ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

Assessment of Behavior Factor recommended in EC8 for Reinforced Concrete Planar Frame Using Nonlinear Analysis

A Thesis submitted to Addis Ababa University in Partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structures stream)

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Declaration

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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Above all, I would like to thank the Almighty GOD, who gives me the courage and strength to do this work from the discoveries of invisibles clouded at perceptions. Next, I would like to convey my deepest and sincere appreciation to my advisor Dr. Adil Zekaria for proposing such an interesting research topic and for always willing and being available to help, support and guide. I am also thankful for his invaluable and constructive comments.

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ABSTRACT

According to current seismic codes the structures are calculated using the capacity design procedure based on the concept of at the shear base depending on several parameters including behavior factor which is consider to be the most important parameter. The behavior factor allows designing the structure when it is at ultimate limit state taking in to account its energy dissipation through its plastic deformation. The aim of the present study is to assess the basic parameters of the behavior factor among of these the reduction factor due to ductility, redundancy and over strength for reinforced concrete frames of different height and bays. Analysis are conducted on these frames using the nonlinear statics pushover analysis method where the effect of some parameters on behavior factor, such as the number of stories, the number of bays and the member section are taken in to account. The results show that the behavior factor is sensitive to the variation of the number of stories and bays. Also, the value of behavior factor recommended by the Euro Code 8 (EC8) is underestimated, mostly for low-rise (storey four) frames.

Keywords: Behavior factor, ductility factor, over strength factor, redundancy factor, base shear, pushover analysis.

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CHAPTER 1 : INTRODUCTION

1.1 BACKGROUND:

Today's seismic design philosophy for buildings, as outlined in different codes and guidelines, such as Euro-code 8 [1], and ASCE7 [2], assumes nonlinear response in selected components and elements when subjected to an earthquake of the design intensity level. However, these codes and guidelines do not explicitly incorporate the inelastic response of a structure in the design methodology. The equivalent static lateral force method, which has been used from the early days of engineering seismic design, is still the most preferable method to a structural design engineer, because it is conceptually simple and less demanding from a computational point of view. Most of the codes used for seismic deign of buildings use the concept of behavior factor to implicitly account for the nonlinear response of a structure. In this approach, the behavior factor q is derived from the equation;

$$q = R_{\mu} R_{\Omega} R_{\rho} \qquad \textbf{(1-1)}$$

Where, q is termed as the ''behavior factor'' in Euro-code 8 (EC8).

Currently, many seismic design codes, such as [1] and [2], permit a reduction in design base shear force, taking advantage of the fact that the structures possess significant reserve strength, redundancy, damping and capacity to dissipate energy, which are incorporated in structural design through a reduction factor called behavior factor (q-factor) in EC8 and response modification factor (R-factor) in the American codes. EC8 specifies a single value of behavior factor for all building structures of a given framing type, irrespective of building dimensions (stories height and width bay). This approach, however, under certain circumstances may lead to values of the behavior factor not always appropriate as compared with the actual dissipative features of the building structure [2-5].

Nonlinear analysis is an important tool for the seismic design of structures. Nonlinear static analysis is an efficient and a simple method to assess the nonlinear behavior of structures under seismic loads, is the inelastic static pushover analysis which is used here in this thesis work.

1.2 OBJECTIVE

1.2.1. General Objective:

The main objective of this study is to verify the behaviour factor (q) for planar RC Frame structure and to compare the assumed behaviour factor during design to actual behaviour factor obtained from non -linear pushover analysis.

1.2.2. Specific Objectives:

- To evaluate behaviour factor according to Euro code 8 and American code;
- To compare the calculated behavior factor with assumed on and also with different parameters of the behavior factor;
- To evaluate ductility reduction factor of study frames;
- To evaluate over strength factor of study frames;
- To verify the redundancy factor of the RC Frame structure

1.3 STATEMENT OF THE PROBLEM

In our seismic code (Euro code 8) the behavior factor is depending on only ductility class, structural type and regularity and it has a constant value for all multi story and multi bay, designed for ductility class medium and high. In this case, our code does not consider the effect of number of stories, number of bays and the performance limit.

1.4 SCOPE OF THE STUDY

The present study is limited to RC plane frames without shear wall. The soil structure interaction effects and bond slip are not taken into account. The flexibility of floor diaphragms is ignored and considered as stiff diaphragm. The column bases are assumed to be fixed. This paper will address only low-rise and mid-rise buildings.

CHAPTER 2 : LITREURE REVIEW

2.1 GENERAL

An extensive literature review was carried out prior to the project. The survey of literature includes classification of RC framed buildings, behaviour factor, redundancy factor. ductility factor and pushover analysis.

2.1.1 Previous Studies on calculation of Behaviour Factors of Existing Buildings:

Kadid and A. Boumrkik, proposes a methodology to develop force reduction factors that are appropriate for the evaluation nuclear facilities. These force reduction factors are functions of acceptable limit state; the structural system, material, and detailing for each individual element, structure's natural frequency; and the influence of higher modes and soft stories. The acceptable limit state, structural system, material and detailing is used to develop allowable element ductility. Individual element ductility's are modified to account for either multi-degree of freedom or soft story effects. This modified element ductility's are combined with the structures natural frequency and an appropriate single degree of freedom dynamic model to develop the force reduction factor [6].

Mohammed Irfan., conducted an evaluation of seismic response of symmetric and asymmetric multi-storied buildings. In this study, 3D analytical model of four and nine storied buildings have been generated for symmetric and asymmetric building models and analyzed using structural analysis tool "ETABS Nonlinear". For the nonlinear static procedure (NSP) which is described in FEMA-273/356 and ATC-40 documents are used. It is concluded that Base shear and displacement at first hinge are less in asymmetric building compared to symmetric buildings. Ductility ratio is maximum for bare frame structures and it gets reduced when the effect of infill wall is considered. It indicates that bare frame structures will shows adequate warning before collapse. Bare frame structures are having highest response reduction factor as compared to the infill frame structures. It indicates that bare frame structures are capable of resisting the forces still after first hinge formation. [7]

Mehmet et al., explained that due the easiness of Pushover analysis, the structural engineers have been using the nonlinear static method or pushover analysis. Pushover analysis is performed for various nonlinear hinge characters available in certain programs based on the FEMA-356 and ATC-40 guidelines and he pointed out that Plastic hinge length has significant effects on the displacement capacity of the structures. The alignment and the axial load degree of the columns cannot be considered properly by the default-hinge properties [8].

Tinkoo Kim and Hyunhoo determine the strength reduction factors for structures with added damping and stiffness device. For the structural period between 0.50 seconds to 5 seconds, the strength reduction factors for TADAS device with ductility equal to 6 varies from 8.30 to 10.70 [9].

Mohd Zulham Affandi Mohd Zahid et al, have assessed an evaluation of overstrength factor of seismic designed low-rise RC buildings. Six frame models regular and irregular in elevation, each are designed to gravity load only and designed to resist seismic load with medium ductility and high ductility class. The nonlinear static analysis or also known as pushover analysis (POA) is used to determine the performance of the buildings. Based on their work, the seismic designed building has greater load carrying-capacity, top displacement capacity and ductility supply compared to the gravity load designed buildings and the over strength factor increases as the ductility supply of the building increases. [8]

Greg Mertz and Tom Houston evaluated the performance of RC framed buildings under future expected earthquakes, a non-linear static pushover analysis has been conducted. To achieve this objective, three frame buildings with 5, 8 and 12 stories were analyzed. The results obtained from this study show that properly designed frames will perform well under seismic loads [7].

Devrim Ozhendekci, Nuri Ozhendekci and A. Zafer Ozturk evaluate the seismic response modification factor for eccentrically braced frames. Conclusion was made that one constant R-value cannot reflect the expected inelastic behavior of all building which have the same lateral load resisting system. In the analysis they used over strength factor, ductility factor and redundancy factor for the evaluation of R-values to the EBF systems [10].

$$R = R\Omega * R\mu * RR$$

2.1.2 Previous Studies on calculation of over strength factor of Existing Buildings:

Freeman had reported over strength factors for 3 three stories moment resisting frames, two constructed in seismic zone 4 and one in seismic zone 3 were 1.9, 3.6, and 3.3 respectively [11].

The study by Kappos on five R/C buildings, with one to five stories, consisting of beam, columns, and structural walls examined and as a result over strength factors 1.5 to 2.7 are obtained [12].

According to the investigation by Lee, Cho and Ko, over strength factors and plastic rotation demands for 5, 10, 15 stories RC buildings designed in low and high seismic regions utilizing three-dimensional pushover analysis. One of their conclusions is that the over strength factors in low seismicity regions are larger than those of high seismicity regions for structures designed with the same response modification factor. They have reported factors ranging from 2.3 to 2.8 [13].

Elnashai and Mwafy developed the relationship between the lateral capacity, design force reduction factor, the ductility level and the over strength factor [14]. The lateral capacity and over strength factor are estimated by means of inelastic static pushover as well as time-history analysis of 12 buildings of various characteristics representing a wide range of contemporary RC buildings. Conclusion was made that the recommendations of FEMA 273 [15] and Paulay and Priestley [16] underestimate the inelastic period.

2.1.3 Previous studies on Calculation of Ductility Factors of Existing Buildings:

The ductility factor **(**Rμ) of Newmark and Hall was determined to be a function of displacement factor (μ). It was observed that in the long period range, elastic and ductile systems with the same initial stiffness reached almost the same displacement. As a result, the response factor can be considered equal to the displacement ductility. This is referred to as 'equal displacement' region. For intermediate period structures, the ductility is higher than the response factor and the 'equal energy' approach may be adopted to calculate force reduction [17]. The relationship derived for Rμ as a function of μ, for short, intermediate and long period structures is presented below.

Short period T < 0.2 seconds $R_\mu = 1$

Intermediate period 0.2 < T < 0.5 seconds $R_\mu = \sqrt{2\mu - 1}$

Long period T > 0.5 seconds $R_\mu = \mu$

Krawinkler and Nassar developed a relationship for the force reduction factor derived from the statistical analysis of 15 western USA ground motions with magnitude between 5.7 and

7.7 [18]. The influence of response parameters, such as yield level and hardening coefficient α, were taken into account. A 5% damping value was assumed. The equation derived is given as:

$$R_U = \left[c(\mu - 1) + 1\right]^{1/C}$$

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

Where, c is a constant which is dependent on period (T) and α which is the strain hardening parameter of the hysteretic model and a and b are regression constants. Values of the constants in above equations were recommended for three values of hardening α as in Table 2-1 below.

Table 2- 1: Model parameter constants for Krawinkler and Nassar

HARDING VALUE	MODEL PARAMETERS	
α	a	b
0	1.00	0.42
2	1.01	0.37
10	0.8	0.29

The equation for reduction factor introduced by Miranda and Bertero was obtained from a study of 124 ground motions recorded on a wide range of soil conditions [19]. The soil conditions were classified as rock, alluvium and very soft sites characterized by low shear wave velocity. A 5% of critical damping was assumed. The expressions for the period-dependent force reduction factors Rμ are given by:

$$R_\mu = \frac{\mu - 1}{\Phi} \pm 1 \text{ ……… Equation 2- 1}$$

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

$$\Phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T}\exp\left[-1.5(\ln T - 0.6)^2\right] \text{…for rock site…………Equation 2- 2}$$

$$\Phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T}\exp\left[-2(\ln T - 0.2)^2\right] \text{…for alluvium site……..Equation 2- 3}$$

$$\Phi = 1 + \frac{T_1}{3T} - \frac{3T_1}{4T}\exp\left\{-3\left[\ln\left(T/T_1\right) - 0.25)\right]^2\right\} \ldots\text{for soft site} \ldots \text{Equation 2-4}$$

Where, T1 is the predominant period of the ground motion. The latter corresponds to the period at which the relative velocity of a linear system with 5% damping is maximum within the entire period range [16]. Paulay and Priestley in their work divided the time period of the structure for calculating ductility reduction factor:

R_μ = 1.0 for zero-period structures

$R_\mu = 2\mu - 1$ for short-period structure

$R_\mu = \mu$ for long period structure

$R_\mu = 1 + (\mu - 1)\, T/0.70$ ($0.70\ s < T < 0.3$)

Mahmoudi developed the relationship between over strength and member ductility of RC moment resisting frames having one, two, three, four, five, six, eight, ten and fifteen stories with three spans [20]. The results indicate that the over strength depends on member ductility considerably and its amount is not equal for structures having low, medium and high ductility.

2.1.4 Nonlinear static pushover analysis:

It is the incremental analysis used by SAP 2000. It divides the load applied and the target displacement to the predefined number of steps of steps. Each steps of load will be applied to the structure. The steps are increased or decreased so that the target incremental displacement is achieved. The target incremental displacement and corresponding sum of lateral forces is recorded. The stress and deformation output from previous step will be imposed to next step of loading. The process is repeated till the instability of structure or target displacement. The performances of reinforced-concrete buildings evaluated by nonlinear static pushover analysis and nonlinear time history analysis were compared. The result shows the nonlinear pushover analysis is accurate enough for practical application [21].

A.Kadid and A. Boumrkik had used a nonlinear pushover analysis to evaluate the performance of framed buildings under expected earthquakes in Algeria. The results obtained from this study show that properly designed frames will perform well under seismic loads [6]. Moreover, Gergely, White, and Mosalam used static nonlinear pushover analysis for evaluation and modeling of in filled frames buildings. Conclusions are made that elastic seismic analysis methods are inadequate for the estimation of the internal force and displacement distributions.

CHAPTER 3: METHODS OF ANALYSIS

3.1 Introduction:

To meet the objectives of the study, a methodology has been developed stepwise with the following brief description of each step:

a) The six regular in elevation and in plan planar frame is modelled and check stability index. After that design of all models using ETABS 2016.2.0 was carried out and ensured ductility class medium for selected frame models.
b) By taking section property, reinforcement and loading from design, calculate the ultimate moment, yield moment and ultimate curvature using SAP2000 program.
c) Determine scale factor for moment and rotation by using the parameters on step b.
d) Define and assign the hinge properties using user-define M3 for the beam and P-M3 for column of each elements.
e) Define the gravity and lateral load cases for inelastic properties of the frame.
f) Run the non-linear models to get the pushover curve and idealized the capacity curves to get yielding shear forces.
g) Then the required behavior factor's parameters are obtained from pushover curve.
h) Finally calculate the behavior factors for each the given parameters.

3.2 Frame Modeling and Design:

In this study a total of six reinforced concrete plane frames are used with selected of 4, 7, and 10 stories. In all cases a variable bay of 3 and 6 are considered as shown in Figure 3.1-3.6. The stories height and spacing between columns of all frames are 3m, and 6m respectively. The dead and live loads are calculated as per EN Euro-Code 1 (2004) and seismic loads are calculated as per Euro code 8, since new code ESEN: 2015 which is the same of Euro code 8 designs manual. The regular building considered in this study is assumed to be an office building, located in seismic zone III. Material properties of C20/25 grade of concrete and S-400 grade of reinforcing steel have been used for all the models. The seismic design load is assumed on the base of a soil type C, with a maximum ground acceleration Υ= 0.1g. The models are created using finite element ETABs 2016.2.0. Each model is designed as ductility class medium (DCM) with a behavior factor of 3.9 in compliance with EN1998-1-1. The section dimension and reinforcement detailing provisions are tabulated in the Table 3.1.

