vectors for models

ØX	ØY	Normalize X	Normalize Y
0.0262	0.0263	1.00	1.00
0.0251	0.0251	0.96	0.96
0.0231	0.023	0.88	0.88
0.0203	0.0201	0.77	0.77
0.0168	0.0165	0.64	0.63
0.0121	0.0118	0.46	0.45

The lateral load patterns that will be used for pushover analysis are defined in SAP2000 as shown in Figure (18).

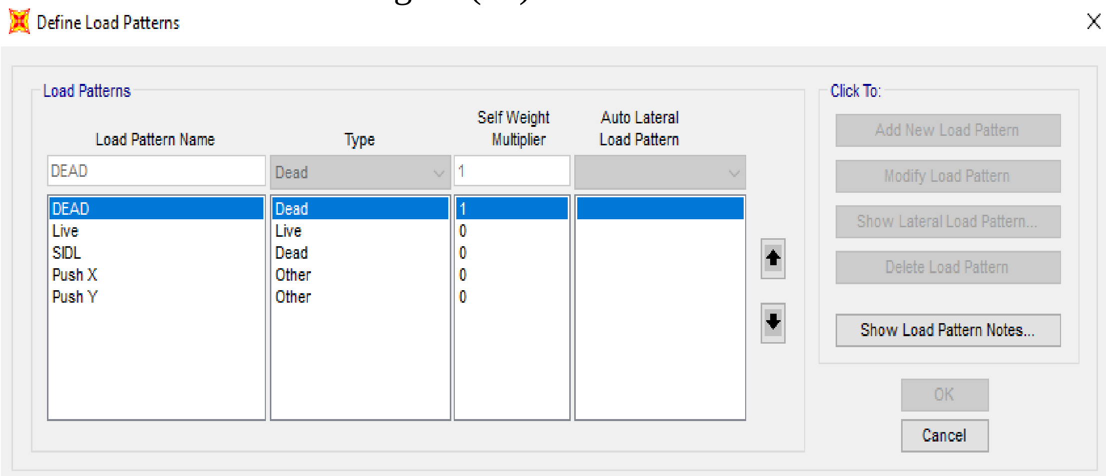


Figure 19: Load Pattern for Lateral Loads

The next step is to assign lateral loads on the models. The loads are assigned on each floor according to the modal shapes at the joints connecting beams with columns such that the center of the loads match with center of mass for each floor.

3.6.2.3. Define Load Cases for Pushover

The lateral loads assigned in the previous step are to be defined in a pushover load case, and this pushover load case should continue after initial load case, which is (DL nonlinear).

The analysis in pushover is displacement controlled with monitoring the roof floor movement. The maximum allowable displacement of the master node is assigned 1m, where the analysis stops.

3.7. Results of pushover analysis

3.7.1. Introduction

In this section, the results of the pushover analysis will be presented for the proposed model of the case study building. The results will be in terms of base shear force versus the top displacement of the building. These results represent the behavior of the building under the influence of lateral forces within the assumptions that were explained earlier. The results can be used to calculate Response Modification Factor (R) and either for showing weaknesses and collapse sequence in the building through the shape of the curve and distribution of resulting plastic hinges and the amount of displacement.

3.7.2. Base Shear vs. Top Displacement

Base shear force and top displacement (P-Δ) was taken as a result of the pushover analysis using SAP2000 program for the model. The following figures (20 and 21) shows the (P-Δ) in X-directions for the model. The performance points for the model is also shown in the figure. This point calculated automatically by using SAP2000 according to ATC-40 iterative procedure(ATC40, 1996).

The performance point represents the point where the expected lateral force (demand from response spectra curve) coincides with the expected capacity from load deflection curve of the building.

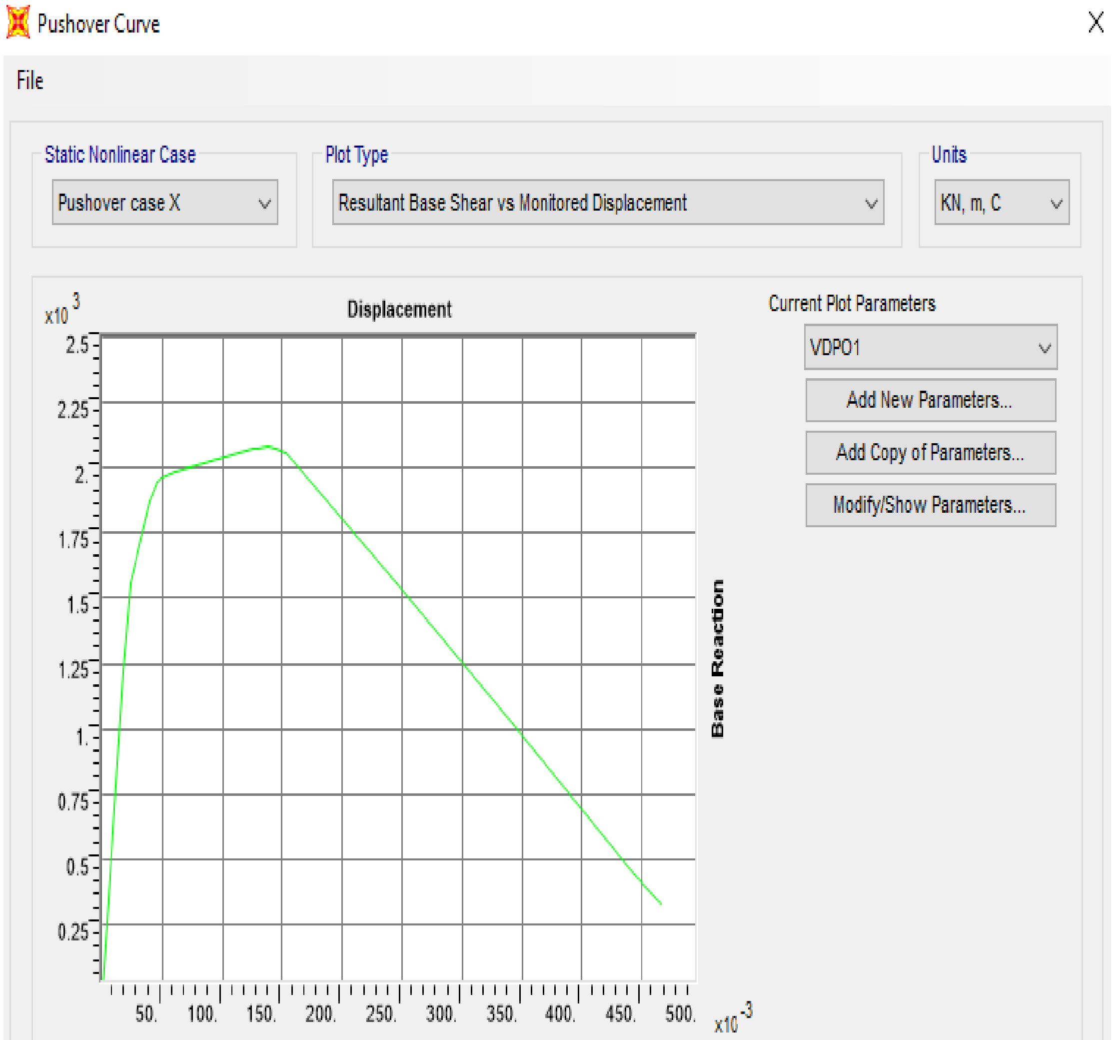


Figure 20: Model Pushover Curve in terms (P-Δ)

