



An-Najah National University
Faculty of Graduate Studies

**DETERMINATION OF R-FACTOR USING
NONLINEAR ANALYSIS FOR R.C SPECIAL
FRAMES CONSIDERING SPAN
VARIABILITY**

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Dedication

The researcher is dedicated to my dear mother, who has encouraged and supported me.

Also, to my great wife and sisters.

Acknowledgements

I would like to express my gratitude to my primary supervisor, Dr. Mahmoud Dwaikat, who guided me throughout this thesis. I would also like to thank my mother who supported me and encourage me to keep on until I succeed.

Declaration

I, the undersigned, declare that I submitted the thesis entitled:

**DETERMINATION OF THE RESPONSE MODIFICATION FACTOR R-
FACTOR OF R.C MOMENT-RESISTING FRAMES USING NONLINEAR
STATICS PUSHOVER ANALYSIS CONSIDERING BEAM TO COLUMN
RELATIVE STIFFNESS**

I declare that 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.

Student's Name: _____

Signature: _____

Date: _____

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DETERMINATION OF R-FACTOR USING NONLINEAR ANALYSIS FOR R.C SPECIAL FRAMES CONSIDERING SPAN VARIABILITY

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Abstract

Background: The Response Modification factor (R-factor) consider as the most important seismic design parameters, it is describing the amount of damping and inherent ductility in the structural system during an earthquake and it is used in general to reduce the actual seismic force to a design force, due to the inelasticity of the structure. Each of the global building codes gives a different procedure to describe the R-factor and give a different value, moreover, the R-factor is a nonlinear factor and it is very complicated thus, any change in the structural system leads to a different value of the R-factor.

Objectives: This thesis gives a comprehensive procedure to calculate the R-factor and then highlights the relationship between the R-factor and the relative stiffness between beam and column. To date, there is no guideline as to how R-Factor would change due to variation of the relative stiffness between beam and column. This thesis comes as a step toward check the code values of the R-factor under the changing of relative stiffness between beam and column.

Methodology: To achieve the above mention goal, a nonlinear static pushover procedure is used to obtain the inelastic behavior of the structure. SAP2000 program is used to generate the nonlinear capacity curves through incremental displacement control nonlinear analysis with concentrated plasticity in the plastic hinge within the structural element, XTRACT program is used to generate all the moment curvature curves for all structural members that are used to define the plastic hinges in SAP2000 program. Two building layouts were used in the study, one is a uniform span length with a different number of floors and a different number of bays, and the other is a nonuniform span length with a different length ratio between spans.

Results: The results show that the R-factor increase as the number of floors increases, also, as the number of bays increases the R-factor will increase and get close to a recommended code value, on the other hand, increasing the length of spans has a negligible effect on R-factor. The R-factor is minimum in the nonuniform cases, for example, the case with a nonuniform span length and three bays (6, 2, 6) meter length and with five floors has a value of R-factor 6.22 which is less than 8 (the IBC 2018 code R-factor for all SMRF). This means that using the IBC 2018 value of the R-factor would give lower induced seismic force for the design and may lead to a detailing level that does not warrant the realistic R-factor for the building being designed.

Conclusions: The study gives a methodology to study the relationship between the R-factor and any variable that affected the structure's response. As a future topic, it is recommended to repeat the work using a nonlinear dynamic procedure like time history analysis using actual earthquake records to compare the result with the actual earthquake loads in these building.

Keywords: R-Factor, Pushover method, nonlinear procedure, Plastic hinge definition, ductility.

Chapter One

Introduction and Literature review

1.1 Motivation

Performance-based seismic design is considered as one of latest approach to earthquake resistant design. Seismic performance can be defined by considering the maximum allowable damage for a specific seismic hazard level. In force-based design, earthquake is considering as a static force added on the structure, this means that both the analysis and design of structure is simple and the same as the gravity load.

Most of the international seismic codes are force-based design and using the structural displacement as an indicator for performance level. Force-based approach is suitable for design forces that are permanently on the structure like gravity load. The effect of these forces is considered as the main design parameter. seismic design under force-based approach is using the same philosophy, except that in seismic design inelastic deformations happened and some of seismic energy is absorbed inside the structure members leading to a reduction in the seismic design forces.

What Building codes trying to do is to define parameters to roughly calculate the amount of the absorbed and induced seismic force and these parameters depend on the complexity of detailing, reinforcement ratio and structural system. Unfortunately, Engineers in Palestine, don't have the needed resources and still in the age of development, so, taken these parameters from international building codes and use it in the seismic design. This procedure is not correct all the time, because some parameters may not be valid for some typical building systems in Palestine.

Buildings in Palestine suffer from a lack of uniformity and irregularity, especially in the spacing between columns which has a main effect on the relative stiffens between beams and columns in the frames. Therefore, it is necessary to look into the validity of building codes parameters for seismic design here in Palestine.

Response modification factor (R) is considered as the most important parameter in seismic design. R-factor is represented the nonlinear response of the structures during an earthquake with strong action. The response modification factor is mainly based on

expert's judgment, and no code technique or method to calculate its value for all structures systems. [1]

Buildings in Palestine are mainly irregular, columns distribution and span lengths are random and serve variable architectural purposes. Therefore; the relative stiffness between column and beam will noticeably change. Till today, there are no instructions to see how the change in column-to-beam relative stiffness effected R-factor values. This thesis comes as a first step to investigating and connect the code given value for R with various values of column-to-beam relative stiffness.

Palestine is highly exposed to the risks posed by nature, the most important of these are earthquakes. The region often faces disasters that are small or medium but sometimes have high potential for large-scale (urban) disaster. The geodynamic processes of the seismic activities occurring in Palestine are largely influenced and controlled by the Dead-Sea Transformation (DST). The DST is a side fault between the Arabian Peninsula and the tectonic plates in Sinai, which carries 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 105km from many of the previous features of myosin ology.

The seismic history 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 seismic activities in the area, an earthquake with a magnitude more than 6 degrees is expected soon. Considering the devastating earthquake of 1927(6.25 degrees and 15km north of the dead sea), a large earthquake is expected to occur every 100-120 years in the dead sea area, which in turn would cause significant damage and losses due to the severe weakness of most of buildings. On the other hand, according to other studies in the region it can be expected an epic earthquake entered in the southern part of dead sea

According to Seismic Zone Factor map, Z of Palestine, which is also used by Israel Building Code, Palestine is divided in to four areas: 1, 2A, 2B and 3 and Palestine is considered as moderate to strong seismic zone.

1.2 Research significance

The seismic design philosophy that each code adopted is based on reducing the design seismic force to consistent with available ductility, each frame is categories according to the amount of reinforcement and level of detailing. Different codes adopting almost the same procedure to categories the frames but with different reduction percentage. The American codes use the response modification factor to lower the seismic force to the design force for different frames category. These categories are common for all international building codes but with different name and values for R-factor.

Generally, a change in the structural parameters such as irregularity of spans, relative stiffness and detailing may cause a change in the value of the R-factor, investigation these parameters to check if these changes are significant.

In this study, a comprehensive procedure is adopted to calculate R-factor, which can be used to investigate any parameter.

The main parameter of this thesis is to study the relationship between the relative stiffness between beam and column and it is effect on the response modification factor and no study yet consider this parameter. The main goal of this thesis is to find the relationship between relative stiffness and R-factor considering any parameters that may be connected to the study.

1.3 Objective

The main target of this thesis is to reevaluate code given R-Factor for reinforced concrete special moment resisting frames (SMRF), by studying the effect of variation in relative stiffness between beam and column. To achieve this goal many tasks will be performed, starting by selecting representative RC-SMRF which is intended to encompass common frames in Palestine where relative column-to-beam may be variable. Then the selected frames will be designed according to ACI/IBC codes with the level of code-specified ductility requirement. Then a nonlinear static pushover analysis is conducted to assess the response of such frame indeed conforms to the code specified level of ductility and to get a sense of how the R-factor may change for such a frame. The output of this analysis is used to estimate the value of the R-Factor. Finally,

comment on the result and compare it with code values, leading to practical guidelines that are useful for practicing engineering during the design phase.

1.4 Thesis outline

The following chapter and its content represent the main point in this thesis:

Chapter one, presents the significance of the problem and the primary objectives to be followed to achieve the research and literature review for R-factor and its components, talking about the history of R-factor and its values in different codes. Also, a quick review is presented to the main studies that is related to our topic.

Chapter two, talks about modeling the nonlinear behavior of the selected frames. In this chapter, the type of analysis used and the different types of nonlinear behavior are described.

Chapter three, covers parametric study. This chapter discusses the parameters in the study and describes the selected frames for analysis.

Chapter four, presents the analysis and the results. Then chapter five, talks about conclusions and future recommendations.

1.5 Literature review

This section provides a quick review to the main two procedure that is used in seismic analysis and design and their major limitation for computing R-factor and its components, moreover, a revision of previous studies and codes will be presented.

1.5.1 Force-based design (FBD)

codes all over the world were originally developed based on elastic and static analysis procedures. The seismic action is presented as a static seismic force and members behave elastically. In reality, this is incorrect because buildings can undergo a large inelastic deformation during earthquake events, which is not considered properly in FBD. The large inelastic deformation depends on the level of detailing, reinforcement ratio and structure type and system. Generally, building codes utilize an ever-fixed reduction factor to modify the elastic force that endured by the building during an earthquake.

all of the international building codes use equivalent lateral force (ELF) methods for the seismic design of regular building. This method is correct when applied on structure with low natural periods like buildings up to 7 floors. For buildings with a relatively long natural period, the ELF procedure becomes incorrect and the structure must be designed using the Response Spectrum Method. [2] Also, ELF requires some degree of regularity in both horizontal and vertical directions.

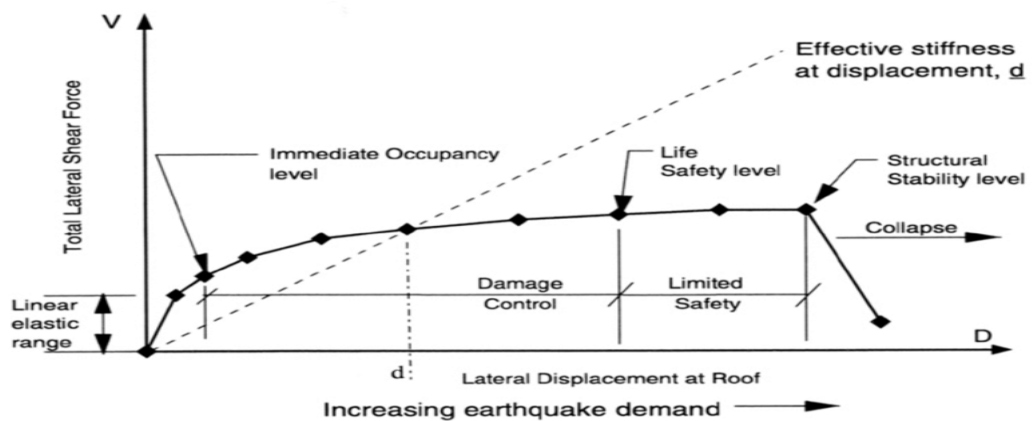
Both vibration properties and the site classification identify the design lateral forces acting on any structure. Equivalent lateral base shear force is calculated based on the fundamental period of the structure, the next step is to distribute these forces along the height of the building normalizing to the fundamental mode of the structure. Finally, structure is analyzed statically for these set of forces to find the internal stress. [2]

1.5.2 Performance-based seismic design (PBD)

This second and modern method named also limit state design and it is contained three performance requirements or Performance level, immediate occupancy, life safety or collapse prevention (structural stability) and it is defined the state of the building under an earthquake, see Figure 1.

Figure 1

Capacity curve [ATC-40]



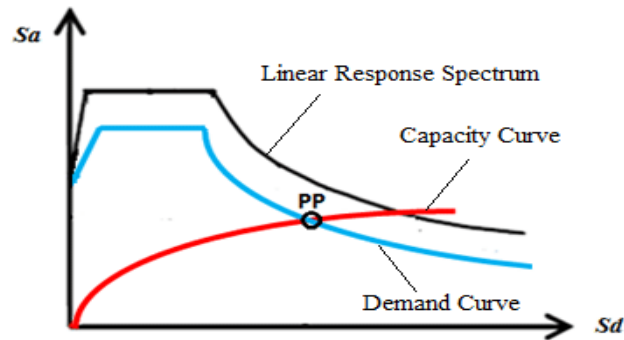
There are two types of design in the PBD method, the capacity spectrum methods and seismic demand approach. [3] The seismic demand approach is represented the real motion of the ground under an earthquake and it can be represented as spectral

acceleration imposed on structures by earthquakes. This requires a nonlinear time-history analysis of several EQ records and an average of the effect on each member.

On the other hand, static nonlinear pushover analysis can be used to find both the level of earthquake excitation force that can be imposed on the structure and the performance of the structure, through the capacity spectrum method.

Figure 2

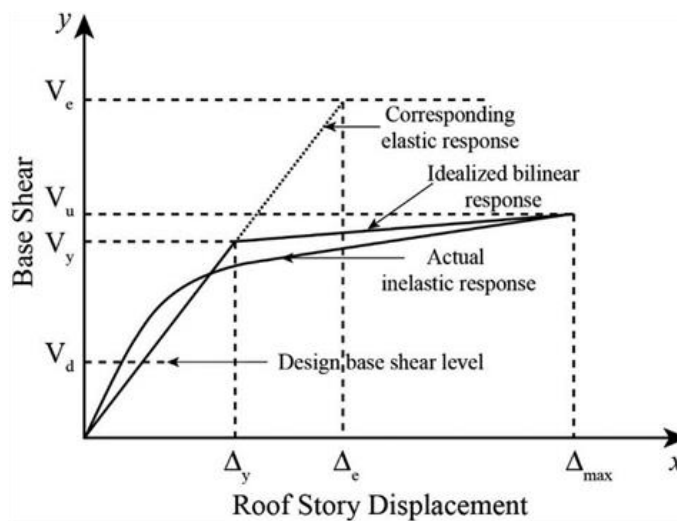
performance point interaction on the capacity curve



Generally, the seismic capacity spectrum curve is obtained for a given structure by converting base force versus top displacement that results from static NL P.O analysis, into spectral acceleration or spectral displacement versus period for an equivalent single degree of freedom system SDOF. The resulting capacity spectrum is then superimposed on the response spectrum curve for a given seismic hazard where the expected performance point can be identified. Figure 3. Shows the elastic and inelastic response.

Figure 3

Typical force-displacement response including elastic and inelastic behavior



From an economic point of view, if the structure continues to behave elastically during the earthquake, the induced seismic force will be large and equal to V_e up to the failure. This becomes uneconomic, although some buildings need to design in such a way because of the damage that happened to the structure, the code allows for reducing the force from the earthquake. Building with sufficient detailing that provides the needed ductility to permit the structure to behave in a nonlinear way can be designed using lower force V_y and sustain large lateral displacement. As seen in figure 3 the ratio between the elastic force and the reduced force is called the response modification factor R .

1.6 Components of R-Factor

Raheem, 2013 defines R-Factor as a “factor used to measure the ductility, damping and inherent energy in structural systems at the maximum displacement of the structural systems just before the actual failure”. [4]

Values for R-factor were appeared for the first time in ATC-3-06 1978 based on the observed performance of structures during previous earthquakes and the rough calculation of damping and over strength of the structures. [1]

Components of R-factor including R_S , R_μ , and redundancy RR , are based on their first definition in ATC-19 1995 and ATC-34 1995. Plenty of research are done since the first appear of R-factor. Later research, like ATC-34, gives a new method to calculate R-factor, this equation is used define R-factor as the products of different three factors:

$$R = R_S * R_\mu * RR \quad 1.1$$

where:

R_S is the structural over strength value

R_μ is the ductility factor

RR is the redundancy factor

Figure 4

Force displacement response of actual inelastic systems

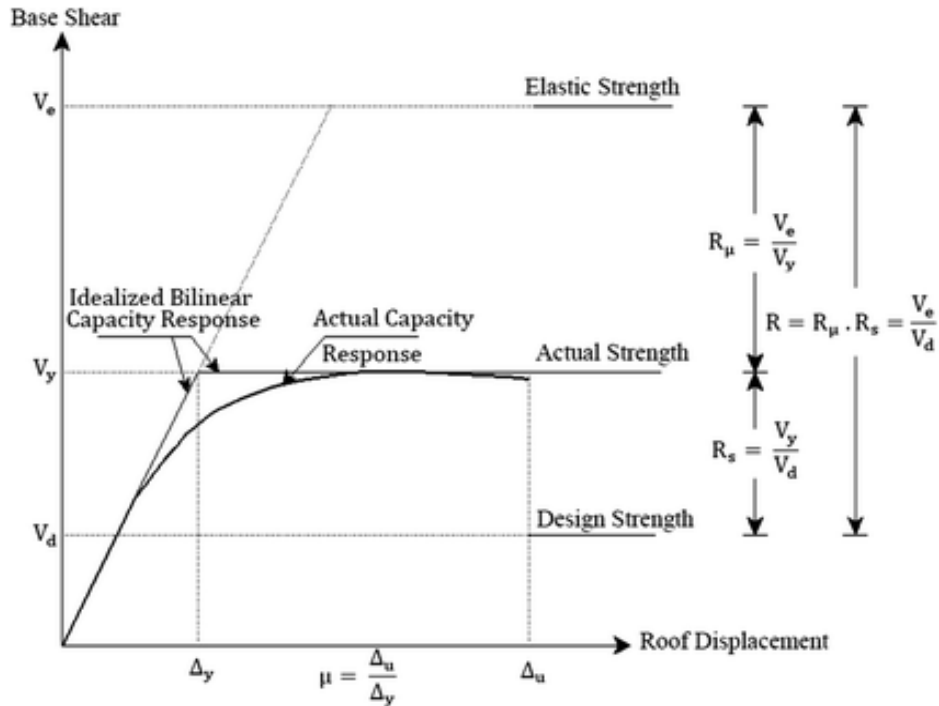


Figure 4, the nonlinear performance curve illustrates the formation of R-factor and its components, moreover the products of this factors, in the next section these components will be discussed.

1.6.1 Ductility factor R_μ :

It also named the strength reduction factor and it define as the reduction in demand force assign to inelastic behavior, R_μ factor used to lower the elastic force that comes from earthquake V_e to the yield strength of the structure V_y , under this definition R_μ factor is defined as follow.

$$R_\mu = \frac{V_e}{V_y} \quad 1.2$$

The nonlinear behavior of the structure that is caused by the hysteretic energy is represented the ductility factor, which mainly depend on the characteristics of imposed earthquake ground motion and the properties of the structural like the fundamental period of vibration mode.

Newmark and Hall [5] developed equations to relate $R\mu$ with the natural period, the maximum drift in the structural before failure (Δ_{max}) and the drift corresponds to the idealized first yield in structure (Δ_y):

$$\begin{aligned}
 \text{Newmark and Hall: } R\mu &= \mu & \text{for } T > 0.5 \text{ sec} \\
 R\mu &= \sqrt{2\mu - 1} & \text{for } 0.1 < T < 0.5 \text{ sec} \\
 R\mu &= 1 & \text{for } T < 0.03 \text{ sec}
 \end{aligned} \tag{1.3}$$

Where T represent the fundamental period of the structure and μ is the displacement ductility factor and it can be defined as.

$$\mu = \Delta_{max} / \Delta_y \tag{1.4}$$

Where Δ_{max} represent the maximum displacement before failure and Δ_y is displacement corresponding to the first yielding in the structure.

It is important to separate between three related but different parameter, ductility ratio, ductility and displacement capacity. It does not mean that as the ductility and ductility factor increase the structural capacity will increase in fact it may have structure with large displacement capacity and small ductile and ductility ratio.

There are two different approaches to calculate the ductility ratio μ , element and story levels. At the story levels, the ductility ratio is calculated based on displacement and it is used to estimate the ductility factor μ . At the element level, the ductility ratio is calculated in terms of curvature or rotation ductility.

[5] conducted important studies on R factors and connect it with ductility, $R\mu$ is calculated in term of ductility and the fundamental period of the structure. They give five different ranges of period and suggest five equation to calculate the ductility factor Eq 1.5, while Figure 4 shows $R\mu-\mu-T$ for many ductility ratios and different periods.

Periods ≤ 0.03 sec:

$$R\mu = 1.0$$

Periods $0.03 < t < 0.12$ sec:

$$R_{\mu} = 1 + \frac{(T-0.03) \cdot (\sqrt{2\mu-1} - 1)}{0.09}$$

Periods $0.12 \leq T \leq 0.5$ sec:

$$R_{\mu} = \sqrt{2\mu - 1}$$

Periods $0.5 < T < 1.0$ sec

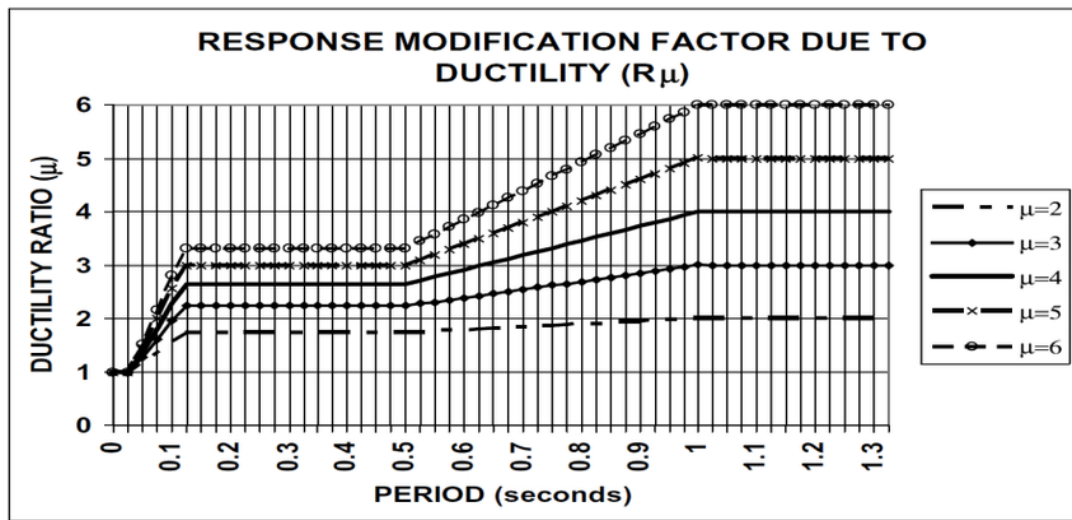
$$R_{\mu} = \sqrt{2\mu - 1} + 2(T - 0.5) \cdot (\mu - \sqrt{2\mu - 1})$$

Periods $T \geq 1.0$ sec:

$$R_{\mu} = \mu \qquad \qquad \qquad 1.5$$

Figure 5

R_μ – T – μ Curves (Newmark & Hall 1982)

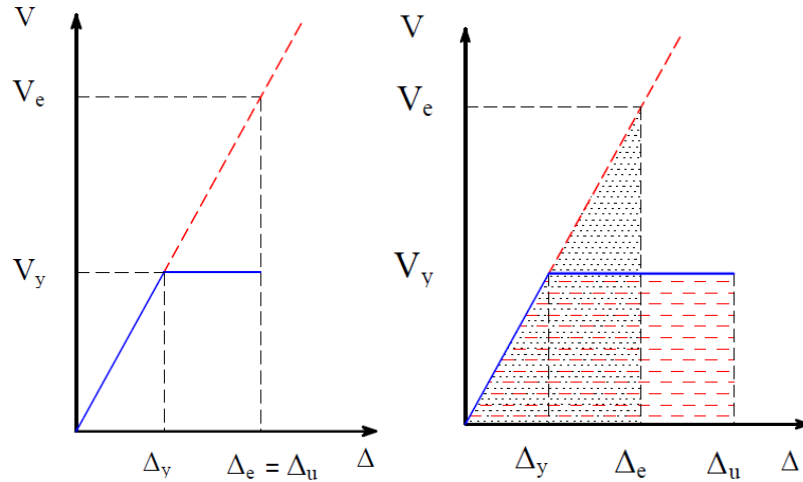


1.6.1.1 equal displacement Vs equal energy

a frame with a natural fundamental period less than 0.5 second is follow the equal energy role with an ultimate displacement larger than the target displacement and ductility factor is given as $R_{\mu} = \sqrt{2\mu - 1}$, while in the equal displacement role the ultimate displacement is very closed to the target displacement and the R-factor is given as $R_{\mu} = \mu$ for the structural with a natural period more than 0.5 second see figure 6.