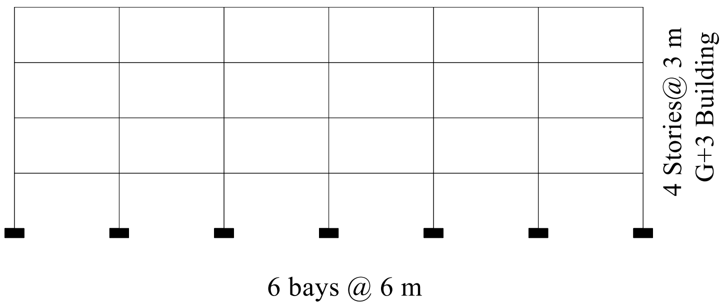


Figure 3- 1: Sample modeling for study frames (6 Bay 4 Stories)

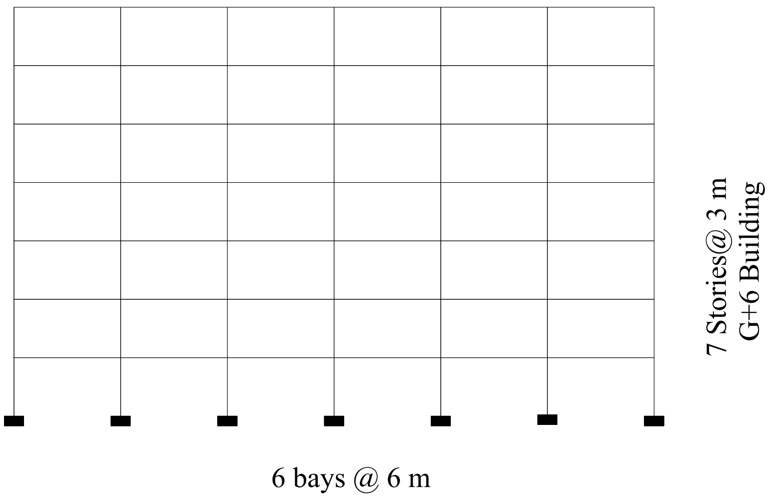


Figure 3- 2: Sample modeling for study frames (6 Bay 7 Stories)

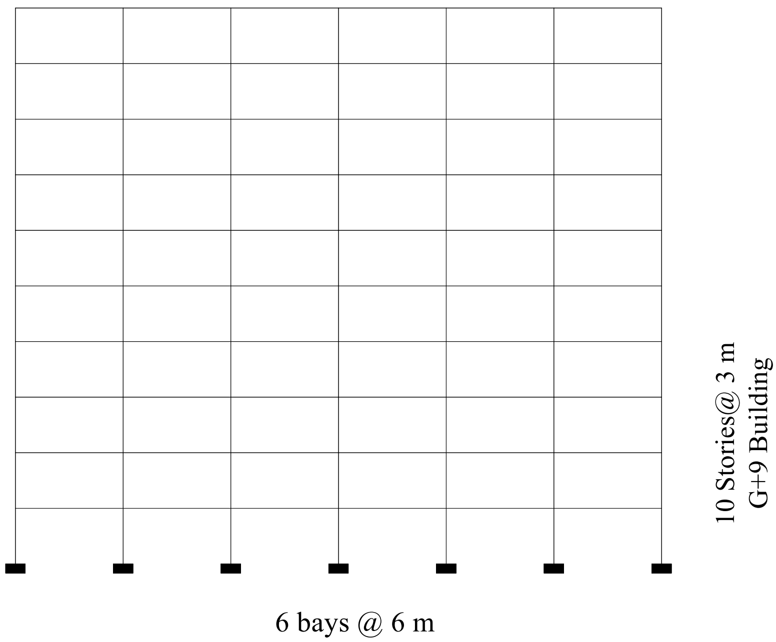


Figure 3- 3: Sample modeling for study frames (6 Bay 10 Stories)

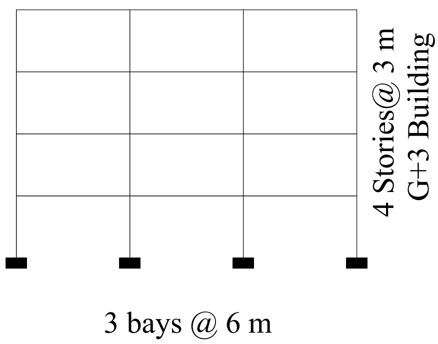


Figure 3- 4: Sample modeling for study frames (3 Bay 4 Stories)

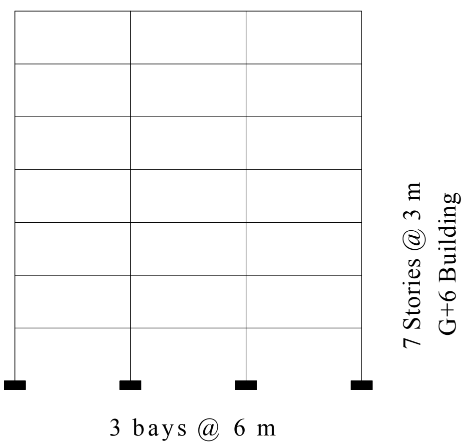


Figure 3- 5: Sample modeling for study frames (3 Bay 7 Stories)

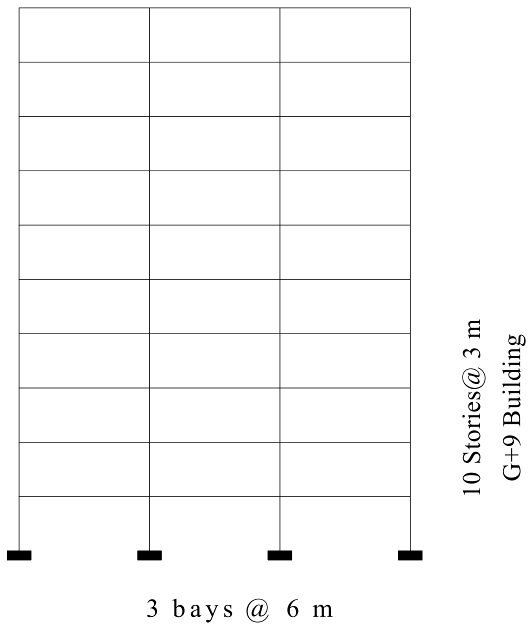


Figure 3- 6: Sample modeling for study frames (3 Bay 10 Stories)

3.2.1 Parameters checked:

To verify the behavior factor four parameters are selected and each parameter is clearly defined. The selected parameters are number of storeys and bays, column dimension and the column size gradually is reduced in story levels with varying longitudinal bar size, and group into four cases, as indicated in the following Tables3-1,3-2,3-3, and 3-4 respectively.

3.2.1.1 Paramer-1 Numbers of storeys (Case-1)

Models with three different storeys that is 4,7,10 are selected representing low-rise and medium rise building.

Table 3- 1 Section dimensions and detailing for Case-1

	Storeys	Dimension(cm)	Reinforcement(mm2)
Case-1	G+3	B30X60	$3\Phi20$ *top and* $4\Phi20$ *bottom*
		C60X60	$12\Phi20$ uniform distribution
	G+6	B30X60	$3\Phi20$ *top and* $4\Phi20$ *bottom*
		C60X60	$12\Phi20$ uniform distribution
	G+9	B30X60	$3\Phi20$ *top and* $4\Phi20$ *bottom*
		C60X60	$12\Phi20$ uniform distribution

3.2.1.2 Parameter's -2 numbers of bays (Case-2)

For each of the above mentioned three models, two bays that is 3 and 6 are checked.

Table 3- 2:Section dimensions and detailing for Case-2

	Bay and storey	Storeys	Dimension(cm)	Reinforcement detailing
Case-2	3 bays	G+3	B30X60	$3\Phi 20$ $top\ and\ 4\Phi 20\ bottom$
			C60X60	12Φ20 uniform distribution
		G+6	B30X60	$3\Phi 20$ $top\ and\ 4\Phi 20\ bottom$
			C60X60	12Φ20 uniform distribution
		G+9	B30X60	$3\Phi 20$ $top\ and\ 4\Phi 20\ bottom$
			C60X60	12Φ20 uniform distribution
	6 bays	G+3	B30X60	$3\Phi 20$ $top\ and\ 4\Phi 20\ bottom$
			C60X60	12Φ20 uniform distribution
		G+6	B30X60	$3\Phi 20$ $top\ and\ 4\Phi 20\ bottom$
			C60X60	12Φ20 uniform distribution
		G+9	B30X60	$3\Phi 20$ $top\ and\ 4\Phi 20\ bottom$
			C60X60	12Φ20 uniform distribution

3.2.1.3 Parameter-3 Column Dimension (Case-3)

One of the above-mentioned models i.e. three bays seven storey model (Figure 3-5) is used to investigate the effect of the column dimension. To do these, three different columns dimensions are considered. Hence this model is done three times with three different column dimensions (45X45,50X50 and 55X55). In all cases the column dimension will remain constant throughout the height of the frame and so does the reinforcement size.

Table 3- 3 Section dimensions and detailing for Case-3

	Models	Dimension	Reinforcement
Case-3	1	B30X60	$3\Phi 20\ top\ and\ 4\Phi 20\ bottom$
		C45X45	$12\Phi 20$ uniform distributed
	2	B30X60	$3\Phi 20\ top\ and\ 4\Phi 20\ bottom$
		C50X50	$12\Phi 20$ uniform distributed
	3	B30X60	$3\Phi 20\ top\ and\ 4\Phi 20\ bottom$
		C55X55	$12\Phi 20$ uniform distributed

3.2.1.4 Parameter-4 column dimension and reinforcement bar size (Case-4)

The above selected model, once again, is used to investigate this one. This time the three bays seven storey model is done three times with the following conditions.

Model-1: The first condition is, that the model is done with columns of uniform dimension (50X50) throughout its height and uniform reinforcement size ($12\Phi 20$) throughout as well.

Model-2: The second condition is done with varying column dimension and uniform reinforcement size throughout storey height.

Model-3: The last condition is conducted model-2 using varying column dimension and reinforcement size as indicated model-3. in Table 3-4.

Table 3- 4 Section dimensions and detailing for case-1 study frames

	Models	Floors	Dimension	Reinforcement
Case-4	1	G-6	B30X60	3Φ20 top and 4Φ20 bot
			C50X50	12Φ20 uniform
	2	G-6	B30X60	3Φ20 top and 4Φ20 bot
		G-2	C50X50	12Φ20 uniform distributed
		3-4	C45X45	12Φ20 uniform distributed
		5-6	C40X40	12Φ20 uniform distributed
	3	G-6	B30X60	3Φ20 top and 4Φ20 bot
		G-2	C50X50	12Φ20 uniform distributed
		3-4	C45X45	12Φ16 uniform distributed
		5-6	C40X40	12Φ14 uniform distributed

3.2.2 CRITERION OF THE SECOND ORDER EFFECTS:

First checked the given all models is there second order effect or not before design the model to consider the second order effect during the pushover analysis. The criterion for taking into account the second order effect is based on the inter storey drift sensitivity coefficient θ , which is defined with equation 3.1.

$$\theta = \frac{p_{tot} * d_r}{v_{tot} * h} \qquad (3\text{-}1)$$

Where,

$d_r = d_i - d_{i-1}$ is the inter-story drift, which evaluated as the difference of average lateral displacements di at the top and bottom of the story under consideration and calculated.

- h- is the story height;
- V_{tot} is the total seismic story shear at considered level;
- P_{tot} is the total gravity load at and above the story considered in the seismic design situation;

The sensitivity coefficients along the elevation for the sample model are described in Table 3-5. In this case of the investigated building, the second order effects need not be taken into account, because the inter-story drift sensitivity coefficient θ is smaller than 0.1 in all stores. In Table 3-5 determine the inter-storey drift sensitivity coefficient θ:

Table 3- 5 Second order effect calculation

Level	p_{tot} **(kN)**	**h(m)**	v_{tot} **(kN)**	d_r **(mm)**	$\theta = \frac{p_{tot} * d_r}{v_{tot} * h} < \mathbf{0.1}$	**Remark**
3	268	3	23.84	1.77	0.0234	ok!
2	778	3	76.70	3.15	0.0429	ok!
1	1288.2	3	108.61	3.88	0.0585	ok!
Ground	1798.34	3	124.56	2.47	0.0468	ok!

So, the second-order effects can be neglected.

3.2.3 **Selection of ductility class medium for Sample model:**

The chosen ductility class for design is "DCM". So, designing, dimensioning and detailing must ensure a ductile behavior of the elements meaning that ductile modes of failure should precede failure modes with sufficient reliability. The plastic hinges which are developed in response to the seismic excitation must be able to dissipate a medium amount of energy in a stable manner. See the detail sample calculation on appendix C.

3.3 PUSHOVER ANALYSIS

3.3.1 MODELLING APPROACH

Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the beams and columns.

ETABS provides user-defined hinges, P-M3 hinges for columns, and M3 hinges for beam as described in FEMA-356. The incremental loads are applied after applying the gravity loads. Flexural characteristics of beams are defined by moment-rotation relationships assigned as moment hinges at beam and column ends. Moment-curvature relationships of beams and columns are calculated based on the section and material properties for each member. Where, the sample calculation of the Scale factors (SF) for rotations and moment capacities of the beams and the columns and performance limits are discussed in detail in appendix A.

Basically, a hinge represents localized force-displacement relation of a member through its elastic and inelastic phases under seismic loads. For example, a flexural hinge represents moment-rotation relation of a beam of a typical one is represented in Figure. 3-7.

Moment-rotation relation for a section consists of plastic rotation and corresponding moments as ratio of yield moment. This relation affects the behavior of a section when a hinge is formed there.

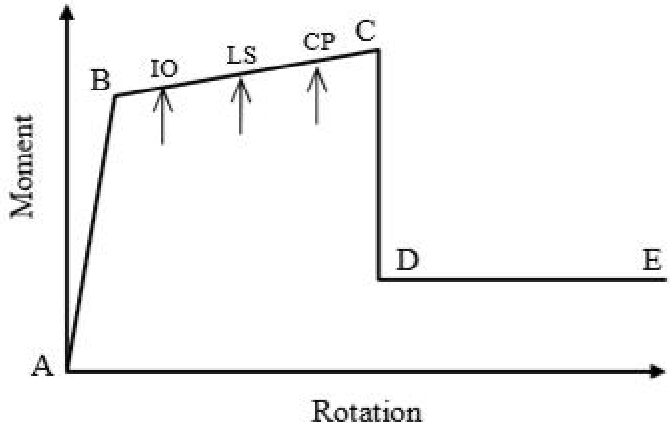


Figure 3- 7 Typical force-deformation curve showing performance levels [20]

As shown in Figure 3-7, five points labeled as A, B, C, D, and E are used to define the moment-rotation behavior of the hinge and three points labeled as IO, LS and CP indicate the acceptance criteria for the hinge; where IO, LS and CP denote Immediate Occupancy, Life Safety and Collapse Prevention respectively.

- Point A denotes unloaded condition (origin).
- Point B corresponds to yielding of the element. No deformation occurs in the hinge up to point B, the portion A to B shows linear response of the structure. The slope of B to C portion is very small and it represents strain hardening phenomenon.
- The ordinate at C represents ultimate strength and abscissa at C corresponds to the deformation at which significant strength degradation begins.
- When, point drop from C to D represents the initial failure of element and resistance to lateral loads beyond point C is usually unreliable. D corresponds to the residual strength.
- The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, gravity load can no longer be sustained and the strength of the components is reduced to zero, a large value for the deformation at point D may be specified.

3.3.2 Steps used in Pushover Analysis:

1. The planar frame is modeled using ETABS2016.2.0 and the hinge properties are defined and assigned as per FEMA 356 and ATC 40 guidelines.
2. First gravity pushover is applied incrementally under force control for the combination of DL+ ψEiLL.
3. Then lateral pushover is applied that starts after the end conditions of gravity pushover under displacement control to achieve the target ultimate displacement or final collapse.
4. The lateral load pattern to be used in the pushover may be in the form of a specified mode shape, uniform acceleration in specified direction, or a user defined static load case. Here the distribution of lateral force employed is in form of the first mode shape.
5. In the model, beams and columns were modeled using frame elements, into which the hinges were inserted.
6. The structure is pushed until global collapse is reached that means when sufficient numbers of plastic hinges are formed to develop a collapse mechanism under the target displacement.

3.3.3 Bilinear Approximation of Pushover Curve (Idealization of pushover curve)

Most pushover methods adopt a bilinear approximation of the actual push-over curve to obtain an idealized linear response curve, as shown in Figure: 3-8.

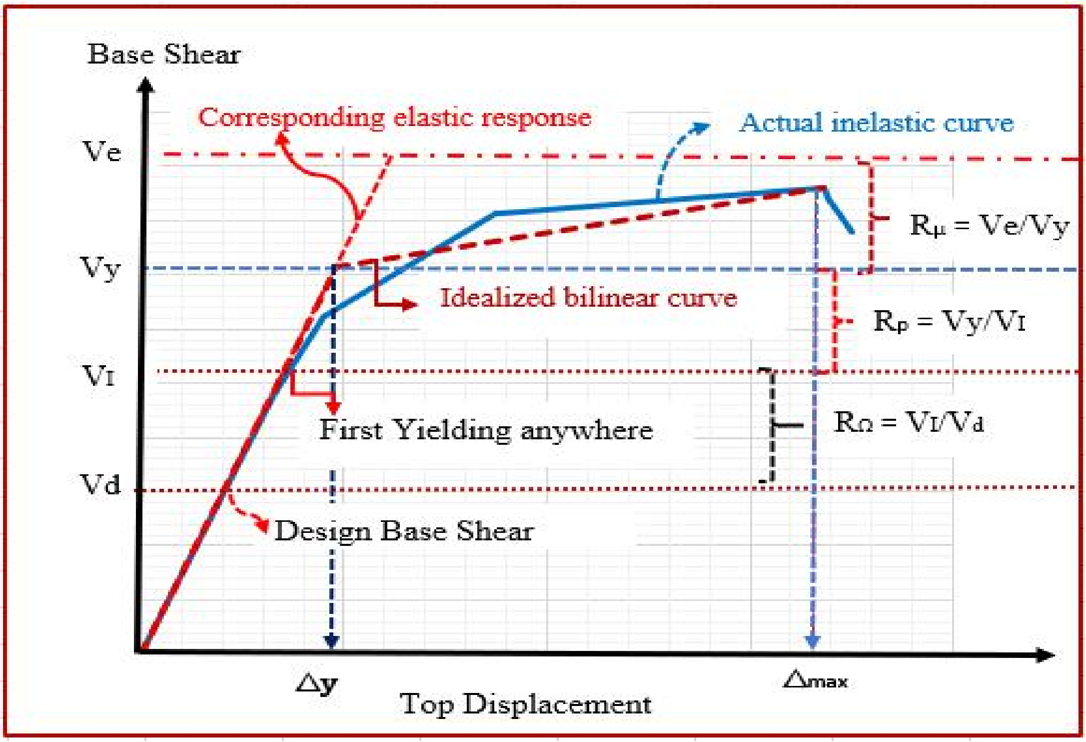


Figure 3- 8 Bilinear Approximation of pushover curve

This is done in such a way that the area under the actual curve will be equal to the area under the bilinear approximate curve.

Bi-linear idealization provides the essential components, which are ultimate shear force, first yielding shear force, the significant yield displacement as well as the ultimate displacement. With the help of these data, the key elements of behaviour factor are calculated, which are ductility factor, Over-strength factor, and redundancy.

3.3.4 **Structural performance limits:**

The performance levels are defined based on the structure type and its intended functions. According to, ATC-40[20] the performance limits can be grouped into two categories: global/structural limits and local/element/component limits.