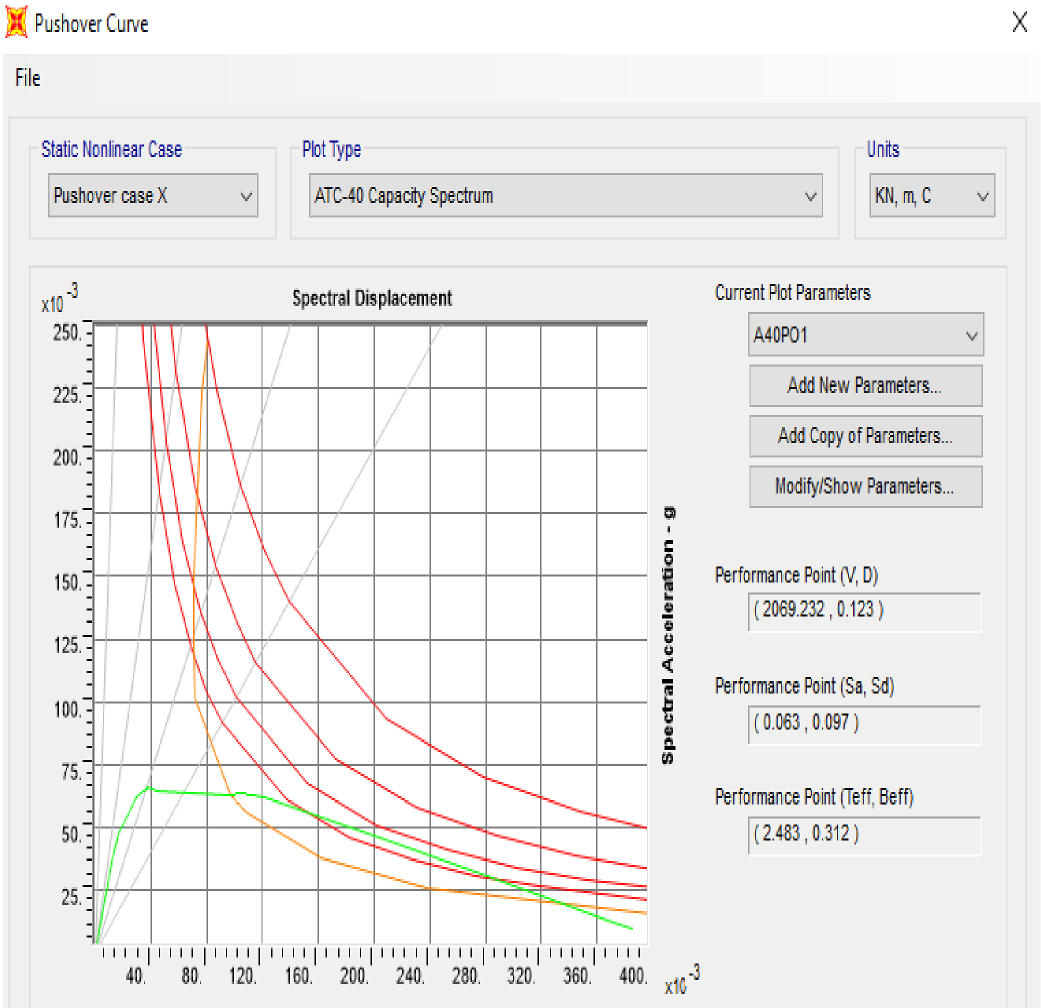


Figure 21: Model Pushover Curve & Performance Point

From figure 21 the performance point is the point that located on the intersect of the capacity curve (green curve) and the demand curve (orange curve).

From the push over curve we can calculate response modification factor (R-Factor) which equal to: Elastic Force ($V_{elastic}$) / Yielding Force (V_{yield})

From the curve above $V_{elastic}$ = 8232.496 kN

To determine V_{yield} there are two methods that can be used:

- Method (1): from first yield point and this gives:

  V_{yield} = 1559.489 KN. Then, response modification factor

  (R) = ($V_{elastic}$) / (V_{yield}) =8232.496/1559.489 = 5.279

- Method (2): from FEMA bilinear curve and this gives:

  V_{yield} = 1672.43 KN. Then, response modification factor

  (R) = ($V_{elastic}$) / (V_{yield}) =8232.496/1672.43 = 4.922

Previous results show that the R-Value depending on V_{yield} taken from FEMA bilinear curve is smaller than that taken from first yield point and this is because the value of V_{yield} from FEMA is greater than V_{yield} from first yield.

According to IBC 2012 code for reinforced concrete intermediate moment resisting frame structures R = 5.

In the next chapter a parametric study is conducted to evaluate the effect of column disorientations with R-Factor for intermediate RC-MRF system.

Chapter Four

Analysis &Estimation of R-Factors for Prototype Buildings

4.1. introduction

To study the effect of column orientation on the R-Factor of framed R.C Structures, two building layouts were selected and used to generate prototype models that cover many scenarios of parameters. The two layouts represent two situations, one with square uniform properties in both direction and the other is a rectangular layout with two district principle directions.

These two layouts were analyzed and designed according to ACI-318 code and then pushover analysis was done to generate load-deflection curves for different cases of floor numbers and column orientations. This chapter presents the models and the resulting curves for these cases and shows how these curves are used to compute R-Factors. Then a comparison of the reality R-Factors is shown.

4.2. Simulated Buildings Layouts:

4.2.1. Square Layout:

Three buildings of square layout were used with 4, 8 and 12 storey, each with 3 bays on each side, the bay width is 4m, and the storey height is 4m.

The column was varied starting with square to rectangular shape in two orientations as shown in Figures 22-23 and Table 24.

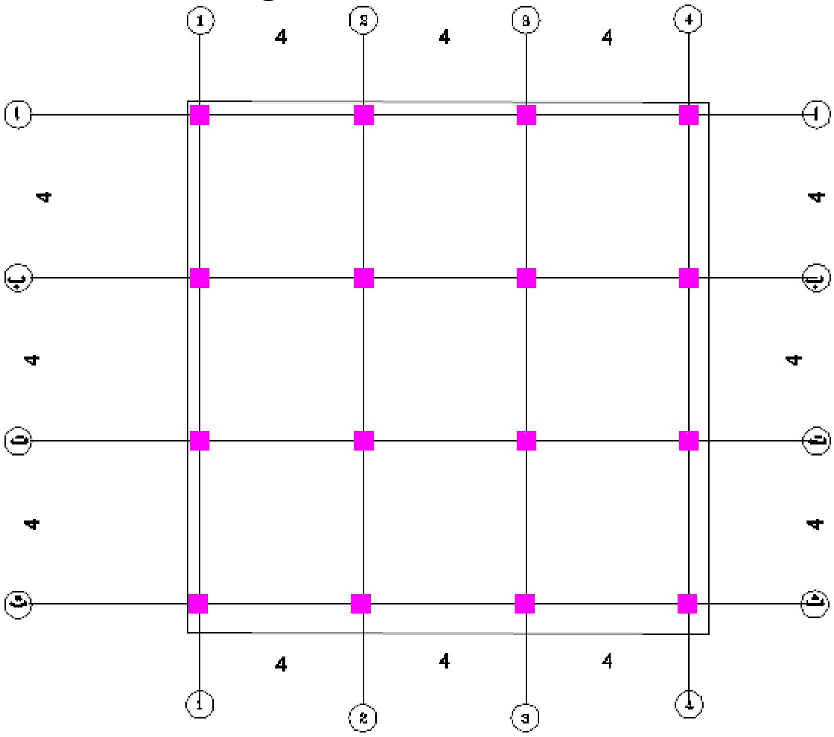
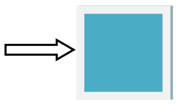
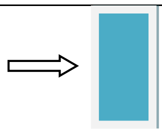
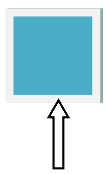
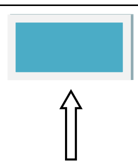
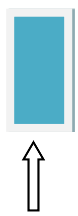


Figure 22: Columns Grid for all Square Shape Prototypes

Because the direction of the earthquake force is generally unknown, building codes allow for assuming that the quake force to be applied in the principal directions of the building. The ASCE/IBC2016 codes demand that the earthquake force to be simultaneously applied in both principal directions. The primary or main direction is assumed to receive all the earthquake load, while the other secondary direction receives only one third of the peak earthquake load expected in that direction. This gives two scenarios for loading the structures. These loading scenarios are indicated by X and Y in Table 24. For the X-axis case, the primary load is applied in full in the strong, (also known as principal or main) direction of the building while 30% is applied in the weak direction. For the case Y, the full force is applied in the weak direction while 30% is applied in the strong direction

Three categories of columns orientations were used to see their effect on R-Factor values as shown in Table 24. For example, Case 1A-S or R means (1: the column is square, A: the primary load is applied in X direction and S or R: square building layout or rectangular building layout).

Table 24: Columns Orientations for all Prototypes.

Category	Description
Case: 1A-S or R 	Square columns and the main load direction in X axis
Case: 2A-S or R 	Rectangular columns in X axis direction and the main load direction in X axis
Case: 3A-S or R 	Rectangular columns in Y axis and the main load direction in X axis
Case: 1B-S or R 	Square columns and the main load direction in Y axis for
Case: 2B-S or R 	Rectangular columns in X axis direction and the main load direction in Y axis
Case: 3B-S or R 	Rectangular columns in Y axis and the main load direction in Y axis

4.2.2. Rectangular RC MRF Building Layout:

A rectangular intermediate RC MRFs building layout is used to create three buildings of 4, 8 and 12 storey's for the analyses as shown in Figures 22 and 23.

