ASSESSMENT OF RESPONSE REDUCTION FACTOR OF RC BUILDINGS IN KATHMANDU VALLEY USING NON-LINEAR PUSHOVER ANALYSIS

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By

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In

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Declaration

I, HEM CHANDRA CHAULAGAIN, hereby certify that the work which is being presented in this dissertation entitled **"ASSESSMENT OF RESPONSE REDUCTION FACTOR OF RC BUILDINGS IN KATHMANDU VALLEY USING NON-LINEAR PUSHOVER ANALYSIS"** in partial fulfillment of requirements for the award of degree of Master of Engineering in Earthquake submitted to the **POST GRADUATE DEPARTMENT OF EARTHQUAKE ENGINEERING AT KHWOPA ENGINEERING COLLEGE, BHAKTAPUR** under **PURBANCHAL UNIVERSITY, NEPAL** is an authentic record of my own work. The matter presented in this dissertation has not been submitted by me in any other University / Institute for the award of M.E in Earthquake.

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This is to certify that the thesis entitled **"ASSESSMENT OF RESPONSE REDUCTION FACTOR OF RC BUILDINGS IN KATHMANDU VALLEY USING NON-LINEAR PUSHOVER ANALYSIS "** by **"HEM CHANDRA CHAULAGAIN"** submitted to the **POST GRADUATE DEPARTMENT OF EARTHQUAKE ENGINEERING** at **KHWOPA ENGINEERING COLLEGE, BHAKTAPUR** under **PURBANCHAL UNIVERSITY, NEPAL** towards partial fulfillment of the requirements for the award of the Master of Engineering in Earthquake is a bonafide record of the work carried out by him/her under my /our supervision and guidance.

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Abstract

This study addresses the issue of response reduction factor which is used in modern codes to scale down the elastic response of the structure. The level of ductility and overstrength of RC buildings in Kathmandu valley are investigated. The ductility and overstrength factors are estimated by analyzing the buildings using non-linear pushover analysis for 12 engineered designed RC buildings of various characteristics representing a wide range of RC buildings in Kathmandu valley. Finally, the response reduction factor of RC building in Kathmandu valley is evaluated by using the relation of ductility and overstrength factor.

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LIST OF ABBREVIATIONS

Q1= Lateral force act on cg of first floor

Q2= Lateral force act on cg of second floor

Q3= Lateral force act on cg of third floor

Q4= Lateral force act on cg of fourth floor

Q5= Lateral force act on cg of fifth floor

GF = Ground floor

FF = First floor

SF = Second floor

TF = Third floor

TF = Top floor

[K]= Stiffness matrix

[M]= Mass matrix

Δe = Elastic displacement

Δu = Ultimate displacement

Δy = Yield displacement

Ah = Design horizontal acceleration spectrum value

A_k = Design horizontal acceleration spectrum value

As' = Area of compression steel

As = Area of tension steel

ATC = Applied Technology Council

b = Width of section

C0= Modification factor to relate spectral displacement

C1= Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C2= Modification factor to represent the effect of hysteresis shape on the maximum displacement response.

C3= Modification factor to represent increased displacements due to second-order effects.

d' = Distance from extreme compression fibre to centroid of compression steel.

d = Effective depth of member.

Ec = Modulus of elasticity of concrete

EQX = Earthquake push on X-direction

EQY = Earthquake push on y-direction

Es = Modulus of elasticity of steel

f = Load vector

fc' = Stress in concrete

FEMA = Federal Emergency Management Agency

fr = Modulus of rupture

fy = Yield strength of steel

h_i = Height of the floor i, measured from base

I = Building occupancy importance

I = Moment of inertia

jd = Distance of compressive forces in the steel and concrete

K = Horizontal force factor

k = Neutral axis depth factor

Ke = Effective lateral stiffness of the buildings in direction under consideration.

Ki = Elastic lateral stiffness of the building in the direction under consideration.

Mcrack = cracking moment

n = Number of storey's in the building n = number of stories

Pk = Modal participation factor

Q_i = Design lateral force at floor i.

R= Response reduction factor

R_R = Redundancy factor

Ω = Overstrength factor

μ = Displacement ductility factor

Rμ = Ductility reduction factor

Rξ = Damping reduction factor.

S = Soil profile coefficient

S1 = Site coefficient

Sa= Spectral acceleration

T = Fundamental period of the building.

Te = Effective fundamental period of the structure.

Te = Elastic lateral stiffness of the building in the direction consideration

Ti= Elastic fundamental period (in seconds)

V_b = Base shear

V_d = Design base shear.