The global limits typically include requirements for the vertical load capacity, lateral load resistance and lateral drift. For example, the various performance levels in ATC-40 [20] are specified in terms of the maximum inter-storey drift ratio as shown (Table 3-6).

Table 3- 6: Deformation limits for different performance levels, as per ATC-40

	Performance level			
	Immediate occupancy	Damage Control	Life safety	Structural Stability
Maximum Inter-story drift ratio	0.01	0.01-0.02	0.02	$0.33V_i/P_i$

Among these performance levels, the Structural Stability level corresponds to the ultimate limit state of the frame, which can be used for obtaining q for a selected frame. The maximum total inter-story drift ratio in the i^{th} story should not exceed $0.33V_i/P_i$,

Where, V_i - is the total lateral shear force demand in the i^{th} story and

P_i -is the total gravity load acting at that story.

The local performance levels are typically defined based on the displacement, rotation or curvature responses of different elements (beams, columns, shear walls, floors, etc.). The limits on the response of structural elements, such as beams and columns, are many times governed by non-structural and component damages as well. For example, the 'local' deformation limits specified by ATC-40 in terms of plastic hinge rotations of beam elements and column elements for reinforced concrete frame see in Table (3-7 and 3-8) respectively.

Table 3- 7 Plastic rotation limits for RC beams controlled by flexure, as per ATC-40.

$\frac{p_{bot} - p_{top}}{p_{bal}}$	Transverse Reinforced		$\frac{V}{b_w d \sqrt{f_c^{'}}}$	Immediate occupancy	Life safety	Structural stability
				Plastic rotation limit		
≤ 0	C		≤ 3	0.005	0.020	0.025
≤ 0	C		≥ 6	0.005	0.010	0.020

C-indicates that transverse reinforcement meets the criteria for ductile detailing

Table 3- 8 Plastic rotation limits for RC columns controlled by flexure, as per ATC-40.

$\frac{N}{A_g f_C^{'}}$	Transverse Reinforced	$\frac{V}{b_w d \sqrt{f_c^{'}}}$	Immediate occupancy	Life safety	Structural stability
			Plastic rotation limit		
≤ 0.1	C	≤ 3	0.005	0.020	0.02
≤ 0.1	C	≥ 6	0.005	0.010	0.015
≥ 0.4	C	≤ 3	0.00	0.005	0.015
≥ 0.4	C	≥ 6	0.00	0.005	0.010

C-indicates that transverse reinforcement meets the criteria for ductile detailing.

These limits are for flexural failures of an element. Therefore, to use these limits, one should ensure that the failure of a member (structure) is governed by flexural demands, and shear failure, for example, does not take place before these rotational limits are reached. The shear detailing provisions specified in Euro code 2 ensures that shear failure does not initiate before the formation flexural plastic hinges at member ends. On the basis of these background information, it is decided to consider an ultimate limit state based on flexural failure at both local and global levels in this paper.

3.4 Computation of behavior factor (q):

The evaluation of q is made by the use of nonlinear static analysis on RC frames including ductility, redundancy and over strength factors. By this method the overall behaviors of the structure are described by the capacity response curve obtained under the effect of an increasing monotonic lateral loading. The capacity response curve then idealized by bilinear curve of type perfect elastic plastic. The two curves are illustrated Figure 3-8.

Evaluating the seismic strength of structures is usually carried out by the capacity design approach taking into account the non-linear response of structure through the behavior factor and in seismic design codes actual seismic load is reduced by this factor that takes into account several parameters including the capacity to dissipate energy, reserve strength and redundancy. Thus, the need for identifying behavior factor in relation to its importance in the seismic design of frames seems indispensable. In this study two methods to compute the behavior factor (q) used. First discussed the relationship between the two codes.

3.4.1 The relationship between EC8 and ASCES

A typical value of the behavior factor (Response modification factor) specified Euro code8 and American Society Civil Engineers(ASCE-7) codes varies depending on the type of the structural system as well as the ductility class of the structure under consideration. For regular RC frame, value of R as specified EC8 and ASCE-7 are provided in Table 3-9and 3-10 respectively.

Table 3- 9 Values of the behavior factor for RC frames, as per EC8

Structural Systems	**Behaviour Factor(q)**
Ductility Class Medium	$3\alpha u/\alpha y$
Ductility Class High	$4.5\alpha u/\alpha y$

Table 3- 10 Values of the R for RC frames Structure, as per ASCE-7

Structural Systems	**Response Modification Factor, (R)**	**System of overstrength factor(Ω_o)**
Ordinary Moment frame	3.0	3.0
Intermediate Moment frame	5.0	3.0
Special Moment frame	8.0	3.0

EC8 gives the q for regular RC framed structure for two ductility classes: ductility class medium and ductility class high. The ductility and overstrength components are properly incorporated in the formulation of this factor. ASCE-7 categories RC frame into three ductility classes (Table.3-10). The design member forces are therefore, obtained by multiplying the member forces corresponding to the design shear force with the system over strength(Ωo). No such specification exists in EC8.

The European Codes do not address the quantitative influence of redundancy and consider a uniform factor q that includes all the parameters of inelastic behavior of structures, while over strength is taken into account within the framework of Capacity Design. On the contrary, in the US Codes there are factors that quantify separately the redundancy and over strength of a structure. However, these factors lack of the appropriate generality due to the fact that the influence of both redundancy and over strength are not fully clarified. whereas the response reduction factors of ASCE 7 is twice as high as those of Eurocode 8.

3.4.2 **In EC8 the behavior factor for building structures is defined as follows:**

The EC8 (EN1998-1-1,2004) definition of the behavior factor for reinforced concrete structures explicitly accounts for the effects of ductility, redundancy and member over strength. The reference behavior factors assigned to reinforced concrete frames in EC8 is $3\alpha_u/\alpha_1$ for ductility classes medium (DCM), the α_u/α_1 ratio being the ultimate-to first plasticity resistance ratio, which is mostly related to the redundancy of the structure as well as to the collapse mechanism exhibited by the structure depending on the adopted hierarchy criteria. A realistic estimate of the α_u/α_1 ratio could be obtained from pushover analysis but should not exceed 1.5. In the absence of a detailed evaluation, the approximate values recommended by EC8 are shown in Table 3-9. According to EC 8 the behavior factor calculated for all two and 3D structure as shown equation 3.2.

$$q = 3 * \frac{\alpha_u}{\alpha_1} = q_o * k_w \qquad (3\text{-}2)$$

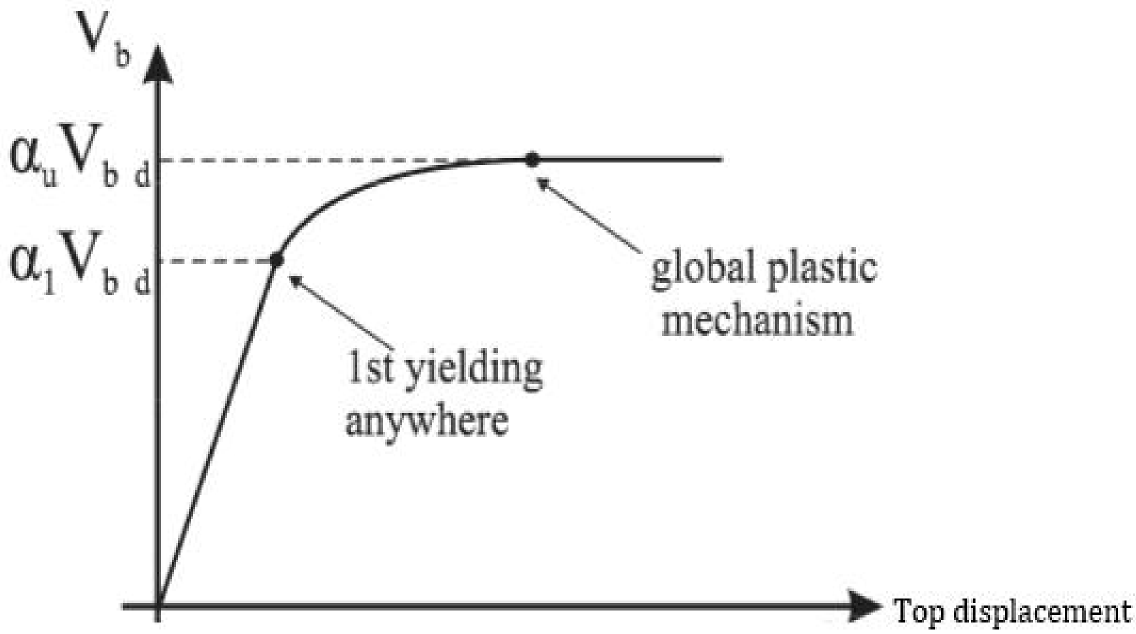
where,

$$q_o = 3 * \frac{\alpha_u}{\alpha_1} = \text{for ductility class medium and}$$

$K_w = 1$, for frame and dual equivalent frame system

- (α_u / α_1) = Multiplication Factor
- α_u & α_1 is obtained from base shear-top displacement curve of a pushover analysis.
- α_u: seismic action at development of global mechanism or represents the strength at the development of a plastic collapse mechanism.
- α_1: seismic action at 1st flexural yielding anywhere or α_1 the force corresponding to first yielding in the structure.
- $\alpha_u / \alpha_1 \leq 1.5$
- Default values given between 1 and 1.3 for buildings regular in plan.

The above expression is explaining as shown in Figure 3-9.



V_{bd} = ***Design base shear***

Figure 3- 9 base shear versus roof displacement

Table 3- 11 detailing of the behavior factor according to Euro code 8:[1].

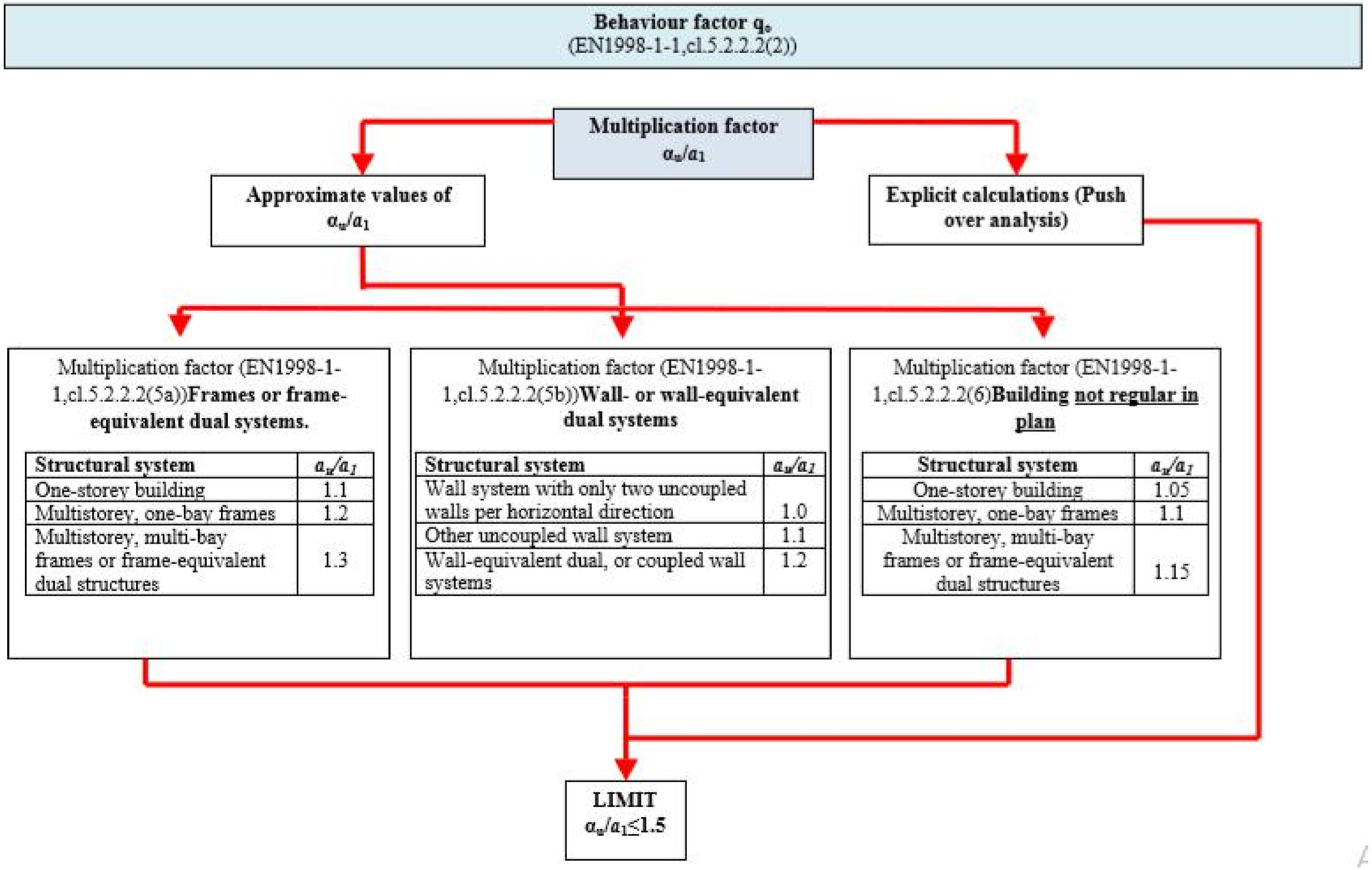


Table 3- 12: Behavior factor q for DCM Structural system [1].

STRUCTURAL TYPES		Regular in plan	Not Regular Structures	
			In plan	In elevation
Frame System, Dual system and Couple Wall System	One Storey (α_u/α_1)	3.3	3.15	2.64
	Multi-storey,one Bay (α_u/α_1)	3.6	3.3	2.88
	multi–storey,multi-bay(α_u/α_1)	3.9	3.45	3.12
System of coupled walls or wall equivalent dual system		3.6	3.3	2.88
Uncoupled wall system, Large light reinforced walls		3.0	3.0	2.4
Torsional Flexible walls		2.0	1.6	1.6
Inverted Pendulum system		1.5	1.2	1.2

3.4.3 Design spectrum for elastic analysis

The seismic analysis of this study is based on EC8(EN-1995-1-1.2004), since ESEN1999-1-1:2015 is the same as EN-1998-1-1:2004. The investigation can also be valid for the new Ethiopian code. As per EC8, the horizontal design forces are defined from maximum acceleration of the structure, under the expected earthquake, that is represented with the acceleration spectrum of the structure. In the horizontal plan, the seismic action acts simultaneously and independently in two orthogonal directions that have the same response. EC8 suggests two different design spectrums.

a) Type 1 for High and moderate seismicity regions (distance EQ, $MS > 5.5$) (southern Europe).
b) Type 2 for Low seismically active regions (local EQs <5.5) (central and northern Europe) and (recommended: PGA on rock ≤0.08g).

In this study, Type1 design spectrum was selected in order to notice the effect of earthquake on the frame systems.

3.4.3.1 Parameters of elastic response spectrum (EN-1998-1-1)

This Table 3-13 shows the parameters of elastic response spectrum.

Table 3- 13 Parameters of Type 1 elastic response spectrum (EN-1998-1-1)

Ground Type	S	T_B(s)	T_C(s)	T_D(s)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

For the horizontal components of the seismic action the design spectrum, Sd(T), shall be defined by the following expressions:

$$0 \le T \le T_B : S_d(T) = (a_g)(S)\left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2.5}{q} - \frac{2}{3}\right)\right]$$

$$T_B \le T \le T_C : S_d(T) = (a_g)(S)\left(\frac{2.5}{q}\right)$$

$$T_C \le T \le T_D : S_d(T) = (a_g)(S)\left(\frac{2.5}{q}\right)\left(\frac{T_C}{T}\right) \qquad (3\text{-}3)$$

$$\ge \beta(a_g)$$

$$T_D \le T \le 4s : S_d(T) = (a_g)(S)\left(\frac{2.5}{q}\right)\left(\frac{T_C}{T}\right)$$

$$\ge \beta(a_g)$$

Design ground acceleration on type C ground: $a_g = \gamma_I * a_gR$

Lower bound factor for the horizontal spectrum: β=0.2

3.4.4 Design Base Shear Force:

Base shear is an estimate of the maximum expected lateral force that will occur due to seismic ground motion at the base of a structure. The design base shear for a building is derived as:

$$V_d = S_d(T_1)\,(m)(\lambda) \qquad (3\text{-}4)$$

Where;

$S_d(T_1)$ is the ordinate of design spectrum(see3.2.2.5) at period T_1

(T_1) is the fundandamental period of vibration of the building for lateral motion in the diraction considered;

m is total mass of the building above the foundation or above the top of a rigid basement. computed in accordance with 3.2.4(2).

λ is the correction factor, the value of which is equal to;

λ = 0.85 if $T_1 \leq 2Tc$ and the building has more than two storeys, or λ=1.0 other wise.be approximated by the following expression;

3.4.5 Fundamental period(EN1998-1-1)

For buildindgs with height of up to 40m the value of T_1 [in sec] may be

$$T1 = C_t H^{\frac{3}{4}} \qquad (3\text{-}5)$$

Where;

C_t: is 0.075 for moment resisting space for concrete frames.

H: is the height of the building in meter. Further details on these planar frames, such as total height of the frame, fundamental period, total seismic weight, and design base shear is provided in Table 3-12.