Figure 6

equal displacement Vs equal energy



1.6.2 Structural over strength factor $R_s(\Omega)$

R_s factor is defined as the ratio between the actual lateral strength V_y to the design lateral strength V_d .

$$R_s = \frac{V_y}{V_d} \quad 1.6$$

In the design phase a lot of assumption and simplification is done and that reflect on the strength of the structure and makes it higher for the real structure. These days it is easy to predict the real strength of the structure by modeled it with the help of the computer program and the real strength of the structure is known. Many factors cooperate to make the overstrength appeared in structures, such as:

1. The ultimate load combination.
2. The real value of gravity load when the seismic events occur
3. The material used in the building generally gives higher strength and better properties.
4. Further reinforcement of the required strength
5. Member size is sometimes greater than the design

1.6.3 Redundancy factor R_R

The effects of reliability and redundancy in the strength of the building is firstly studied in SEAOC (1996); UBC (1997); and IBC (2000) and called the redundancy factor (ρ).

redundancy factor (ρ) is appeared in the formulas of the load combination. Its main effect to decrease the number of parameters that increase the over strength value and make them more realistic.

redundancy factor R_R is defined as the percent between the ultimate strength of a structural system over the strength of the same, but nonredundant structure. And its formula is given as.

$$R_R = \frac{S_u}{S_{nr}} \quad 1.7$$

where S_u =structural system strength considering the redundancy effects, and S_{nr} = structural system strength under nonredundant condition. [6]

This factor is assumed one and will not consider in this study.

1.7 Previous Studies

Ahmad Zaid (2018) studied the effect of column orientation on factor (R) of reinforced concrete intermediate resisting frames, he found that R-factor which is calculated pass on the American code and it approved by the Palestinian standards, gives in some frame's wrong representation of structures response during a strong active seismic effect. It was found that representing R-factor with just one value for such a type of structure could become in particular cases over or non-conservative. His main finding was that, the maximum R-value can be obtained when seismic load direction is coincident with the strong axis of the column. [7]

Zafar (2010) calculate R-factor for common reinforced concrete moment-resisting frames that built in Pakistan, moreover he compared the calculated R-factor with the code given values. To calculate the R-factor for RC-MRF incremental dynamic analysis methods (IDA) is conducted using a seismic record in Pakistan. He studies the effect of different material and geometric properties of RC-MRFs on R-Factor. [8]

Zafar main found that during a seismic event the R-factor could give a wrong representation of the response of the building, moreover unique value of R-factor as given in Pakistan code and UBC97 code could be over or unconservative. [8]

Heshatollah Abdi et al (2019) studied the use and the application of R-factor. They found a connect between the viscous damping effects on R-factor, which represent the inelastic behavior of the building throw dissipate of energy. Base shear is expected to reduce if we add additional damping to the structure. It is well known that; the real lateral strength of the frame is lower than the yielding force according to most seismic codes. Moreover, the overstrength factor change based on the hight of structure, type of frames. According to ATC-19, the number of lines of strength and stiffness effect the redundancy factor. [1]

Mohd. Zameeruddin and Keshav K. Sangle (2021) evaluating the performance of reinforced concrete frame using nonlinear static procedures. For this, fifteen-moment resisting frames designed following the guidelines of Indian seismic codes were subjected to different lateral load patterns. The seismic performance is investigated in terms of fundamental periods, roof displacements, interstory drift ratio, base shear, and modification factor and was compared with various performance limits. They found that response reduction/modification factor has been evaluated for various performance levels defined in PBSD. A component wise calculation of R factor is done, which was not addressed by IS 1893. The values of R obtained at IO, LS, and CP performance levels show that, IS 1893 overestimates R factor. Such an overestimate may lead to the potentially dangerous underestimates of elastic base shear, leading towards a conservative design approach. [9]

Praveen Oggu et al (2021) said that R-factor does not address the effects of changes in structural configurations, which eventually alters the dynamic behavior of the structure. Hence, the adequacy of prescribing a constant factor to account for the variable dynamic characteristics of structural systems always appears contentious. Further, seismic analysis of RC buildings usually ignores the interaction of the infill wall with the structural frame leading to inappropriate evaluation of dynamic characteristics of the structure. Hence, in the present research, it is attempted to investigate the sufficiency of the code-based R-factor in assessment of seismic behavior using non-linear static analysis (NLS) and nonlinear dynamic analysis (NLD) for the structural models considered. And they found that the higher values of R can be observed from IDA than NLS analysis in view of accurate estimation of dynamic characteristics of the structure. Further, these R-values are observed to be significantly higher than those specified by

IS 1893 (Part 1): 2016 for the models considered. This can be attributed to the higher inherent reserve inelastic capacity of the Indian code designed RC frame. [10]

Reham Eltahawy (2020) investigate the variation of seismic response of special RC structures with different configurations, such as elevated metro stations. Consequently, the response reduction factor can be evaluated. Then the calculated response reduction/modification factors (R) for reinforced concrete (RC) structures will be compared to those specified in ECP and the ASCE code. She found that the (R) value does not match those values mentioned in ECP code due to lack of parameters affecting the R value, because the recommended values for (R) factor in the ECP code are not affected by height of RC structures, which affect ductility factor. Moreover, the response reduction/modification factor (R) value is sensitive to both RC statically system and RC geometry. [11]

Somayeh Sharifi and Hamid Toopchi-Nezhad (2018) evaluate the response modification factor, R, of moment-resisting RC-frame structures that are designed based on a limit state design methodology. The study is focused on the ordinary and special RC-frames of 1, 2, and 3 bays at 3, 5, 7, and 9 stories. To evaluate the R-factor of each frame structure, a nonlinear static pushover analysis is performed and the capacity curve of structure until a maximum lateral displacement that is typically prescribed by seismic design codes is plotted. Results of this study indicate that the R-factor is significantly influenced by the number of stories and bays, as well as the maximum lateral displacement that is imposed on structure during pushover analysis. They found that, the design R-factor calculated for ordinary RC-frame structures of this paper is consistent with the values given by ASCE. However, both of these standards overestimate the R-factor of special RC-frame structures as compared to the value of 7 obtained in this research study, or the value of 6.8 given by NBCC. [12]

Momen Mohamed et al (2021) illustrate that the response modification factor (R) is sensitive to many aspects such as overall ductility, over-strength, damping, and redundancy levels. Indeed, these factors are severely affected by geometric irregularity of the structural system. So, R-value does not become a constant number for the all types of structures with the same lateral load resisting system, as many standard codes noted. It depends on types, combination, and degrees of geometric vertical irregularity.

They found that, the seismic force reduction using R factor become highly misleading value for vertical irregular buildings. Indeed, the calculated R value is reduced by 20-40% due to obvious defect in overall ductility and overstrength. [13]

H. F. Wong and J. S. Kuang (2008) studied the effect of the beam to column depth ratio on joint seismic behavior and they found that both strength and ductility of beam-column joints is affected by the ratio of beam-column depth. Exterior beam-column joints with a low beam-column depth ratio show a higher ductility and better hysteretic behavior. Finally, the shear strength of frame joint decreases as the depth ratio between beam and column increases. [14]

Filippou studied and evaluate the R-Factor for an existent multistory structure and look at the effect of different reinforcing ratio on the R-value. [15]

the capacity curve of the building is conducted using linear and nonlinear pushover analyses. The results were studied and the main goal is to obtain the differences between these two different analysis approaches. [15]

Jizhi Su Boquan Liu et al (2020) studied the Influence of the beam-to-column linear stiffness ratio on the failure mechanism of reinforced concrete moment-resisting frame structures, a set of nonlinear dynamic analyses were performed on the program model, considering two situations non-conforming and conforming joints. The test results showed that damage is common to start at the columns in structures with higher beam-to-column linear stiffness. The dynamic analysis recommended to firstly design the beam-to-column joints with higher linear stiffness ratio, so a transition from a column-hinge mechanism to a beam hinge mechanism is obtained. [16].

1.8 R-Factor in Building Codes

The utilize of the response modification factor or its equivalent is common to almost all seismic design codes in the world. A quick summary of response modification factor in seismic design codes and how it varies for different regions considering special moment-resisting frames will be presented in this section.

1.8.1 American codes

The response modification factor (R) in the UBC code went through a series of stages until it reached its current form.

There was no mention of the R-factor in the early stage of UBC code development. The common practice was to apply a percent of the weight of the building as a lateral load, and then continue the design using elastic analysis.

Later, the UBC code introduced an equation to define the base shear (V)

$$V = ZKCW \quad 1.8$$

Where Z represents the seismic zone, K represents the building type, C for the period of the structure and W is the weight of the structure. Again, no use or mention of R-factor in this design equation.

Then, the response modification factor R is considered in seismic design code in the computing of equivalent lateral load. Moreover, the UBC code connects the equivalent lateral load with a new variable like the importance factor (I), the site (soil) factor, period, site-specification ground motion maps.....

The recommended value of R-factor in UBC97 for moment-resisting frame system is shown in the next table.

Table 1

R-Factor values in UBC97 for moment resisting frame system (MRFs)

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

In 1994 Northridge earthquake was the motivate to a large application and considering of the seismic design mandatory use in all of the U.S.A. Then merging of codes and the adopting of the International Building Code (IBC) ensure to have uniform design criteria. However, IBC standards is updating continually. The next table gives R-factor for different frame type. [7]

Table 2

R-Factor values in IBC-2018 for moment-resisting frame system (MRFs)

	Structural system	R	Ω	C_d
Special moment frame	8	3	5.5	
Intermediate moment frame	5	3	4.5	
Ordinary moment frame	3	3	2.5	

So, in the UBC code, a value of 8.5 is used for SMRF, while it is 8 in the IBC-2018 code

1.8.2 National Building Code of Canada (NBCC 2005)

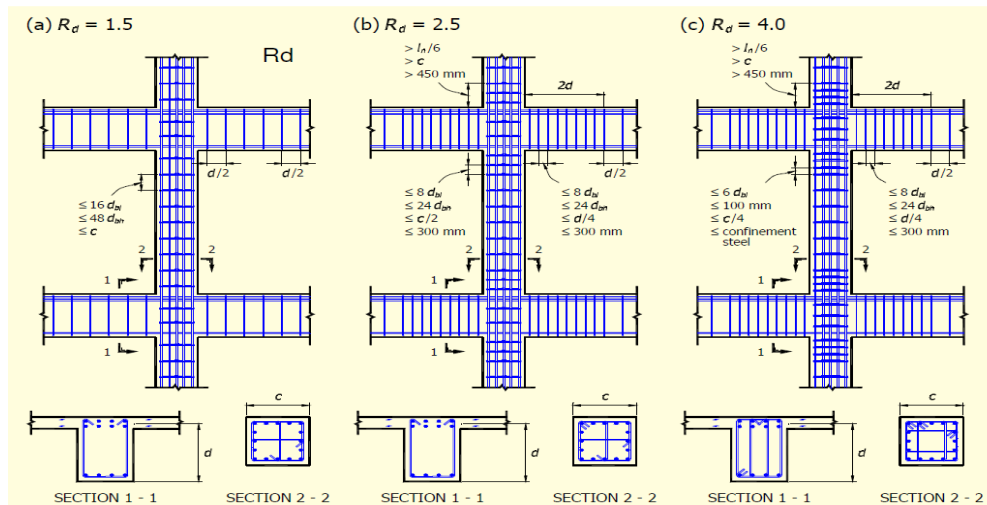
The base shear force used in the Canadian code is given as:

$$V = \frac{S \cdot T a \cdot M v \cdot I}{R_d \cdot R_o} * W \quad 1.9$$

Where R_d is a ductility factor that ranges from 1.5 to 4, R_o depends on the system use for the structure and it is between 1.3 and 1.7

Figure 7

R_d for a different level of ductility



When applied to a SMRF of a similar character to that in IBC code we obtain an equivalent value of R as 6.8 which is smaller than the value of IBC code.

1.8.3 Europe (Euro code 8)

Behavior factor q is responding to R-factor in the seismic design for Euro code it is used to reduce elastic demand force to the strength design force and it depends on the period of the structure. q factor depends on four factors: ductility, building strength, structural system and stiffness regularity, equation 1.10 is used to calculate the behavior factor:

$$q = q_0 * k_D * k_R * k_W \quad 1.10$$

q_0 represents the basic value for the response factor.

k_D represents ductility class.

k_R represents the irregularity in the elevation.

k_W represents the failure mode of the structure.

Based on EC the horizontal seismic force F_a is given as:

$$F_a = S_a * W_a * \lambda_a / q_0 \quad 1.11$$

where F_a is the horizontal seismic force, acting at the center of mass of the structural structure in the most unfavorable direction.

W_a is the weight of the structure.

S_a is the seismic coefficient applicable to non-structural elements.

λ_a is the important factor.

If we try to compare the R-factor from EC to that of IBC, the equivalent EC value is in the order of 5 compare to 8 for SMRF in IBC.

1.8.4 Japan

Two-methods for the seismic design of building are suggested in Japanese building standard law (BSL). Firstly, the design follows the force base methods and elastic forces are computed to consider the seismic action. Secondly, both strength and ductility are considered to check if the design is sufficient for severe ground shaking. BSL consider the response modification factor with a different name compare with the IBC code. R-

factor is replaced with a ductility factor ($1/D_s$) and it is used for the design of all building systems and it is range from 1.8 to 4. In IBC code a value of 8 is represented for a high ductile frame while in BSL, the ductility factor ($1/D_s$), for ductile and special frames is equal to 4 and it is half the IBC code.

1.8.5 Egypt

R factor is considered In Egyptian code, it is value range between 5 to 7 and the ductility of frames is the main parameters to assign it is value. based on the level of details, the maximum number of plastic hinges and I is effect on the collapse mechanism two different ductility level is considered sufficient and non-sufficient. Generally, high ductility frame like SMRF in the IBC code a value of 7 is consider.

1.8.6 Summary

This journey through the international code all over the world enabled us to see how the codes are inconsistent in specifying unique R-factor for an identical building type. Moreover, the R-factor change in some code with each update, so it's important to investigate how one of the most important factors for seismic design is affected by some changes in the properties of the frame.

Typically, R-factor depends on the characteristic of each building and cannot be unique for many buildings under the same category. Restricting the R-value to overall ductility and natural period is only a simplification. To be more precise more factors must be included in the calculation of R-factor and these factors must reflect the physical properties of the building and not only ductility which required sophisticated nonlinear analysis to determine it.

Chapter Two

Non-Linear Modeling

2.1 Nonlinear procedures

Generally, there are four level of analysis to design any building:

- 1-Linear static analysis (LSP)
- 2-Nonlinear static analysis (NSP)
- 3- Linear dynamic analysis (LDP)
- 4- Nonlinear dynamic analysis (NDP).

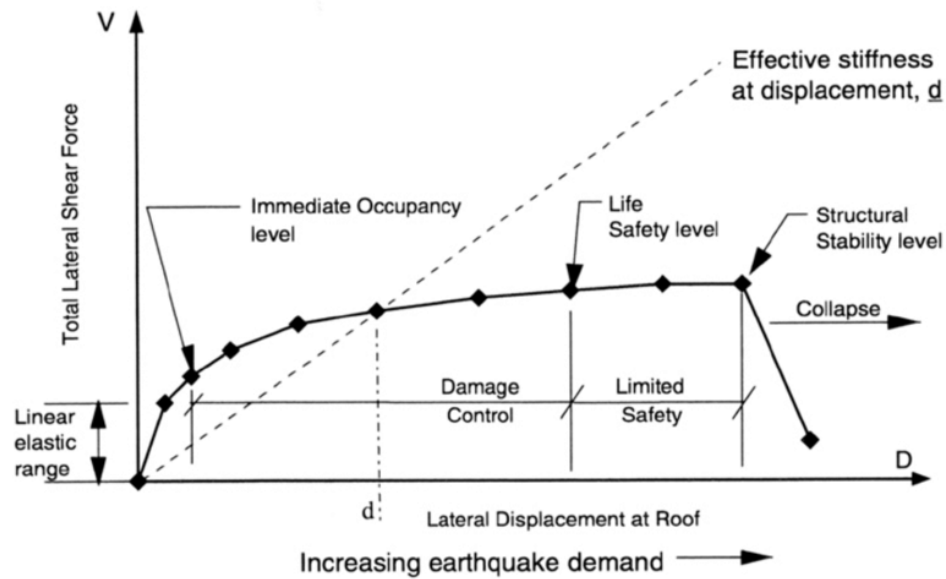
The first and simplest methods is the linear static which is suitable in the analysis of uniform structures with relative short height. Linear dynamic analysis is done using the response spectrum method. Both LSP and LDP are categories as elastic analysis methods to calculate internal stress. The different between static and dynamic linear analysis is related to the forces level and how to distribute this force along the height of structure. The nonlinear static analysis taken the inelastic behavior of the structure.

It is important to consider the nonlinear behavior of the structure in the seismic design and the pushover procedure is consider as a nonlinear static procedure and it happened by pushing the structure with an incremental lateral load or displacement, until the failure of the structures. The nonlinear dynamic analysis is based on combining the inelastic behavior of the structure and nonlinear time history load. The finite element software SAP 2000 considering the nonlinear static behavior of structure through incremental pushover analysis assuming elastic behavior between each increment while plastic hinges are forming at each step of incremental load or displacement. [7]

The response curve represented the main output of the pushover load analysis. The characteristics and shape of this curve depend mainly on the stiffness of the structure, sequence of the formation of plastic hinges and the distribution of ductile member in the structure. displacement can be used as an indication of the damage level inside the structure. Figure 8 shows a capacity response curve and the distribution of the level of damage along the pushover load analysis.

Figure 8

Capacity curve with different performance levels



The distribution of the pushover lateral loads is generally following the dominant vibration modal shape of the structure.

2.1.1 Pushover method

Pushover analysis is a way better than linear analyses (both static and dynamic), it is simple, also pushover analysis shows the actual response of the structure, moreover the formation of plastic hinge effects the total response of the structure. Generally, for the analyzed structure, the following points can be showed using pushover analysis: [17]

- Initiation and sequence of degradation of the structure in each number of floors and in each load step
- The reduction in the capacity of each member of the frames, especially columns and beams due to plastic spread.
- Location of the weak elements in the structure and their effect on the response
- The effect of week points and failure of members on the overall structural stability and behavior.
- The load bath adequacy

The pushover analysis methods (PO) that are used can be divided into three general groups: The Conventional pushover methods, the Adaptive pushover methods, and the Energy-Based pushover methods. The Conventional PO methods are the following:

- Capacity Spectrum Method, CSM
- Improved Capacity Spectrum Method, ICSM
- N2 method, adopted by the EC
- Displacement Coefficient Method, DCM
- Modal Pushover Analysis, MPA

Mohssen Izadinia et al (2011) studied the Response modification factor for steel moment-resisting frames by different pushover analysis methods, Among the main conclusions drawn, is that the maximum relative difference for response modification factors was about 16% obtained by the methods of conventional and adaptive pushover analyses this shows the various techniques of P.O analysis can be trusted to produce similar results. [18]

Generally, the pushover methods have some limitations. First, it is assumed that the applied push overload conforms in distribution to the domination vibration mode. Generally, the first modal shape of the structure must be determined and dominates the behavior. If the first mode was not the dominating mode in the structure, then a single point to represent the displacement of the structure like the roof displacement may not be a good representative of the behavior of the structure. Also, an interaction of different modes may affect the response. Second, the distribution of the maximum internal forces changes at the time, each earthquake has its unique characteristics and the acceleration is changing with time also the effect of load reversal. However, the distribution of inertia forces and lateral load pattern is assumed to be uniform and constant in pushover analysis during the earthquake and each step of pushover load analysis. Therefore, the load pattern distribution is very important to be representative as the important of chose a target displacement location. [7]

The pushover analysis can produce a correct result if the building is controlled by a major fundamental mode during the entire EQ.

2.1.2 Load-controlled vs displacement-control in pushover analysis

In the load control regime, the force is incremented through steps until a specified value is reached, and the resulting displacement is recorded at each step. This method is used only when the total applied load is known (i.e. gravity load). In the displacement control analysis, the displacement is increased and the needed equilibrium force is recorded. This method allows the capturing of the softening behavior of the building.

2.2 Local nonlinear behavior

The definition of local nonlinear behavior of the structure can be done through two different approaches:

1. Continuous plasticity, where the material behavior is updated at every location in the member according to the material stress-strain law. To achieve this level of modelling, either 3D modelling of the element is done, or a simplifying section fiber approach is used. In the fiber element approach, the nonlinearity is obtained on the cross-sectional level and then the overall nonlinear behavior is interpolated along with the member.

This approach is computationally expensive but very accurate.

2. Concentrated plasticity, in this approach the plastic behavior is assumed to occur in a specified location and all inelastic deformation is concentrated at the location of the plastic hinge.

If non-linear analysis is conducted for reinforced concrete frames the moment-curvature relationships for all members cross-sections is needed. real moment-curvature relationships need the property of material and the constitutive models for confined concrete and unconfined concrete, and reinforcing steel properties. [19] The moment curvature relationship for reinforced concrete cross-section under different load condition like moment or moment plus axial force can be calculated and found for each step of axial load. [20] Mander model is used to calculate the moment-curvature relationship using the equilibrium and compatibility equations while stress-strain curves of steel and concrete is known and defined. [21]

FEMA 356 gives parametrized curves to definition the plastic hinges for different detailing slandered and for different applied loading conditions. These curves are used in the generation of the capacity curve.