There are many factors that affect the value of response modification factor (R) and in this thesis we focus on three variables which are: the columns orientations, the number of storey's and building layout.

The building layout factor can tell us about the possible effect of the building overall lateral stiffness as framed structure as compare to individual column stiffness on the R-Factor. The column orientation factor covers the effect of both stiffness and the strength of the columns with the frames. The number of storey factor covers the effect of overall building flexibility on the fundamental period and hence on the expected seismic force.

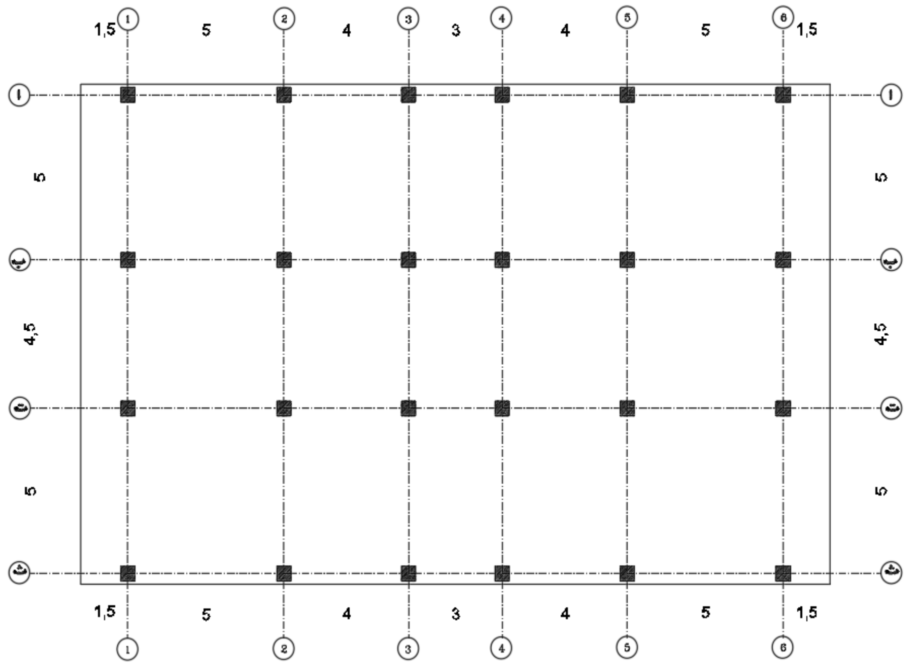
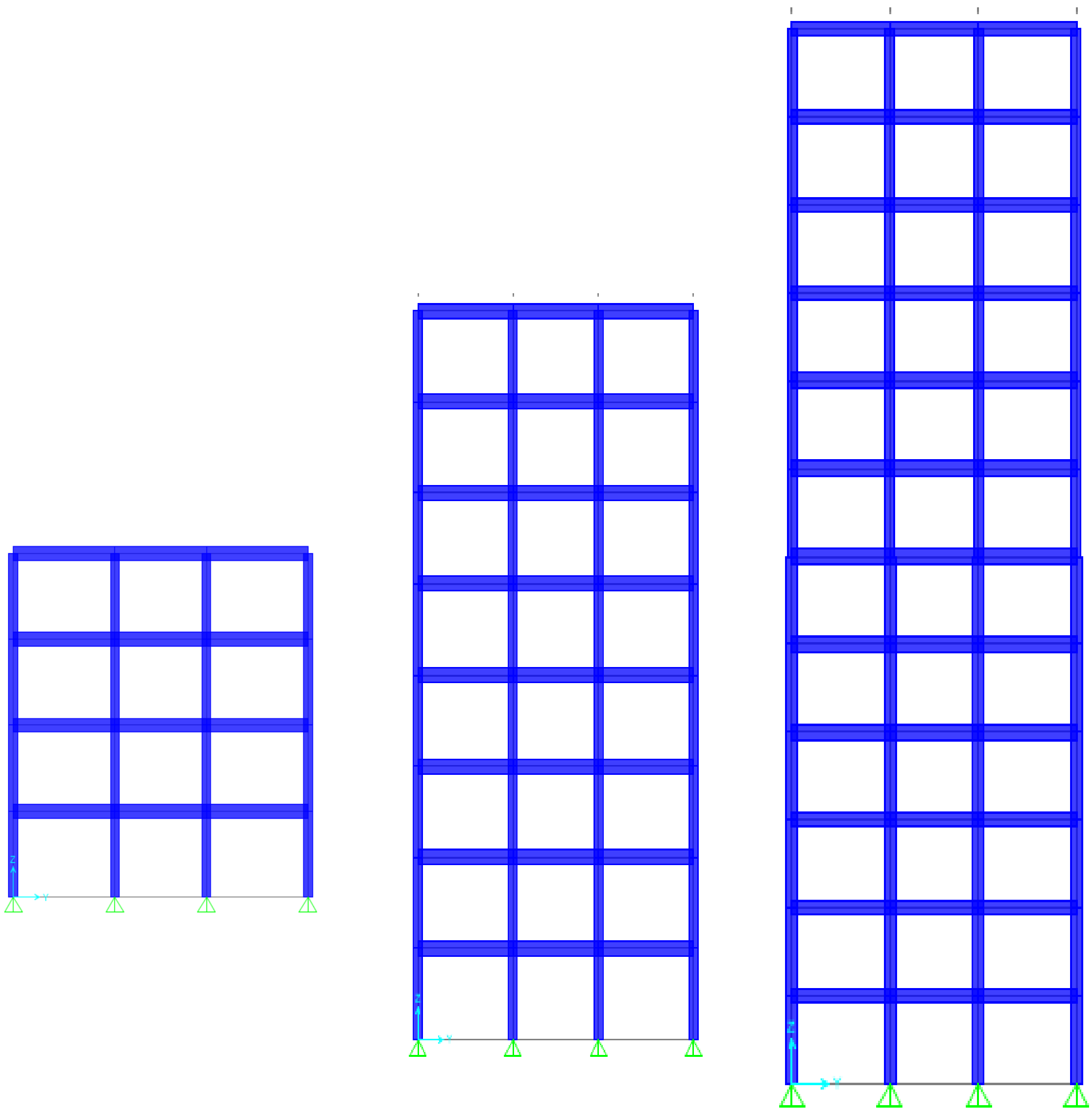


Figure 23: Columns Grid for all Rectangular Shape Prototypes.



4 Storey’s Model 8 Storey’s Model 12 Storey’s Model

Figure 24: Number of Storey’s

4.3. Generating the Pushover Curves:

The push over load-deflection curves are necessary to compute the R-Factor. To obtain these curves, the following procedure is used for all models. First, every model building is designed in accordance with the

ACI-318 code as an intermediate moment resisting frame. Code requirements for detailing of reinforcement is followed and the code-specified value of R = 5 was used in the design process. Then the plastic hinges are defined for each member according to FEMA 356 definitions. These hinges include flexural hinges for beams and flexural-axial hinges for columns. Then for each model, a modal analysis is performed where the fundamental modal shapes are obtained in each direction. The modal shapes are used to define the load distribution along the building. After that, a primary (or main) direction is selected. The primary (main) direction is the direction that is assumed to be exposed to the full seismic excitation. In contrast, the secondary direction is assumed to have a portion of the seismic excitation. To specify the portion of earthquake load that goes into the secondary direction, we follow the ASCE7/IBC2012 code specifications, which allow for 30% of the earthquake load to simultaneously affect the building in the secondary direction. Therefore, a loading scenario is presumed where the lateral loads in both directions are applied simultaneously but keeping the ratio of 1:1/3 fixed all over the push over analysis. This gives a load-deflection curve for each model. Then the loading scenario is switched where the primary direction becomes secondary and the secondary becomes primary.

The following figures show the pushover curves for all models.