Table 3- 14 Design Base shear result of RC plane frames:

Storey	No. bays	Height(m)	$T1(s)$	W(kg)	$Sd(T1)$	$V_d(kN)$	Vd/W
3-storey	3	13.5	0.528	1998.61	0.723	147.3	0.0737
	6	13.5	0.528	3888.83	0.723	286.6	0.0737
6-storey	3	24	0.813	3679.15	0.534	200.3	0.0544
	6	24	0.813	7086.57	0.534	385.75	0.0544
9-storey	3	34.5	1.067	5359.68	0.41	224.00	0.0417
	6	34.5	1.067	10448.37	0.41	436.68	0.0417

3.4.6 The formula proposed by American codes for determining the behavior factor is given by:

In the force based seismic design, the force is extracted from spectra based on linear behavior together with the use of a reduction factor that modifies the linear system to an equivalent one to account approximately for the nonlinear effects. This force reduction factor or response modification factor (often called q-factor or R-factor) has an important role in the estimation of design force of a structure. Its value depends on the parameters that directly affect the energy dissipation capacity of the structure: ductility, added viscous damping and strength reserves coming from its redundancy and the over strength of individual members. An appropriate definition of the R-factor is based on a ductility dependent component, an over strength-dependent component, and a redundant dependent component as shown Figure 3-11.

$$q = \frac{V_e}{V_d} = \frac{V_e}{V_Y}\frac{V_Y}{V_I}\frac{V_I}{V_d} = R_\mu R_\rho R_\Omega = R_\mu R_S \quad (3.6)$$

Where, V_e, V_y, V_1 and V_d correspond to the structure's elastic response strength, the idealized yield strength, the first significant yield strength and design base shear (design strength), respectively.

In Eqnation.3.6, R_S is an overall over-strength factor defined as the ratio between the real lateral strength of the structure and the design lateral strength. The ductility factor R_μ is the ratio of the elastic base shear to the real lateral strength of the structure. This ductility factor is a measure of the global inelastic response of the structure and is expressed as a function of the displacement ductility.

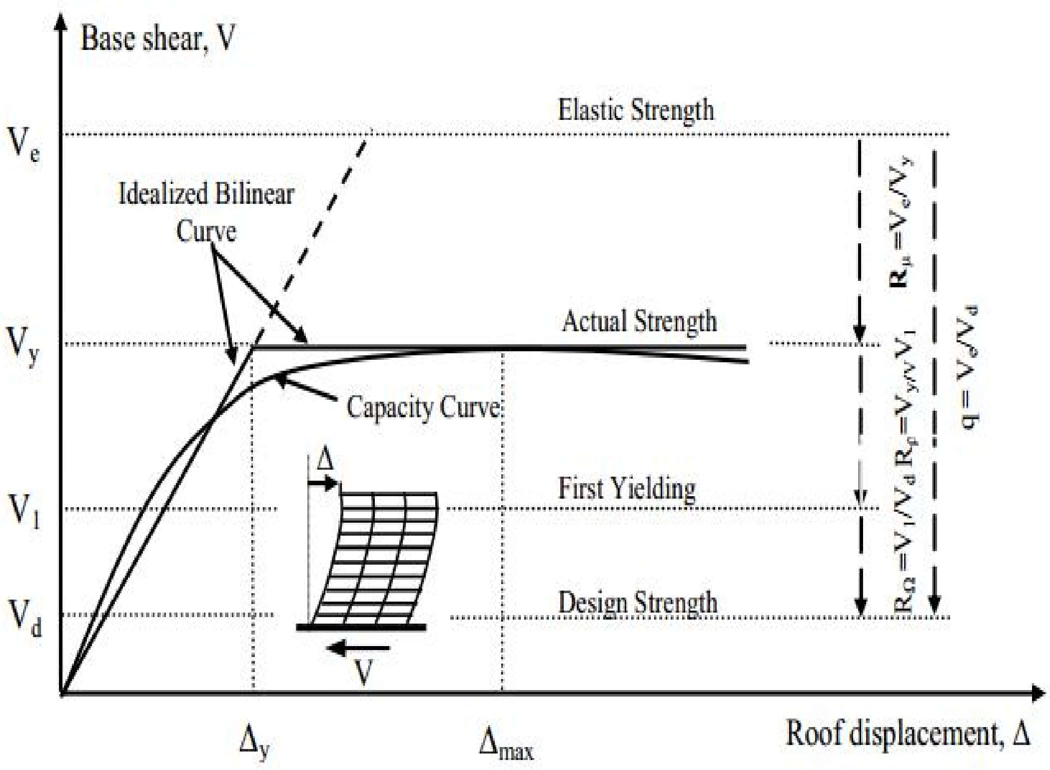


Figure 3- 10 capacity curve of a structure [27]

In the case of using pushover analysis the relation expressing the global behavior of the structure is the capacity curve, that is a base shear versus roof displacement relation obtained under monotonic increasing lateral loads. The force-displacement response curve obtained from pushover analysis is generally idealized by a bilinear elastic–perfectly plastic response curve, as shown Figure 3-11.

3.4.6.1 Ductility Factor

Ductility of a structure, or its members, is the capacity to undergo large inelastic deformations without significant loss of strength or stiffness. Ductility is a very important property, especially when the structure is subjected to seismic loads. High ductility allows a structure to undergo large deformations before it collapse resulting in the dissipation of large amount of energy. It is represented by the ratio of maximum absolute displacement to its idealized yield displacement expressed as follow.

$$R_\mu = \frac{\Delta_{max}}{\Delta_y} \tag{3-9}$$

Where, Rμ is the ductility factor, Δ_u (Δ_{max}) is the ultimate deformation at failure and Δ_y is the yield deformation. Yield deformation is obtained from idealized bilinear pushover curve. It can be expressed in terms of displacements, rotations (for members) and curvatures (for members).

3.4.6.2 Over strength Factor:

The structure has finally reached its strength and deformation capacity. The additional strength beyond the design strength is called the over strength. Most structures display considerable over strength. Sequential yielding of critical regions, material over strength, strain hardening, capacity reduction factors are the sources of over strength (Ω).

$$R_{\Omega} = \frac{V_I}{V_d} \tag{3-10}$$

Where, V_u is the ultimate base shear and V_d is the design base shear.

3.4.6.3 Redundancy Factor

Redundant is usually defined as exceeding what is necessary or naturally excessive. Building should have a high degree of redundancy for seismic load resistance. More redundancy in the structure leads to increased level of energy dissipation and more over strength. Generally, over strength, redundancy and ductility together contribute to act that an earthquake resistant structure can be designed for much lower force than is implied by the strong shaking.

$$R_p = \frac{V_y}{V_I} \tag{3-11}$$

Generally, from above expression the relation of both the formula proposed by American codes and in Euro-code 8 is shown as equation 3-12:

$$q = R = q_O \frac{\alpha_u}{\alpha_1} = R_\mu R_\rho R_\Omega \tag{3-12}$$

where,

$R\rho$ = $\boldsymbol{\alpha_u/\alpha_1}$ **is** the redundancy factor and

q$_O$ = R_μ R_Ω is the basic behavior factor.

3.5 Sample Calculation of user -defined hinge properties:

Sample calculation of user-defined hinge for typical beam and column element

- To calculate user-defined hinge properties of the members, first determine the moment -curvature relation of each section. See the sample calculation of the typical column and beam section as the following.

Table 3- 15 : Parameters used for moment-curvature from elastic analysis

Member	Size(mm)	P(KN)	Reinforcement
Beam	300X600	35	Top- 3-ϕ20 bars and bottom-4-ϕ20
Column	600X600	1598	12-ϕ20 uniform distribution

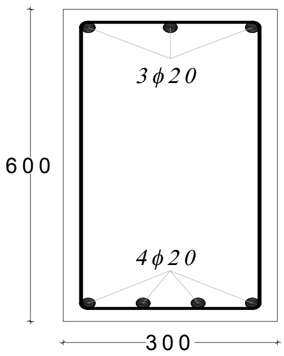


Figure 3- 11 Typical Beam cross-section

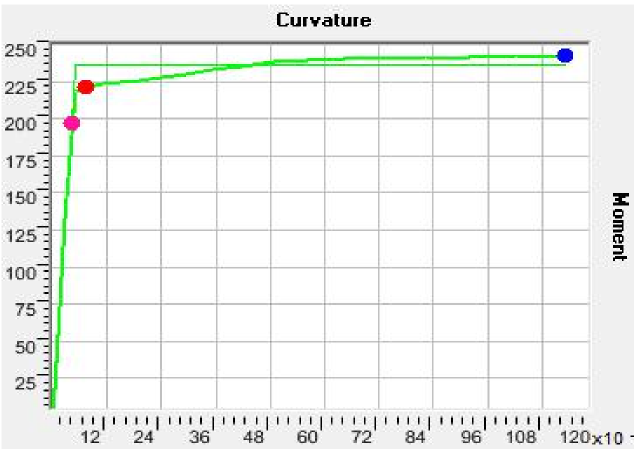


Figure 3- 12 Moment curvature for sample beam

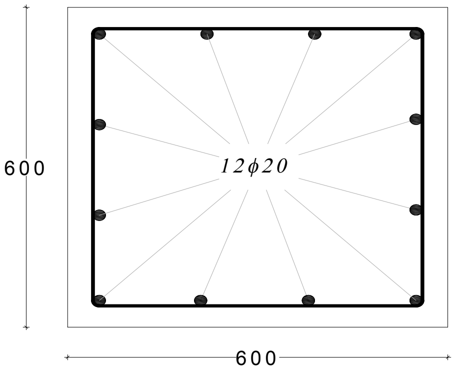


Figure 3- 13 Typical Column Section

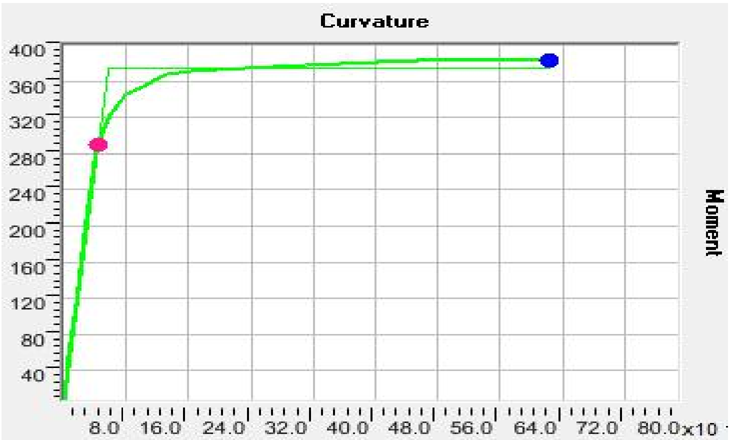


Figure 3- 14 Moment-curvature for typical column section

Moment capacity:

- Maximum moment (Mu) = 240kNm
- Maximum Curvature (φ_u) = 0.065
- Yield moment (My) = 198 kNm
- Yield Curvature (φ_u) = 0.0059

Scale factor for rotation:

- Rotation at A = 0
- Rotation at B = 0
- Rotation at C = φu *Lp = 0.065*0.275 = 0.0175
- Rotation at D = Rotation at C = 0.0175
- Rotation at E = 2* Rotation at D=0.035

Scale factor for moment:

- At point A =0, B = My/My = 1.0, C = Mu/My = 240/198 = 1.21, D=E= 0.2

Table 3- 16 Cross sectional properties of R/C members

member	Size	Mu	My	φ_u	φ_y	Θc	Mu/My
Beam	30x60	240	198	0.065	0.0059	0.016	1.21
Column	60x60	364	296	0.052	0.0047	0.0143	1.23

Where, Θ_c is rotation factor at point **C**

Table 3- 17 Moment-Rotation relationship of user-defined M3 hinges

	Beam(B30X60)	
point	Moment/SF	Rotation/SF
E-	-0.2	-0.035
D-	-0.2	-0.0175
C-	-1.21	-0.0175
B-	-1	0
A	0	0
B	1	0
C	1.21	0.0175
D	0.2	0.0175
E	0.2	0.035

Where, SF is scale factor for moment and rotation

Table 3- 18 Moment-Rotation relationship of user-defined P-M3 hinges for sample frame

	Column (C60X60)	
Point	Moment/SF	Rotation/SF
A	0	0
B	1	0
C	1.23	0.0143
D	0.2	0.0143
E	0.2	0.0286

Where, SF is scale factor for moment and rotation

After defining the above points in ETABs2016.2.0 software the scale factors of the moment and rotation are displayed.

3.5.1 Failure Mechanism:

Under the application of pushover loads the members in a structure initially remain elastic up to a certain moment that is the maximum moment of resistance of a fully yielded section. Any further increase of moment will cause the beam to rotate with little increase in load. The rotation occurs at that particular moment. So, these expected locations of damage caused by a yielded zone having large inelastic rotation capacity at constant restraining moment are called plastic hinges. The combination of inelastic hinges at the ends of beams and columns which formed in a frame eventually makes it unstable and causes its collapse, hence called collapse mechanism. The mode of failure in the form of sequence, location and number of plastic hinges in a sample frame is as shown below in Figure 3-16. The plastic hinges should be distributed throughout the structure for a good failure mode so that maximum number of members will be involved in energy dissipation. If the damages are concentrated only at a few locations then members in those locations will collapse even before the other members get into the inelastic range. So, sequence and distribution of hinge formation is very important to have a good failure mode in the structure. The hinge formations started from the lower storey and proceeds to upper stories. The final step of hinging at failure after attaining the target displacements are shown in the Figure 3-17.

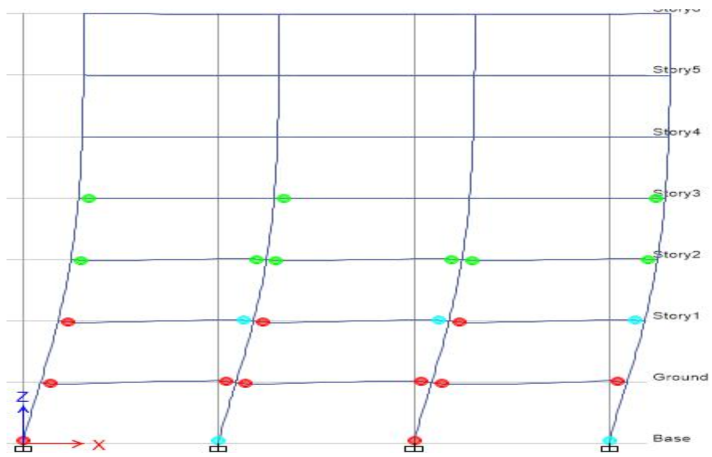


Figure 3- 15: Hinge Formation at each damage limit Failure

3.6 FRAME Verification Data:

This verification is done by hand calculation with assigning hinge in each member and by modeling the given frame on SAP2000. See the given properties, loading and frame in the following figures below. Figure 3-18 shows the element allocation with its joint assignment of the frame elements which is used during section capacity estimation and moment curvature relation.

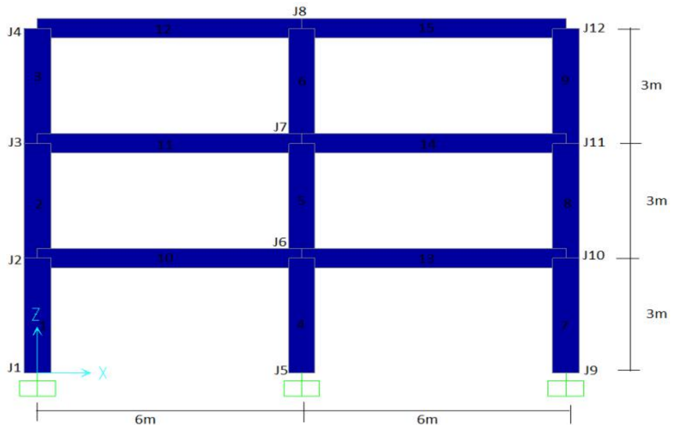


Figure 3- 16 Frame element and joint allocation [24]

3.6.1 **Verification Frame Loading and Modeling**

3.6.1.1 Loading on SAP2000

The frame is loaded with gravity load and lateral load. A super imposed dead load of magnitude 10KN/m at third story and 15KN/m at first and second story and service live load of 2KN/m on every story. The lateral load is not known at first so it is increased at every step until mechanism is formed. For the pushover analysis the following loading is used DL + 0.3LL + EQ and it is named “PUSH”.

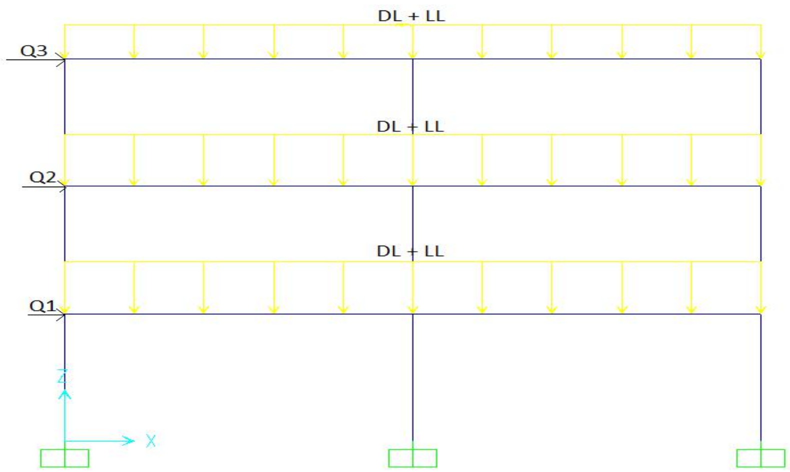


Figure 3- 17: Frame Loading [24]

Material Property:

C-55 concrete and S-570 steel is used and concrete cover of 50mm is used.