Vu = Ultimate shear force

W = Total gravity load

W_i = Seismic weight of the floor i,

y = Depth of neutral axis

z = Distance of critical section to the point of contraflexure

Z = Zone factor

α = Ratio of post yield stiffness to effective elastic stiffness δt = target displacement

μd = Displacement ductility demand

μs = Displacement ductility supply

$\phi crack$ = Curvature before cracking

ϕy = Curvature at first yield of the tension steel.

Ω = Overstrength factor

$\Omega default$ = Overstrength factor when default hinge was used.

$\Omega user$ = Overstrength factor when user defined hinge was used.

ϕ_{ik} = Mode shape coefficient at floor i in mode k,

T_1 = Predominant period of the ground motion.

lp = Plastic hinge length

ϕult = Ultimate curvature

θp = Plastic rotation

L= Member lengt

ϕy= Curvature at yield

θy= Rotation at yield

1 INTRODUCTION

1.1 Background of the Study

1.1.1 Seismic Hazard of Nepal and Kathmandu Valley

Nepal and the Himalayan range that forms its northern border with China were formed as a result of the collision of the Indian plate with the Tibetans plate about 50 million years ago. This collision still continuous which results in subduction of Indian plate below the Tibetan plate makes Nepal and the entire Himalayan range seismically active.

Nepal lies in a very high seismic hazard zone. Global seismic hazard map shows Nepal in Zone 4 as possible shaking of MMI IX or above with 10% probability of exceedence in 50 years [46]. Probabilistic seismic hazard mapping of Nepal conducted during building code development in Nepal has shown PGA of 0.36 g in Kathmandu Valley in 500 years return period [49]. In summary, Nepal including Kathmandu valley lies in a very high seismic hazard zone.

Looking at the urbanization of Kathmandu valley now, if similar earthquake as that of 1934 A.D was to occur today, the scenario would be devastating, and the fatalities would be very high. For that earthquake scenario, Japan International Cooperation Agency [46] estimated up to 59000 houses destroyed, 18000 deaths and 59000 seriously injured. Another study carried out in the frame work of the Kathmandu valley Earthquake Risk Management Project [47] estimates a total of 40000 deaths, 95000 injuries and 600000 or more homeless.

Seismic hazard map of Nepal and Eastern Himalaya (figure 1.1) also gives justify that Kathmandu valley is in highly vulnerable due to earthquake. So, it is urgent need to assess the non-linear behaviour of RC buildings in Kathmandu valley during earthquake.