Each material in construction work, such as concrete has classified according to FEMA 356. The next figure illustrates the general capacity curve parameters and the properties of plastic hinge.

Figure 9

RC members Force-Deformation relation

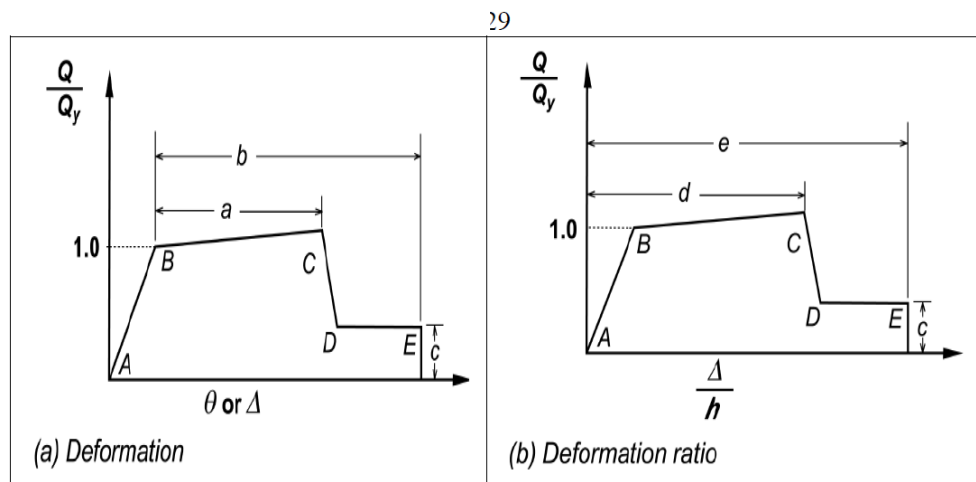


Figure 9 shows the plastic hinge behavior for a certain section, point A represent the zero-force condition while point B represented the first yielding point of the section then point C is the maximum and the design strength of the element with significant deformation and just after that the degradation is expected to start. point D represents the first failure of the section, from D to E this stage represents the final gravity loading on frame element. After point E the deformation is no longer withstand and the element is considered as failure.

Figure 10

RC beams FEMA 356 nonlinear parameter modeling.

Conditions	Modeling parameters ³			Acceptance criteria ³			
	Plastic rotation angle, radians	Residual strength ratio		Plastic rotation angle, radians			
				Performance level			
	a	b	c	IO	Component type		LS
Primary					Secondary		

i. Beams controlled by flexure¹

$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse Reinforcement 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 shear¹

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 span¹

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 joint¹

	0.015	0.03	0.2	0.01	0.01	0.015	0.02
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Notes:

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the most appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming. A transverse reinforcement 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 is at 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.

Ductility of reinforced structures is a desirable property where resistance to brittle failure during flexure is required to ensure structural integrity. Ductile behavior in a structure can be achieved through the use of plastic hinges positioned at appropriate locations throughout the structural frame. These are designed to provide sufficient

ductility to resist structural collapse after the yield strength of the material has been achieved. The available ductility of plastic hinges in reinforced concrete is determined based on the shape of the moment-curvature relations.

Ductility may be defined as the ability to undergo deformations without a substantial reduction in the flexural capacity of the member and this deformability is influenced by some factors such as the tensile reinforcement ratio, the amount of longitudinal compressive reinforcement, the amount of lateral tie and the strength of concrete. Theoretical moment-curvature analysis for reinforced concrete structural elements indicating the available flexural strength and ductility can be constructed providing that the stress-strain relations for both concrete and steel are known. Moment-curvature relationship can be obtained from curvature and the bending moment of the section for a given load increased to failure.

In this study, reinforced concrete columns and beams were designed and the effect of the longitudinal reinforcement ratio, axial force level and transverse reinforcement ratio on the behavior of these models were considered. In last decades various stress-strain relationships for unconfined and confined concrete were proposed by different researchers. A concrete model proposed by Mander which is widely used, has been used to determine the moment-curvature relationships of reinforced concrete members based on the XTRACT. Moment-curvature relations were obtained and presented in graphical form using XTRACT Software which takes nonlinear behavior of materials into consideration. The examined behavioral effects of the parameters were evaluated by the curvature ductility and the cross-section strength.

XTRACT offers six material models. Mander models for Unconfined and Confined concrete, Bilinear Steel, Bilinear with Parabolic Strain Hardening, Menegotto Pinto, and a User Defined. Material parameters can be input very quickly and stress strain diagrams can be viewed for correctness. Any number of material models can be added. Each one is defined by the name inputted by the user. This works well for section comparisons involving different transverse reinforcing ratios (different levels of confinement) as well as sensitivity studies on input parameters. The material models can be changed and/or modified at any time. They show up on the material in user-defined colors to verify that everything is where it should be. XTRACT also offers a User Defined material model page where stress strain points can be import from a file. The User Defined material model can be saved and will then available on subsequent uses of XTRACT.

2.3 Plastic Hinge Length

Plastic hinges form at the location of the maximum moment for both beam and column. The estimation of the length of the plastic hinge is a major point in the calculation of the load-deflection of the structure. It is more accurate to estimate the plastic hinge length by using complex nonlinear tools but computationally uneconomic, so, simplified empirical equations can be used to overcome this step. The length of plastic hinge effected with several factor, such as:

- 1) amount of axial load.
 - 2) moment slope.
 - 3) the level of shear stress in the plastic hinge location.
 - 4) longitudinal and transverse reinforcement ratio
 - 5) ultimate concrete strength and amount of tie steel provided in the plastic hinge joint.
- [22]

Table 3 shows many expressions proposed by many researchers. These expressions were derived or fitted to experiment data on a column or beam subjected to different load types.

Table 3

Empirical expressions for the plastic hinge (Xuemei Zhao et al, 2011).

Research Reference	Plastic hinge length expression L_p
Baker (1956)	$K (z/d)^{1/4} d$ for RC beam and column
Sawyer (1964)	$0.25d+0.075z$
Corley (1966)	$0.5d + 0.2 \sqrt{d \left(\frac{z}{d}\right)}$ for RC beam
Mattock (1967)	$0.5d+ 0.05z$ for RC beam
Priestley and Park (1987)	$0.08z + 6d_b$ for RC column
Paulay and Priestley (1992)	$0.08z+ 0.022d_b f_y$ for RC beam and column
Sheikh and Khouy (1993)	$1.0 h$ for RC column
Coleman and Spacone (2001)	$G_f / [0.6f_c (E_{20} - E_c + 0.8 f_c / E_c)]$
Panagiotakos and Fardis (2001)	$0.18z+0.021 d_b f_y$ for RC beam and column
Bae and Bayrak (2008)	$[0.3 P/P_0 + 3 A_s/A_g - 1] z + 0.25$

Interestingly, Berry et al, 2008. share a paper discuss the length of plastic hinge and presented equation for computing the length of the plastic hinge.

$$l_p = 0.05L + \frac{0.1 \cdot f_y \cdot d_b}{\sqrt{f_c}} \text{ (MPa)} \quad 2.1$$

The proposed equation includes the strength of concrete, type and amount of longitudinal steel but it does not consider the parameters considered by the authors. Berry 2008 make comparison between this formula and equation proposed by Corley 1966 and Paulay and Priestley 1992 (see table 5), and found that it provides sufficient value for the force-displacement response when comparing with 37 experimental tests on bridge circular columns. [23]

2.4 Program for Nonlinear Modeling

The concentrated plasticity approach can easily be implemented in the SAP2000 program which is a common and validated program. The procedure involves modelling the frames with elastic members and assigning plastic hinges of different kinds to regions of expected maximum moment. The SAP 2000 program generally conducts incremental static displacement-controlled analysis and updates the behavior of the plastic hinges based on their moment-curvature relations until the collapse or instability failure of the frame.

The local plastic hinge definition can be obtained either through the default code-based simplified curves such as the FEMA table (see table 3) or can be inputted from another software that produces moment-curvature curves from sections with various properties. One of the best programs that can produce moment-curvature curves is the XTRACT software.

XTRACT Software takes nonlinear and confinement effects and behavior of materials into consideration. XTRACT offers material models that is already defined on the program. [21] Concrete models with confined and unconfined condition, reinforced Steel definition, and a user defined model. The characteristic of the material is easy to add and the resulting stress strain curve is drawn. the confinement steel is easy to add and the program offers different transverse reinforcing ratios. The material models can be changed or modified at any time of the analysis. The User Defined material model can be saved and it can be used in the subsequent uses of the program. Also, XTRACT allows the input of axial force level on the section which leads to producing moment-curvature curves that vary depending on the axial stress level.

Chapter Three

Parametric Study

3.1 Introduction

the effect of beam-to-column relative stiffness on the R-factor for a special moment-resisting frame is studied, a parametric study will be conducted. In the parametric study, the main parameter will be identified and then a generic model will be constructed using SAP2000. The generic model will be varied and the resulting R-factor from each set of parameters will be computed and then compared. In the following section, the characteristics of the generic model and the parameters will be discussed.

3.2 Characteristics of the Generic Model

In this study, the generic model of a typical RC-SMRF building will be designed to fit the IBC 2018 code requirements and using $R=8$.

The plan of the frames is rectangular with 3 or 4 spans and each is (4, 6 and 8 meter) in length, some frames have variable span lengths. All frames have 3.5-meter floor height and each case is consider using five and ten floors.

3.3 Structural Systems

The gravity loads are transmitted by the slab to beam to the columns and then to the foundation, which in turn takes the load to the ground, while the seismic load is resisted by the beam and the column in the frame.

The type of the frame is a special moment-resisting frame (SMRF) with a solid slab and rectangular column with a different dimension of drop beams

Each case in this analysis needs to be designed as a special moment resisting frame and the strong column weak beam philosophy is one most important parameter to be taken into consideration in the design of special moment frames.

3.4 Materials

All the frames are made of reinforced concrete. The same concrete for all elements with f_c 25 MPa and with unit weight equal to 25 kN/m³. The steel is ASTM Grade 60.

3.5 Gravity loads

According to ASCE standards the live loads were taken and the superimposed dead load were calculated under the general weight of finishes in Palestine.

Super imposed load = 3.5 kN/m^2

Live load = 3 kN/m^2

3.6 Demonstration case

Let us take one of the cases and firstly, design it according to the previous assumption and secondly, analyze it to find the response modification R-Factor by applying nonlinear pushover analysis on the selected frame. the first and the typical case is a five-story SMRF with four spans in each direction and each span is a four-meter length. See figure 13 in

Characteristics of structural element for the demonstration case:

Table 4

Characteristics of structural element for the typical frame

Name	width	depth	Longitudinal reinforcement		Ties
Columns	50 cm	50 cm	8 Φ 20		1 Φ 10 /10
Beams	30 cm	30 cm	Top 4 Φ 20	Bottom 3 Φ 20	1 Φ 10 /10
slab	-	20 cm	-		-

3.6.1 Definition of Plastic Hinges

To define the properties of the plastic hinge in SAP 2000 you have two options, either, automatically according to FEMA 356 tables, or manually by defining the moment-curvature relationship.

SAP2000 gives default-hinge properties according to FEMA table and suggests axial load plus bending moment hinges for columns and bending moment hinges for beams. Firstly, the structure is design and it is member diminution and reinforcement ratio is known then the model is built with nonlinear load applied on it and assign all default hinges to the elements (PMM for columns and M3 for beams). In this study, the definition is user-input hinges by defining the moment-curvature curve for each section. The modified Kent and Park model for confined concrete and the typical steel stress-

strain model with strain hardening for steel are implemented in the moment-curvature analyses.

XTRACT program is used to make a section analysis in order to calculate the moment curvature for beam and column, in beam analysis plastic hinge is simply define by calculating the moment curvature for the section in positive and negative direction, while in column we need to add an axial load effect on each moment curvature.

For PMM plastic hinge, SAP2000 program asked for the number of axial loads for each PMM hinge and the value of the axial force. In this study four levels of axial loads are used to create the PMM hinge properties with the following values (0.0Pu, 0.1Pu, 0.4Pu, 0.8Pu). The section analysis is done using XTRACT program to draw the moment-curvature relationship for the SMRF section. Figure 14 in Appendix E describe the definition of the plastic hinge for the demonstration case.

3.6.2 Pushover Load Definition

The first step is to define an initial load condition, a nonlinear gravity loads is applied as an initial load condition then nonlinear lateral pushover load start acting on the structure member that carrying the gravity nonlinear load. The gravity load is defined as a nonlinear load case including full dead load (structural weight and superimposed dead load) and 25 % of the live load (IBC, 2018). See figure 13 Appendix E

According to FEMA 356, fundamental mode shapes is used to normalized the applied nonlinear pushover load, this step make the analysis as closed as the expected earthquake actions, because the structure is expected to response during an earthquake events same as the fundamental mode. The table below illustrates the vectors for the first mode fundamental shape and normalization to the top displacement and figure 15 in appendix E show the pushover load.

Table 5

normalization for pushover load in the demonstration case

ΦX	Normalize X
0.0465	1
0.0424	0.91
0.0362	0.78
0.0274	0.59
0.0158	0.34

Then the pushover load is defined as nonlinear with displacement controlled and case and continues from the end of gravity case by monitoring the roof displacement as a single degree of freedom indicator.

The result of the analysis is the pushover curve, also called the P- Δ curve or force top displacement curve. This curve is the key to the calculation of the response modification R-Factor and for understand the behavior of the building. With it we can predict the collapse sequence and understand the weakness of the structure and the sequence of the plastic hinges and its order and its location moreover the ultimate displacement before the collapse and the actual load needed for collapse. See Figure 16 in Appendix E.

From figure 17 in Appendix E P- Δ curve for our demonstration case five floors with four spans each with four-meters in length is generated using SAP2000 program, the final output is to calculate the R-factor.

R-Factor is given as the ratio of Elastic Force (V_e) to Yielding Force (V_y) 4.1

$V_e = 4550$ kN the elastic force corresponding to a displacement equal 0.35m ($0.02h$).

V_y from the first plastic hinge formed = 609kN . $R = 8.03$.

According to the IBC code the R-factor is equal to 8 for SMRF.

Newmark and Hall equation for computing R-factor is related with periods, for flexible structure and period more than 1 second:

$$R\mu = \mu = \Delta_{max} / \Delta_y = 0.394 / 0.0517 = 7.62$$

And this follow the equal displacement with natural period more than 1 second and and the ultimate displacement 0.394m is very closed to the target displacement which can be calculated by IBC 2018 code by $0.02H$, and H is the hight of the structure, and this gives a target displacement equal to 0.35m .

Each of IBC 2018 code, Newmark and Hall equation, equal energy and equal displacement role and our procedure gives nearly the same value for R-factor and it is fit with all assumption. Figure 18 in Appendix E gives the distribution of plastic hinges before the collapse in the demonstration case

In the next section, we will focus on our parameter (relative stiffness between beam and column) and generate all the models and finally find the effect between R-Factor and the relative stiffness.

3.7 Models Generation

To study the effect of column-to-beam relative stiffness on the R-Factor of SMRF, two-building layouts are chosen and used to generate models that cover many parameters. The two layouts represent two scenarios one with uniform span length and the other with variable span length. These models were designed according to ACI code and then all the nonlinear moment-curvature relationships for all cross-sections are generated using EXTRACT section analysis. Finally, pushover analysis is done to generate all the capacity load-deformation curves, which are used to calculate the R-Factor. A comparison between the results and the IBC code for the R-factor is conducted.

The symbol G will be adopted to represent the sum of ratio of flexural stiffness between column and beam for each model. Specifically, in our study, this ratio equals:

$$\Sigma G = \Sigma \{I_c / L_c^3 \div I_b / L_b^3\} \times \frac{L_{min}}{L_{max}} \quad 4.2$$

The value of G is calculated for each joint in the ground floor. Where L_{max} is represented the longest span in the frame, L_{min} is represented the shortest span in the frame, I is the moment of inertia for the column and the beam, and L is the length for the column and the beam and the summation on ground floor. The ΣG is defined in this way because based on our results we found that R-factor does depend on the ratio $\frac{L_{min}}{L_{max}}$, as will be seen later.

In this section, more information is given about the models used in the nonlinear analysis. Overall, the models are grouped into two main categories:

- a- uniform span lengths.
- b- non-uniform span lengths.

3.7.1 Models of uniform span lengths

In this category of models, the span lengths of all bays are kept constant for each model. The number of bays, the number of floors and the length of spans are the main parameters that were varied to study the effect on R-factor.

Firstly, the number of floors is varied by considering models with 10 and 5 floors. Realistically as the number of floors is increased the column cross-section will increase and this will increase the G-factor. Thus, by varying the number of floors, the effect of gravity loads on the columns and the vibration modal shape with it is consequence.

Secondly, the number of bays in each building is also a parameter in this study. This parameter affects the summation of the G-factor (ΣG) and is expected to affect R-factor by increasing the redundancy and the available load path. Buildings with two, three and four bays are studied to investigate the effect of the number of bays on the R-factor. Thirdly the span of the beam is varied into three realistic values of 4, 6 and 8 meters for each model and it is kept constant for all bays of uniform spans.

Finally, to study the effect of plastic hinge behavior on the R-factor, one of the cases is studied using the definition of the plastic hinge based on the FEMA 365 tables, while in the rest of the models, the plastic hinge is defined manually using XTRACT moment-curvature.

3.7.2 Models of non-uniform span length

Most of the buildings suffer from irregularities because of various architectural demands. Thus, the distribution of columns could be irregular leading to non-uniform spans. This is common in Palestine where residential buildings have corridors. Therefore, new models of similar characteristics to the earlier models of uniform spans are generated but this time with variable span lengths. An example is where the interior frame has a smaller span compared to the exterior spans, as illustrated in the figure 19 in Appendix E.

Table 6 in Appendix F contains all the: uniform and non-uniform spans. The model name for each case contains four symbols with their meaning:

B: indicates the number of bays.

N: indicates the number of floors.

L: indicates the length of the span.

V: indicates non-uniform span length.

3.8 Pushover Load Generation

The pushover curve is very important to calculate the R-Factor. Each case is designed according to ACI-318 code as a special moment frame and then the resulting cross-section with reinforcement inside is used to generate the moment-curvature curve which in turn helps in the definition of the plastic hinges in the structure. The distribution of pushover load on the structure is followed the shape of the fundamental vibration modes. Finally, the pushover curve is generated.

3.8.1 Effect of Definition of Plastic Hinge (FEMA VS XTRACT).

The effect of changing the definition of plastic hinges on the resulting load-deflection curve is investigated. By comparing the R-factor from two cases. first case, the plastic hinge was defined using the FEMA tables second case plastic hinge is defined using a plastic hinge extracted from sectional analysis using XTRACT software. Two cases are studied using model N5 B3 L4. Figure 18 in Appendix E gives the pushover curves for the two cases.

It can be seen from Figure 20 in Appendix E that both FEMA and XTRACT definitions of plastic hinges give almost the same load-deflection curves, but the XTRACT definition gives more ductility. This can be explained by the fact that XTRACT software incorporates the hardening behavior better than the FEMA definition. In general, both load-deflection curves give almost the same R-factor of 7.5 for XTRACT program and 7.4 for FEMA tables.

3.8.2 Effect of Number of Bays on R-Factor.

Firstly, in order to investigate the effect of the number of bays on the R-factor both the number of floors and the span length are considered constant. Results from 4m span length and 5 floors models will be discussed.

It can be seen from Figure 21 and Figure 22 in appendix E that, as the number of bays increases both plastic capacity and ductility increase for the same number of floors and

equal spans lengths, in turn will increase the value of the R-factor from 7 to 7.5 to 8. Secondly, keep the length of spans 4m but this time increase the number of floors to ten.

Thirdly, frames with 6 and 8m spans lengths are used to investigate the effect of the number of bays on the R-factor. See Figure 23 and Figure 24 in appendix E.

All the previous cases investigate the effect of the number of bays on the load-deflection curve and on the R-factor, increasing the number of bays increases the ductility and the plastic capacity moreover the R-factor for all cases.

3.8.3 Effect of Number of Floors on R-Factor

To investigate the effect of the number of floors on the R-factor and load-deflection curve frames with five and ten floors are studied with a different number of bays and different spans lengths.

Firstly, five floors frame is studied against a ten floors frame with three bays and four-meter span lengths. Secondly, a six-meter spans with 4 bays is investigated to see the effect of increasing the number of floors from five to ten.

It is clearly notable that, the higher number of floors gives more plastic capacity and increases the ductility of frames. In figure 25 in Appendix E R-factor is increased from 7.5 to 7.9 by increasing the number of floors and in figure 26 in appendix E also, the R-factor increase from 8.1 to 8.5.

3.8.4 Effect of Length of Spans on R-Factor

Considering ten floors frames with 4 bays and different spans lengths to investigate the effect of the span length on the load-deflection curves and R-factor. See figure 27 in Appendix E.

the plastic capacity increase as the span length increase but the ductility will be affected and decrease with keep increasing the span length. also, there is no significant effect on R-factor it increases from 8.2 to 8.4 to 8.5.

3.8.5 Effect of Non-Uniform Span Length

In the non-uniform span length, 3 and 4 bays are used with 5 and 10 floors, with different spans lengths. Firstly, a five floors frame with three bays is used to investigate the effect of

non-uniform span length, the different span is either an exterior or an interior span with a smaller length than the other two spans. See figure 28 in Appendix E.

It's easy to say that both nonuniform cases give almost the same load-deflection curves, but the case 6-6-2 gives more plastic capacity and the curve is smoother and both cases give almost the same R-factor of 6.3, the distribution of moment different for the 2m span and this clearly seen when comparing the results with uniform case N5 B3 L6 that has a more plastic capacity and ductility and also a higher R-factor of 7.8.

From Figure 29 in Appendix E, the moment distribution at the collapse load is almost constant all the beam and for the case 6-2-6 it is full constant, and this effect the distribution of plastic hinges and reduce the R-factor.

The next case is the same as the previous case but with smaller section for the two-meter span length to confirm with the real design. Although, the common practice is to have a continues beam dimension but this case will illustrate how the span length ratio does affect R-factor. See Figure 30 in appendix E

Although, the R-factor increase and the capacity curve for the smaller inner dimension beam section have a higher plastic capacity and higher ductility still under the code requirements.

Figure 31 in Appendix E showed the effect of number of bays for the nonuniform span length ratio

Increasing the number of bays will increase the plastic capacity and the ductility and also R-factor is increased from 6.2 to 6.8.

Figure 32 in Appendix E discuss the case of 8-4-8 which gives R-factor equal to 7.7, while case 6-2-6 gives R-factor equal to 6.22, and this is because of the length of the interior span, and the ratio between the smallest and larges span in the frame. The final comparison will focus on this point.