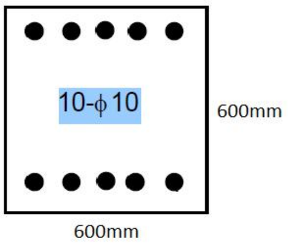


Figure 3- 18 Column cross-section

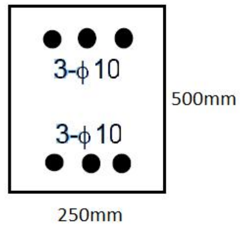


Figure 3- 19 Beam Cross-section

3.6.1.2 Modeling using SAP2000

The moment curvature relation used for the section capacity determination for both SAP2000 and for the increasing load approach (the handmade pushover analysis with an increasing load for the verification frame) is shown in Table 3-19.

Table 3- 19 Moment Curvature Relation for beams and columns on the sample frame

Member	Element type	My KNm	My KNm(verify)	θ_y rad/m	θ_y rad/m(verify)
1	Column	124	122	0.0055	0.0046
2	Column	115.5	114	0.0056	0.0054
3	Column	107.5	107	0.0057	0.0058
4	Column	166	163.4	0.00519	0.0061
5	Column	143	140.12	600.00	0.0063
6	Column	119	117.55	0.0060	0.0062
7	Column	133.5	131.68	0.0056	0.0055
8	Column	122	0121	0.0057	0.0056
9	Column	110	109.6	0.0054	0.0055
10	Beam	49	47.6	0.0073	0.0068
11	Beam	50	50.8	0.0069	0.0062
12	Beam	53	53.5	0.0069	0.0062
13	Beam	49	47.6	0.0073	0.0068
14	Beam	50	50.5	0.0069	0.0062
15	Beam	53	53.5	0.0069	0.0062

3.6.2 Results of SAP2000 and hand calculation (increasing lateral load in each step) Analysis

From the verification analysis show Figure 3-18, it is seen that in both cases the sequence of hinge formation is similar and the graph of the SAP2000 results from original and verification are almost the same curves, but the difference is occurred may be the using material properties of for hinge property.

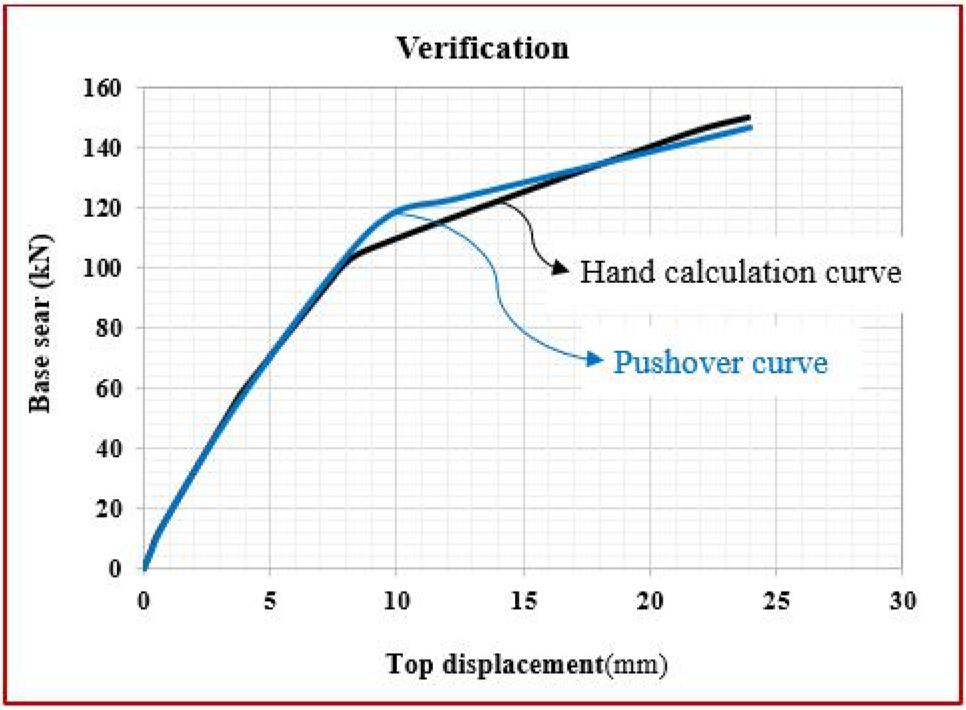


Figure 3- 20: Capacity Curve for hand calculation and SAP2000

CHAPTER 4: RESULT AND DISCUSSION

In this chapter results of the analysis are compiled and presented with narration and graphics including capacity curves accompanied with various evaluation results.

4.1 Push Over Curve:

The main output of pushover analysis is in the form of base shear versus roof displacement curve called pushover curves. Pushover curves for all frames are presented in Figure.4-1. These curves represent the global behavior of the frame with stiffness and ductility. The slope of pushover curves is gradually reduced with increase in the lateral displacement of the frames. This is due to the progressive formation of plastic hinges in beam and column throughout the structure.

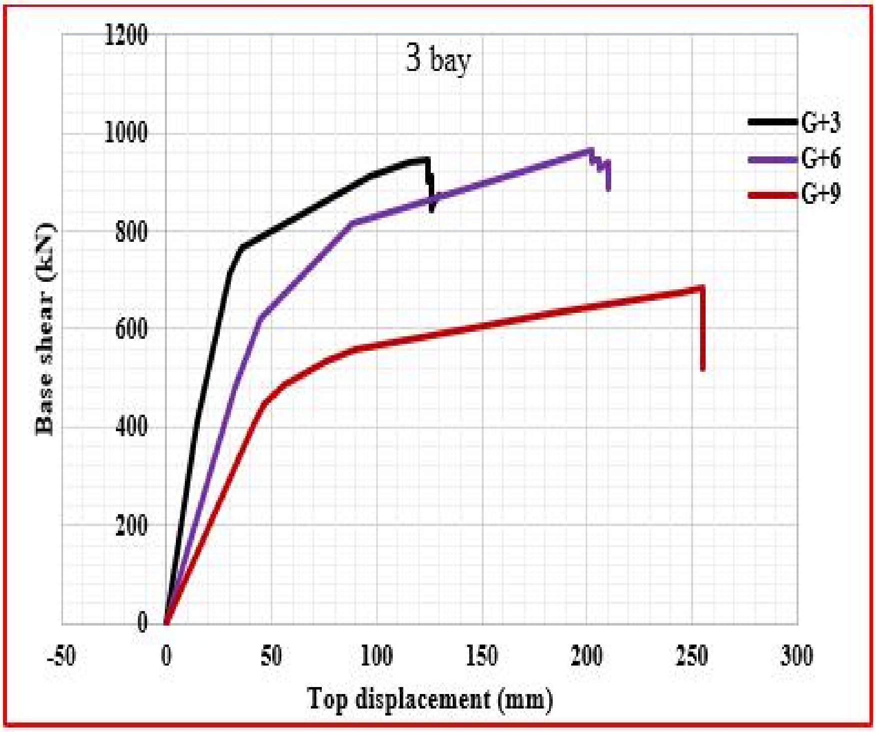


Figure 4- 1 Pushover curve with increasing height for 3 bay

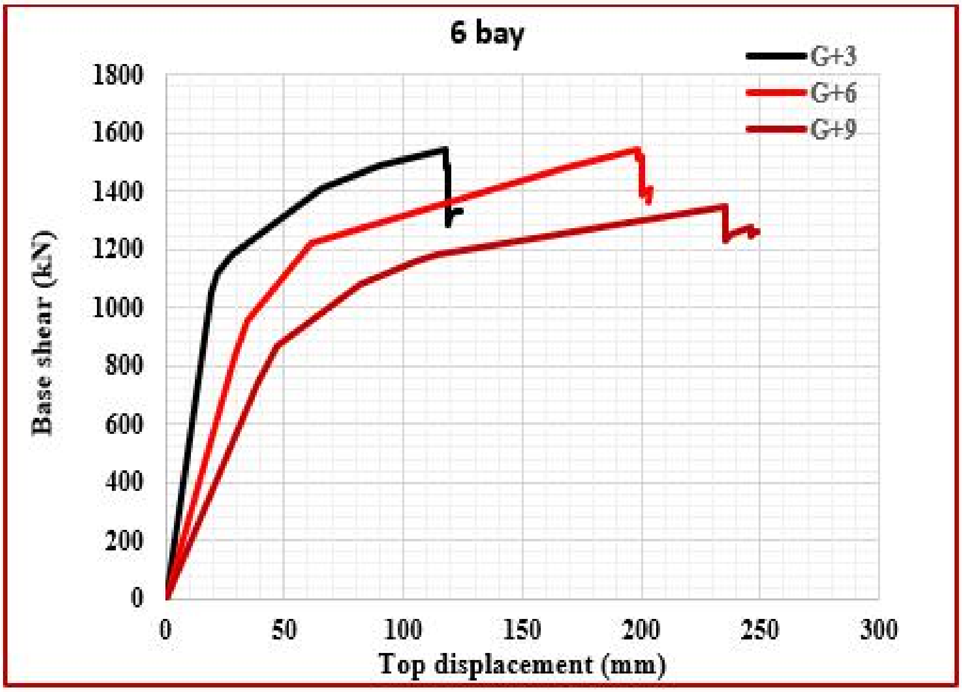


Figure 4- 2 Pushover curve with increasing height for 6 bay

Figure 4- 3 Pushover curve with increasing bay storey four

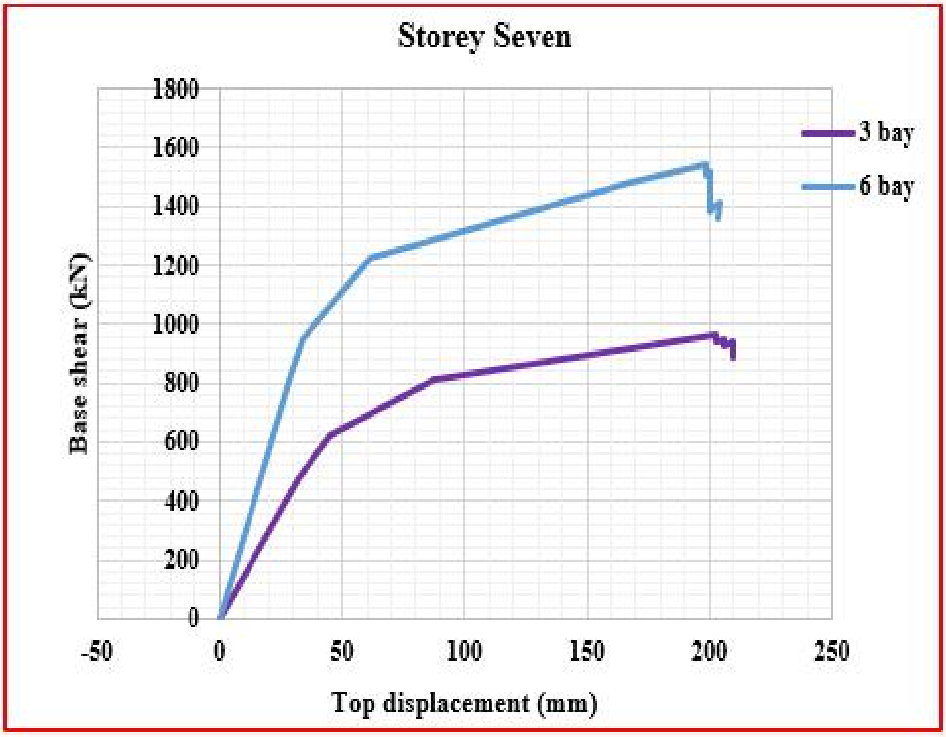


Figure 4- 4 Pushover curve with increasing bay Storey Seven

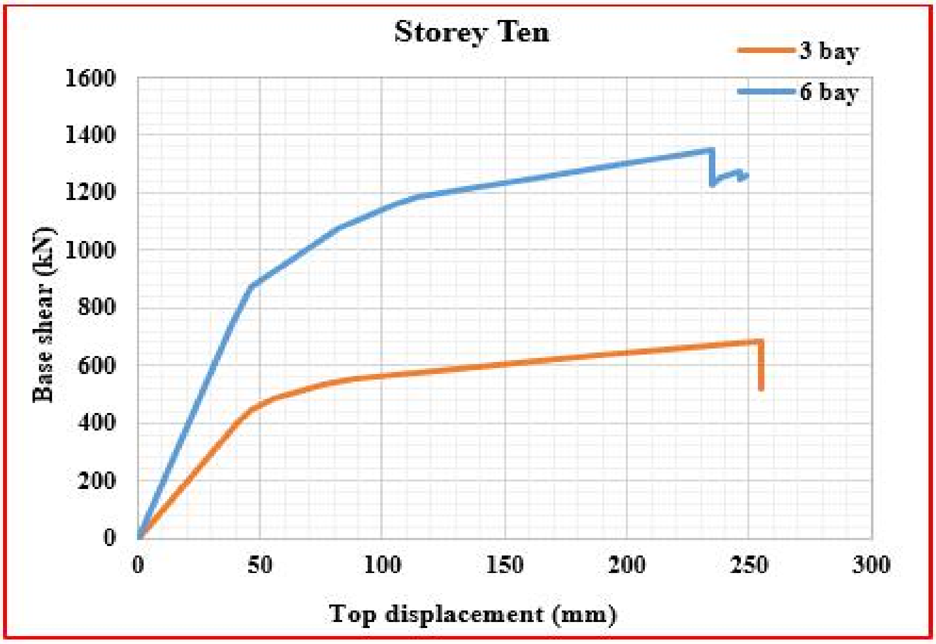


Figure 4- 5: Pushover curve with increasing bay storey ten

In Figure 4-1 to 4-5 when increasing number of storey with increasing top displacement of the frame, then once increasing number of bays by reduced the displacement.

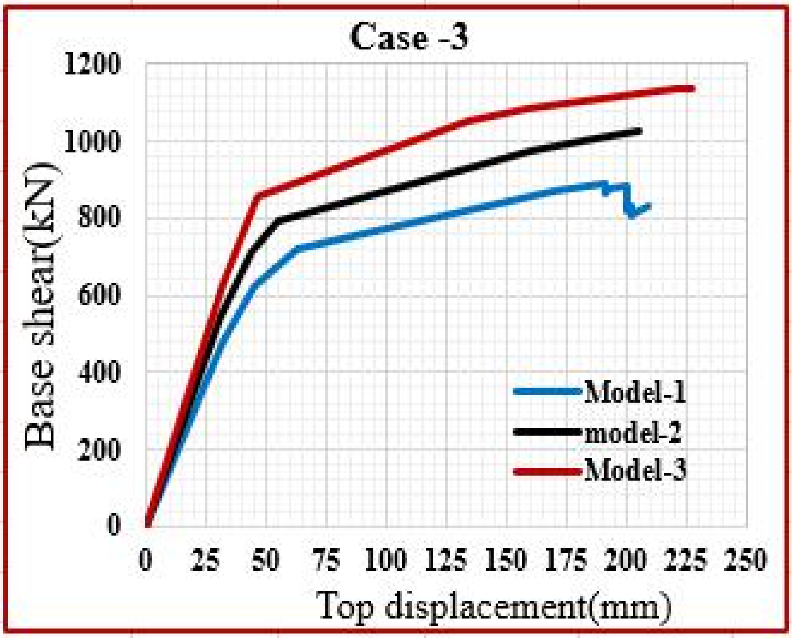


Figure 4- 6: Pushover curve with increasing column dimension

In Case-3 frames, the base shear force increases with the increase of "Column/Beam" capacity factor. This is due to the increase in the dimensions of columns cross-sections (hence an increase of their lateral resistance), which will absorb more forces.

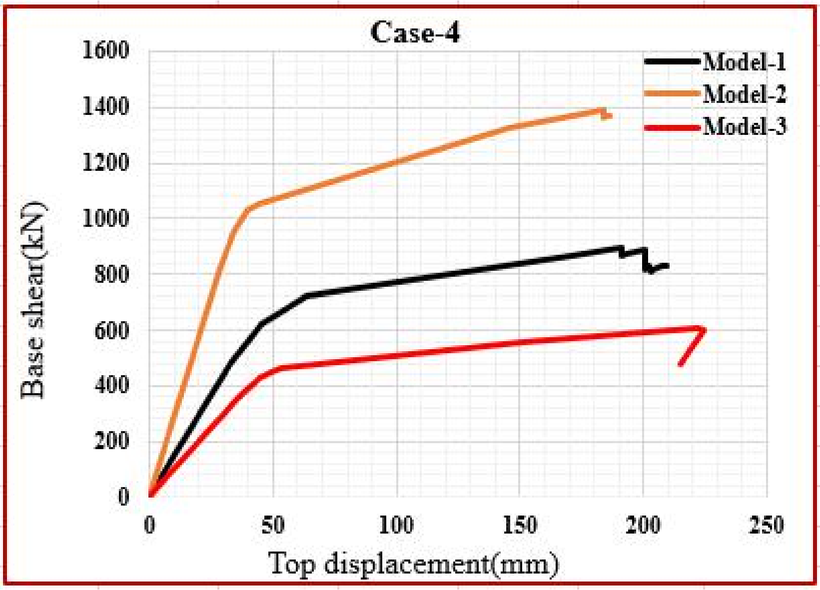


Figure 4- 7: Pushover curve for case-4

As shown Figure 4-7 in Case-4 a uniform cross-section and reinforcement size along the storey, which has maximum base shear and small displacement from reducing gradually through the height of the frame.