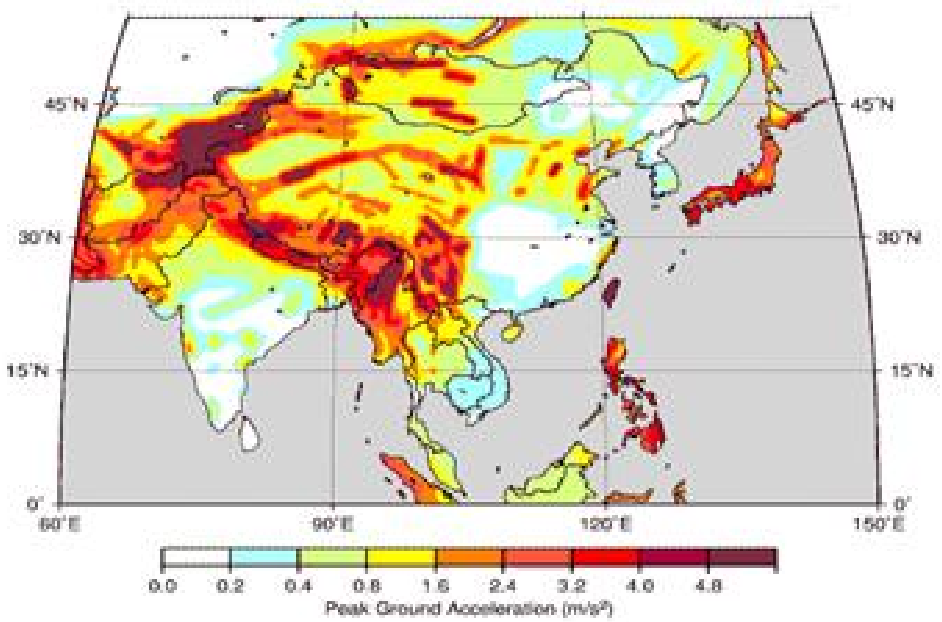


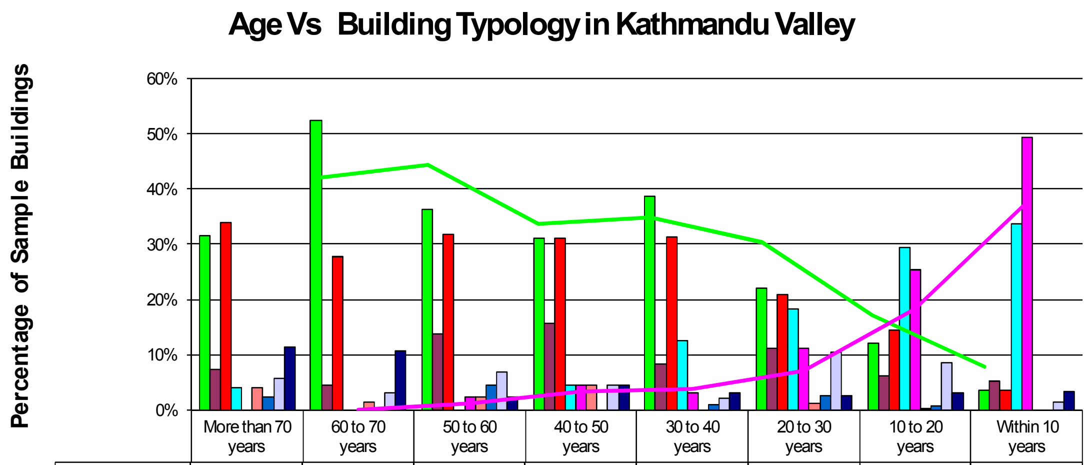
Figure 1.1. Seismic hazard map of Nepal and Eastern Himalayan

Source: Global Seismic Hazard Assessment Program in Continental Asia (GSHAP). International Lithosphere Program (ILP), 1999

1.1.2 Trend of Building Design and Construction

Most of the casualty from earthquakes is due to collapse of buildings. More than 80 % of the total people killed in developing countries during earthquakes are collapse of buildings.

But now-a-days, the trends of RC building construction are rapidly increased [46]. Figure 1.2 shows the trend of building construction in Kathmandu valley and clearly shows that the trends of RC buildings construction is rapidly increasing from the past some years.



	More than 70 years	60 to 70 years	50 to 60 years	40 to 50 years	30 to 40 years	20 to 30 years	10 to 20 years	Within 10 years
Adobe	31%	52%	36%	31%	39%	22%	12%	4%
Stone	7%	5%	14%	16%	8%	11%	6%	5%
BM	34%	28%	32%	31%	31%	21%	14%	4%
BC	4%	0%	0%	4%	13%	18%	29%	34%
RC	0%	0%	2%	4%	3%	11%	25%	49%
Stone and Adobe	4%	2%	2%	4%	0%	1%	0%	0%
Stone and BM	2%	0%	5%	0%	1%	3%	1%	0%
BM and BC	6%	3%	7%	4%	2%	10%	9%	1%
Others	11%	11%	2%	4%	3%	3%	3%	3%

Age

Figure 1.2. Trend of Building Construction in Kathmandu Valley

Source: "The Study on Earthquake Disaster Mitigation in the Kathmandu Valley Kingdom of Nepal". Japan International Cooperation Agency (JICA) and Ministry of Home Affairs, His Majesty's Government of Nepal, 2002.

Now-a-days the number of engineer designed buildings is also increasing. The design of RC building mainly based on seismic coefficient method which gives approximate design base shear. The value of Response Reduction Factor (R) = 5 is taken in all the times, considering the building is special moment resisting frame with expectation of very high ductility. To meet these expected very high ductility capacity of structural members has to go very high inelastic deformation. The capacity governs the structural behaviour and damageability of buildings during earthquake ground motions. Sometimes Response spectrum method was also used to determine the design base shear of the structure but it is also far to address the actual base shear which is generated during earthquake.

Although, the current practice for earthquake resistant design is mainly governed by the principles of force-based seismic design, there have been significant attempts to incorporate the concepts of deformation- based seismic design and evaluation into earthquake engineering practice. In general, the study of the inelastic seismic response of buildings is not only useful to improve the guidelines and code provisions for minimizing the potential damage of buildings, but also important to provide economical design by making use of the reserved strength of the buildings as it experiences inelastic deformations. In recent seismic guidelines and codes in Europe and USA, the inelastic response of the building are determined using nonlinear static methods of analysis known as the pushover analysis methods but such trends does not established in South Asian region [24].

1.1.3 Research need

Past evidence had shown that the structures in Kathmandu valley are vulnerable due to earthquake [46]. The RC building construction trends increases day by day. Buildings are designed based on linear elastic methods which are considered only elastic range. Assumption was made that Non-linearity of the structure is incorporated by response reduction factor R. The effect of ductility, over strength, load path and column beam capacity ratio on performance of structure is essential to study through non linear analysis.