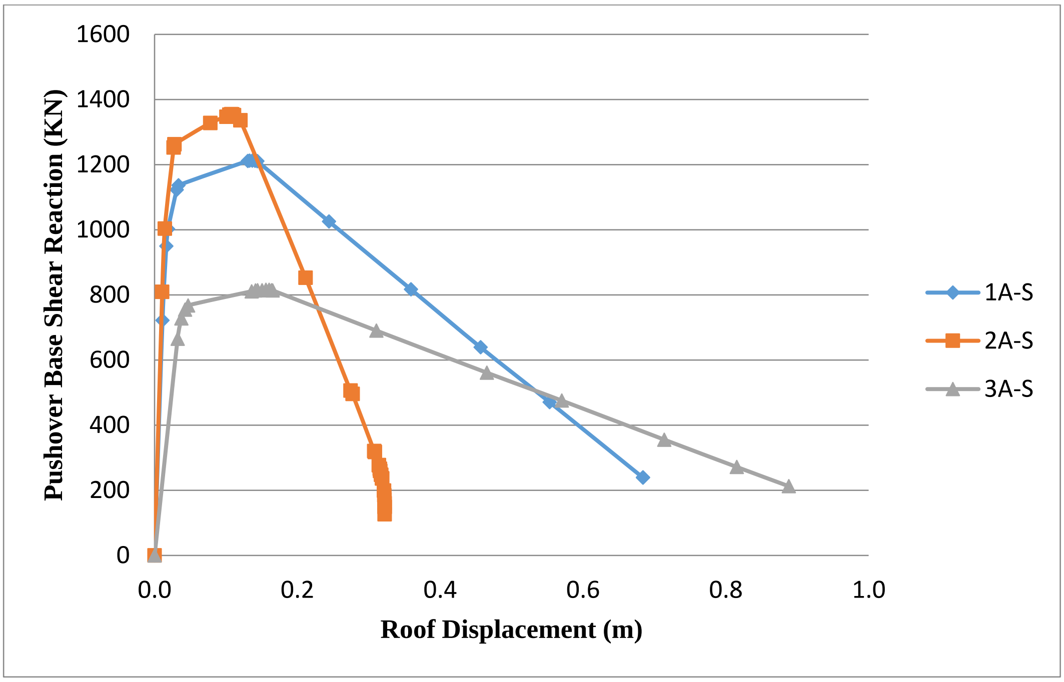


Figure 25: Pushover Curves for 4 Storey's Square Shape Prototype

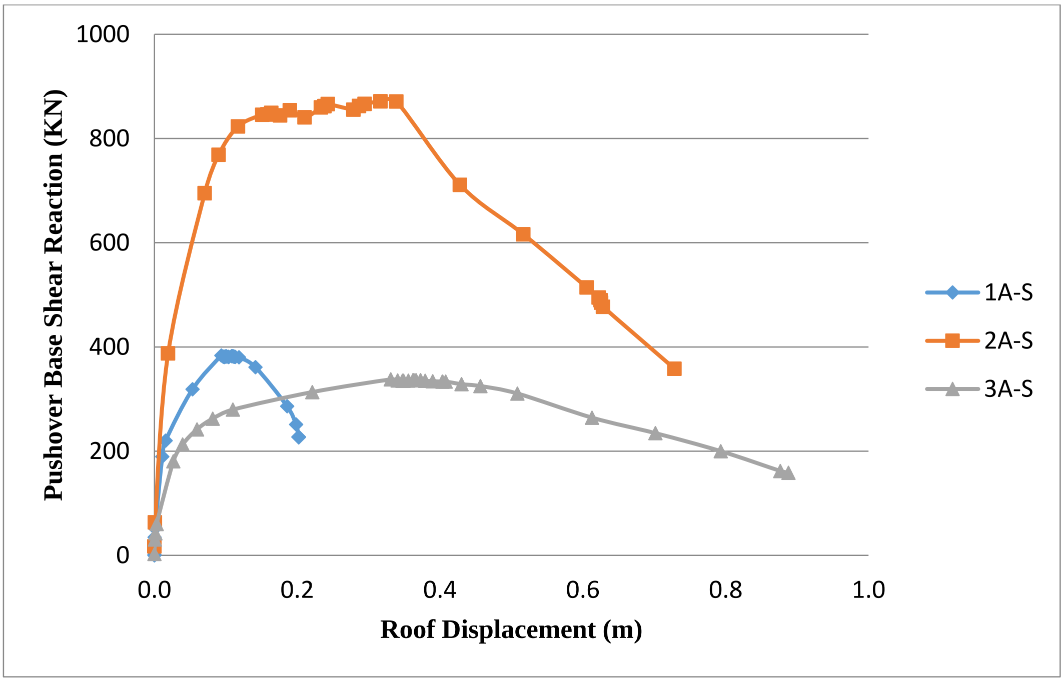


Figure 26: Pushover Curves for 8 Storey's Square Shape Prototype

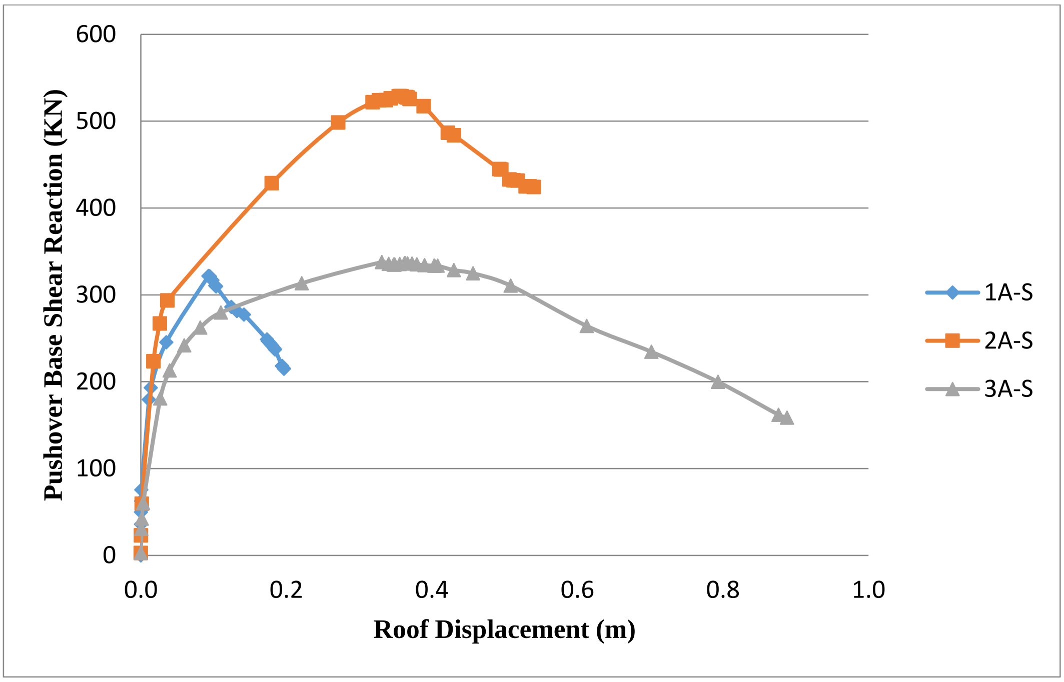


Figure 27: Pushover Curves for 12 Storey Square Shape Prototype

It can be seen from figures 25-27 for the square building layout that for the same number of storey's: case 2A has the maximum plastic capacity because the primary load direction coincides with the strong direction of the columns, and case 3A has the minimum pushover curve because the primary load direction coincides with the weak column's direction. Also, it can be seen that case 1A has a brittle failure type with small ductility as compared to case 2A and this happened because of when the number of storey's reach up to 8 and more the torsional mode of the model gets close to the main mode and this cause that failure.