4.1.1 The performance limit states of the structure

As mentioned earlier in Sect.3.3.4, two performance limits are considered in pushover analysis. The first one corresponds to structural stability limit. This limit state is defined at global structure (in terms of lateral load resistance) as well as at the story level (in terms of the maximum inter-story-drift ratio). The second limit is based on plastic hinge rotation capacities that are obtained for each member depending on its cross-sectional geometry.

4.1.1.1 STOREY DISPLACEMENT

The displacement of the frame obtained from the non-linear static pushover analysis used to understand the displacement capacity and stiffness of the frames. At every deformation step, pushover analysis determines plastic rotation hinge location in the elements and which hinges reach the performance limit state, which are IO, LS, and CP using colors for identification see in appendix B.

4.1.1.2 STORY DRIFT:

Figure 4-8 to 4-10 shows inter story drift ratio for all cases at the collapse point. From this information, which can know the maximum drift ratio and which, model has maximum inter story drift ratio and its performance level according to Table 3-6. The expected performance level of a given structure is evaluated by comparing the drift of the frame with the limit drift as shown below Table 4-1. So, it is important to accurately quantify the story drift in order to have a clear image of the damage on the frame and the attained performance level of the frames.

Inter-story drifts ratio:

Inter-storey drift ratio is defined as the ratio of relative horizontal displacement of two adjacent floors and corresponding storey height (h).

Inter-storey drift = $\Delta/h = (\Delta_i - \Delta_{i-1})/h$

Where, Δ_i = displacement at ith storey

h = each storey height

Table 4- 1: Sample global limits for different performance levels as per ATC-40

Stories	V_i(kN)	P_i(kN)	$0.33V_i/P_i$	Max IDR	Limitation
Ground	683.97	8595.66	0.0263	0.01854	ok
1st floor	581.05	7263.06	0.0264	0.0192	ok
2nd floor	475.33	5930.47	0.0264	0.0131	ok
3rd floor	369.00	4597.87	0.026	0.0073	ok
4th floor	262.68	3265.23	0.026	0.0035	ok!
5th floor	156.36	1932.68	0.026	0.0017	ok!
6th floor	50.04	600.08	0.026	0.00066	ok!

Note: Max IDR – Maximum Inter Storey Drift Ratio

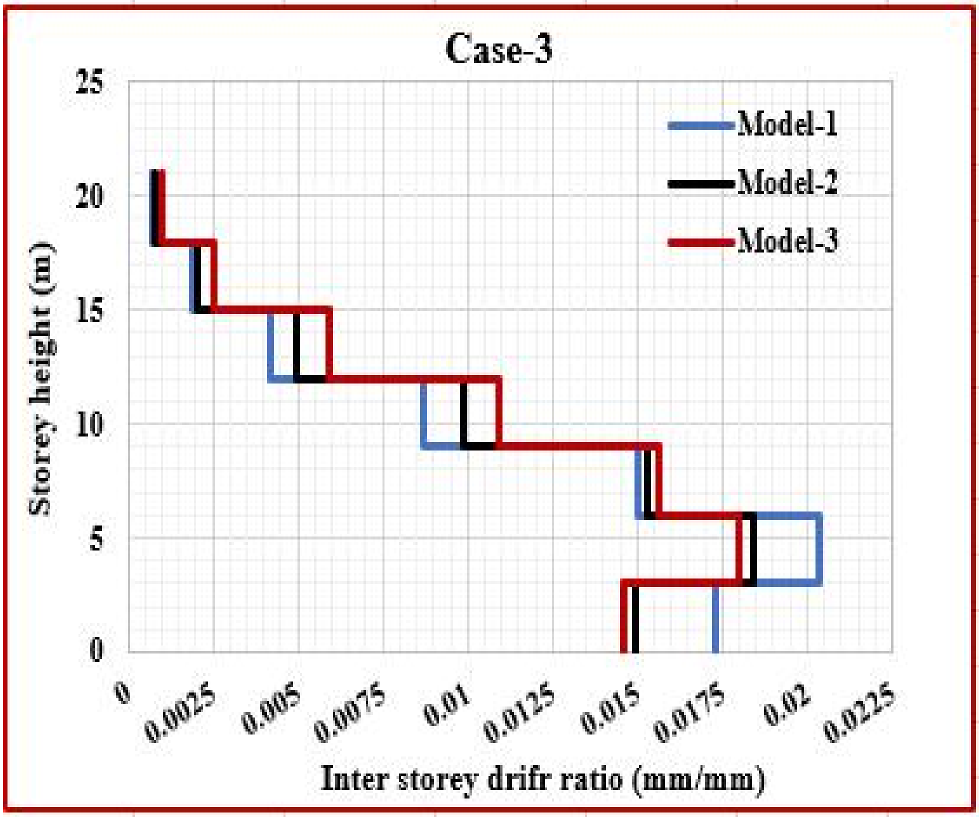


Figure 4- 8: Inter Storey Drift Ratio for Case-3

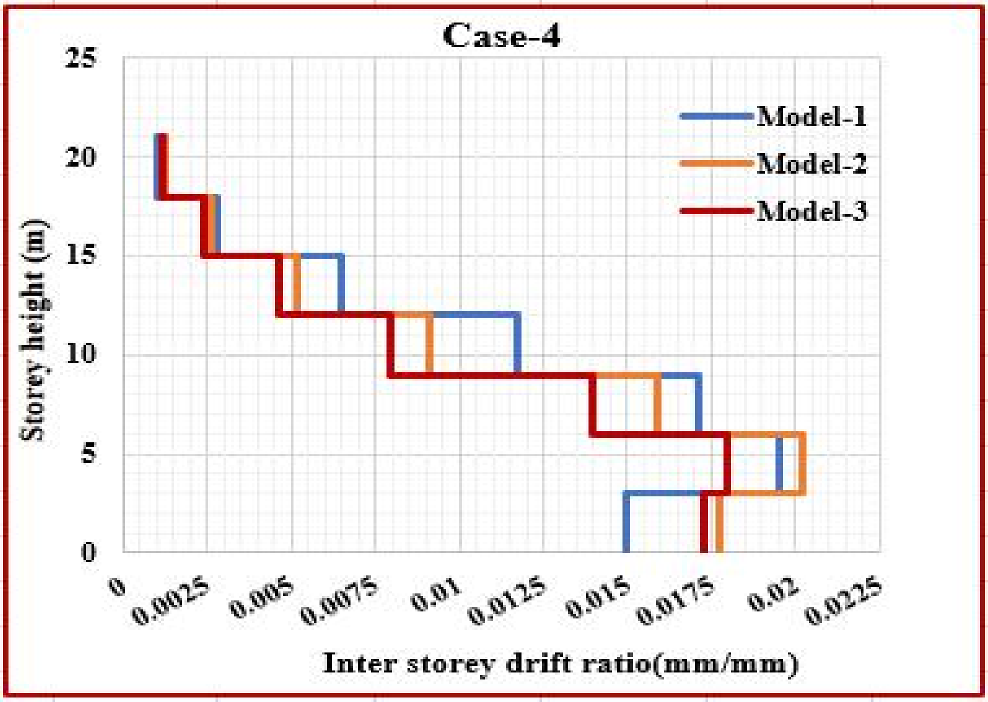


Figure 4- 9: Inter Storey Drift Ratio for Case-4

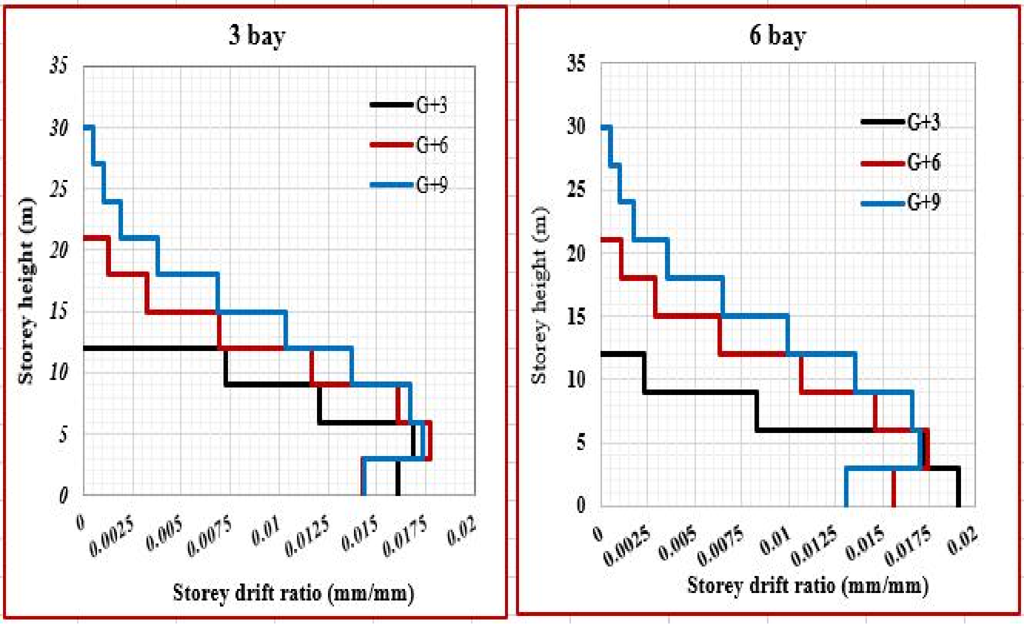


Figure 4- 10 Inter Storey Drift Ratio for 3 and 6 bays

4.1.2 Behavior factor parameters:

The results obtained from pushover curve of the base shear and roof displacements and the evolution of different parameters of the over strength factor, ductility factor, redundancy factor, and behavior factor for all cases are shown in Table 4-2, to 4-8 except

Table 4-4. Whereas, Table 4-4 indicates the axial load ratio effects on behavior factor with increasing the column cross-section for case-3 models.

Table 4- 2: Results from pushover curve Case- 3

	Model	**V_d**	**V_e**	**V_y**	**V_I**
Case-3	1	182.95	735.38	583.06	343.99
	2	188.15	801.13	614.21	391.27
	3	193.9	859.96	660.95	440.55

Table 4- 3 Parameters of behavior factor with case 3

	Model	**R_u**	**R_p**	**R_o**	**q**
Case-3	1	1.26	1.69	1.86	3.96
	2	1.30	1.56	2.08	4.22
	3	1.34	1.5	2.26	4.41

Table 4- 4: Axial load effects on the sampling frames

		Column Element Numbers				
Dimension	**Model**		**1**	**2**	**3**	**4**
C45X45	1	Axial load	940.62	1925.72	1909.9	1073.88
		Axial load ratio	0.278	0.569	0.564	0.317
C50X50	2	Axial load	975.17	1935.08	1923.87	1108.52
		Axial load ratio	0.233	0.463	0.460	0.265
C55X55	3	Axial load	1008.8	1951.63	1943.66	1142.56
		Axial load ratio	0.199	0.386	0.3845	0.226

The above Table 4-2 and 4-3 present the results of base shear and behavior factor parameters for the given models, that show as the column dimension increased the value of over-strength factor also increased; whereas the ductility factor and the multiplication factor (redundancy factor) found reduced as the column section increased. According to case-3 models, it is clear that the behavior factor values decrease as the axial force increase due to increasing the column size for three models. Also, in Case-3, the column size increases the axial force increases; it leads to limiting the ductility of the frames.

Table 4- 5: Results from pushover curve case-4

	Model	V_d	V_u	V_y	V_I
Case-4	1	189	773.73	576.14	286.1
	2	185	898.54	718.17	391.3
	3	185	854.11	674.35	471.7

Table 4- 6: Parameters of behavior factor case-4

	Model	R_u	R_p	R_o	**q**
Case-4	1	1.34	2.0	1.51	4.04
	2	1.25	1.52	2.54	4.83
	3	1.26	1.43	2.54	4.57

Figure 4-6 shows the result of case-4 over strength factor, ductility factor and redundancy factor; that present the effect of the column and bar size along storey height. In the models of 1 and 2 the only difference in the input variable is column size; then the result in model-1 show higher ductility and redundancy factor, but smaller over-strength factor. Moreover, from model 2 and 3 the obtained result shows similarity in the bar size effect on ductility and over-strength factor for two models. However, the effect on the redundancy factor is large for model-2. According to the result of case-4, the reinforcement size has slight effect while the column size has significant impact on the behaviour factor along the frame height.

Table 4- 7: Push over parameters for the two bay frames

No. of storey	No. bay	**Vd**	**Ve**	**Vy**	**V_I**
3	3-bay	147.3	874.2	699.1	407.00
	6-bay	286.6	1608.72	1329.5	994.00
6	3-bay	200.3	917.67	712.9	480.97
	6-bay	385.75	1428.88	1143.83	826.37
9	3-bay	224	684.42	518.5	403.50
	6-bay	436.68	1337.97	1042.06	744.89

Table 4- 8: Push over parameters for formulation

No. of storey		R_Ω	R_μ	R_ρ	**q**	
					code	**formula**
3	3-bay	2.76	1.25	1.72	3.9	5.90
	6-bay	3.468	1.22	1.21	3.9	5.12
6	3-bay	2.39	1.28	1.48	3.9	4.53
	6-bay	2.14	1.25	1.38	3.9	3.69
9	3-bay	1.779	1.32	1.28	3.9	3.04
	6-bay	1.70	1.28	1.39	3.9	3.03

Table 4-8 shows the variation of the q-factor components according to the stories and bays number. In particular, the over strength factor R_Ω, the redundancy factor R_ρ, and the ductility factor R_μ resulting from pushover analysis. In Table 4-8, it can be noticed that the number of stories influence the value of R_Ω-factor. The greater value of this factor is obtained for low-rise frame. This result could be explained by the fact that the magnitude of design over strength depends on the relative values of the gravity and earthquake loads. Comparison between the earthquake base shear to the total gravity load ratio of the studied Frames (see Table 3-12) shows that the highest V_d/W ratio is observed for the 4-storey frame, reflecting the high stiffness and the efficiency of this frame in resisting lateral forces. The lowest V_d/W ratio is observed for the 10-storey frame as a result of the high total gravity load. Furthermore, the RΩ-factor is little sensitive to the number of bays.

The R_ρ-factor is significantly reduced for 3 bay frames and is increased for bay 6 with increasing the storey height. The average value of R_ρ-factor is 1.49 and 1.33 for frames with 3 and 6 bays, respectively. These values are higher than that recommended by EC8: $(R_\rho(\alpha_u/\alpha_1) = 1.30)$ for ductility class medium frames.

Ductility factor (R_μ) shows an increasing trend when raised the storeys from 4 to 10 but reduced when increasing numbers of bays. The effect of ductility factor is influenced by inter- storey- drift ratio.

The following sections provide and discuss the results of computing q-factor considering the effects of stories and bays number, column dimension and reinforcement bar size. The q-factor value specified by EC8 is represented by a horizontal dashed line (q design = 3.9 for ductility class medium regular Frames).

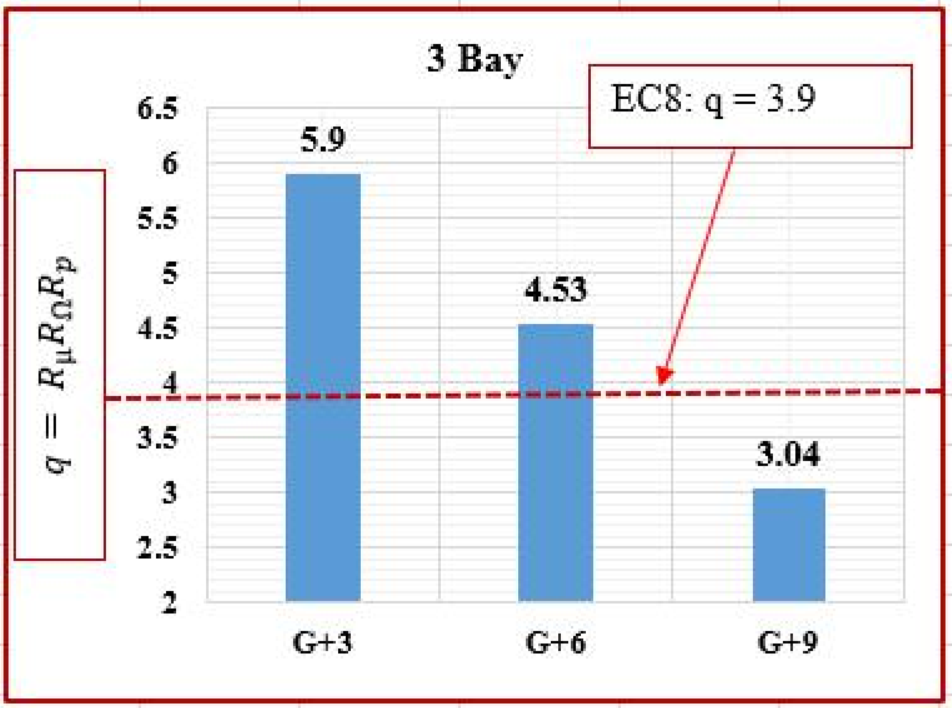


Figure 4- 11 Behavior factor with increasing storey for 3 bay

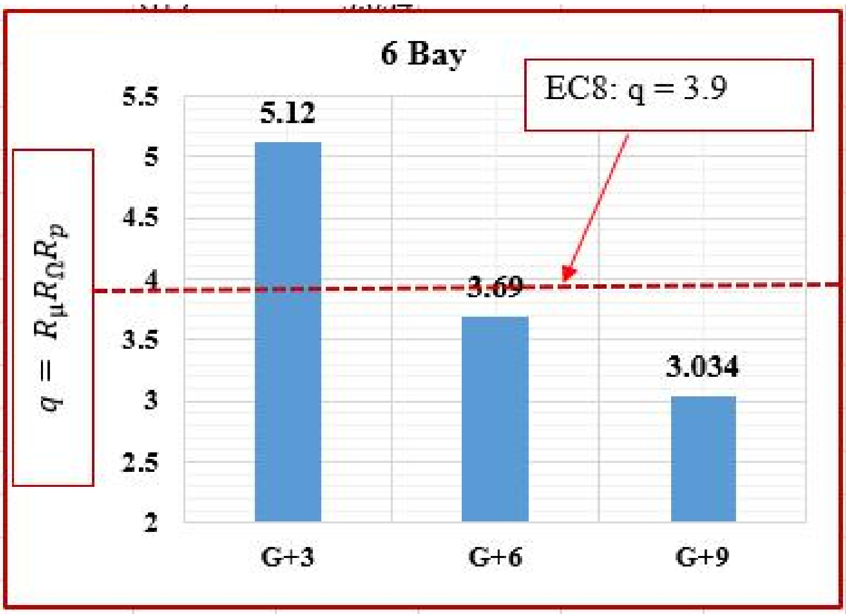


Figure 4- 12 Behavior factor with increasing storey for 6 bay

As shown Figure 4-11and 4-12 the behavior factor is reduced with increasing the numbers of storey. Furthermore, in the case of high-rise frame, the calculated q-factor is less than to the EC8 specified value. It also can be indicated that the obtained results show that the q-factor value depends on the structural performance limits that define the failure criteria, which are not taken into account by EC8.). In general, the number of stories has significant influence on the q-factor value. It is clear that the value of q-factor decreases as the number of stories increases.