Table 7 of IS 1893(Part 1): 2002 gives the value of Response Reduction Factor, R, for lateral load resisting system. IS 13920-1993 gives the ductility requirement for earthquake resistant design. For special moment resisting RC frame structures (SMRF) R value is given as 5. While designing the RC structure R value is taken as 5 in all situations. Code does not explain all necessary circumstances of SMRF. Thus it is essential to study the real behaviours of RC buildings in Kathmandu valley through non-linear analysis and suggest the circumstance which affects the response of the structure.

2 OBJECTIVES, APPROACHES AND METHODOLOGIES

2.1 Objective of the Study

The main objective of this research is to verify the designed R factor of most common engineer designed RC buildings in Kathmandu Valley through comparing the assumed R factor during design to actual R factor obtained from non-linear analysis. The specific objectives of the study are to:

- Select most common engineer designed RC buildings and study different parameters to consider for analysis
- Study different method of non-linear analysis to calculate R factor and select the appropriate method to calculate the R factor
- Conduct non-linear analysis and calculate R factor of more than 10 buildings
- Compare the calculated R factor with the assumed one and also with different parameters of the building
- Evaluate ductility reduction factor of study buildings
- Evaluate Overstrength factor of study buildings
- Check effect of overstrength factor on the ductility factor
- Check effect of load path on response reduction factor
- Check beam column capacity ratio on building ductility
- Check combined effect of beam column capacity ratio and load path on response reduction factor

2.2 Study Approaches

This study will be a combination of both the field work and desk study for analysis. However, the field work is limited to selection of typical RC buildings for analysis. More work is on desk study as it is more an analytical study.

Review of R factor calculation methods and also review of different non-linear analysis methods for calculation of R factor is one of the major parts of the study.

2.3 Research Methodology

To meet the objectives of the study a methodology has been developed and given in Figure 1.1 below. The research will be started from review and study of RC buildings and lastly conclusions are drawn. The brief description of each steps are:

2.3.1 Review and study of engineered design buildings in Kathmandu valley

The trends of construction RC buildings, designing criteria, behaviours of structure during earthquake were firstly studied [chapter 1]. The secondary data are used in this study to fulfil the study needs. Apart from this, some primary data are also collected. The sources of secondary data are:

- Journal and newspapers
- Published and unpublished articles
- Past studies made in this field
- Data from the analysis results from structural analysis program.

2.3.2 Selection of sample building for response reduction factor (R) study

Sampling was done randomly which represent the nature of deigning trends and construction practices in different localities. The buildings of Kathmandu valley which has Plan less than 2000 sq.ft and up to 5 stories are taken as the population of the study. The site soil condition is taken as medium, clay.

2.3.3 Review of response reduction factor methodologies

Response reduction factor is used to scale down the elastic force of the structure. Elastic force generated during the earthquake is divided by force reduction factor(2R) to obtain design base shear which is used for designing of structure. Ductility factor, over strength factor and redundancy are the key factors for the formulation of response reduction factor. Different methodologies and its formulation is presented in chapter [3]

2.3.4 Review of different pushover methods for conducting non-linear pushover analysis

Different procedures used for the analysis of non-linear behaviour of the structure were studied and appropriate method for non-linear analysis of study buildings was chosen. The detailed description of the pushover and modeling of structure is given in chapter [4].