Three different ratio between L_{max} and L_{min} is considered, 6-2-6, 6-3-6 and 6-4-6 with $\frac{L_{min}}{L_{max}}$ ($\frac{1}{3}$, $\frac{1}{2}$ and $\frac{2}{3}$).

From Figure 33 and 34 in Appendix E it is clear to say that as the span length ratio increase the capacity and the ductility for the structure increase and also R-factor increase. This is happened because of the distribution of moment in the small, and the plastic hinges start to get more uniform when the span is more uniform and with higher span length ratio.

3.9 Calculation of R-Factor

The pushover curves are used to calculate R-factor, two values need for computing R-factor, firstly the yielding force which represents the change in the behavior of the structure from the elastic to the inelastic phase. In our study, this force is taken as the force needed to form the first plastic hinge in the structure.

V_y force needed to form the first plastic hinge in the structure

C_s : the seismic response coefficient.

W : the effective seismic weight.

The elastic force represents the maximum sustained force by the structure assuming the behavior to remain elastic during an earthquake up to the maximum drift ratio (0.02 h) ASCE 7-16.

Where h : is the height of the structure

Also, this force must not exceed the intersection between the pushover curve and the capacity spectrum acceleration with 5% damping.

Table 6*R-Factor value for all cases*

Case	Fy	Fe	R	ΣG
N5 B4 L4	609	4891	8	80
N5 B3 L4	349	2621	7.5	66
N5 B2 L4	228	1597	7	55
N5 B4 L6	1009	8166	8.1	114
N5 B3 L6	448	3427	7.65	98
N5 B2 L6	305	2241	7.35	82
N5 B4 L8	1552	12727	8.2	140
N5 B3 L8	977	7711	7.8	120
N5 B2 L8	601	4666	7.7	100
Case	Fy	Fe	R	ΣG
N5 B3 L4 (FEMA)	346	2563	7.4	66
N10 B4 L4	643	5311	8.2	231
N10 B3 L4	486	3791	7.8	198
N10 B2 L4	346	2531	7.3	165
N10 B4 L6	782	6693	8.5	750
N10 B3 L6	587	4650	7.9	642
N10 B2 L6	413	3100	7.5	428
N10 B4 L8	1703	14461	8.5	910
N10 B3 L8	1146	9250	8	780
N10 B2 L8	830	6474	7.8	560
N5 B3 L (V) 6 2 6	866	5390	6.22	63
N10 B3 L (V) 6 2 6	991	6822	6.88	39
N5 B3 L (V) 6 6 2	882	5556	6.3	39
N10 B3 L (V) 6 6 2	1170	8310	7.1	43
N5 B4 L (V) 6 2 2 6	1311	8400	6.4	38
N10 B4 L (V) 6 2 2 6	1472	10011	6.8	40
N5 B4 L (V) 6 2 6 2	1463	8750	6	18
N10 B4 L (V) 6 2 6 2	2469	12312	6.2	20
N5 B3 L (V) 8 4 8	937	7233	7.7	93
(N5, B3, L (V) 6,2(S),6 m)	803	5480	6.82	23

Figures 35, 36 and 37 in Appendix E Illustrates three important facts, firstly as the number of bays increase the R-factor also increase, while increasing the span length has almost no effect on R-factor, finally, increase the number of floors effected R-factor and increased it.

For the non-uniform cases increasing the number of bays also increase the R-factor, and the cases that have the smaller span length in the sides gives a higher value for R-factor than having the smaller span in the middle of the structure.

3.10 Summary

As a summary of the finding in this chapter, increasing the ΣG -factor leads to an increase in the R-factor. In some cases, SMRF with non-uniform spans and with G-factor less than 80 get a value of R less than the code-specified value of 8, as seen in figure 38 in Appendix E.

Overall for frames with uniform span length, increasing the number of bays gives more load bath and increases the R-factor. Also, increasing the number of floors will enhance the R-factor because this will increase the ductility of frames. Increasing the length of the span also increases the R-factor because this increases the plastic capacity for the frame.

Finally, in the non-uniform cases, the R-factor is less than the code-specified because of the effect of the span length ratio, frames with smaller spans like 6-2-6 having a uniform distribution of moment in the smaller span and this effect the generation of the plastic hinges and also effect the R-facto. On the other hand, span length ratio $\left(\frac{L_{min}}{L_{max}}\right)$ also effect R-factor, as this ratio increase and get close to 1, the R-factor get close to the code value.

3.11 Verification of the Model

The models were built on SAP 2000 program and to verify it elastically we need to make a manual calculation

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

Equilibrium: it can be approved by calculating the weight of the structure and comparing it with the base reaction

Stress-strain relationship: it can be approved by calculating the moment or the deformation manually and comparing them to the result from the structural.

1 compatibility check: OK

2 Equilibrium check:

Dead load = slab weight + columns weight + beams weight

$$\text{Slab} = 5 * 16 * 16 * 0.2 * 24.5 = 6272 \text{ kN}$$

$$\text{Columns} = 5 * 25 * 3.5 * 0.5 * 0.5 * 24.5 = 2680 \text{ kN}$$

$$\text{Beams} = 5 * 10 * 16 * 0.3 * 0.1 * 24.5 = 588 \text{ kN}$$

$$\text{Total dead load} = 9540 \text{ kN}$$

$$\text{Total Live load} = 5 * 16 * 16 * 3 = 3840 \text{ kN}$$

$$\text{Total SD load} = 5 * 16 * 16 * 3.5 = 4480 \text{ kN}$$

See figure 39 in Appendix A.

The equilibrium check is OK with % of error equal zero (0.00%).

Stress-strain relationship check

Let us take one of the frames in the middle and calculate the moment by ACI coefficients and compare them with the result from program

The distribution of the live load from the slab = $4 * 3 = 12 \text{ kN/m}$

$$\text{Total moment in the beam} = \frac{w * L^2}{8} = 18.4 \text{ kN.m} \text{ see figure 40 in Appendix A.}$$

The sum of the moment from SAP = $11.5 + 6.9 = 18.4 \text{ kN.m}$ Ok.

Therefore, we can clearly say that the program works good in the elastic calculation.

Definition of the response spectra load:

the seismic load was added to the structure firstly by defined the response spectrum curve and then by added a new load case in two direction full load in the primary direction and 30% of the load in the second direction. See figure 41 and 42 in Appendix A.

Before the starting of the design three check need to be OK

- Check drift
- Check P-delta
- Torsion

In each case of the study, the frame went through all these checks and then the phase of the design started, ensuring that the frame is special with all its elements which is reflect in the definition of the plastic hinges in the next section.

Elastic period: in order to calculate the period of the structure, both mass matrix and stiffness matrix are calculated and the results is compared with SAP2000 result.

Table 7

Calculation Stiffness for the Floors of Case Study Building X or Y Direction

Name	width	length	Hight	I	I/L ³	No	Σ EI/L ³
1	0.5	0.5	3.5	0.003646	8.50286E-05	25	149862.9
2	0.5	0.5	3.5	0.003646	8.50286E-05	25	149862.9
3	0.5	0.5	3.5	0.003646	8.50286E-05	25	149862.9
4	0.5	0.5	3.5	0.003646	8.50286E-05	25	149862.9
5	0.5	0.5	3.5	0.003646	8.50286E-05	25	149862.9

E con = 23.5 MPa

Table 8

Rayleigh Method

Floor No.	F.Disp shape	F.mass	F.K-XorY dir
5	1	280	149862.9
4	0.91	280	149862.9
3	0.78	280	149862.9
2	0.59	280	149862.9
1	0.34	280	149862.9

Table 9*Comparing the results*

	Rayleigh Method	SAP2000	Diff%
X or Y dir	0.95	1.08	12%

3.12 Verification of Plastic Hinge

Saeid Foroughi and Bahadir Yüksel (2020) investigate the moment curvature relationship for reinforced concrete column. The effect of the material model, axial load, longitudinal reinforcement ratio, transverse reinforcement ratio and transverse reinforcement spacing on the behavior of reinforced concrete cross-sections were investigated.

column models with dimensions of 400mm×400mm were designed with the following details and the models were investigated under five different axial forces; $N1=0$, $N2=480\text{kN}$, $N3=960\text{kN}$, $N4=1440\text{kN}$ and $N5=1920\text{kN}$.

In this thesis, the XTRACT program is used to analysis of any cross-section and the generation of Moment Curvatures curves.

Moment Curvature Analysis Methodology in XTRACT:

Moment curvature analysis is often used in earthquake engineering as a necessary step toward assessment of the displacement capacity of reinforced concrete components. A moment curvature analysis establishes the ductile capacity of a cross section by plotting the curvatures against corresponding moments.

Material Models. Analysis begins with the specification of nonlinear material models as stress-strain diagrams. For reinforced concrete, three typical material models must be defined: steel, unconfined concrete, and confined concrete. The stress-strain behavior of steel depends on the material type and strength but generally can be described as a three-part relation: linear, constant zero slopes (the yield plateau), and a strain hardening branch. Numerous models for confined and unconfined concrete have been proposed by many researchers (Kent and Park, 1971; Vallenias et al, 1979; and Sheikh and Uzumeri, 1980). Confined concrete mathematical models incorporate effects of increased compressive strain capacity in addition to an increased compressive strength as a

function of passive confinement from transverse reinforcing steel. One commonly used model in moment-curvature analysis is the Mander Model proposed by Mander et al (1988). The Mander model accounts for the effects of confinement by variation of the input parameters such that the same mathematical formulation can be used to describe unconfined, lightly confined, and heavily confined concrete behavior.

Discretization. With the material models defined, the cross section must be cut into a series of layers, if moments about one axis are considered, or fibers if moments about two axes are considered. Tighter mesh sizes will give more accurate results within the confines of the material models but at the cost of increased computation time. When a loose (or coarse) mesh is used, unconservative or incorrect results may ensue. Each fiber (or layer) within the discretized cross section is associated with a tag identifying it with a specific material type as defined by the material model.

Analytical Methods. With the materials defined and mapped to the fibers (layers) of the cross section, an applied axial load is specified along with the analytical method for finding the moment curvature coordinate pairs. There are two general methods used in moment curvature analysis: displacement control and force control.

applied the first case in XTRACT program, 40*40 cm column with zero axial force and same material used in Saeid Foroughi and Bahadir Yüksel research and the same reinforcement gives:

There is no different between XTRACT program and the research and both give almost the same result so we can use the XTRACT program for the generation of plastic hinges.

3.13 Verification of Pushover Procedure

The procedure used by SAP2000 is hinge to hinge method. This method will be verified by the way of plastic hinge sequence through 2D frame model present in fig1, with assuming plastic hinge with no hardening and $M_p = M_y = 300\text{kN}$. See figure 44 Appendix A

this portal frame is 6m length and 4m hight, its plastic hinge is defined as moment only (M3), with no hardening. See figure 45 Appendix A.

The resulting pushover curve of this case is given in figure 46 and 47 in Appendix A

This curve has two points need to be calculated:

Started by push the frame with 300kN in the X-direction and see the moment distribution on the members: see figure 48 and 49 in appendix A

First moment appeared in the base with moment equal to 330kN.m and the second moment equal to 240kN.m, while the displacement is 0.0261m.

So, first hinge formed in the base with maximum shear force of 273 and displacement of 0.024m with almost zero error in with SAP2000 pushover curve.

Secondly, the base is now hinge and the applied shear force is 50kN see figure 49 in Appendix A.

The needed moment for the formation of the second plastic hinge is 84kN.m and this is done by sheer force of 50kN, and the resulting displacement is 0.019m.

The second point on the curve is 332kN shear force and a cumulative displacement of 0.43m, while the error is almost zero.

Chapter Four

Conclusion and Recommendation

4.1 Conclusion

This thesis study and highlight the variation of the response modification factor (R-Factor) of special moment resisting frames (RC-MRFs). The R-Factor assumed from the seismic design codes is studied and compare against the nonlinear R-Factor obtained from nonlinear static pushover analysis.

The effect of the column to beam relative stiffness on the R-Factor is investigated by considering two buildings layouts, one with a uniform span length and the other with nonuniform spans length. These two situations were designed according to the IBC 2018 code requirements. The output of the design is used to generate all the plastic hinges needed to create the nonlinear analysis using the pushover analysis procedure. The final output was a nonlinear load-deflection capacity curve. The capacity curves were used to calculate the R-Factor for all cases.

This study highlighted many findings and conclusions and there are summarized in the section below.

4.2 Main Result

There are a difference between building code regarding the value of R-Factor. The American code gives in some cases wrong representation of the actual behavior of the structure during an earthquake action.

The code gives a single value of R-Factor, this value may become over-conservative in some cases or under conservative in other cases.

The following point represents the main conclusions of this thesis:

- There is huge variation between building codes about R-factor.
- Increasing the number of floors will increase the R-factor
- R-Factor is sensitive to the number of bays in the frame. Increases the number of bays also will increase the R-factor.

- The variation in the length of spans affected R-Factor.
- Changing the length of the spans have almost no effect on R-Factor
- The nonuniform span length cases give lower value for R-factor. The nonuniform span length with smaller spans in the middle gives the worst case for R-factor and it could be unconservative with the recommended code value of 8. Case 6-2-6 can have R-value of 6.2 (less than 8), moreover Newmark and Hall method seems to comply with our results and the value of 5.84 can be obtained for R-factor ($R\mu = \mu = \Delta_{max} / \Delta y = 0.2157 / 0.0369 = 5.84$)
- A comprehensive methodology to calculate R-factor presented in this thesis.

4.3 Limitations and Future Recommendation

1. Torsional effects are assumed negligible on the structural elements and only bending and axial force is considered.
2. The building was treated as a SDOF in the pushover analysis. This means that only the fundamental mode governs the vibrational modes.
3. All the buildings are regular. Highly random buildings may give different performance and different results and required a different assessments tool.
4. This thesis can be extended throw incremental nonlinear dynamic analysis by repeating the same analysis using nonlinear time-history analysis considering both geometric nonlinearity and material nonlinearity.

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Appendices

Appendix A

Figures

Figure 11

RC columns FEMA 356 table for nonlinear modelling parameter .

Conditions	Modeling parameters ⁴			Acceptance criteria ³						
	Plastic rotation angle, radians	Residual strength ratio		Plastic rotation angle, radians						
				Performance level						
				Component type						
			IO	LS	CP	LS	CP			
a	b	c								
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 ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing ≥ 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 ≤ 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.

Figure 12

3D view of a typical frame

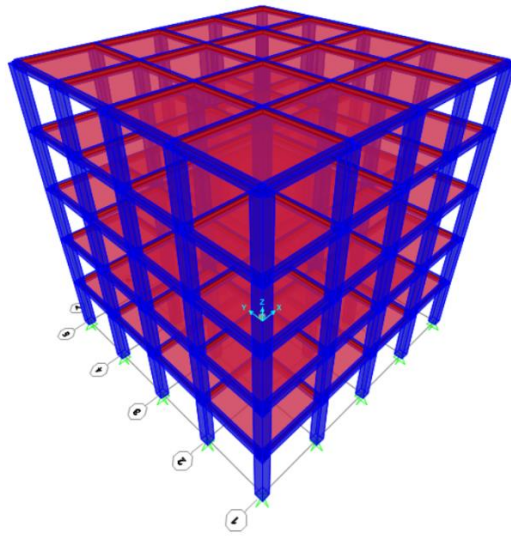


Figure 13

2D frame view for the demonstration case

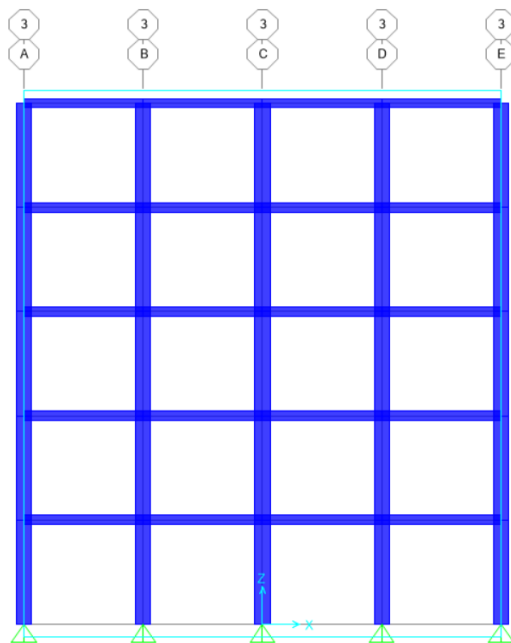


Figure 14

Moment curvature relationship for beam in demonstration case.

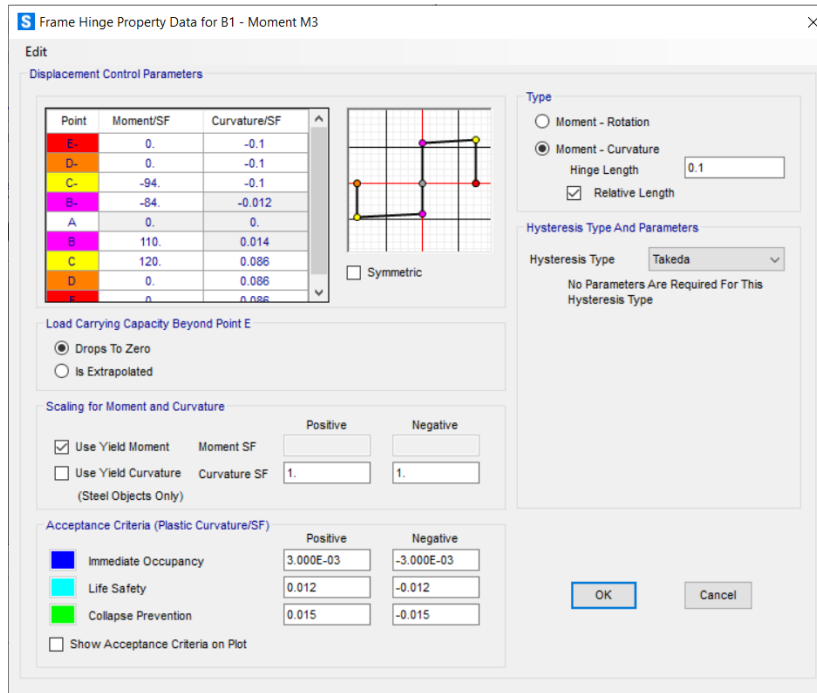


Figure 15

Gravity nonlinear load

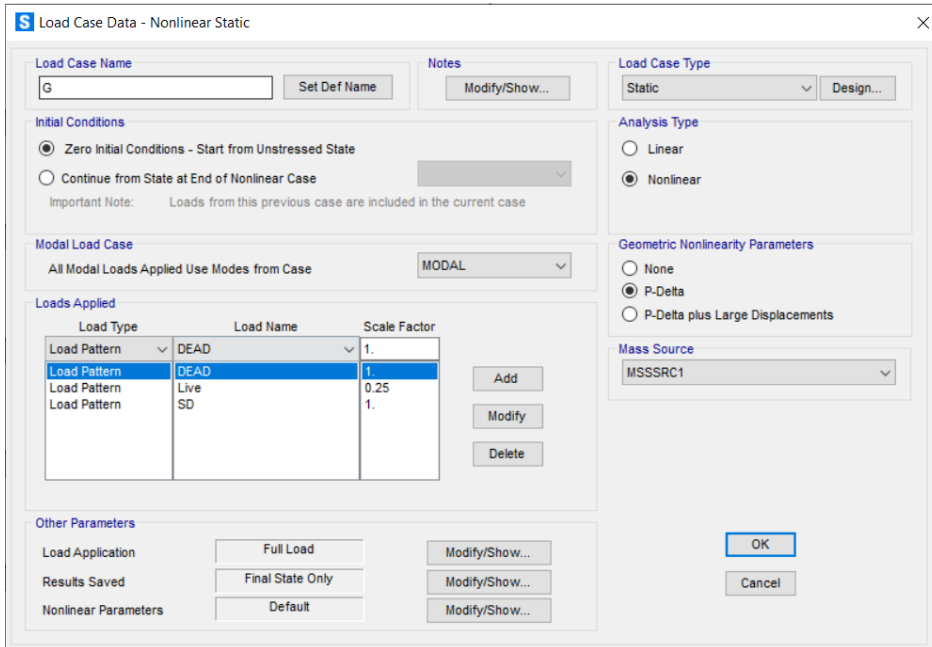


Figure 16

Pushover nonlinear load applied on joints of the frame.

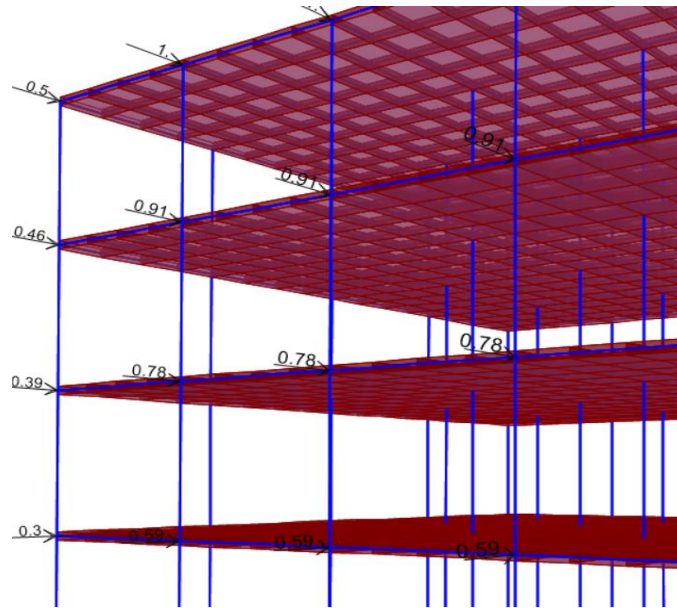


Figure 17

P-Δ curve for demonstration case.

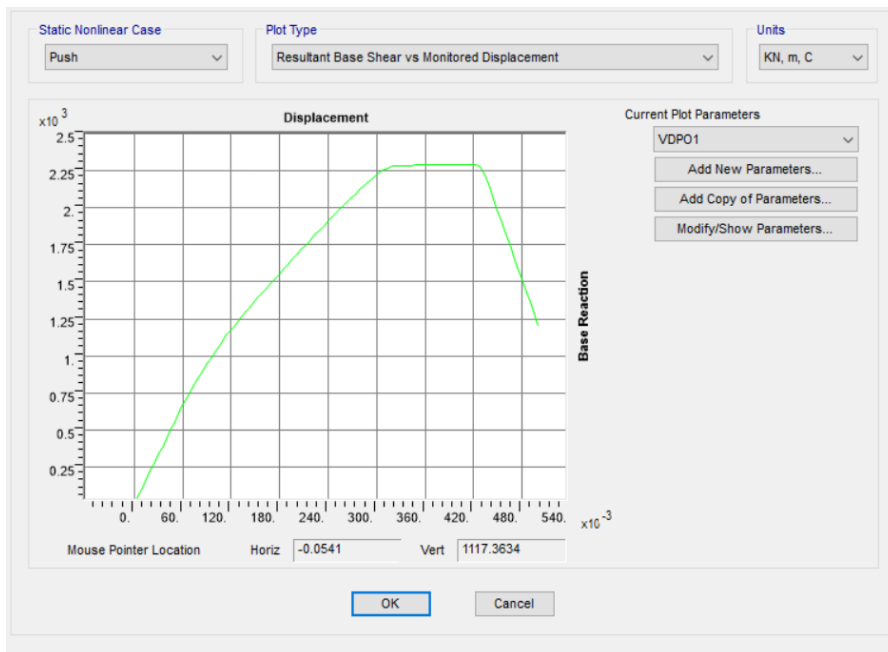


Figure 18

Distribution of plastic hinges before the collapse in the demonstration case.