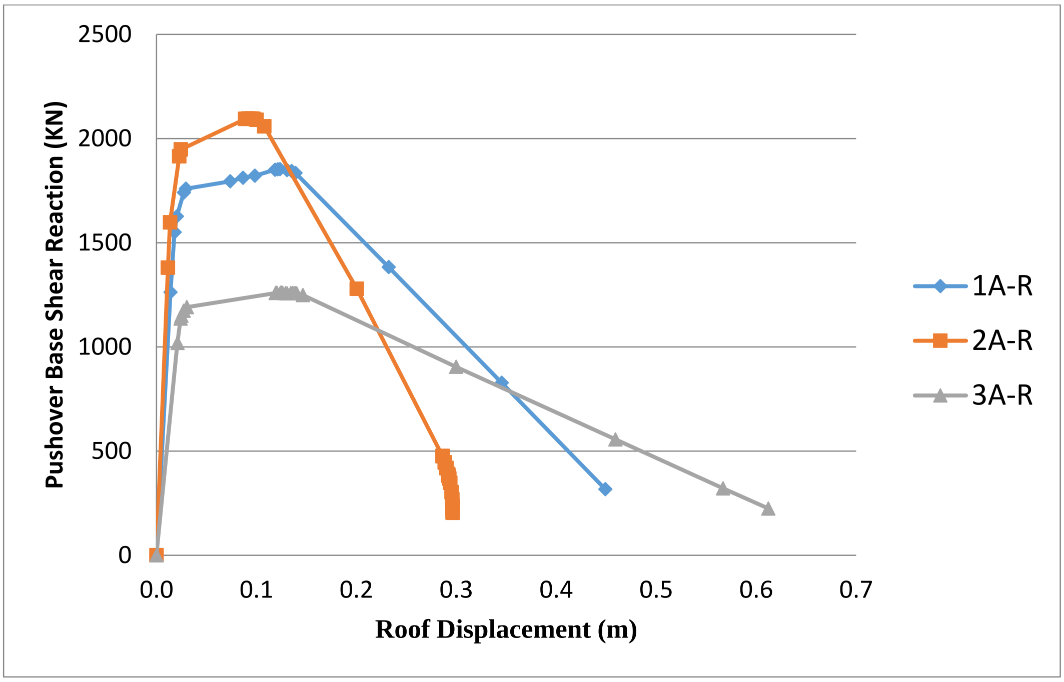


Figure 28: Pushover Curves for 4 Storey's for Rectangular Prototypes with Primary Loads in X Direction

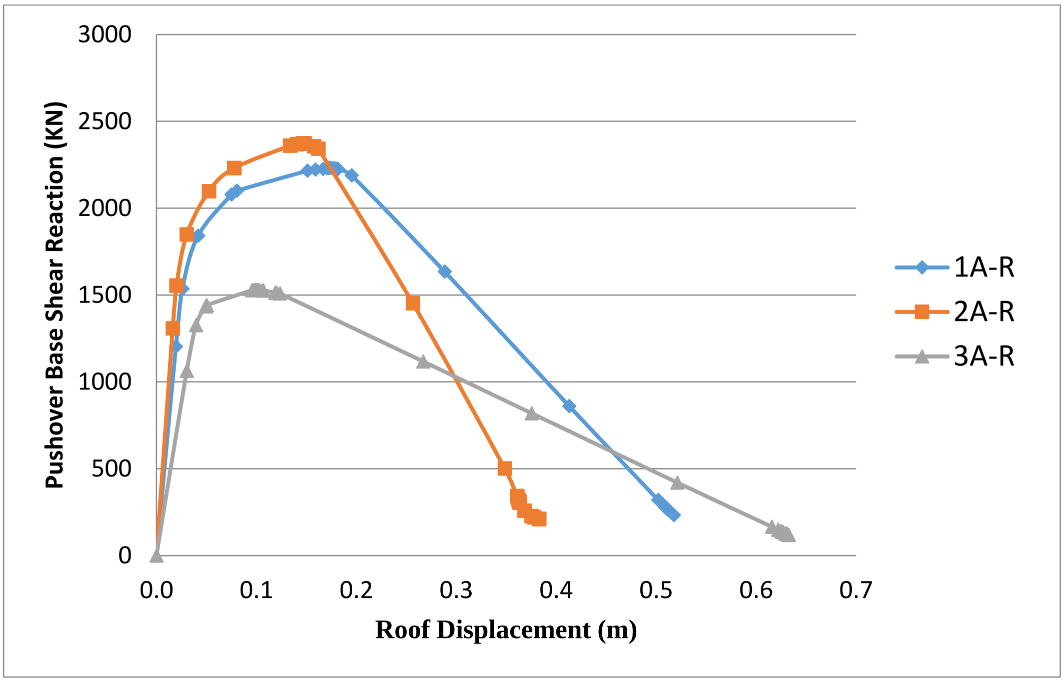


Figure 29: Pushover Curve for 8 Storey's for Rectangular Prototypes with Primary Loads in X Direction

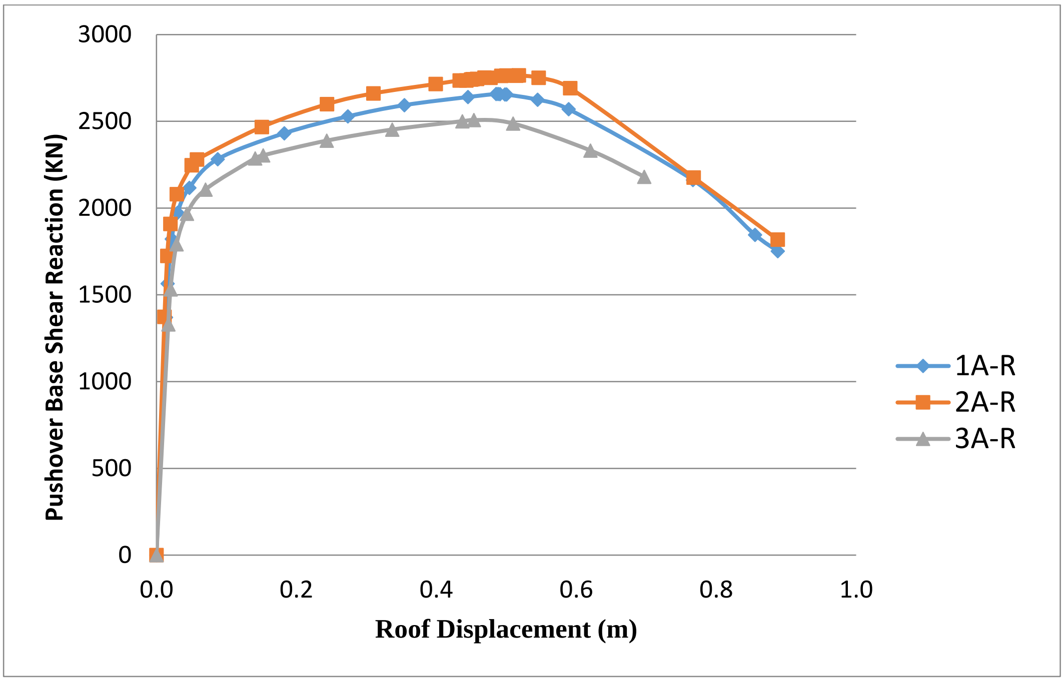


Figure 30: Pushover Curve for 12 Storey for Rectangular Prototypes with Primary Loads in X Direction

Figures: 28, 29 and 30 shows that for rectangular building layout with the primary load in X direction we get the same trend as that of the squared buildings for the same number of storey's. Case 2A has the maximum pushover curve because the main load direction is coincident with strong direction of the columns, and case 3A has the minimum pushover curve because the main load direction is perpendicular to the column's direction. It can also be seen that the higher the building the closer the curves become. For 4 storey building the peak capacity ranged from 1300 kN to 2300 kN while for the 12 storey building it ranged from 2500 kN to 2700 kN.

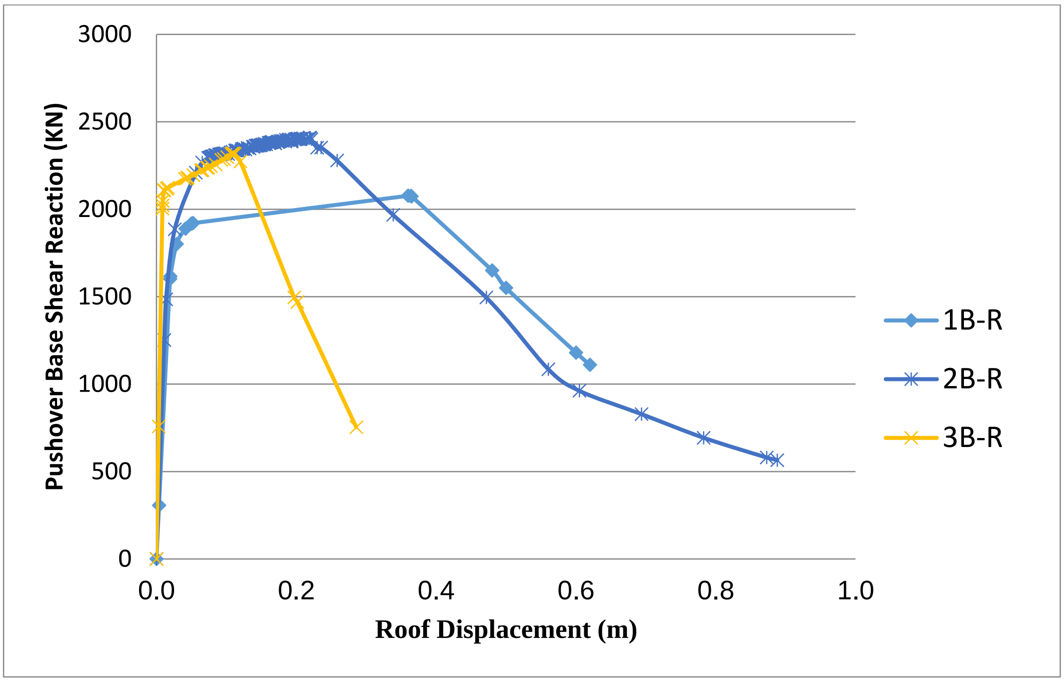


Figure 31: Pushover Curves for 4 Storey's for Rectangular Prototypes with Primary Loads in Y Direction