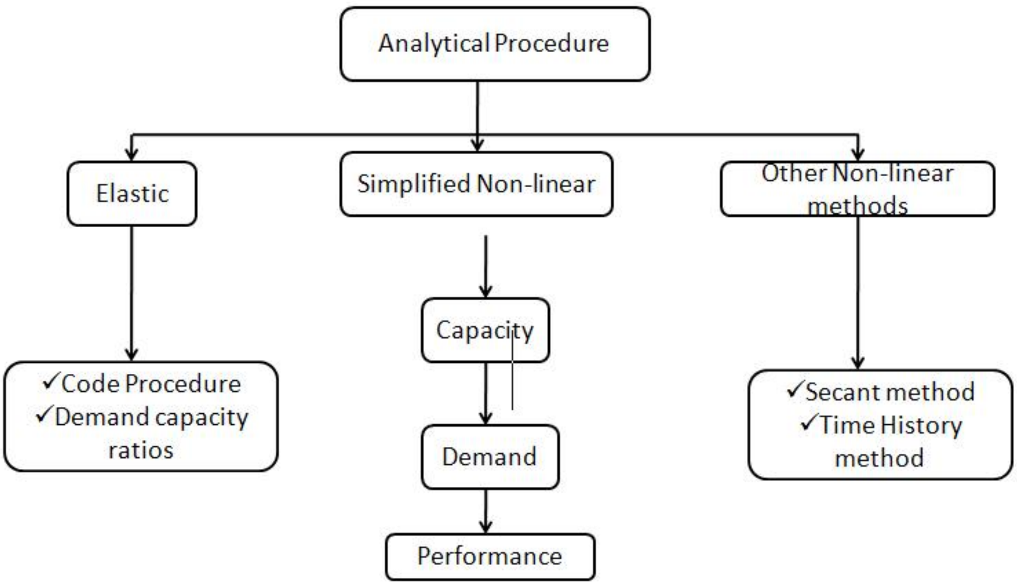


Figure 2.1. Analytical procedure for non-linear analysis

2.3.5 Analysis and interpretation of results

After pushover analysis, capacity curve of the structure is obtained. Capacity curve of the structure gives the ultimate deformation and ultimate base shear. Bilinear idealization of capacity curve gives the yield deformation. With the help of these data, over strength factor, displacement ductility, ductility reduction factor and ultimately response reduction factors are calculated. The calculations are based on the mathematical expressions explained in chapter [3] and [4]. Sample calculation of one model is given in Annex 3.1. The final results and interpolation of the results are presented in chapter [5]. Finally, conclusions were drawn based on results which is presented in chapter [6]