Figure 4- 13 Behavior factor with increasing bays for storey four

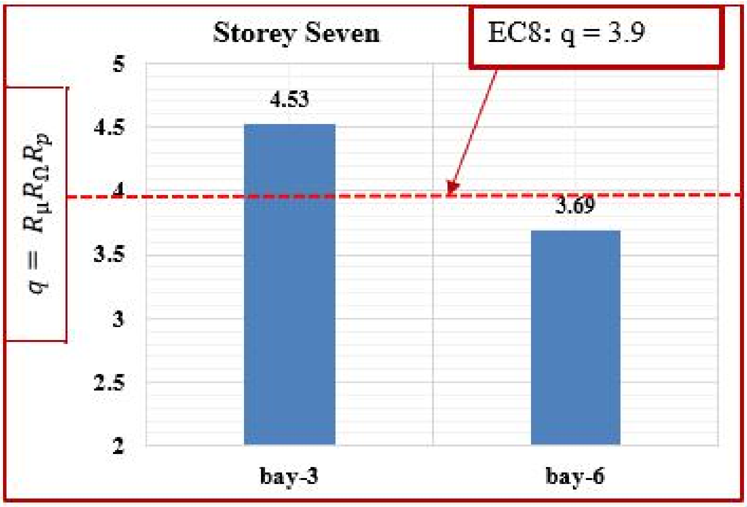


Figure 4- 14 Behavior factor with increasing bays for storey seven

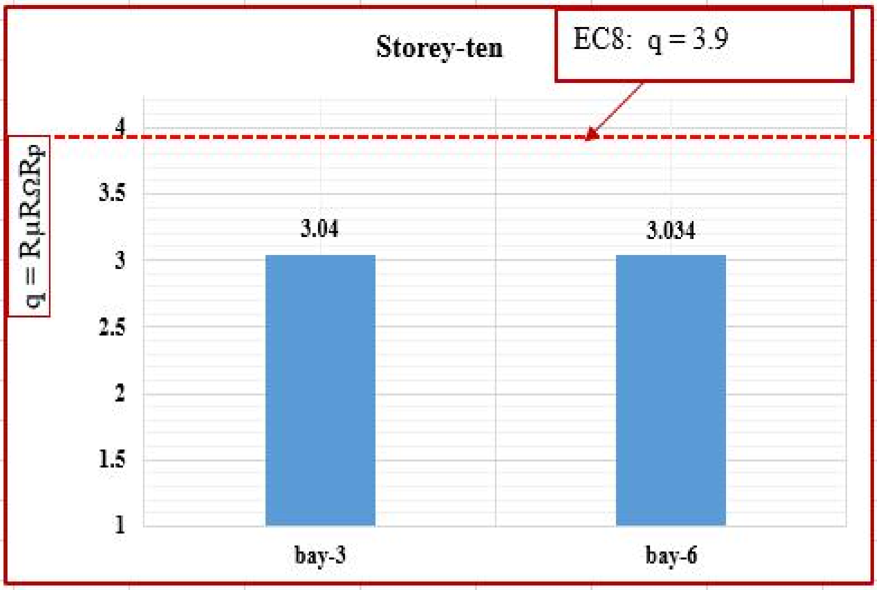


Figure 4- 15 Behavior factor with increasing bays for storey ten

In Figure 4-13 and 4-14 behavior factor is significantly reduced, but as shown Figure 4-15 when increasing numbers of bay behavior factor is almost the same. Generally, the low-rise storey frames have greater behavior factor than the higher rise frames.

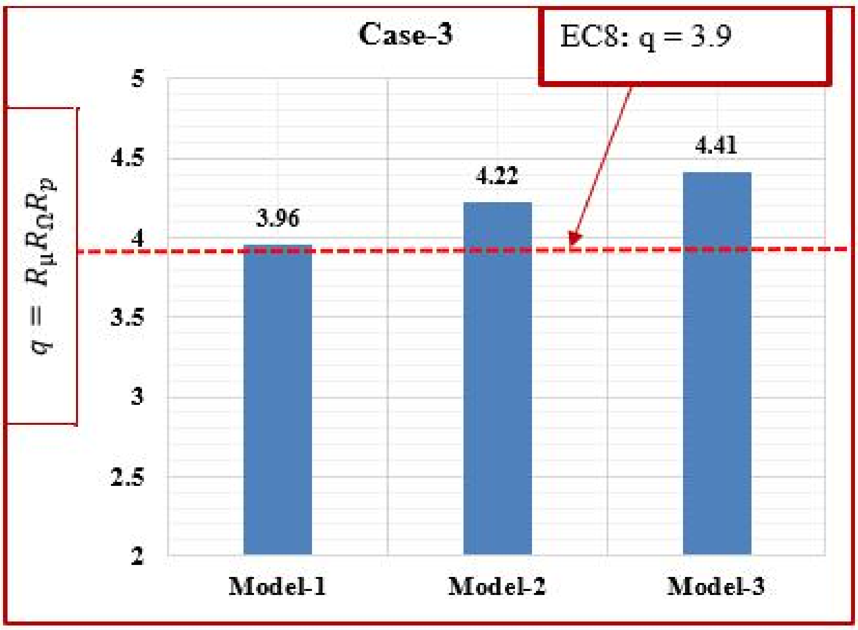


Figure 4- 16 Behavior factor for Case-3

In case-3 as shown three bay frames with six stories are analyzed to investigate the relation of the behavior factor with the column size. Hence, these three frames are taken to be similar except there column size. Column size of 45*45, 50*50 and 55*55 are used, i.e one for each frame. These column sizes are kept uniform from bottom to top of their respective frames.

The analysis shows that the behavior factor has a direct relation with the column size. That implies the behavior factor for a frame with 45*45 column is minimum whereas the behavior factor of a frame with column size 55*55 is maximum.

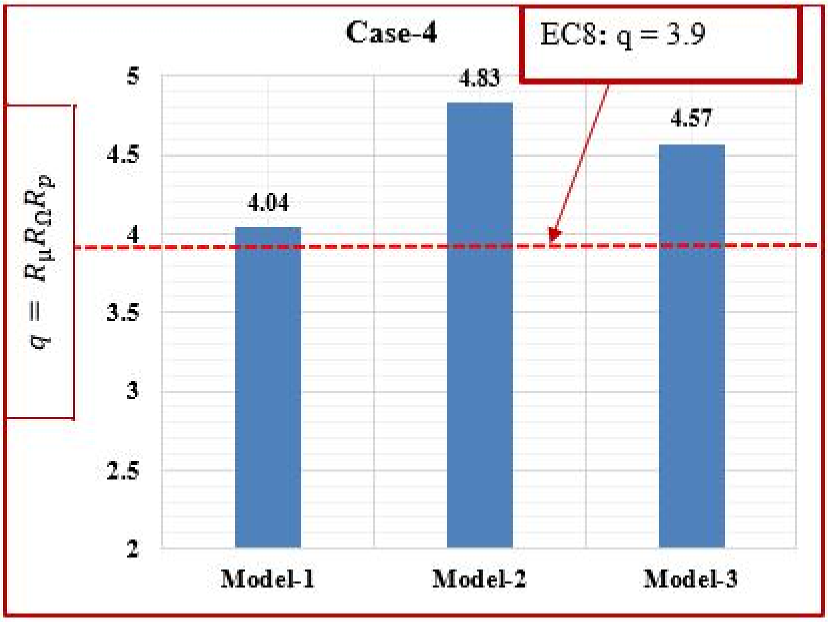


Figure 4- 17 Behavior factor for Case-4

Finally, results of the analysis and evaluations of this study are tried to be summarized.

- Based on this study, overall results of "q" factors obtained for low-rise and regular section of the systems are considerably higher when compared to code values. This result may be issued to some facts such as:
- Frames designed as high ductility, had to fulfill the requirement of columns being stronger than beams. This practice brings high ratios of overdesign since beam design is mostly governed by gravity loads. Especially in story four frames, gravity loads govern the design instead of lateral loads, which results an excess lateral strength, raising the over strength factor
- The decrease in strength of the structure results in a decrease of overstrength.
- The structures with vertical geometric irregularity have lower demands than regular structures

CHAPTER 5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

This study calculated the behaviour factor by using non-linear pushover analysis as described in previous section, which has been done for 12 reinforced concrete planar frame structures. Followings are the conclusions arrived at from the analysis and interpretation of the results.

- In practice, the behavior factor (q) according to EC8 depends on types of structure, ductility type and regularity requirement. However, this value is kept constant for multistory, multi-bay frame with ductility class medium for all structures. This study has obtained different values from EC8 and the values of q not constant for all cases. This is knowing some significant factors affecting behavior factor (which include not only 'types of structure', 'ductility class' and 'regularity'; but also 'frame type', 'over strength' and 'redundancy capacities').
- EC8 specifies approximate a single value of multiplication factor for all multi stories and multi bay frame structures. However, from observed results these values are different for multi- storey and bay frames.
- The value of the behavior factor recommended by the EC8 is 12% smaller than an average values of calculated behavior factor in studied frames.
- For the sample frames considered with increased axial load ratio, the number of redundancy factor increased. So, the impact of gravitational loads was seen to be more important in the design of beams for low-rise RC-frames.
- The major intention of behaviour factor is to utilize the inelastic capacity of the structure. Designing the building for a significantly lower base shear than expected will inevitably lead to inelasticity but in an uncontrolled manner; key components of inelastic behavior such as story drift ratios, overall displacement and plastic rotations will be unknown.

5.2 Recommendation

Recommendations for future studies can be in the following area:

- In this work the behavior factor value obtained from non-linear static analysis, but the behavior factor of the buildings determines by using non-linear time history Analysis.
- This study used only planar frame structures to assess the behavior factor. Furthermore, it can evaluate by considering the effect of infill wall.
- For this study all models were fixed. Effect of soil structure interaction could also be a case to be studied on behavior factor.
- The behavior factor will be determined from global analysis by considering the effect of p- delta.

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APPENDIX A: USER DEFINED HINGES PROPERTIES

Modeling Parameters and Acceptance Criteria for Nonlinear Static Procedures (for User Defined Hinges)

The modeling parameters of frame elements used in Nonlinear Static analysis (Pushover) are all properties of the plastic hinges or plastic zones. These parameters can be determined from ATC-40. According to ATC-40, the properties of sections which can be transferred to plastic hinges depend on the relationship between Force-Displacement or Moment – Rotation. The general relationship can be defined as follows:

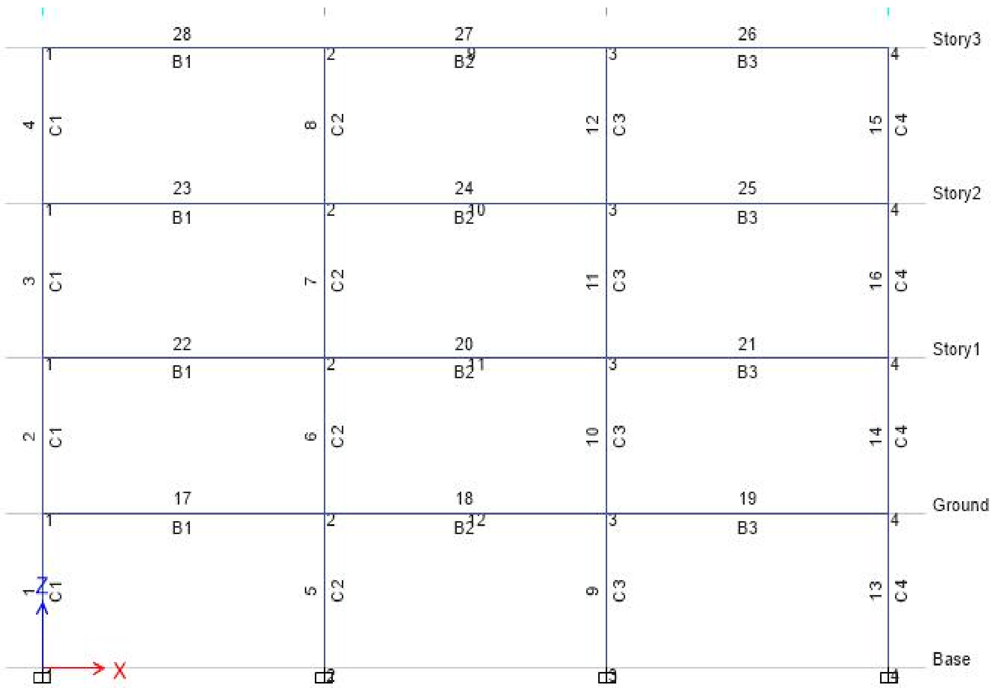

Figure Appendix 1 Sample frame for determine hinge properties

Table Appendix 1 Properties of the plastic hinge for beam section

Beam span	Position of plastic hinge	ρ of top reinforcement area	ρ of bottom reinforcement area	Balanced normalized reinforcement area	$\frac{p_{top} - p_{bot}}{p_{bal}}$	$\frac{p_{bot} - p_{top}}{p_{bal}}$
17(B1)	Left	0.011	0.005	0.02107	0.284	< 0
17(B1)	Right	0.011	0.005	0.02107	0.284	< 0
18(B2)	Left	0.011	0.005	0.02107	0.284	< 0
18(B2)	Right	0.011	0.005	0.02107	0.284	< 0
19(B3)	Left	0.011	0.005	0.02107	0.284	< 0
19(B3)	Right	0.011	0.005	0.02107	0.284	< 0

Table Appendix 2:Modeling Parameters for Negative Plastic Moment (According to FEMA 356 – Table 6.7 – Beams controlled by Flexure)

Beam span	Position of plastic hinge	$\frac{p_{top} - p_{bot}}{p_{bal}}$	Transverse Reinforced	$\frac{V}{b_w d\sqrt{f'_c}}$	Performance Level		
					Primary		
					IO	LS	CP
17	Left	0.285	C	< 3	0.005	0.02	0.025
17	Right	0.284	C	< 3	0.005	0.01	0.025
18	Left	0.284	C	< 3	0.005	0.01	0.025
18	Right	0.284	C	< 3	0.005	0.01	0.025
19	Left	0.284	C	< 3	0.005	0.01	0.025
19	Left	0.284	C	< 3	0.005	0.01	0.025

Table Appendix 3:Properties of the plastic hinge for column section(C2-Axis)

Column of floor	Position of plastic hinge	N-axial force(kN) (Axis-B)	V-shear force(kN)	Gross area of Column Ag(mm2)	$\frac{N}{A_g f_C^{'}}$	$\frac{V}{b_w d\sqrt{f_c^{'}}}$
G	Bottom	647.92	26.34	360000	0.1078	< 3
G	Top	618.66	47.66	360000	0.1029	< 3
1	Bottom	464.85	35.88	360000	0.077	< 3
1	Top	436.63	51..85	360000	0.073	< 3
2	Bottom	282.39	31.80	360000	0.047	< 3
2	Top	255.60	42.98	360000	0.0425	< 3
3	Bottom	100.75	26.50	360000	0.0167	< 3
3	Top	73.74	29.39	360000	0.0122	< 3

Table Appendix 4:Modeling Parameters for Plastic Moment (According to FEMA 356 – Table 6.8 – Columns controlled by Flexure)

Column of floor	Position of plastic hinge	$\frac{N}{A_g f_C^{'}}$	Transverse Reinforced	$\frac{V}{b_w d \sqrt{f_c^{'}}}$	Performance Level		
					Primary		
					IO	LS	CP
G	Bottom	0.1078	C	< 3	0.005	0.01	0.02
G	Top	0.1029	C	< 3	0.005	0.01	0.02
1	Bottom	0.077	C	< 3	0.005	0.01	0.02
1	Top	0.073	C	< 3	0.005	0.01	0.02
2	Bottom	0.047	C	< 3	0.005	0.01	0.02
2	Top	0.0425	C	< 3	0.005	0.01	0.02
3	Bottom	0.0167	C	< 3	0.005	0.01	0.02
3	Top	0.0122	C	< 3	0.005	0.01	0.02

Calculations scale factor for moment and rotation

Table Appendix 5:Hinge properties for typical axis

Column of floor	Position of plastic hinge	N-axial force(kN) for (Axis-B)	Ultimate moment (kNm)	Yield moment (kNm	Yield curvature	Ultimate curvature
G	Bottom	647.92	494.02	408.46	0.00474	0.045
G	Top	618.66	478.22	399.65	0.0047	0.047
1	Bottom	464.85	439.32	358.36	0.00457	0.0501
1	Top	436.63	432.65	364.25	0.00457	0.057
2	Bottom	282.39	392.12	321.06	0.00442	0.0585
2	Top	255.60	389.05	313.002	0.00437	0.062
3	Bottom	100.75	365.35	288.32	0.00427	0.0651
3	Top	73.74	358.23	268.12	0.00422	0.0714

Table Appendix 6:Hinge properties of scale factor of rotation and moment

Column of floor	Position of plastic hinge	Plastic hinge length(mm)	Scale factor at point C	
			Moment(Mu/My)	Rotation **(Lp*φu)**
G	Bottom	275	1.20	0.0124
G	Top	275	1.19	0.013
1	Bottom	275	1.22	0.014
1	Top	275	1.187	0.0156
2	Bottom	275	1.22	0.016
2	Top	275	1.24	0.017
3	Bottom	275	1.26	0.018
3	Top	275	1.33	0.019

APPENDIX B: PERFORMANCE LIMITS CALCULATION

The program displays the pushover and capacity spectrum curves where their intersection defines the performance point. Table below gives the results of the pushover method where for each step a point on the pushover curve of base shear vs. displacement is defined and the total number of plastic hinges in each step and the distribution of this number between performance levels are also listed.