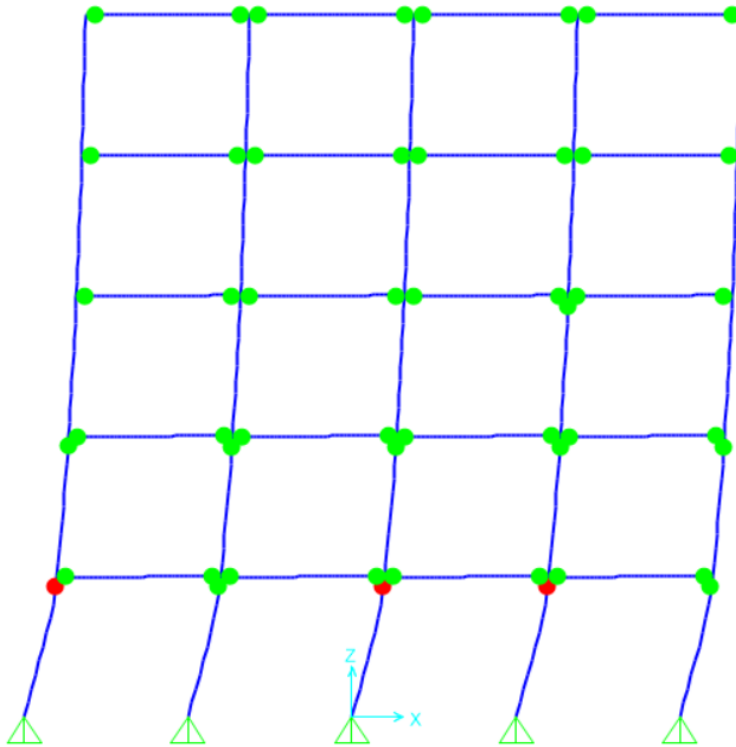


Figure 19

Various length span frame, 6m for exterior and 2m for the interior.