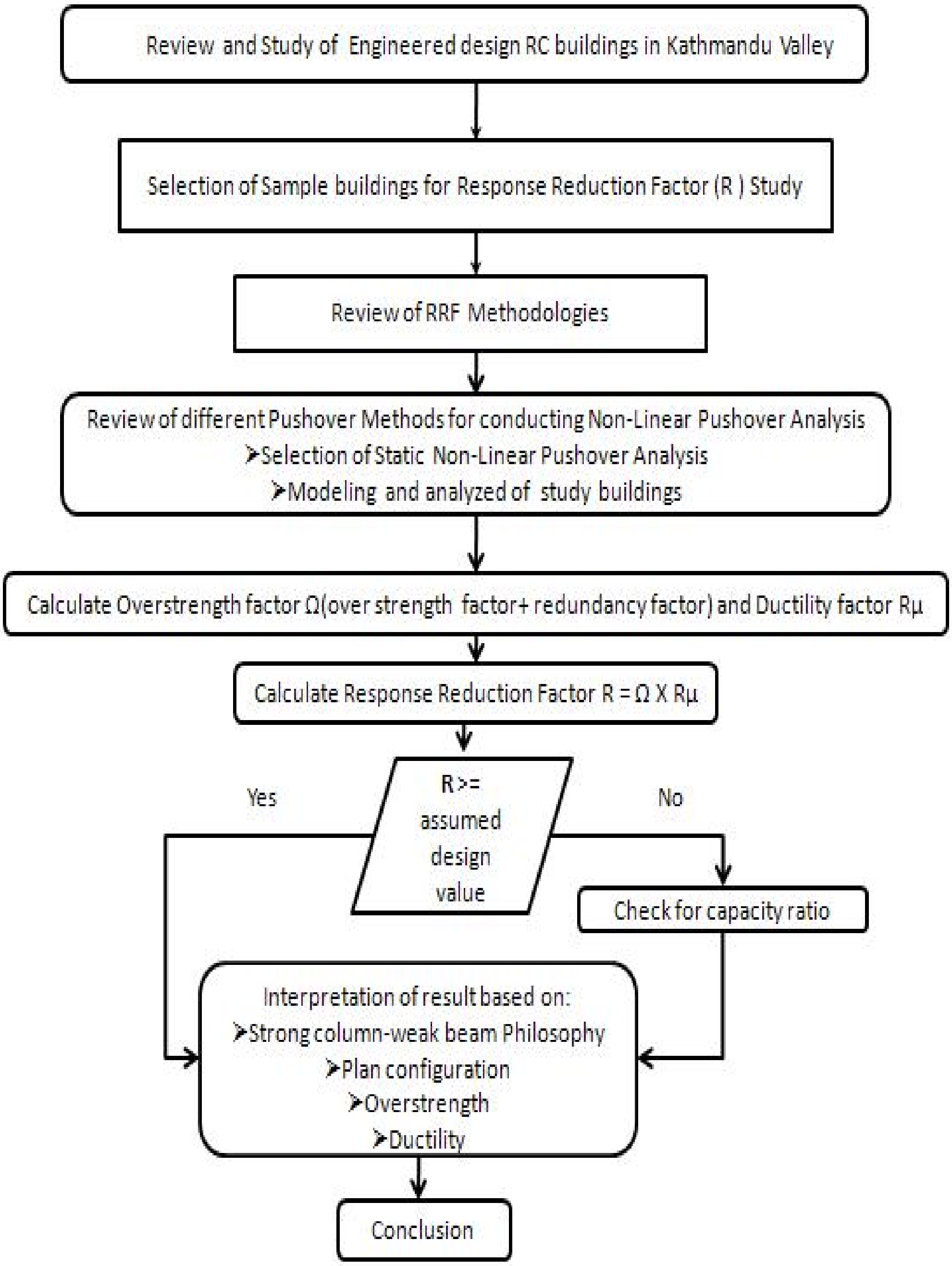


Figure 2.2. Flow Chart of Research Methodology

3 REVIEW ON CALCULATION METHODOLOGIES FOR RESPONSE REDUCTION FACTOR

3.1 Definition of Response Reduction Factor

Response reduction is used to scale down the elastic response of the structure [8]. The structure is allowed to be damaged in case of severe shaking. Hence, structure is designed for seismic force much less than what is expected under strong shaking if the structure were to remain linearly elastic.

It is simply represents the ratio of the maximum lateral force, Ve, which would develop in a structure, responding entirely linear elastic under the specified ground motion, to the lateral force, Vd, which has been designed to withstand. Response reduction factor R, is expressed by the equation:

$$R = Ve/ Vd \tag{1}$$

The factor R is an empirical response reduction factor intended to account for damping, overstrength, and the ductility inherent in the structural system at displacements great enough to surpass initial yield and approach the ultimate load displacement of the structural system [1]. The concept of a response reduction factor was based on the premise that well-detailed seismic framing systems could sustain large inelastic deformations without collapse(ductile behavior) and develop lateral strength in excess of their design strength(often termed reserve strength)[2]. R factor is first introduced in 1978[3], used to reduce the elastic shear force (Ve) obtained by elastic analysis using a 5% damped acceleration response spectrum for the purpose of calculating a design base shear(Vd). Major static analysis routines are Equivalent Lateral Force Method and Response Spectrum Method; in both procedures R factors are utilized to calculate the design base shear.

Now, the IS code provides the realistic force for elastic structure and divides those forces by (2R) [16].

$$\text{Force reduction factor (2R)} = \frac{\text{Elasticstrengthdemand}}{\text{Designstrength}} = R\mu\ \Omega \tag{2}$$

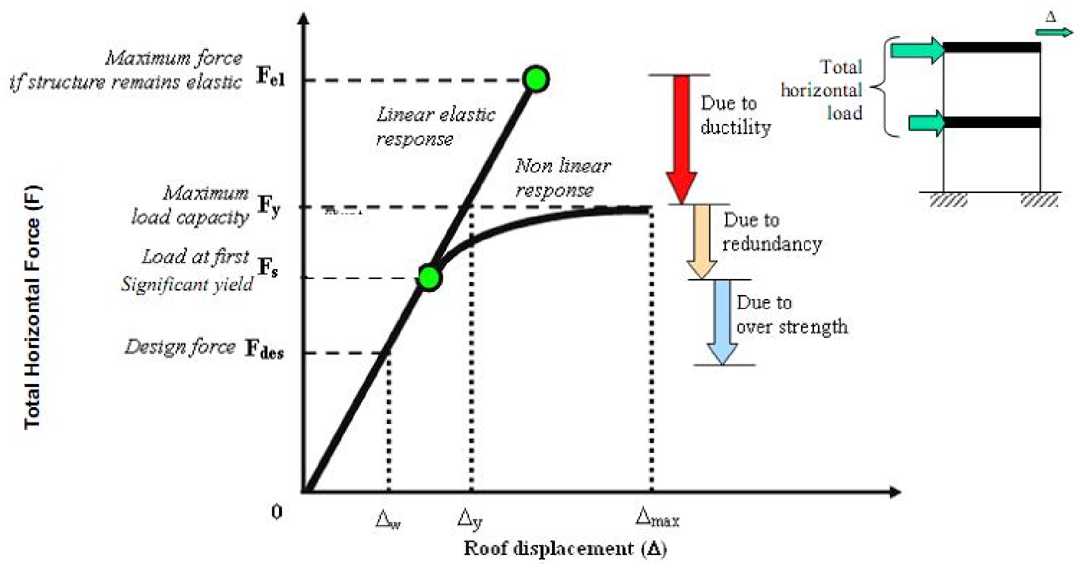


Figure 3.1. concept of response reduction factor

Source: proposed draft provisions and commentary on Indian Seismic Code IS 1893 -Part 1

3.2 Response Reduction Factor Formulation

In the mid-1980s, Berkeley [1] described R as the product of three factors that accounted for reserve strength, ductility, and added viscous damping

$$R = R_S \cdot R_\mu \cdot R_\xi \qquad (3)$$

Rs stands for overstrength and calculated to be equal to the maximum base shear force at the yield level (Vy) divided by the design base shear force (Vd).