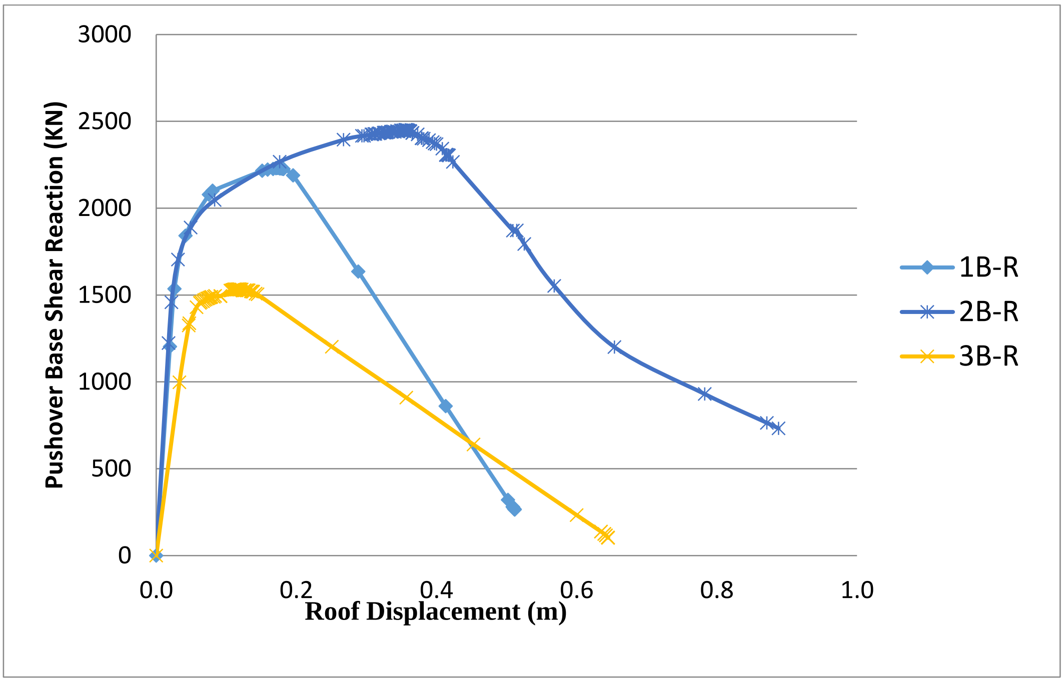


Figure 32: Pushover Curves for 8 Storey's for Rectangular Prototypes with Primary Loads in Y Direction

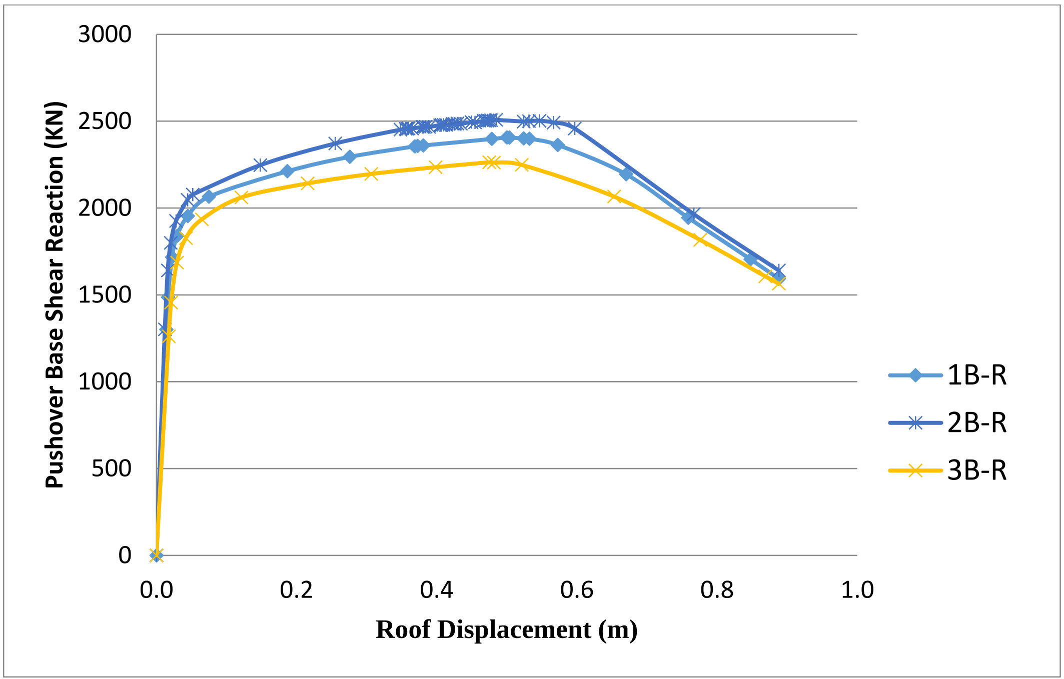


Figure 33: Pushover Curve for 12 Storey Rectangular Prototypes with Primary Loads in Y Direction

Figures: 31, 32 and 33 for pushover curves for rectangular shape buildings with main load in Y direction show the opposite trend of the previous types of buildings pushover curves for the same number of storey. For example, case 2B has the maximum pushover curve and case 3B has the minimum one and this change because of the main load comes perpendicular to the building.

4.4. Computation of R-Factors:

The push over curves are used to compute the R-Factor. To do so, the yield point on the curves must be first determined. This point represents when the overall behavior of the building changes from elastic to inelastic. There have been variations on how to define this point. For our study, two approaches will be used to define this point. The first approach is based on the formation of the first plastic hinge in the building. This approach is logical because this really represents the start of plastic behavior of the building. The second approach is using FEMA 356 bi-linear transformation of the push over curve. The FEMA approach converts the nonlinear load-deflection curve to a bi-linear curve based on equality of energy under both curves. Using the FEMA approach generally gives a higher value for yield force of the building as compared to the first-hinge approach. The yield force obtained from the two approaches is called V_y.

Using yielding Force (V_y) the R-Value can be computed using equation (4.1):

$$R = V_e / V_y \tag{4.1}$$

Where,

R: response modification factor

V_e: elastic force as estimated using equivalent lateral force method.

V_y: yielding force

Equivalent Lateral Force Method (Static Method) were used to determine the value of elastic force (V_e):

$$V_e = C_s * W \tag{4.2}$$

Where,

W: the effective seismic weight

C_s: the seismic response coefficient

The seismic response coefficient was determined using the Equivalent Lateral Force Method in IBC2012:

The seismic response coefficient, C_s, shall be determined by:

$$C_S = \frac{S_{SD}}{R/I_e} \qquad 4.3$$

Where:

I_e= the importance factor determined from section 11.5.1ASCE7

R= the response modification factor in Table 12-2-1 ASCE 7

The value of C_s need not exceed the following:

$$C_S = \frac{S_{D1}}{T(\frac{R}{I_e})} \quad \text{for } T \leq T_L \qquad 4.4$$

$$C_S = \frac{S_{D1}T_{L1}}{T^2(\frac{R}{I_e})} \quad \text{for } T > T_L \qquad 4.5$$

C_s shall not be less than:

C_s= 0.044 S_{DS} $I_e \geq 0.01$

In addition, for structures located where S_1 is equal to or greater than 0.6g, C_s shall not be less than ASCE (12.8-6)

$$C_S = \frac{0.5 * S_1}{(\frac{R}{I_e})} \qquad 4.6$$

Where:

T= the fundamental period of the structure

T_L=mapped long-period transition period determined in section 11.4.5 of IBC2012.