Table Appendix 7 Etabs Result hinge properties state

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	-3.898E-05	0	98	0	0	0	0	98	0	0	0	98
1	32.58	480.975	97	1	0	0	0	98	0	0	0	98
2	45.114	621.596	84	14	0	0	0	98	0	0	0	98
3	87.349	811.794	71	27	0	0	0	81	17	0	0	98
4	88.347	814.357	70	28	0	0	0	81	17	0	0	98
5	202.736	962.569	64	33	1	0	0	70	12	8	8	98
6	202.746	938.375	64	33	0	1	0	70	12	8	8	98
7	203.594	943.019	64	33	0	1	0	70	12	8	8	98
8	206.013	946.85	64	32	1	1	0	69	13	8	8	98
9	206.023	924.168	64	32	0	2	0	69	13	8	8	98
10	209.792	938.404	64	31	1	2	0	69	13	7	9	98
11	209.801	889.251	64	30	0	4	0	69	13	7	9	98
12	212.454	907.19	64	30	0	4	0	69	13	5	11	98
13	213.567	910.446	64	29	1	4	0	69	13	5	11	98
14	191.123	589.46	64	29	1	4	0	69	13	5	11	98

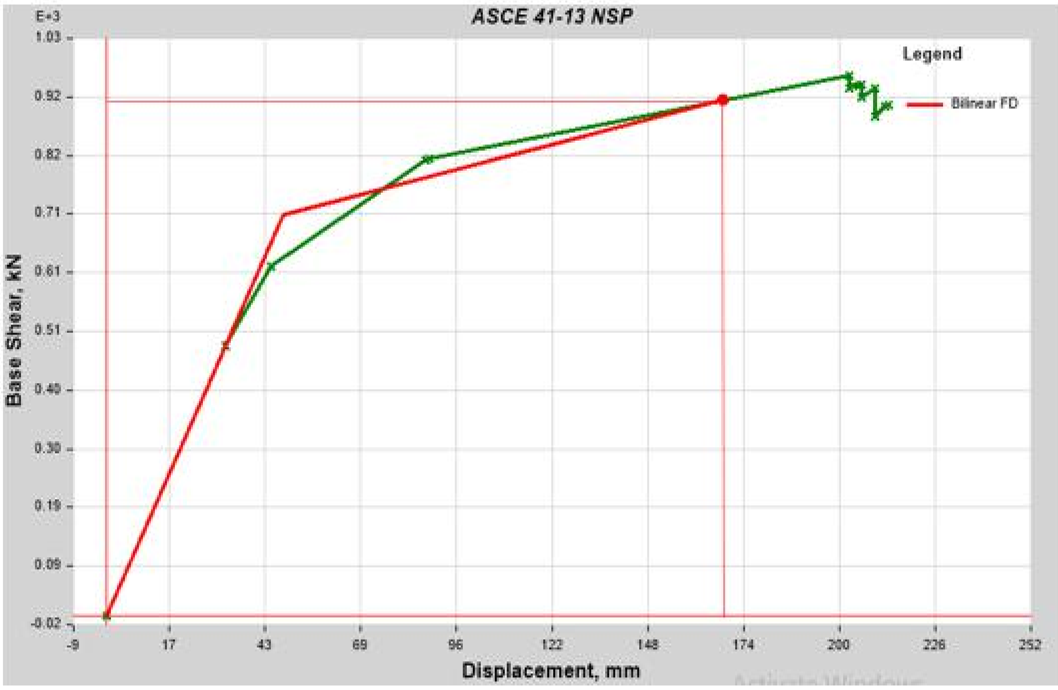

Figure Appendix 2 Capacity curve

The performance point occurred at step numbers between 4 and 5 at a displacement of 168mm and a base shear 917.68 kN. Referring to Table Appendix 7, the table shows that for each step of pushing the number of plastic hinges that occurred in members increases for each performance level till total collapse of structure. This can also be observed visually in the deformed shapes of Figure Appendix 2. In addition, deformation limits can be checked at the performance point level as follows: Total displacement at the performance point = 168.32 mm Total height of structure = 21 m = 21000 mm Ratio of performance point displacement / total height = 168.32/21000 = 0.008 Referring to Table Appendix 7, drift limitations are met for the immediate occupancy performance level.

Table Appendix 8 Deformation limits

Performance level				
Inter storey limit	Immediate occupancy	Damage Control	Life safety	Structural Stability
Maximum Total drift	0.01	0.01-0.02	0.02	$0.33V_i/P_i$

APPENDEX C: SELECTION OF DUCTILITY CLASS FOR SAMPLE MODEL

The chosen ductility class for design is “DCM”. So, designing, dimensioning and detailing must ensure a ductile behavior of the elements meaning that ductile modes of failure should precede failure modes with sufficient reliability. The plastic hinges which are developed in response to the seismic excitation must be able to dissipate a medium amount of energy in a stable manner.

Material checks:

Concrete:

- In accordance to 5.4.1.1 – EC8 [1], for ductility class “DCM” the use of concrete class which is lower than C16/20 is not allowed in primary seismic elements. So, by choosing the concrete class C25/30 for all models. The condition is met.

Flexural reinforcement steel:

- In accordance to 5.4.1.1 – EC8 [1], only ribbed bars are allowed as reinforcing steel in critical sections of primary seismic elements. The reinforcing steel class S400, the high ductility steel that satisfies the additional requirements in critical regions concerned in table C.1, annex C – EC2 [2], is chosen.

Design and detailing requirements of EC8 – Primary Beams

Specific measures for the flexural reinforcement.

• Min/max reinforcing steel - In accordance to 5.4.3.1.2

– EC8 [1], minimum tension reinforcement ratio shall not exceed the value:

$$\rightarrow \rho_{\min} = 0.5\frac{f_{ctm}}{f_{yk}} = 0.5*2.2/400=0.00275$$

→ The reinforcement content is satisfactory.

$$\rightarrow \rho_{maax} = \rho^{'} + \frac{0.0018}{\mu_{\phi}\varepsilon_{sy,d}} * \frac{f_{cd}}{f_{yd}} = 0.02311$$

- According to 5.4.3.1.2 – EC8 [1], within the critical regions, the tension reinforcement ratio shall not exceed the value below:

where, $\mu_{\phi} = 1 + 2(q_o - 1)\frac{T_c}{T}$ = 6.104

$$\varepsilon_{ys,d} = \frac{f_{yd}}{E_s} = 2*10E\text{-}3$$

So, $\rho_{\max}$ =0.02311

→ρ_{max} = 0.02311>0.01759>0.00275→ The reinforcement content is satisfactory.

- Critical region length (EN1998-1-1,cl.5.4.3.1.2(2)) = $2h_w$ = 1000mm

• **Longitudinal bar diameters:**

- According to 5.6.2.2 – EC8 [1], to prevent the bond failure, the diameter of longitudinal bars of the beams is limited as the following conditions:

Those equations developed in order to ensure that the area is sufficient joint region through the beam-column joint where are existing high rate of change of reinforcement stress.

For interior beam – column joints:

$$\rightarrow \frac{d_{bL}}{h_c} \leq \frac{7.5 * f_{ctm}}{\gamma_{Rd} * f_{yd}} * \frac{1 + 0.8\nu_d}{1 + 0.75 * K_D * \frac{\rho'}{\rho_{\max}}} = 0.045$$

→ The chosen reinforcement is satisfactory.

For exterior beam – column joints:

$\rightarrow \frac{d_{bL}}{h_c} \leq 4.0 * \frac{f_{ctm}}{f_{yd}}(1 + 0.8\nu_d)$ =0.042 → The chosen reinforcement is satisfactory.

Where: h_c: – is the width of the column parallel to the bars, so h_c = 600mm.

f_{ctm}: is the mean value of the tensile strength of concrete → f_{ctm} = 2.2N/mm2.

f_{yd} = 347.8 MPa.

ν_d – is the normalized design axial force in column, taken with its minimum value for seismic design situation.

$$\nu_d = \frac{N_{sd}}{A_c * f_{cd}}$$

Where,

N_{Ed} = -648568N; f_{cd} = 16.67MPa; A_c = 600x600=360000mm^2.

$$\rightarrow \nu_d = \frac{N_{sd}}{A_c * f_{cd}} = 0.139$$

→k_D – is the factor reflecting the ductility class equal to 2/3 for DCM.

→ρ' – compression steel ratio → ρ' = 0.01759

→ρ_{max} = 0.02311

Design and detailing requirements of EC8 – Primary Columns:

Specific measures for the flexural reinforcement:

- According to 5.4.3.2.1(3) – EC8 [1], in primary seismic columns the value of the normalized axial force ν_d shall not exceed the value of 0.65.

$$\nu_d = \frac{N_{sd}}{A_c * f_{cd}} \leq 0.65 = (581.84*1000)/\ (16.67*500*500) = 0.139 < 0.65.$$

So, the condition is met!

- According to 5.4.3.2.2(1P) – EC8 [1], The total longitudinal reinforcement ratio shall be not less than 0,01 and not more than 0,04. → The condition is met.

- According to (EN1998-1-1, cl.5.4.3.2.2(2) P) – EC8 [1], at least one intermediate bar shall be provided between corner bars along each column side, to ensure the integrity of the beam column joints. → The condition is met.
- In accordance to 5.4.3.2.2 – EC8 [1], in the critical sections of the primary seismic columns the diameter of hoops or cross-tie is at least of 6mm. The diameter of the hoops is of 8mm, so this condition is satisfactory.
- In accordance to 5.4.3.2.2 (11)– EC8 [1], the maximum spacing of the hoops:

s_{max} = min (b0/2; 175; $8d_{bL}$) = min (412/2;175;8*20) =160mm;

s = 120mm, so this condition is met.

- d_{bL} ≥{8mm}.
- Maximum spacing between restrained bars (EN1998-1-1,5.4.3.2.2(11b))

  ≤{200mm}.

CHECK FOR RESISTANCE BETWEEN COLUMNS AND BEAMS

- According to 4.4.2.3 (3) – EC8 [1], in multi-storey buildings formation of a soft storey plastic mechanism shall be prevented, as such a mechanism might entail excessive local ductility demands in the columns of the soft storey.
- According to 4.4.2.3(4) – EC8 [1], the following condition should be satisfied at all joints of primary seismic beams and primary seismic columns:

$$\sum M_{RC} \geq 1.3 * \sum M_{Rb}$$

Where:

ΣM_{Rc} – is the sum of the design values of the moment resistance of the columns framing to the joints.

ΣM_{Rb} – is the sum of the design values of the moment resistance of the beams framing to the joints.

Table Appendix 9:moment and axial forces from analysis for column elements

Beam-Column Joint	$\sum M_{RC}$	$\sum M_{Rb}$	$\frac{\sum M_{RC}}{\sum M_{Rb}} \geq 1.3$	Remark
1	354	51.9	4.88	Ok!
2	394	261.9	1.51	Ok!
3	394	228.52	1.72	Ok!
4	354	-204	1.74	Ok!

- It is just necessary to check for the first floor because the reinforcing areas of columns are not changed over their length and the reinforcing areas of the first-floor beams are greater than the other beams.

- ΣM_{Rc} = 236*2=472KNm
- 1.3* ΣM_{Rb} = 1.3* (-185.5 + 137.32) = 1.3*(185.5 +137.32) = 419.66KNm

→ ΣM_{Rc} = 472KNm > 1.3* ΣM_{Rb} = 419.66 KNm. So, the condition is met.

Local Ductility

The length of critical regions l_{cr}:

- According to 5.4.3.2.2 (4) – EC8 [1], l_{cr} may be computed as following expression: l_{cr} = max {h_c; l_{cl} / 6; 0,45} = max {0.5, 3/6;0.45} = 0.45m
- According to 5.4.3.2.2 (6P and 7P) – EC8 [1], The amount of hoops at the critical regions of the base of primary seismic columns should be satisfy be this equation:

$$\alpha\omega_{wd} \geq 30^* \mu_\phi {}^* \nu_d {}^* \varepsilon_{sy,d} {}^* \frac{b_c}{h_c} - 0.035$$

Where:

- ω_{wd} – is the mechanical volumetric ratio of confining hoops within the critical regions. So, ω_{wd} is calculated as follows:

- α - is the confinement effectiveness factor, equal to $\alpha = \alpha_n \alpha_s$, with:

- $\alpha = \alpha_n \alpha_s$ = 0.849*1= 0.849

ω_{wd} – = (82,874.62/ 20,369,280.00)*(347.8/16.7) = 0.085

- μ_φ - is the required value of the curvature ductility factor, μ_φ = 6.104
- $\alpha_n = 1 - \sum b_i^2 / 6 b_o h_o$ = 0.849
- $\alpha_s = \dfrac{1 - \dfrac{s}{2 * b_o}}{1 - \dfrac{s}{2 * h_o}}$ = 1

➢ $\alpha\omega_{wd} \geq 30*\mu_{\phi}*\nu_{d}*\varepsilon_{sy,d}*\frac{b_c}{h_c}-0.035$

$$= 0.072 \geq 30*6.104*0.139*0.00217*\frac{500}{500}-0.035 = 0.020$$

- So, the condition is met.

Hinge formation with increasing the storey height

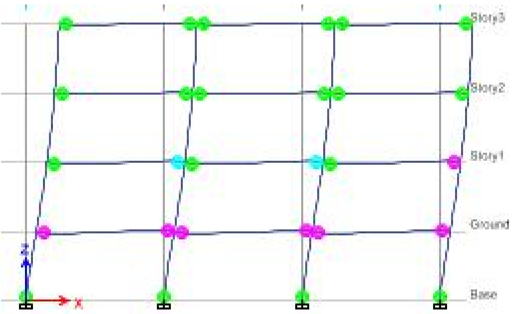

Figure Appendix 3 Hinge Formation for G+3

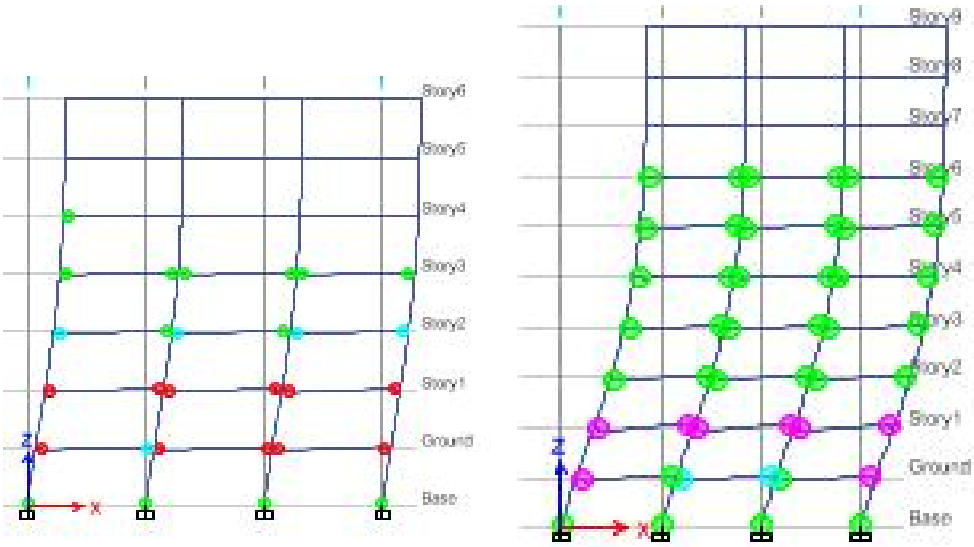

Figure Appendix 4:Hinge Formation for G+6 and G+9