Rμ stands for ductility factor and calculated as the base shear (Ve) for elastic response divided by the yield base shear (Vy). The damping factor was set equal to 1.0.

ATC 19 [27] splitting R into three component factors.

$$R = R_S \cdot R_\mu \cdot R_R \qquad (4)$$

Where $R\xi$ is replaced by R_R(redundancy factor). Differences in the values of the behavior factors specified in various codes for the same types of structure.

3.3 Previous Studies on calculation of Response Reduction Factors of Existing Buildings

Tinkoo Kim and Hyunhoo Choi [4]

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.

Bhavin Patel[1] and Dhara Shah[2] [23]

Formulate the key factors for seismic modification factor of RCC framed staging of elevated water tank. The analysis revealed that three major factors, called reserved strength, ductility and redundancy affect the actual value of response modification factor. Conclusion was made that the water tank which is well design by using codal procedure has the response reduction factor 4.0.

Greg Mertz[1)] and Tom Houston[2)] [6]

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 ductilities. Individual element ductilities are modified to account for either MDOF or soft story effects. These modified element ductilities are combined with the structures natural frequency and an appropriate SDOF dynamic model to develop the force reduction factor.

A. Kadid and A. Boumrkik [7]

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.

Devrim Ozhendekci, Nuri Ozhendekci and A. Zafer Ozturk [5]

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 overstrength factor, ductility factor and redundancy factor for the evaluation of R-values to the EBF systems.

$$R = R_{\Omega} * R_{\mu} * R_{R} \tag{5}$$

3.4 Overstrength Factor

3.4.1 Overstrength

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

Overstrength can be employed to reduce the forces used in the design, hence leading to more economical structures. The main sources of overstrength are [13]:

- The difference between the actual and design material strength
- Conservation of the design procedure and ductility requirements
- Load factors and multiple load cases
- Serviceability limit state provisions

- Participation of nonstructural elements
- Effect of structural elements not considered in predicting the lateral load capacity
- Minimum reinforcement and member sizes that exceed the design requirements
- Redundancy
- Actual confinement effects
- Utilizing the elastic period to obtain the design forces.

Member size or reinforcement lager than required, strain hardening in materials, Confinement of concrete, strength contribution of non-structural elements and special ductile detailing are also the sources of overstrength [24]

Overstrength factor (Ω) = apparent strength/design strength [9]

$$\Omega = V_u/V_d \tag{6}$$

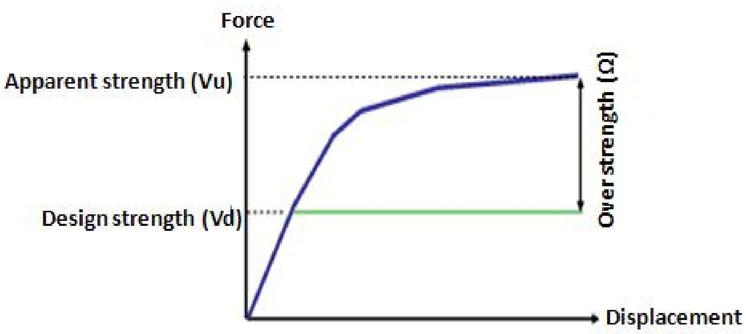


Figure 3.2. Force Displacement relationship for overstrength

3.4.2 Previous Studies on calculation of Overstrength factor of Existing Buildings

In this section, some of the previous studies about overstrength factor are reviewed

Freeman [10]

The author reported overstrength factors for 3 three storey moment resisting frames, two constructed in seismic zone 4 and one in seismic zone 3 were 1.9, 3.6, and 3.3 respectively.

Kappos [11]

In this study five R/C buildings, with one to five stories, consisting of beam, columns, and structural walls are examined and as a result overstrength factors 1.5 to 2.7 are obtained.

Lee, Cho and Ko [12]

In their study the authors investigated overstrength factors and plastic rotation demands for 5, 10, 15 storey R/C buildings designed in low and high seismic regions utilizing three dimensional pushover analysis. One of their conclusions is that the overstrength 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.

A.S Elnashai[1] and A. M. Mwafy[2] [13]

Develops the relationship between the lateral capacity, design force reduction factor, the ductility level and the overstrength factor. The lateral capacity and overstrength 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[14] and Paulay and Priestley [15] underestimate the inelastic period.