After computing elastic forces and determining yielding forces from pushover curves for all models, the R-Factors are computed and determined as listed in the following tables using both FEMA and first hinge approaches:

Table 25: R-Values Based on FEMA Approch for Square Shape Building models

Model Case	R-value for 4 storey	R-value for 8 storey	R-value for 12 storey	R-value IBC 2012	R-value UBC 97	R-value EC8	R-value Japan	R-value Egypt
1A-Sqr	4.6	4.9	5.43	5	5.5	5	4	5-7
2A-Sqr	4.82	5.07	5.81					
3A-Sqr	4.31	4.51	5.11					
Avg.	4.58	4.83	5.45					

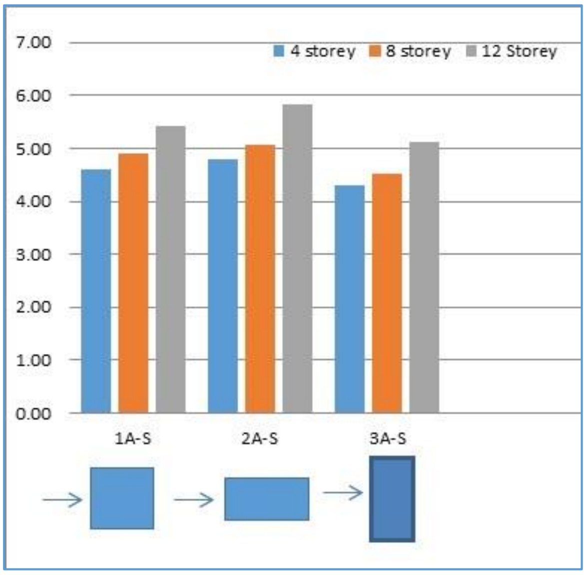


Figure 34: R-Values based on FEMA Approch for Square Shape Building Models

From Table 25 and Figure 34 for square building shape and by using FEMA it can be seen that, R-Factor increases by increasing the number of storey's. For the same number of storey's the maximum R-value is in case 2A-Sqr and minimum value is for case 3A-Sqr, this means that R-vale is minimum when the primary loading is in the weak direction of the columns. It is also evident that the code is not conservative for these cases.

Table 26: R-Values based on First-Hinge Formation for Square Shape Building Models

Model Case	R-value for 4 storey	R-value for 8 storey	R-value for 12 storey	R-value IBC 2012	R-value UBC 97	R-value EC8	R-value Japan	R-value Egypt
1A-Sqr	4.87	5.03	5.60	5	5.5	5	4	5-7
2A-Sqr	5.06	5.21	6.03					
3A-Sqr	4.49	4.83	5.18					
Avg.	4.81	5.02	5.60					

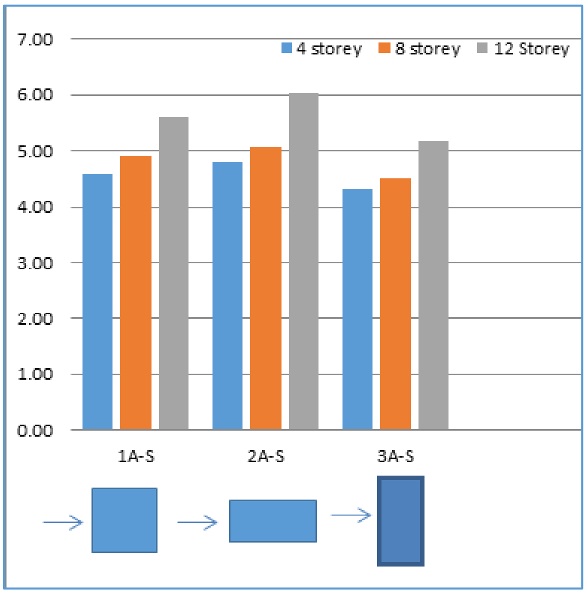


Figure 35: R-Values Based on First Hinge Formation for Square Shape Building Models

From Table 26 and Figure 35 for square building shape and by using V_y based on first hinge formation in pushover curves the R-Factor values were found as shown in Table 26 and the following can be noticed: R-Factor also increases by increasing the number of storey's, and for the same number of storey's the maximum R-value is for case 2A-Sqr and the minimum is for case 3A-Sqr.

The difference between R values from FEMA and first hinge formation comes from higher Vy values from FEMA method and this gives Lower R-value.

Overall, using FEMA bi-linearization to estimate the yield force gives lower values of (R) as compared to first-hinge approach. This can be explained by the fact that the FEMA approach gives higher yielding force than the force needed to cause first-hinge. Thus the FEMA approach gives conservative values for R which is lower than the code-spicified values.

The same trend of results is obtained for the rectangular layout as seen in the following Tables:

Table 27: R-Values Based on FEMA Approch for Rectangular Shape Building Models

Model Case	R-value for 4 storey	R-value for 8 storey	R-value for 12 storey	R-value IBC 2012	R-value UBC 97	R-value EC8	R-value Japan	R-value Egypt
1A-Rec	4.64	4.99	5.73	5	5.5	5	4	5-7
1B-Rec	4.42	5.12	4.62					
2A-Rec	4.82	5.52	6.07					
2B-Rec	4.39	5.86	4.03					
3A-Rec	4.07	4.78	4.88					
3B-Rec	3.70	5.13	4.22					
Avg.	4.34	5.23	4.93					

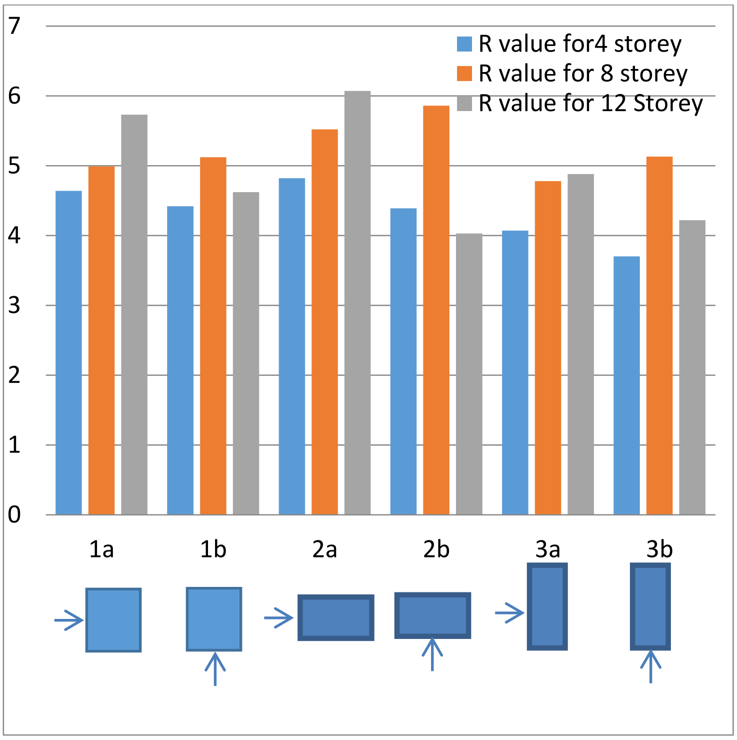


Figure 36: R-Values Based on FEMA Approch for Rectangular Shape Building Models

Table 28: R-Values Based on First-Hinge Formation for Rectangular Fhape Building Models

Modal Case	R-value for 4 storey	R-value for 8 storey	R-value for 12 storey	R-value IBC 2012	R-value UBC 97	R-value EC8	R-value Japan	R-value Egypt
1A-Rec	4.81	5.16	5.91	5	5.5	5	4	5-7
1B-Rec	4.72	6.10	6.28					
2A-Rec	5.30	5.70	6.14					
2B-Rec	4.96	6.34	6.75					
3A-Rec	4.57	4.97	5.01					
3B-Rec	4.20	6.20	6.38					
Avg.	4.76	5.75	6.08					