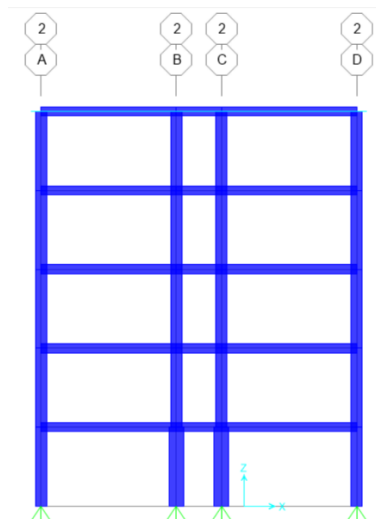


Figure 20

Effect of Varying Plastic Hinge definition on Load-Deflection Curve

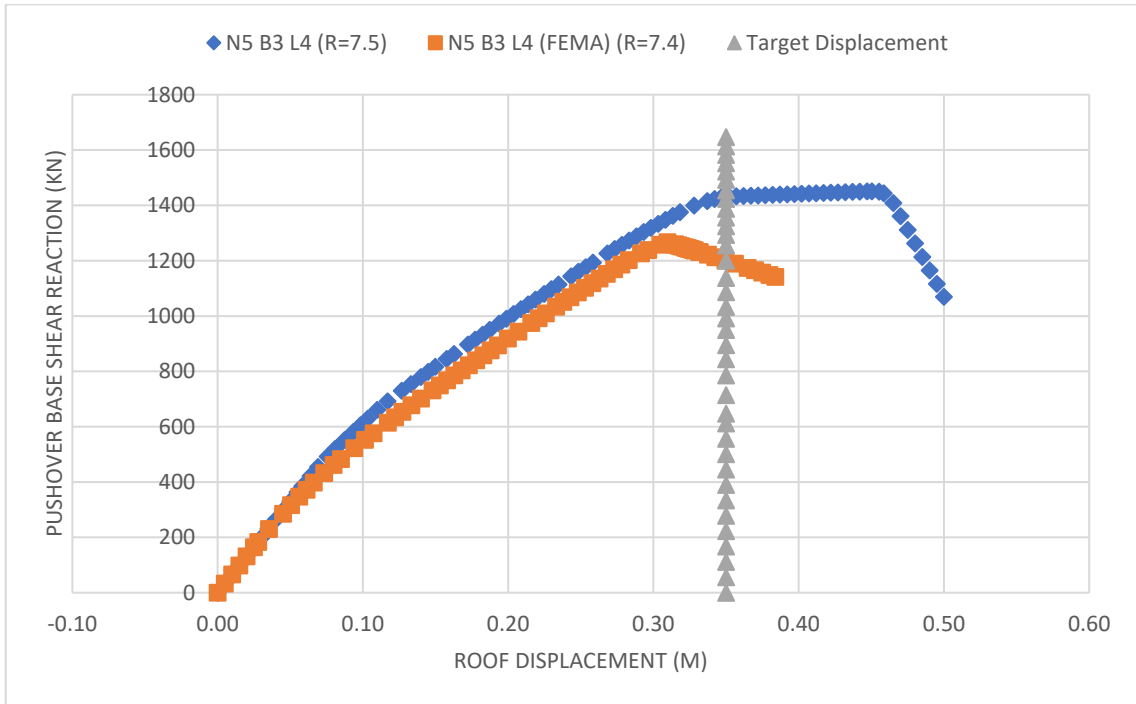


Figure 21

Effect of number of bays on 5 floors frames with 4m span length

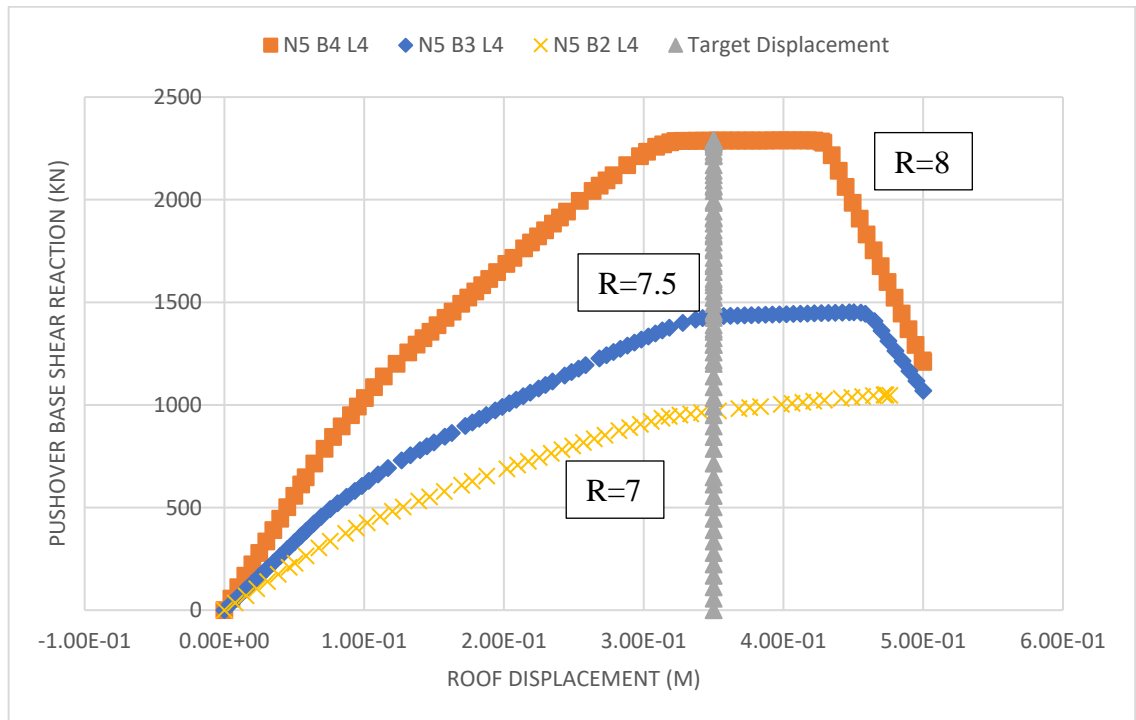


Figure 22

Effect of number of bays on 10 floors frames with 4m span length

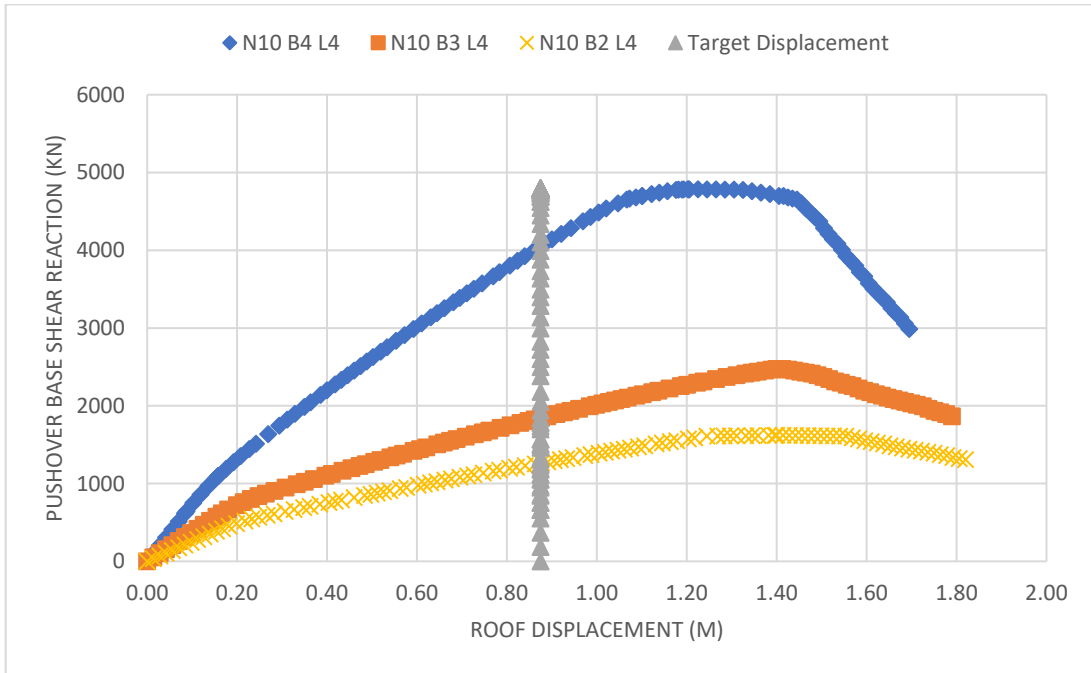


Figure 23

Effect of number of bays on 5 floors frames with 6m span length

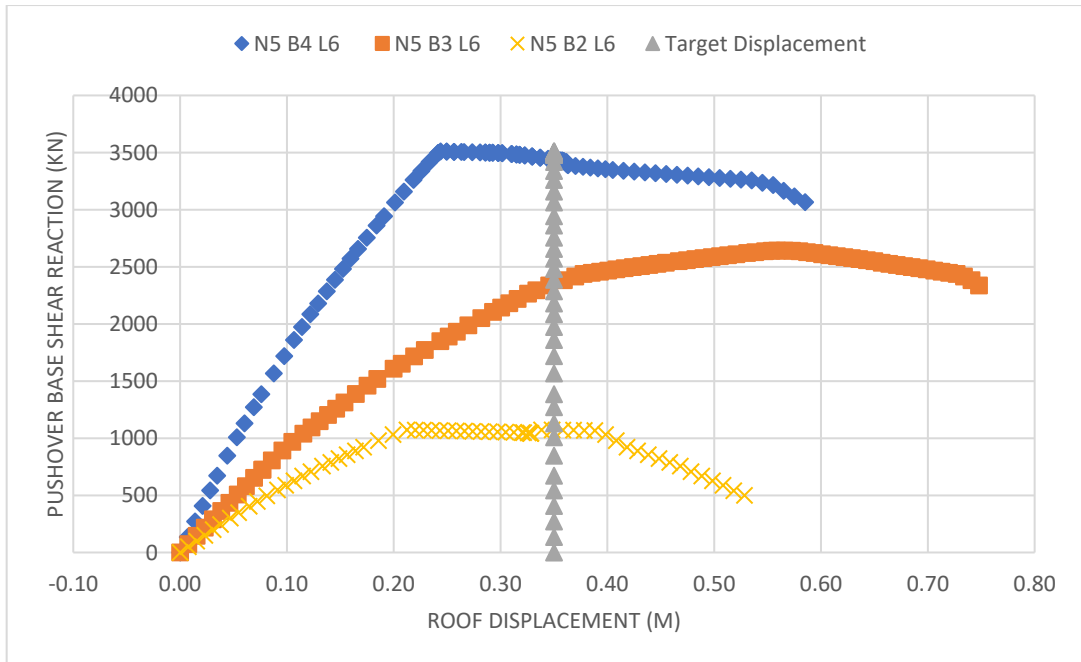


Figure 24

Effect of number of bays on 5 floors frames with 8m span length

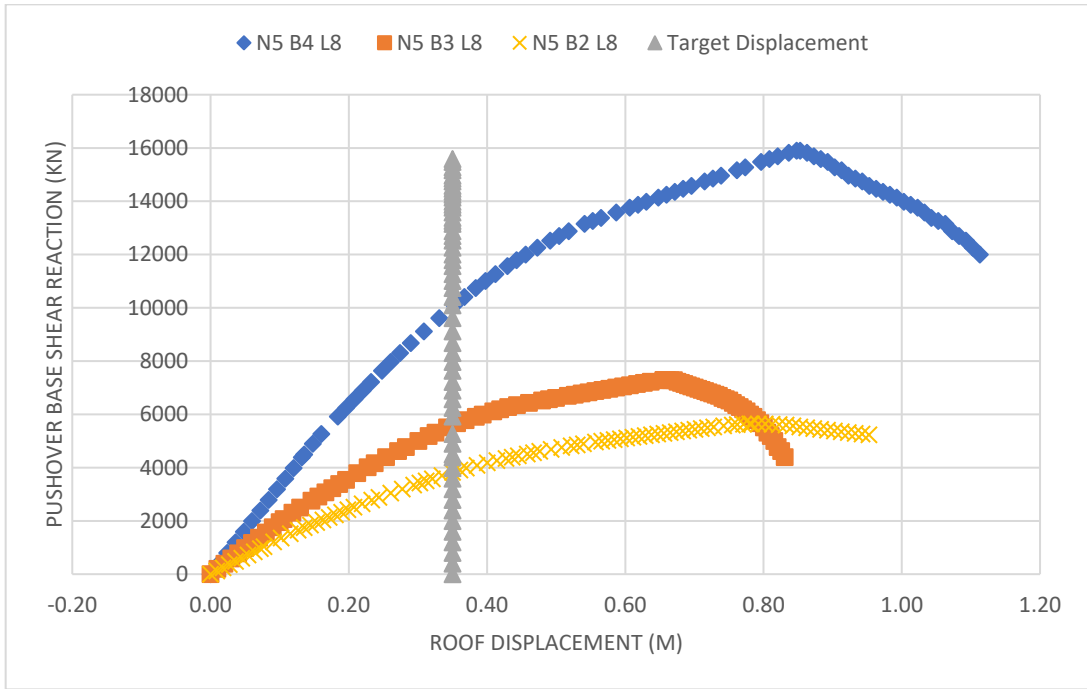


Figure 25

Effect of number of floors on 3 bays frames with 4m span length

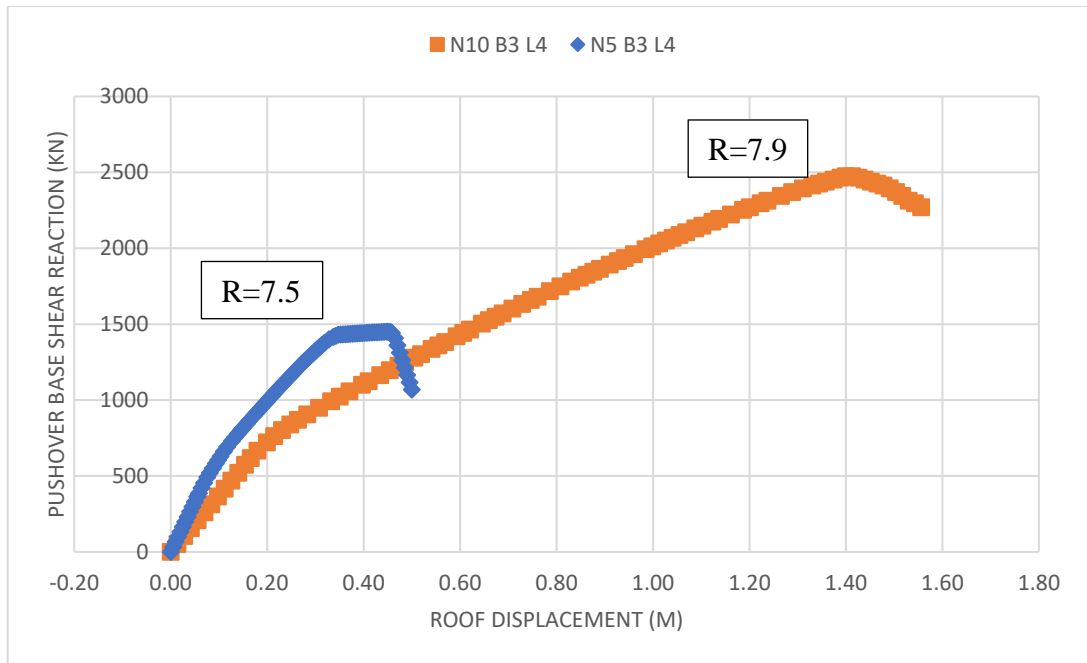


Figure 26

Effect of number of floors on 4 bays frames with 6m span length

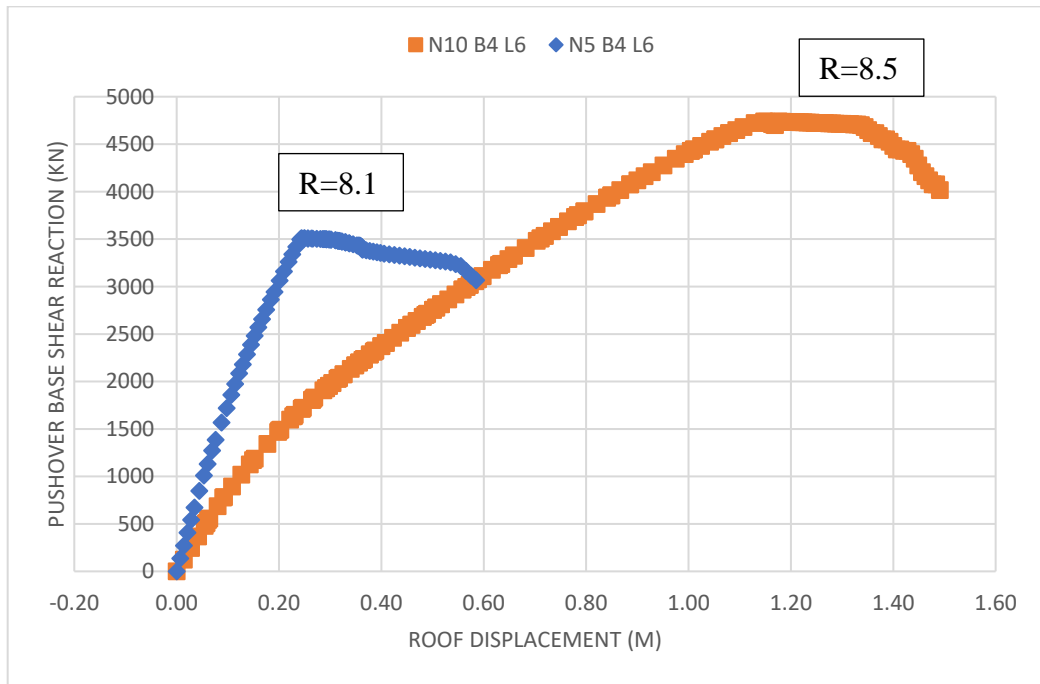


Figure 27

Effect of length of spans for 10 floors frames with 4 bays

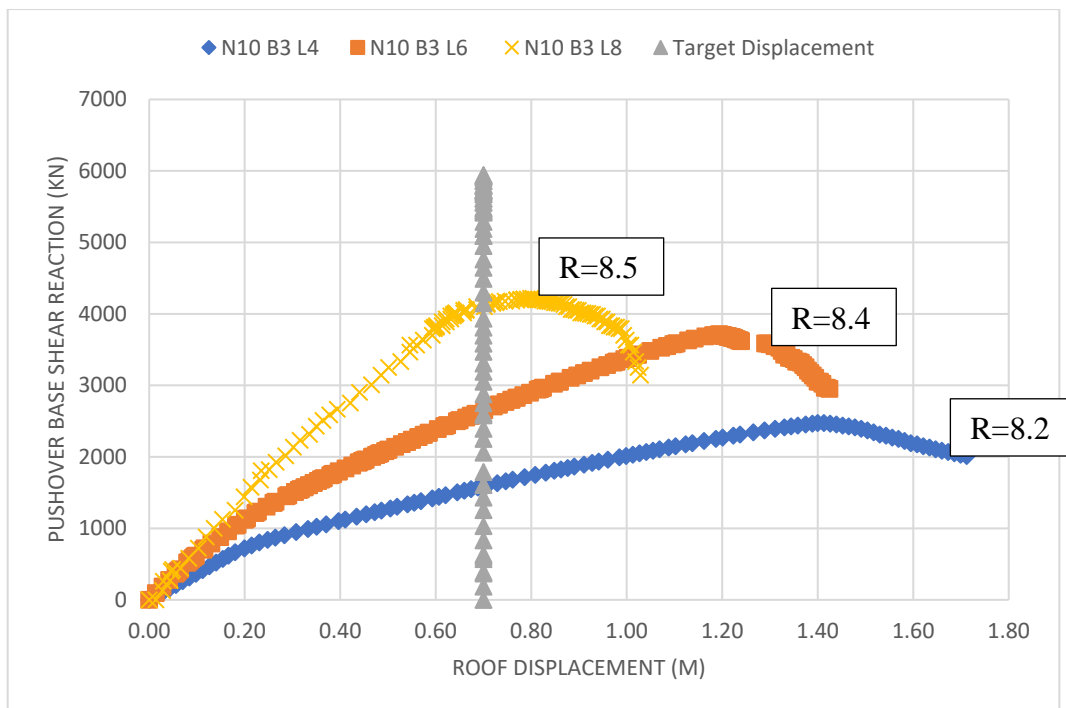


Figure 28

Effect of non-uniform span length for 5 floors frames with 3 bays

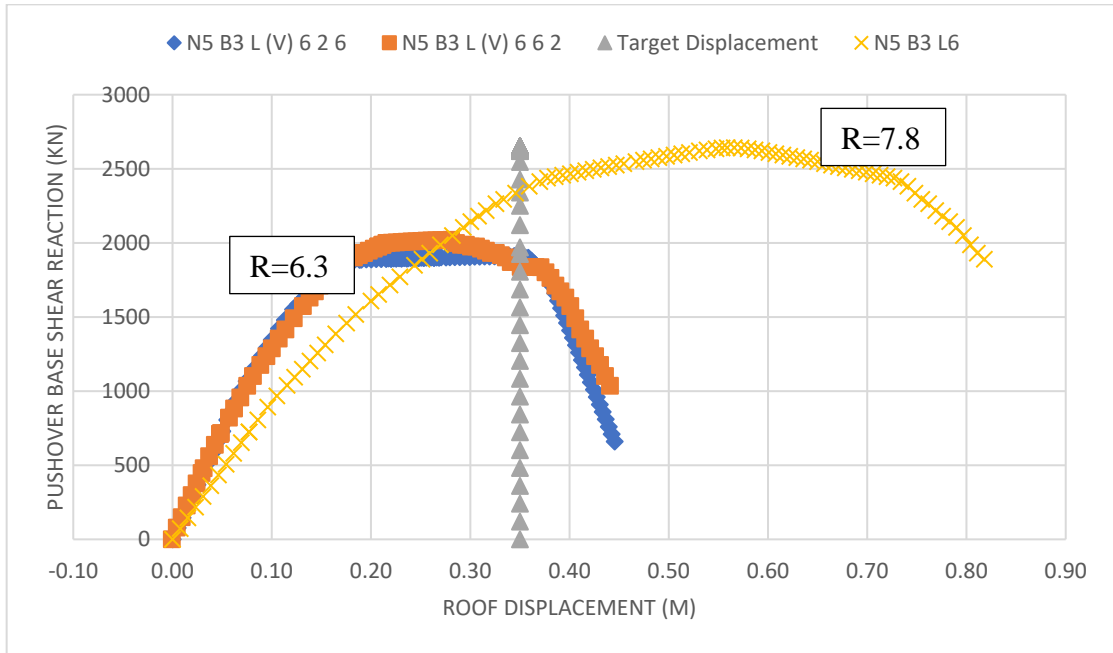


Figure 29

moment diagram for the 2m span length in case 6-6-2

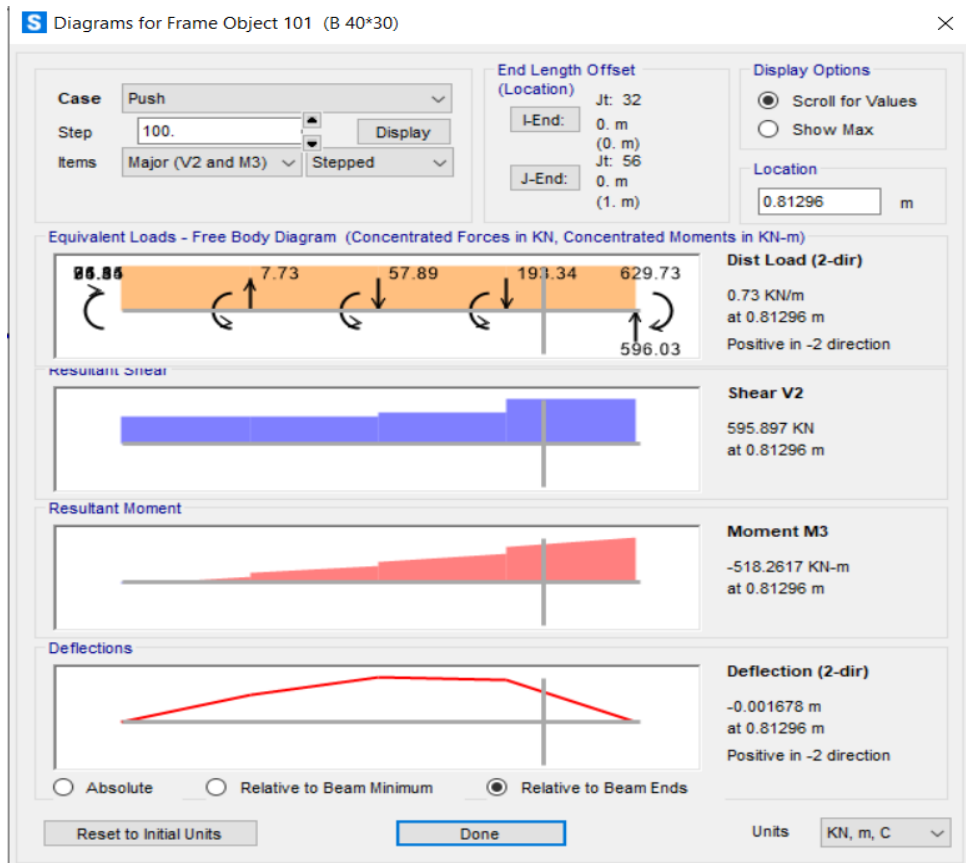


Figure 30

Effect of non-uniform span length for two different inner beam dimensions.

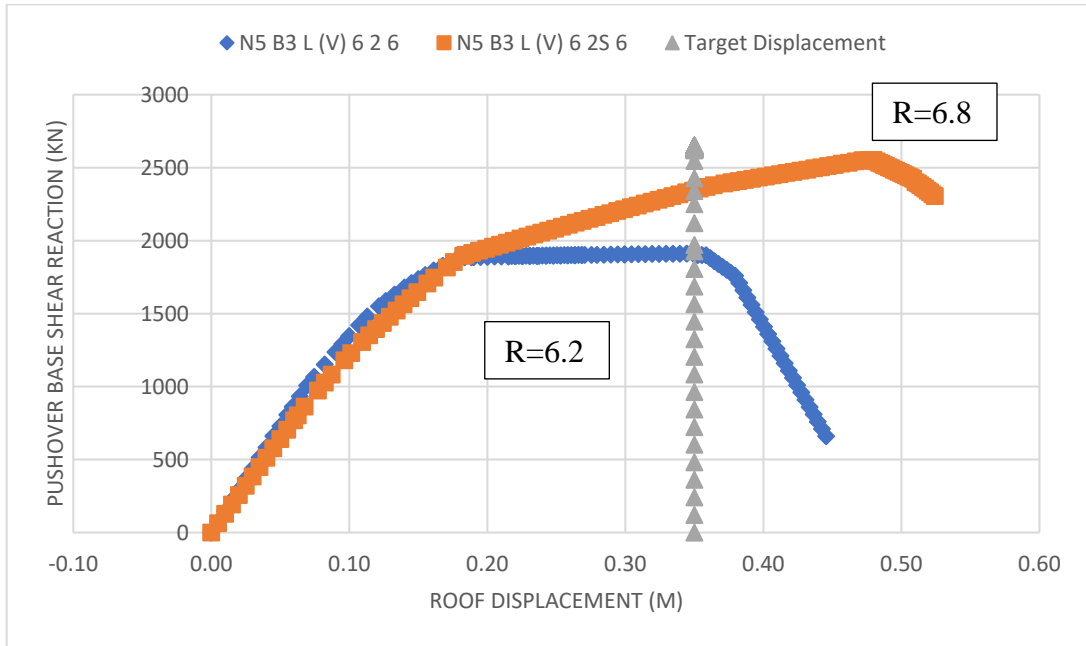


Figure 31

Effect of non-uniform span length for 5 floors frames with 3 and 4 bays

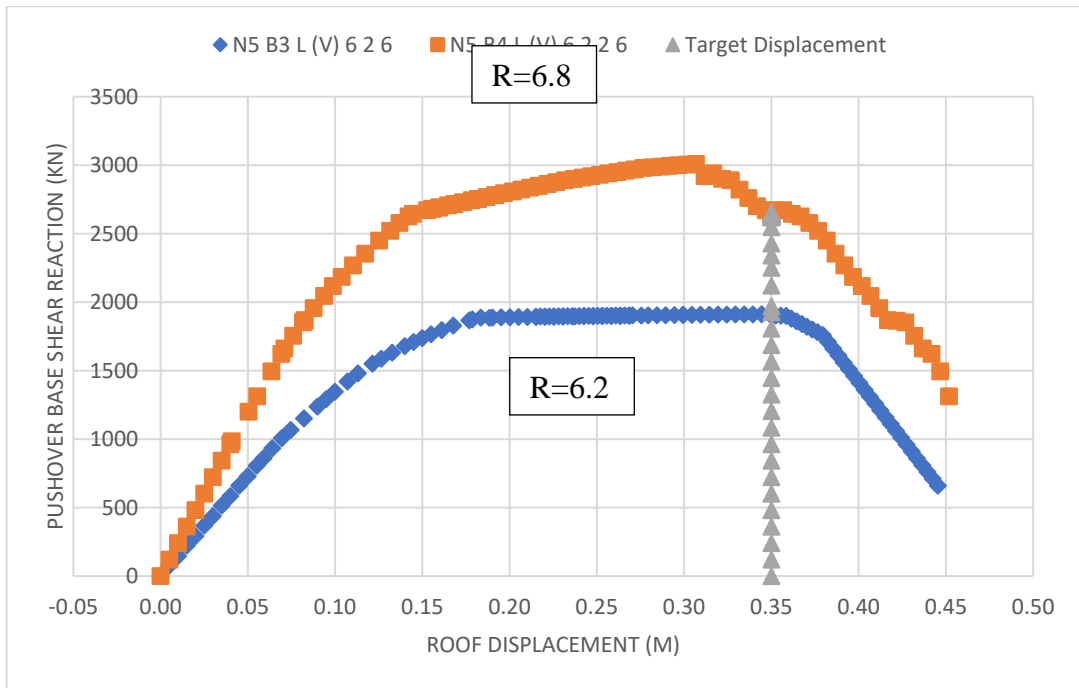


Figure 32

Effect of non-uniform span length for 5 floors frames with 3 bays

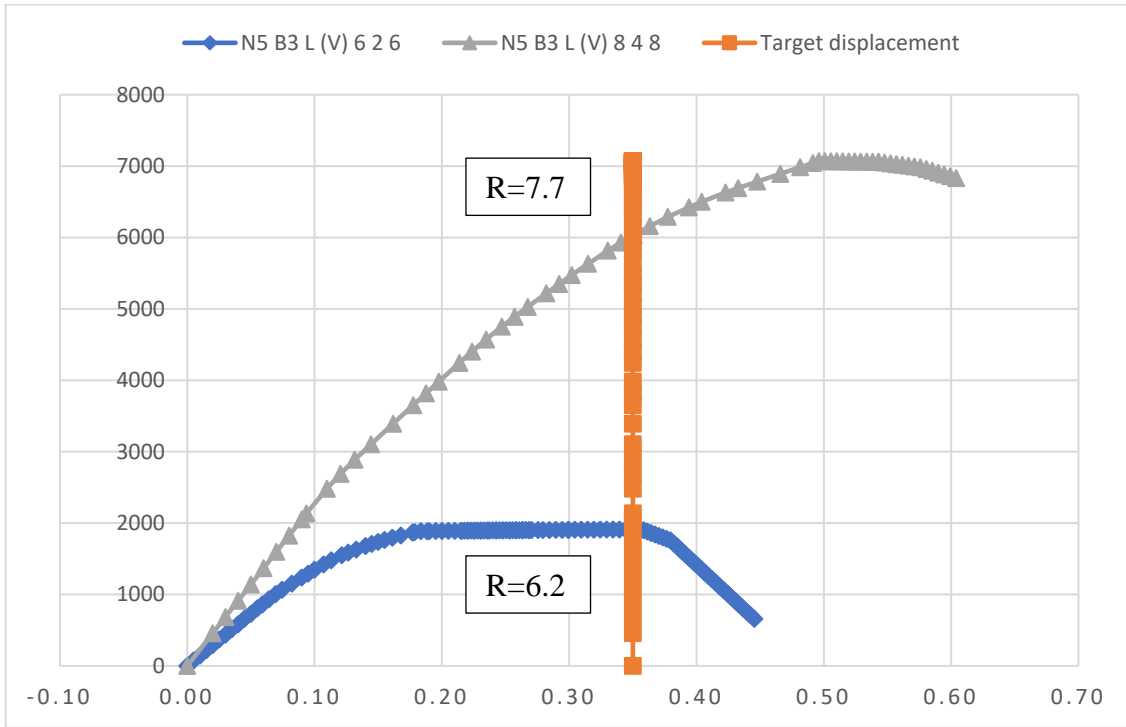


Figure 33

Effect of span length ratio on nonuniform span cases

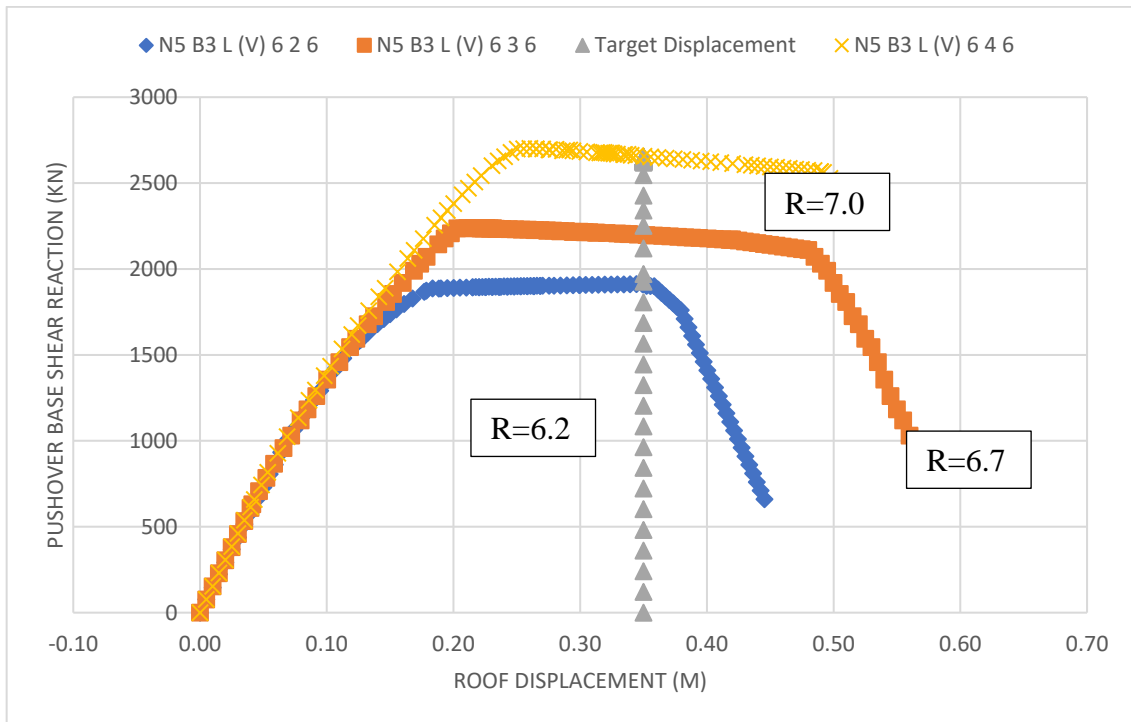


Figure 34

R-factor for different span length ratio

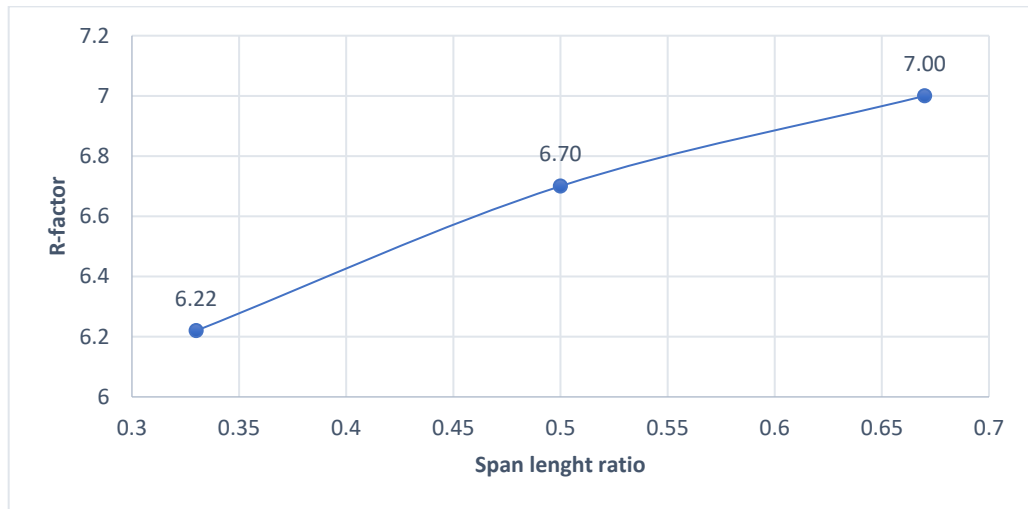


Figure 35

R-factor for 5 Floors frames with different spans lengths and different number of bays.

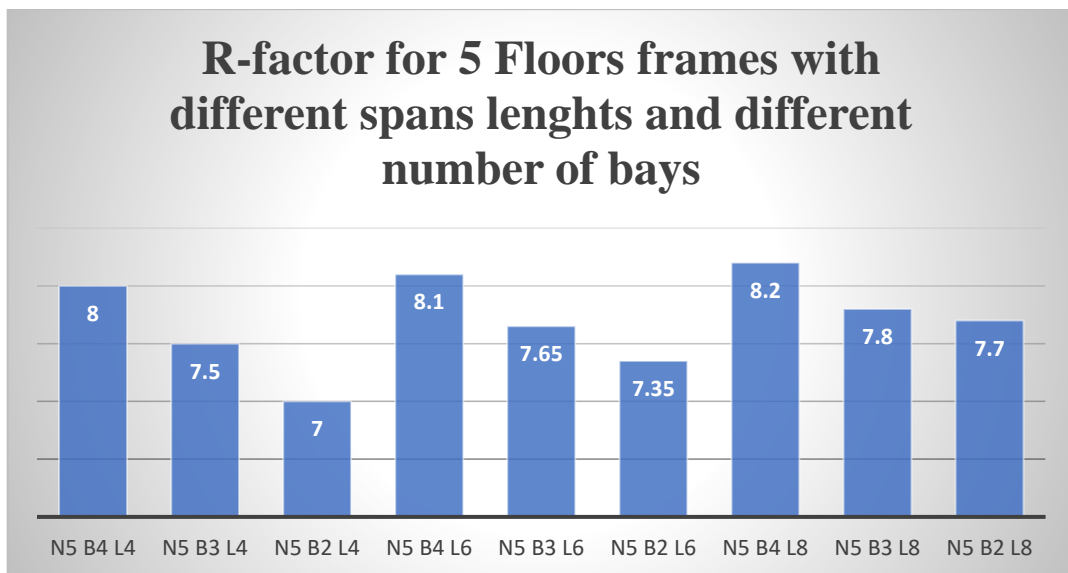


Figure 36

R-factor for 10 Floors frames with different spans lengths and different number of bays

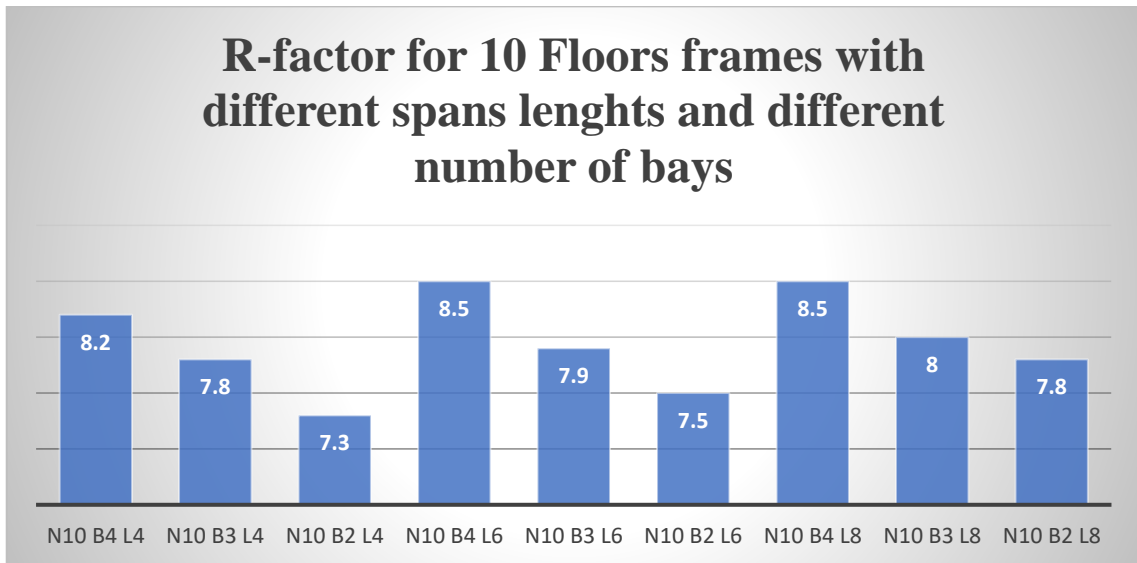


Figure 37

R-factor for non-uniform spans lengths

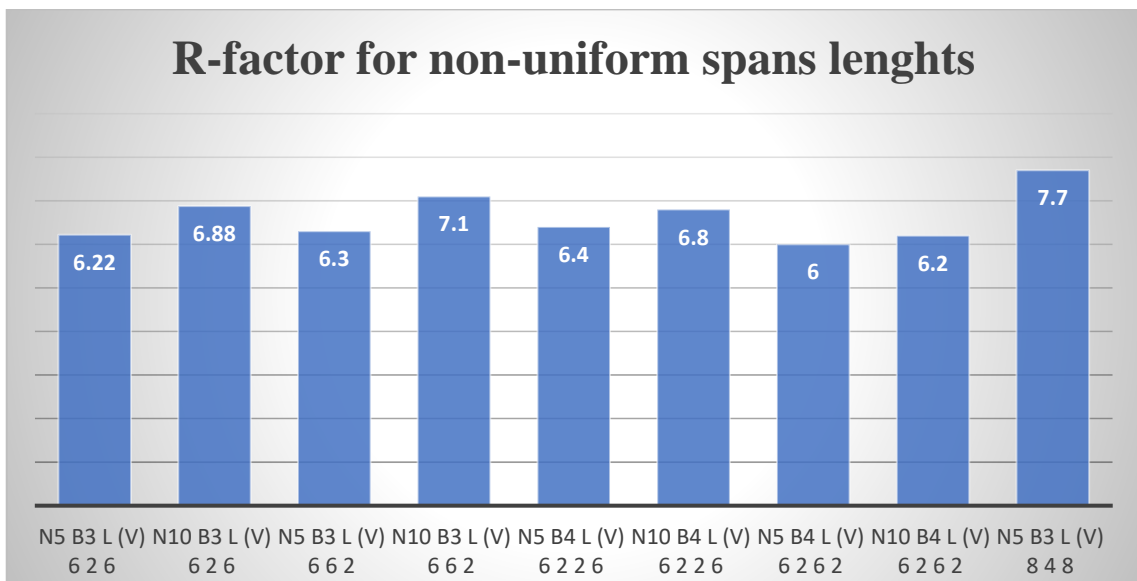


Figure 38

R-factor for all cases

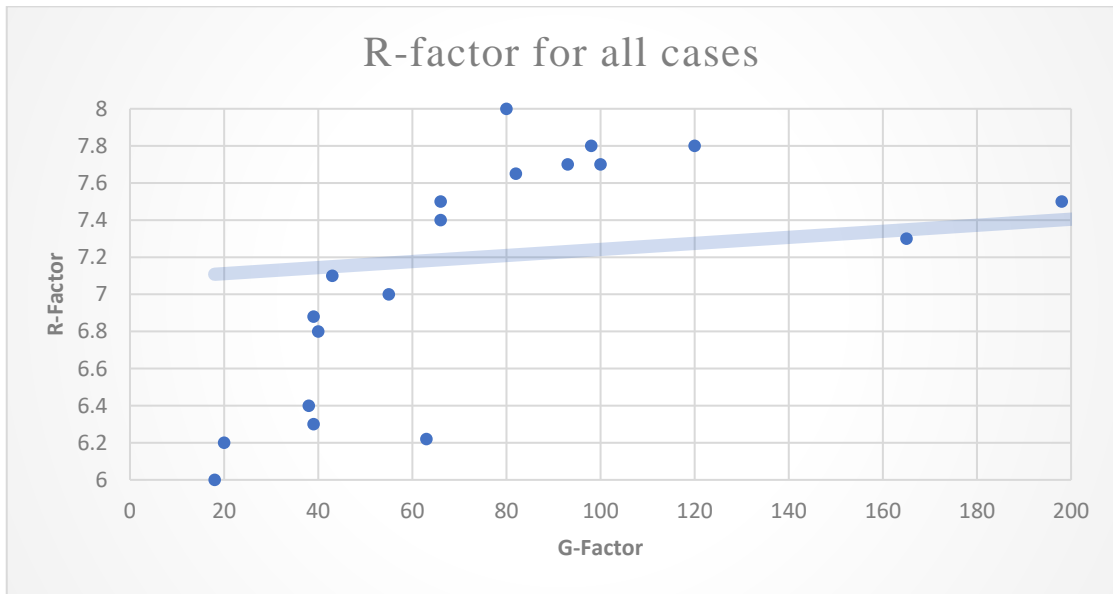


Figure 39

Base shear reaction from SAP 2000

S Base Reactions

File View Edit Format-Filter-Sort Select Options

Units: As Noted Base Reactions

Filter:

	OutputCase	CaseType Text	StepType Text	StepNum Unitless	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m	GlobalX m
▶	DEAD	LinStatic			1.057E-13	1.066E-14	9539.687	2.024E-11	-1.023E-11	-4.228E-13	0
	MODAL	LinModal	Mode	1	430.287	3.633	8.348E-07	-43.276	5125.0694	-3.309E-07	0
	MODAL	LinModal	Mode	2	-3.633	430.287	9.956E-07	-5125.0694	-43.276	-2.24E-07	0
	MODAL	LinModal	Mode	3	-2.742E-07	1.644E-07	6.883E-06	8.856E-06	-4.077E-05	3896.9643	0
	MODAL	LinModal	Mode	4	1724.249	10.538	-0.0006409	67.3629	-11017.7903	-0.0002471	0
	MODAL	LinModal	Mode	5	10.538	-1724.249	-0.0005984	-11017.7931	-67.3362	-9.002E-05	0
	MODAL	LinModal	Mode	6	0.0002417	0.0002477	0.00281	-0.0206	0.0261	15519.5742	0
	MODAL	LinModal	Mode	7	-4347.944	-834.004	0.0002305	874.4507	-4558.7066	0.0001438	0
	MODAL	LinModal	Mode	8	834.005	-4347.946	-0.002501	4558.5663	874.3336	0.0042	0
	MODAL	LinModal	Mode	9	-0.0006218	0.006456	0.041	1.0248	0.0957	40487.6374	0
	MODAL	LinModal	Mode	10	-7988.082	-0.327	-0.000913	-0.3576	8661.1923	-0.0014	0
	MODAL	LinModal	Mode	11	0.327	-7988.082	-6.133E-05	-8661.194	-0.3559	0.0003791	0
	MODAL	LinModal	Mode	12	-0.001584	-0.001533	-0.036	0.2413	0.1218	74381.1631	0
	SD	LinStatic			6.706E-14	-1.776E-15	4480	7.162E-12	-8.981E-12	-2.292E-13	0
	Live	LinStatic			4.774E-14	3.331E-15	3840	6.082E-12	-8.242E-12	-2.132E-13	0

Record: << < 1 > >> of 18

Add Tables... Done

Figure 40

Moment distribution on beam from live load

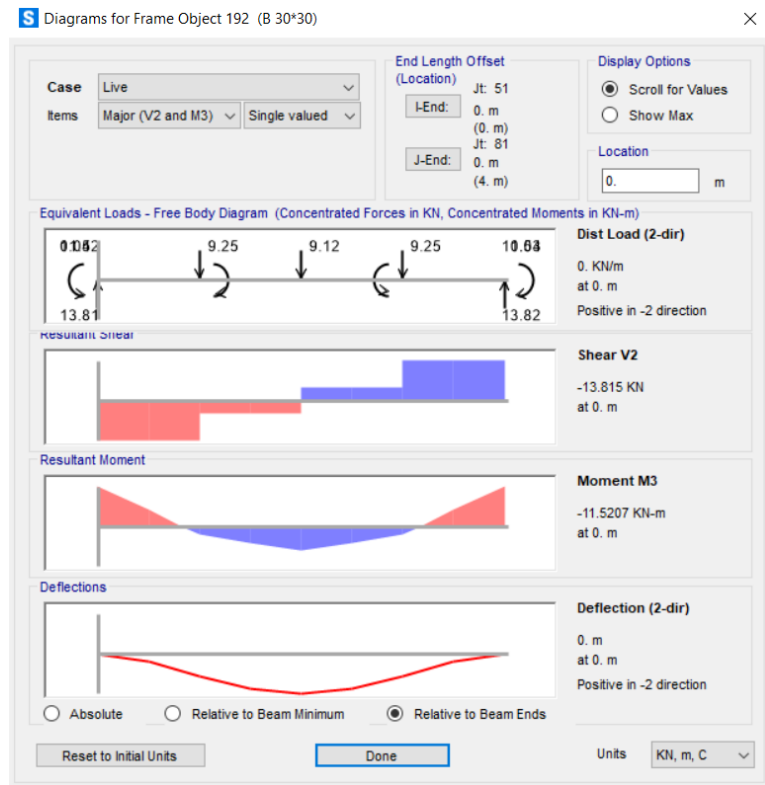


Figure 41

Response spectrum curve definition

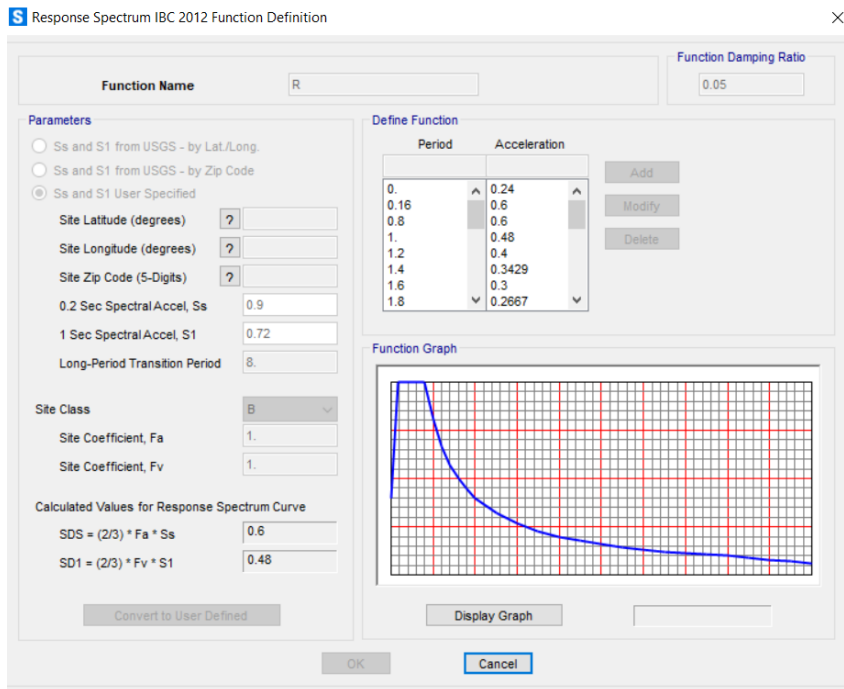


Figure 42

Response spectrum load definition

Load Case Data - Response Spectrum

Load Case Name: Resp

Modal Combination: CQC (selected), SRSS, Absolute, GMC, NRC 10 Percent, Double Sum

Directional Combination: SRSS (selected), CQC3, Absolute

Mass Source: Previous (MSSSRC1)

Diaphragm Eccentricity: Eccentricity Ratio: 0.0

Loads Applied:

Load Type	Load Name	Function	Scale Factor
Accel	U1	R	2.8672
Accel	U1	R	2.8672
Accel	U2	R	0.8614

Other Parameters: Modal Damping: Constant at 0.05

Figure 43

The moment-curvature relationships for C1

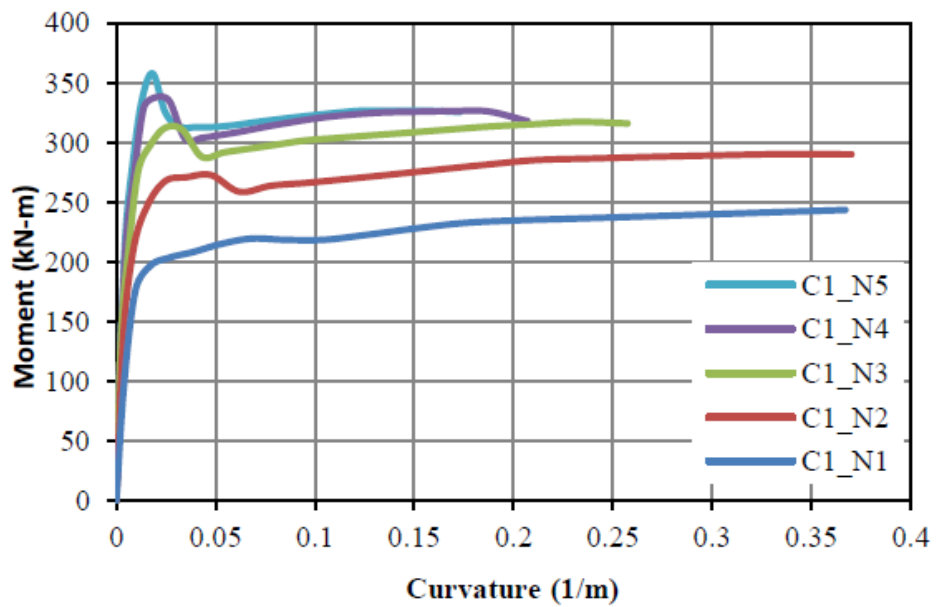


Figure 44

2D frame model

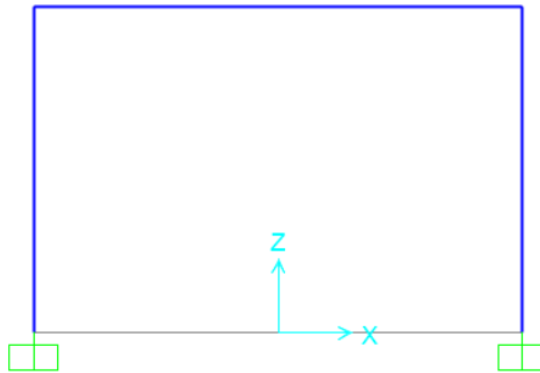


Figure 45

Plastic hinge definition

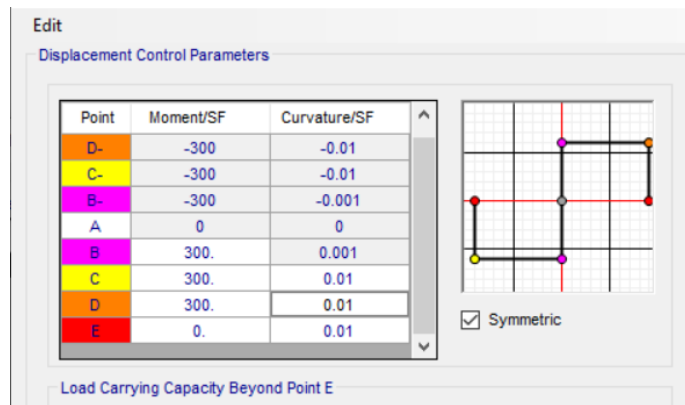


Figure 46

Pushover curve 1

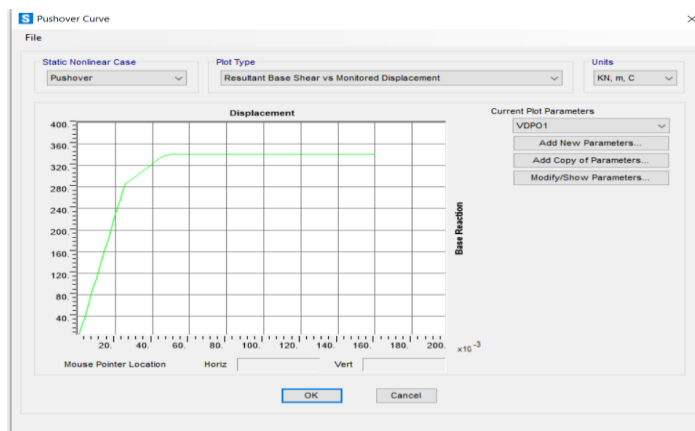


Figure 47

Pushover curve 2

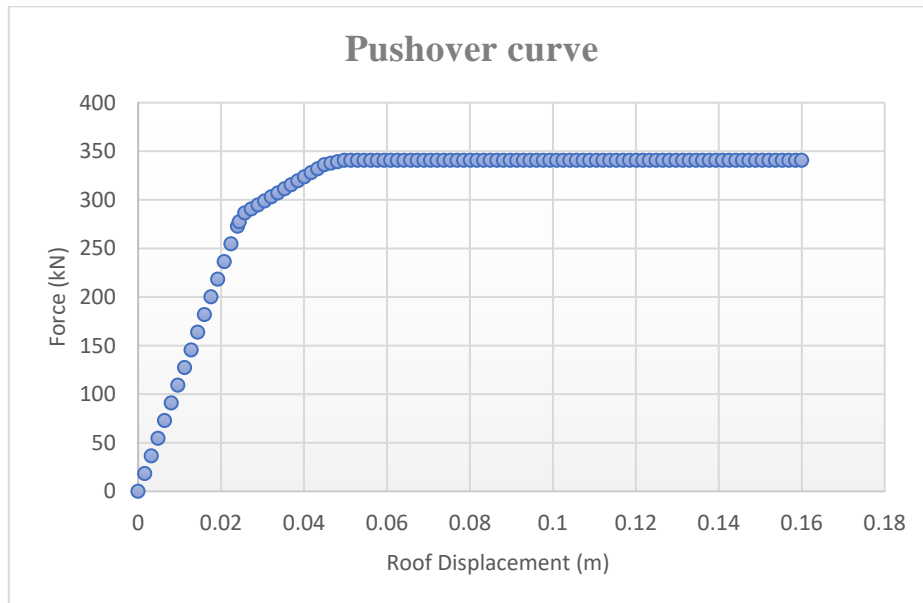


Figure 48

Moment distribution on frame

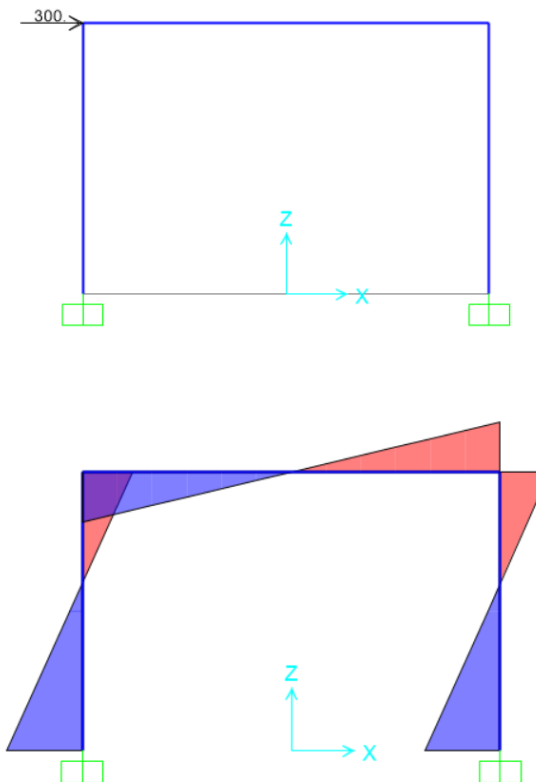
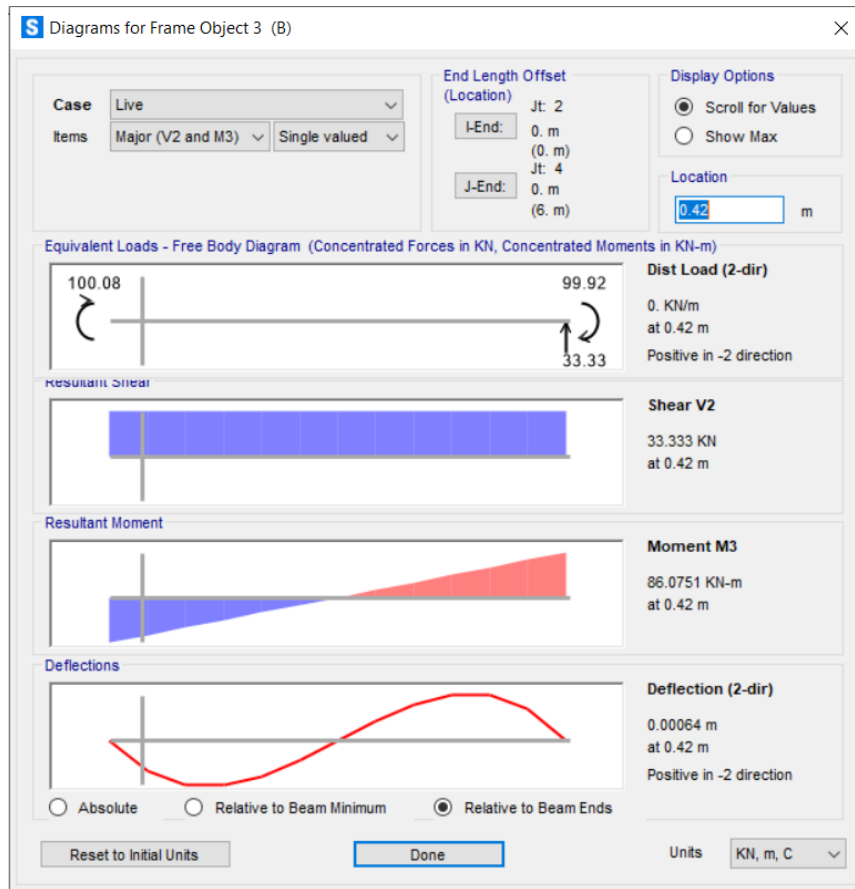


Figure 49

Moment distribution on frame for second analysis



Appendix B

Tables

Table 10

Calculation results of bending moment-curvature

Kesit No	Axial Load Level N/N_{max}	M_y (kN.m)	ϕ_y (1/m)	M_u (kN.m)	ϕ_u (1/m)	μ_ϕ
C1	0	165.03	0.0088	243.32	0.4060	46.1
	0.1	218.78	0.0101	290.0	0.3258	32.3
	0.2	278.27	0.0116	317.0	0.2574	22.2
	0.3	322.15	0.0132	337.54	0.2070	15.7
	0.4	345.35	0.0153	357.52	0.1913	12.5

Table 11

Data from pushover curve.

Point number	Force (kN)	Displacement (m)
1	275kN	0.024
2	333kN	0.046

Table 12

first point in the pushover curve.

	capacity	Demand	D/C	First hinge
M1	330	300	1.1	300
M2	240	300	-	218
D	0.0261	-	-	0.024
F	300	-	-	273

Table 13*List of models IDs and it description.*

Cases	category	description	ΣG
1	(N5, B4, L4)	It's a five floors frame with four bays each of 4m length	80
2	(N5, B3, L4)	It's a five floors frame with three bays each of 4m length	66
3	(N5, B2, L4)	It's a five floors frame with two bays each of 4m length	55
4	(N5, B4, L6)	It's a five floors frame with four bays each of 6m length	114
5	(N5, B3, L6)	It's a five floors frame with three bays each of 6m length	98
6	(N5, B2, L6)	It's a five floors frame with two bays each of 6m length	82
7	(N5, B4, L8)	It's a five floors frame with four bays each of 8m length	140
8	(N5, B3, L8)	It's a five floors frame with three bays each of 8m length	120
9	(N5, B2, L8)	It's a five floors frame with two bays each of 8m length	100
10	(N5, B3, L4) FEMA	It's a five floors frame with three bays each of 4m length and the plastic hinge is auto defined according to FEMA	66
11	(N10, B4, L4)	It's a ten floors frame with four bays each of 4m length	231
12	(N10, B3, L4)	It's a ten floors frame with three bays each of 4m length	198
13	(N10, B2, L4)	It's a ten floors frame with two bays each of 4m length	165
14	(N10, B4, L6)	It's a ten floors frame with four bays each of 6m length	750

15	(N10, B3, L6)	It's a ten floors frame with three bays each of 6m length	642
16	(N10, B2, L6)	It's a ten floors frame with two bays each of 6m length	428
17	(N10, B4, L8)	It's a ten floors frame with four bays each of 8m length	910
18	(N10, B3, L8)	It's a ten floors frame with three bays each of 8m length	780
19	(N10, B2, L8)	It's a ten floors frame with two bays each of 8m length	650
20	(N5, B3, L (V) 6,2,6 m)	It's a five floors frame with three various bays such 6 , 2 , 6 m length	24
21	(N10, B3, L (V) 6,2,6 m)	It's a ten floors frame with three various bays such 6 , 2 , 6 m length	25.67
22	(N5, B3, L (V) 6,6,2 m)	It's a five floors frame with three various bays such 6 , 6 , 2 m length	27.33
23	(N10, B3, L (V) 6,6,2 m)	It's a ten floors frame with three various bays such 6 , 6 , 2 m length	28.67
24	(N5, B4, L (V) 6,2,2,6 m)	It's a five floors frame with four various bays such 6 , 2 , 2 , 6 m length	24.67
25	(N10, B4, L (V) 6,2,2,6 m)	It's a ten floors frame with four various bays such 6 , 2 , 2 , 6 m length	26.33
26	(N5, B4, L (V) 6,2,6,2 m)	It's a five floors frame with four various bays such 6 , 2 , 6 , 2 m length	12
27	(N10, B4, L (V) 6,2,6,2 m)	It's a ten floors frame with four various bays such 6 , 2 , 6 , 2 m length	13.33
28	(N5, B3, L (V) 8,4,8 m)	It's a five floors frame with three	93

29	(N5, B3, L (V) 6,2(S),6 m)	various bays such 8 , 4 , 8 m length It's a five floors frame with three various bays such 6 , 2 , 6 m length and the inner span with section different than the 6 meter spans	23
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Appendix C

Special Frame Design

Seismic Design

Step 1: Select the most appropriate risk category

Figure 50 . Risk category of building for Earthquake loads

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent a low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life. Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure.	III
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	
Buildings and other structures designated as essential facilities.	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is sufficient to pose a threat to the public if released. ^a	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	

^aBuildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the substances is commensurate with the risk associated with that Risk Category.

Step 2: Set the importance factor I

Importance factor is a function from the risk category. It points to the level of danger to human life, health, and welfare concerning damage to property, loss of use or functionality: 1

Figure 52. Short-Period site Coefficient F_a and Long-Period site Coefficient F_v

Table 11.4-1 Short-Period Site Coefficient, F_a

Mapped Risk-Targeted Maximum Considered Earthquake (MCE _R) Spectral Response Acceleration Parameter at Short Period						
Site Class	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_S .

Table 11.4-2 Long-Period Site Coefficient, F_v

Mapped Risk-Targeted Maximum Considered Earthquake (MCE _R) Spectral Response Acceleration Parameter at 1-s Period						
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2 ^a	2.0 ^a	1.9 ^a	1.8 ^a	1.7 ^a
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1 .

^aAlso, see requirements for site-specific ground motions in Section 11.4.8.

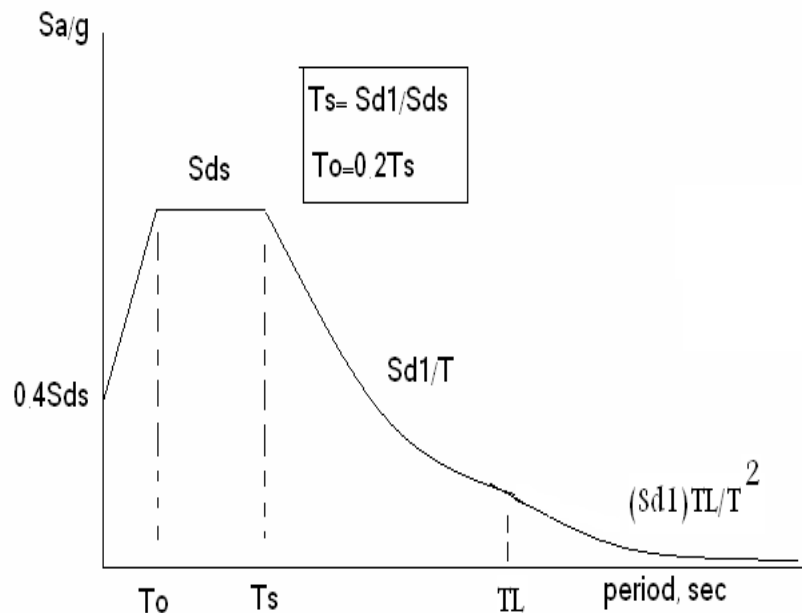
Step 6: Calculate the maximum considered earthquake (MCER) Spectral Response Acceleration Parameters S_{MS} and S_{M1} using both site class and site coefficients

- S_{MS} = the MCE_R , 5 percent damped, spectral response acceleration parameter at short periods adjusted for site class effects.
- S_{M1} = the MCE_R , 5 percent damped, spectral response acceleration parameter at a period of 1 s adjusted for site class effects.
- $S_{MS} = F_a * S_s = 0.6$
- $S_{M1} = F_v * S_1 = 0.48$

Step 7: Calculate the design spectral acceleration S_{DS} & S_{D1}

- Despite that fact that seismic hazard in ASCE 7 is quantified in terms of 2% of being exceeded in broadening time of 50 years (2500 years return period), the basic ground motion parameters corresponding to 10% probability occurs of being exceeded in 50 years (475 years return period).
- $S_{DS} = S_{MS} = 0.6$
- $S_{D1} = S_{M1} = 0.48$

Figure 53. Response spectrum curve



- Period Limiting Values
 - $T_0 = 0.2 S_{D1}/S_{DS}=0.16\text{sec}$
 - $T_S = S_{D1}/S_{DS}=0.8\text{sec}$
 - $T_L=4\text{sec}$
- S_a , design spectral response acceleration
 - S_a is a function of structure period, T_n

Buildings are classified into three seismic design categories according to their seismic hazard:

- Category A (regions of negligible seismicity: $S_{DS} < 0.167$ and $S_{D1} < 0.067$): no spectral values are required. Use a minimum lateral force 1% of the dead load. ASCE 7-16 11.6.
- Categories B and C (low to moderate seismicity): use S_s and S_1 map values.
- Categories D, E and F (high seismicity)

Step 8: Pick out the most appropriate seismic design category: SDC D is extracted

Determination of analysis method:

- ELF for regular and some irregular
- Modal for some irregular
- Site specific required in Site Classes E or F

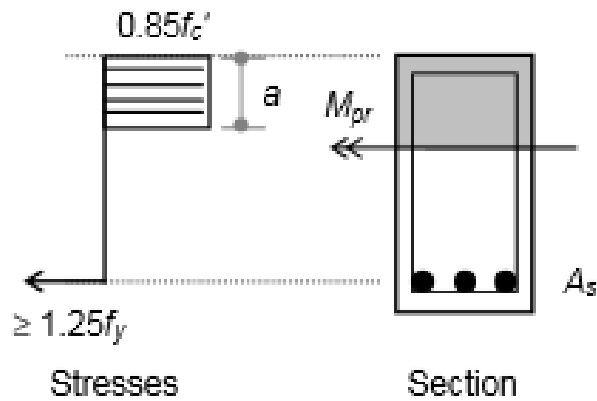
Modal Response Spectrum analysis will be used to design the frames

- Response modification factor (R) is 8 to determine the base shear
- system over strength, Omega factor (Ω_0) is 3 to determine element design forces.
- deflection amplification factor (C_d) is 5.5 to determine design story drift.

Special Frame design:

Probable moment, M_{pr} : is the nominal moment using stress in steel equal to at least $1.25f_y$ and the strength reduction factor = 1.0. The stress distribution is shown. M_{pr} is given as a minimum and not an exact quantity. This moment accounts for the overstrength effect. This action is needed to keep brittle elements elastic. See next figure.

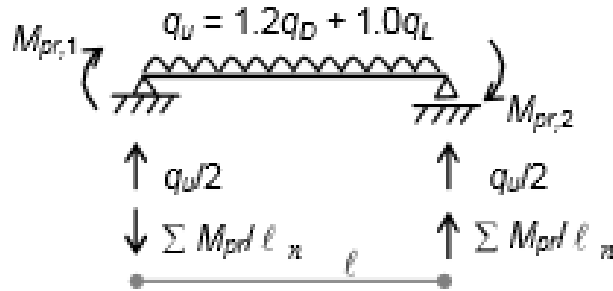
Figure 54. Probable moment calculation



Special Beams: Special beams have special requirements in terms of design shear, dimensions, main reinforcement and lateral reinforcement as follows.

Design shear, V_e : the design shear force including seismic effect, V_e , shall be evaluated according to the maximum probable shear that can develop in the section due to probable moments. This shear force includes the effect of gravity loads and the over-strength effect insofar as the moment is the function of the ultimate strength of reinforcement.

Figure 55. Probable Shear Force



Consequently, the required design shear strength may be expressed as:

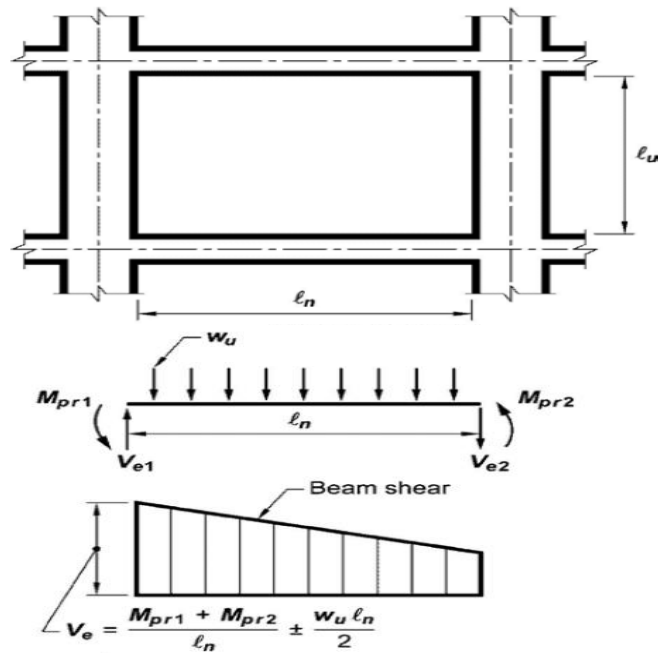
$$V_e \geq \frac{\sum M_{pr}}{L_n} + V_u \text{ (gravity)}$$

The design shear above must be resisted by the nominal shear strength of the member, V_n , which consists of concrete contribution (V_c) and steel contribution (V_s). However, the contribution of the concrete shall be ignored under the following two conditions when they occur simultaneously:

$$V_c = 0 \text{ If } P_u \leq 0.05 \cdot f'_c \cdot A_g \quad \text{and } V_E \geq \frac{1}{2} V_e$$

where V_E is the shear force due to earthquake effect only.

Figure 56. Shear force due to earthquake effect



Dimension limitations

The additional requirements for dimensions are:

1. The web width shall not be less than 0.3 times its height but not less than 250 mm.
2. The transverse projection of the beam over the supporting columns shall not exceed three fourths the dimensions of the supporting column.
3. The clear span of the beam shall not be less than four times its effective depth. This requirement is intended to guarantee the flexure behavior of the beam without significant shear deformations (plane section hypothesis).

Main reinforcement

The additional requirements for main reinforcement design are:

1. The positive moment at the face of the column must exceed one half the negative moment strength provided at the face of the supporting column
2. The minimum positive and negative moments at midspan must exceed one fourth the maximum moment strength provided at the face of the supporting column.
3. At least two bars at the top and bottom faces of the beam must be continuous.

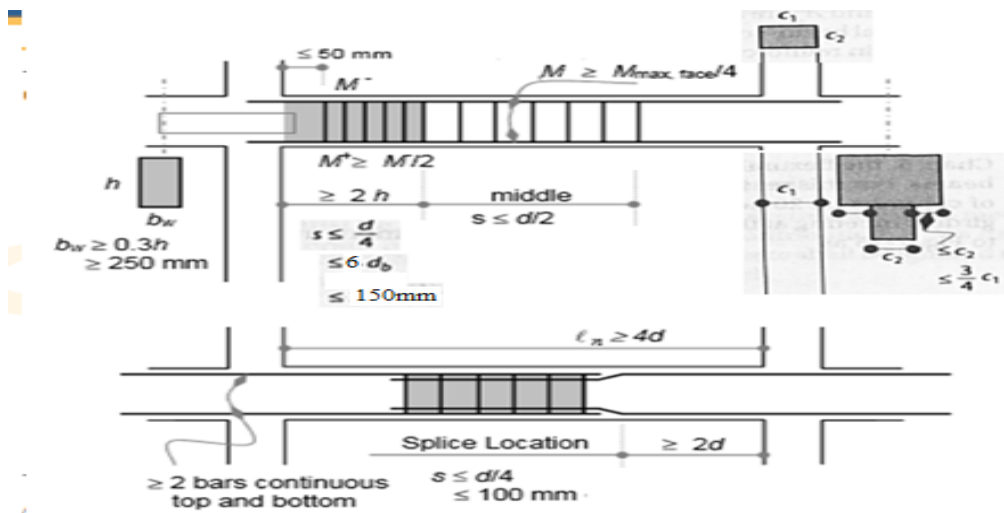
4. The splice location shall not be less than $2d$ from the face of the support or from the critical section of any plastic hinge.
5. Maximum reinforcement ratio must not exceed 0.025

Lateral reinforcement

The additional requirements for lateral reinforcement are:

1. The potential plastic hinge region is assumed to extend a distance of $2h$ from the face of the support. Therefore, over this distance ($2h$), the maximum spacing of stirrups is given in terms of the effective depth of the member (d), the diameter of main reinforcement (d_b) and 150 mm. The first stirrup shall start at least 50 mm from the face of the support.
2. The spacing of stirrups in the middle of the beam shall not exceed $d/2$.
3. The spacing of stirrups over the splice length shall not exceed $d/4$ or 100 mm.

Figure 57. Distribution of reinforced steel in the special beam



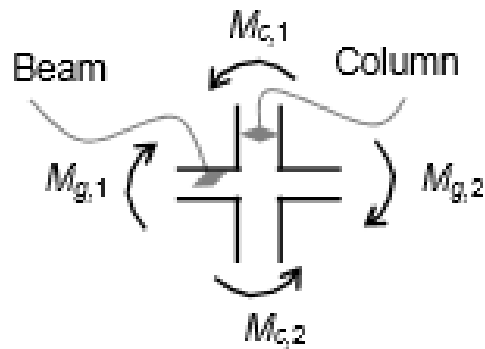
Special Beam-Columns: have special requirements in terms of design forces, dimensions, main reinforcement and lateral reinforcement as follows.

Design Forces:

Design Moment: to force the plastic hinges in the beams to achieve a weak beam-strong column mechanism, the flexural strength of the columns must exceed the flexural strength of the beams. For this reason, ACI 318 requires that the summation of the design moments of columns be 20 percent larger than the summation of the design moments of the girders meeting at the same joint.

$$\Sigma M_{nc} \geq \frac{6}{5} \Sigma M_{nb}$$

Figure 58. weak beam-strong column mechanism



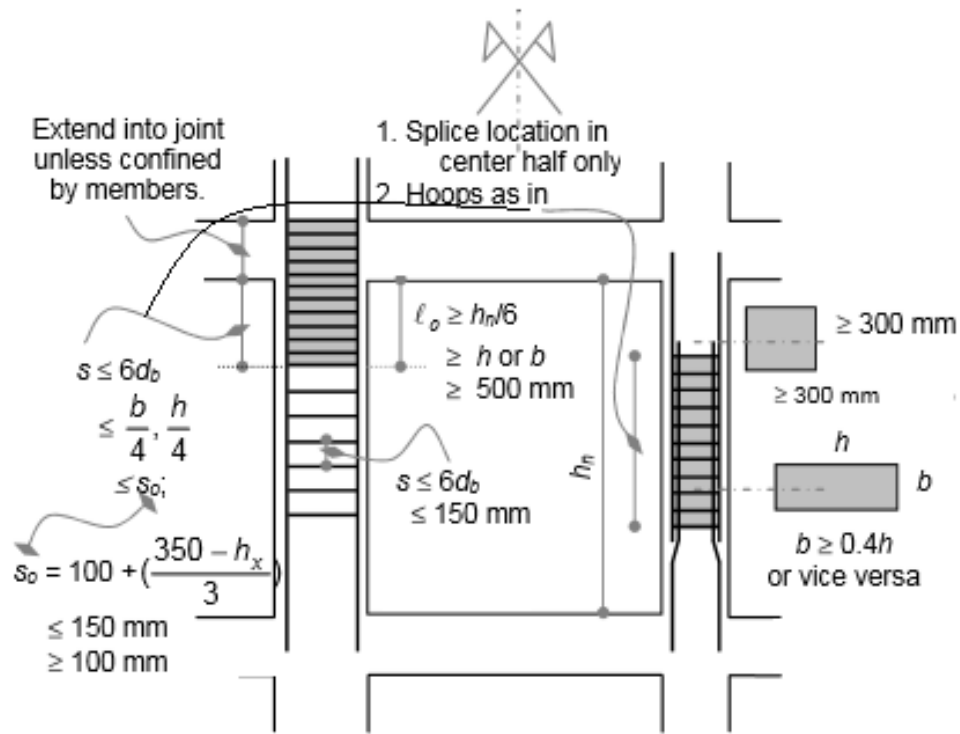
Dimension limitations

The width of one side of the section not be less than 0.4 times its other side, but not less than 300 mm as shown next.

Main reinforcement:

1. The gross reinforcement ratio, ρ_g , is limited to a maximum 0.06 (preferably 0.04) and a minimum 0.01.
2. The splice location shall be limited to the center half of the beam column to keep the splice outside of the regions of the plastic hinges. The spacing of the splice hoop shall be the same as in the plastic hinge region

Figure 59. Distribution of reinforced steel in the special columns



All these notes were taken in consideration in the design of the frame cases. The next table gives all cases and the section dimensions for columns and beams.

Table 14. columns and beam dimensions for all cases.

Cases	category	Column (cm)	Beam (cm)
1	(N5, B4, L4)	50*50	30*30
2	(N5, B3, L4)	50*50	30*30
3	(N5, B2, L4)	50*50	30*30
4	(N5, B4, L6)	50*50	30*40
5	(N5, B3, L6)	50*50	30*40
6	(N5, B2, L6)	50*50	30*40
7	(N5, B4, L8)	70*70	30*50
8	(N5, B3, L8)	70*70	30*50
9	(N5, B2, L8)	70*70	30*50
10	(N5, B3, L4) FEMA	50*50	30*30
11	(N10, B4, L4)	65*65	30*30

12	(N10, B3, L4)	65*65	30*30
13	(N10, B2, L4)	65*65	30*30
14	(N10, B4, L6)	70*70	30*40
15	(N10, B3, L6)	70*70	30*40
16	(N10, B2, L6)	70*70	30*40
17	(N10, B4, L8)	80*80	30*50
18	(N10, B3, L8)	80*80	30*50
19	(N10, B2, L8)	80*80	30*50
20	(N5, B3, L (V) 6,2,6 m)	50*50 65*65	30*40
21	(N10, B3, L (V) 6,2,6 m)	65*65	30*40
22	(N5, B3, L (V) 6,6,2 m)	50*50 65*65	30*40
23	(N10, B3, L (V) 6,6,2 m)	65*65	30*40
24	(N5, B4, L (V) 6,2,2,6 m)	50*50	30*40
25	(N10, B4, L (V) 6,2,2,6 m)	65*65	30*40
26	(N5, B4, L (V) 6,2,6,2 m)	50*50	30*40
27	(N10, B4, L (V) 6,2,6,2 m)	65*65	30*40
28	(N5, B3, L (V) 8,4,8 m)	70*70	50*30
29	(N5, B3, L (V) 6,2(S),6 m)	50*50 65*65	30*40 20*20



جامعة النجاح الوطنية
كلية الدراسات العليا

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باعتبار التباين في طول الجسور

إعداد

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قدمت هذه الرسالة استكمالاً لمتطلبات الحصول على درجة الماجستير في هندسة الانشاءات، من كلية الدراسات العليا، في جامعة النجاح الوطنية، نابلس - فلسطين.

2022

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الملخص

خلفية الدراسة: يعتبر معامل خفض القوى الزلزالي احد اهم المتغيرات في التصميم الزلزالي و هو يمثل مقدار الطاقة الضائعة في المبنى خلال الزلزال و يمثل ايضا مقدر الخفض بين القوى المرنة و غير المرنة المؤثرة على المبنى الخرساني ذو الاطار الخاص اخذا بعين الاعتبار ان مراجع التصميم لا تتفق على قيمة واحدة لهذا المتغير مع العلم انه يعتمد على كثير من العوامل و بعض المراجع تحدث قيمته في كل تحديث.

أهداف الدراسة: هذه الرسالة تعطي طرية لحساب معامل الخفض الزلزالي تحت تأثير اي متغير و تدرس بتفصيل تأثير معامل المرونة بين الجسر و العمود و تغير الطول في جسور المنشئ. من خلال مجموعة من التحاليل غير الخطية على منشئات خرسانية فو باستخدام برنامج ساب 2000 و مقارنة النتائج مع الكود الامريكي.

منهجية الدراسة: لتحقيق الهدف السابق مجموعة من عمليات التحليل غير الخطي لمجموعة من المنشئات باستخدام برنامج ساب 2000 تم تنفيذها و الناتج من هذه العمليات غبارة عن منحنى التحمل الخاص بكل مبنى, هذا المنحنى يعتمد بشكل رئيسي على توزيع نقاط الدونة التي تم حسابها باستخدام برنامج الكستراكت لتشكيل نقاط الدونة في المبنى. مجموعتين من المباني تم استخدامها في هذه الدراسة الاولى تمثل الحالة

التي ليس فيها تغير في طول الجسور مع تغير عدد الطوابق و عدد الجسور في المبنى الواحد و الثانية تمثل تغير في طول الجسور و نسب مختلفة لهذا التغير.

نتائج الدراسة: نتائج التي حصلنا عليها تبين ان زيادة عدد الطوابق يزيد قيمة معامل المرونة و زيادة عدد الجسور ايضا يزيد قيمة معامل المرونة لكن لا يوجد تغير يذكر على معامل المرونة عند زيادة طول الجسر، في حالة الجسور غير المنتظمة في الطول لوحظ وجود تاثير على معامل المرونة وكل ما زادة نسبة التغير في طول الجسور زادت قيمة معامل المرونة واقتربت من القيمة المعتمدة في المرجع الامريكي و اقل قيمة تم الحصول عليها في حالة الجسور غير منتظمة الطول لحالة تمثل جسور بطول 6 و 2 و 6 متر و بنسبة 0.33 لفارق بين اطوال الجسور.

الاستنتاجات: هذه الرسالة تقدم طريقة لحساب معامل المرونة للمباني تحت تغير اي عامل قد يؤثر على قيمة معامل المرونة و كدراسة مستقبلية لتطوير البحث يرجى اعادة العمل باستخدام طريقة تحليل ديناميكية وغير خطيا تحت تاثير احمال زلزالية حقيقية لتأكيد نتائج البحث.

كلمات مفتاحية: معامل المرونة, طريقة الدفع التحليل المباني, طريقة التحليل غير الخطي, تعريف نقاط الدونة للمبنى, المرونة.