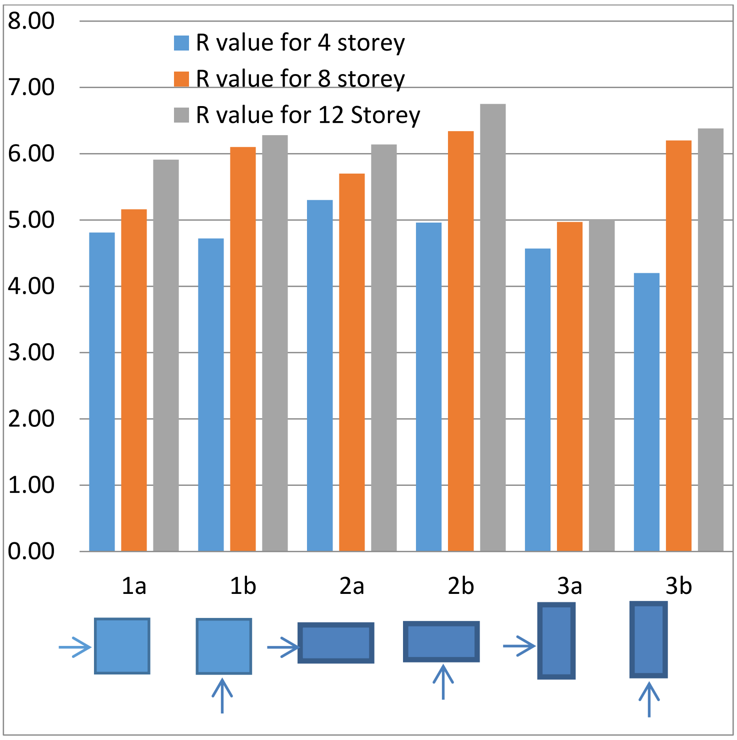


Figure 37: R-Values Based on First Hinge Formation for Rectangular Shape Building Models

Tables (31 and 32) and Figures (36 and 37) show the values of R-Factor using from FEMA method and first hinge formation. From these figures and tables, we see that for case's A and B the R-value increase by increasing the number of storey and the maximum R-value for case 2B-Rec for 12 storey and the minimum R-value is for case 3B-Rec for 4 storey.

Chapter Five

Conclusion and Recommendation

5.1. Summary:

This thesis explored the variation of response modification factor (R-Factor) of intermediate RC-MRFs. The R-Factor recommended by the seismic design is compared against the obtained R-Factors for different columns orientation.

To study the effect of column orientation on the R-Factor of framed R.C creatures, two building layouts were selected and simulated using Sap2000. The two layouts represent two situations, one with square uniform properties in both directions and the other is rectangular layout. These two layouts were analyzed and designed according to ACI code and then pushover analysis was done to generate load-deflection curves for different cases of floor numbers and column orientations. These curves were used to estimate R-Factor under various parameters.

This thesis can help engineers to evaluate the R-Factor values for buildings by using a simple method.

5.2. Main Findings:

It was found from results that the R-Factor suggested in seismic codes, which has been adopted in Palestine, gives in some cases false representation of building response during a seismic event. It was also found that a single value of R-Factor as suggested in IBC2012 may become in most of cases overly conservative or non-conservative.

The following are the main conclusions of the study:

- R-Factor is sensitive to both column orientation and the building height (number of storey).
- For the same number of storey's, the maximum R-value takes place when the primary load direction is coincident with strong direction of the columns, and the minimum R-value is obtained when the primary load direction coincides with the weak column's direction.
- For the same column orientation, R-value increases by increasing the number of storey's.
- Increasing the stiffness of the column or the frame leads in general to an increase in the R-Factor value.
- The IBC 2012 codes is not conservative for R-Factor in case of four floor buildings, as it gives higher value of R-Factor compared to non-linear pushover analysis.

5.3. Limitations of this Study:

In this study the results are bounded by the limiting assumptions under taken in this study. These limitations include:

- Analysis was done for regular buildings. Highly random buildings may give different results. The regularity must be observed in both plan and vertical directions.

- The building was treated as a SDOF. This indicates that the modes of vibration are governed by one fundamental mode.
- The structural elements are mainly subjected to bending and/or axial forces. Thus, torsional loads are assumed negligible on these elements.

5.4. Future studies:

To extend this study further, some points may be addressed in this research. These points may include:

- Repeat the same analysis but using non-linear time-history analysis with both material and geometric no-linearity.
- Study other types of building configurations and introduce more randomness into the building.
- Introduce the effect of soil as a substructure in the building.

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جامعة النجاح الوطنية
كلية الدراسات العليا

تأثير اتجاه الأعمدة على قيمة معامل خفض القوة الزلزالية للمباني ذات الاطارات الخرسانية المسلحة

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أحمد محمود عبد الله زيد

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قدمت هذه الاطروحة استكمالا لمتطلبات الحصول على درجة الماجستير في هندسة الانشاءات بكلية الدراسات العليا، جامعة النجاح الوطنية في نابلس– فلسطين.

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الملخص

ان معامل تعديل الاستجابة (R–Factor) هو معامل تصميم زلزالي أساسي ، والذي يستخدم عادة لوصف مستوى عدم المرونة المتوقع في الأنظمة الإنشائية خلال الزلزال ويستخدم لتقليل الحمل الزلزالي المتوقع بسبب عدم المرونة الكامن في المبنى. توفر كودات البناء الدولية قيمًا ثابتة لهذا المعامل لكل فئة من فئات أنظمة المباني على الرغم من أن قيمتها تعتمد على تفاصيل النظام الهيكلي وبالتالي يجب أن تختلف لكل مبنى. أحد جوانب المباني في فلسطين هو عدم انتظامها ، وهذا يشمل تباين الأعمدة في محاور البناء القوية لتلائم الاحتياجات المعمارية.

في فلسطين ، تُطبَّق الكودات الدولية عمومًا مع القليل من الإرشادات أو عدم صلاحيتها بشأن صلاحية هذه الرموز للمباني التي يتم تصميمها. حتى الآن ، لا توجد مبادئ توجيهية حول كيفية تغير عامل R هذا بسبب توهين المحاور الرئيسية للأعمدة الحاملة في المبنى. تأتي هذه الدراسة كخطوة نحو التحقق من صحة القيم المحددة بالكودات للمعامل R للمباني ذات الإطارات الخرسانية ذات الأعمدة المشوشة او الغير منتظمة.

لتحقيق الهدف المذكور أعلاه ، يعتبر التحليل بالدفع المتتالي(Pushover Analysis) إجراء غير خطي للتنبؤ بالسلوك الغير مرن للمباني ذات الاطر الخرسانية ، من خلال تعريض المبنى لزيادة في الأحمال الجانبية ، حتى يحدث الفشل. يتم استخدام برنامج العناصر المحدودة SAP2000 لتوليد منحنى السلوك اللاخطي من خلال التحليل المرن للبلاستيك المرنة(elastic–plastic) مع اللدونة المركزة في المفصلات البلاستيكية(Plastic–hinges) داخل العناصر الانشائية. تم

استخدام مخططين للمبنى في الدراسة ، الاول مربع الشكل والآخر مستطيل الشكل ، مع تغيير في اتجاهات الاعمده وعدد الطوابق. تظهر النتائج أن المعامل R يزيد مع زيادة عدد الطوابق ، ويحصل على قيمة قصوى عندما يتزامن اتجاه التحميل مع المحاور القوية للأعمدة. يكون المعامل R في حده الأدنى عندما يتزامن حمل الزلزال الرئيسي مع المحاور الضعيفة للأعمدة. هذه النتائج ثابتة لكل من المخططين المستخدمان في الدراسة. وكذلك ، فقد وجد أن المعامل R الذي أوصت به كودات التصميم الزلزالي (IBC 2012 على سبيل المثال) قد لا يكون مناسب للاستخدام في المباني ذات الأعمدة الغير منتظمه. في الواقع ، تم العثور على أنه بالنسبة للمباني المكونة من 4 طوابق ، فإن قيمة المعامل R من IBC 2012 أعلى من تلك التي تم الحصول عليها من تحليل الدفع(Pushover). وهذا يعني أن استخدام قيمة المعامل R-Factor IBC2012 سيعطي قوى زلزالية أقل للتصميم ، مما قد يؤدي إلى تفصيل المستوى الذي لا يضمن المعامل R الواقعي للمبنى الذي يتم تصميمه.

الدراسة ليست سوى خطوة أولى نحو التدقيق في صحة كودات البناء الدولية لاستخدامها في فلسطين وهناك حاجة إلى مزيد من البحوث لتعزيز هذه الدراسة. كموضوع بحثي مستقبلي ، يوصى بإجراء تحليل زمني غير خطي(Time-history) باستخدام سجلات الزلازل الفعلية لمقارنة السلوك غير المرن لهذه المباني بالأحمال الزلزالية الفعلية المستحثة في هذه المباني.