3.5 Ductility Reduction Factor

3.5.1 Terms used in ductility reduction factors

Strength, stiffness and ductility are the essential structural properties for the seismic protection.

Stiffness

If the deformation under the action of lateral forces is to be reliably quantified and subsequently controlled, designer must make a realistic estimate of the relevant property stiffness. This quantity relates loads or forces to the ensuing structural deformations. A typical non-linear relationship between induced forces or loads and displacements, describing the response of a reinforced concrete component subjected to monotonically increasing displacement. For design computations, one of the two bilinear approximations may be use where V_y defines the yield or ideal strength V_i of the member [16]. The slope of the idealized linear elastic response

$$K = V_y / \Delta_y \quad (7)$$

Figure 3.3 is used to quantify the stiffness.

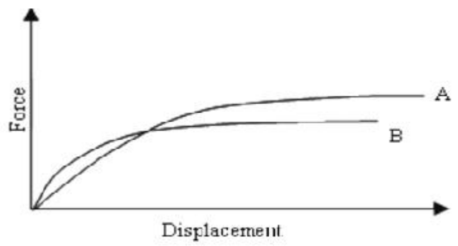


Figure 3.3. Stiffness Vs Strength relationship1

Structure A has higher strength and lower stiffness as compared to structure B.

Strength

If the structure is to be protected against damage a selected or specified seismic event, inelastic excursions during its dynamic response should be prevented.

This means that the structure must have adequate strength to resist internal actions generated during the elastic dynamic response of the structure [16].

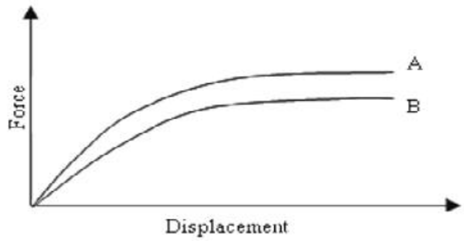


Figure 3.4. Stiffness Vs Strength relationship2

Structure A has higher strength and higher stiffness as compared to structure B.

Ductility

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. Ductile structures have been found to perform much better in comparison to brittle structures [16]. High ductility allows a structure to undergo large deformations before it collapse. Large structural ductility allows the structural to move as a mechanism under its maximum potential strength, resulting in the dissipation of large amount of energy [1].

The extent of inelastic deformation experienced by the structural system subjected to a given ground motion or a lateral loading is given by the displacement ductility ratio "μ" and it is represented by the ratio of maximum absolute displacement to its yield displacement [9].

The inelastic behavior of structure can be idealized as [9]

$$\mu = \Delta u / \Delta y \tag{7}$$

Where μ_Δ is the displacement ductility, Δu is the ultimate deformation and Δy is the yield deformation.

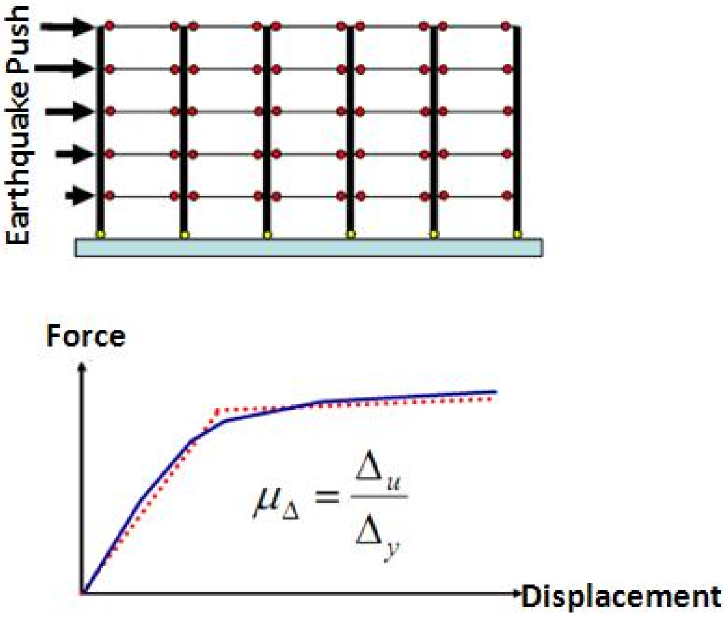


Figure 3.5. Representation of displacement ductility (source FEMA 451).

Yield deformation is obtained as follows [17]

The nonlinear force-displacement relationship between base shear and displacement of the control node shall be replaced with an idealized relationship to calculate the effective lateral stiffness, Ke, and effective yield strength Vby, of the structure shown in figure 4.8. The relation shall be bilinear, with initial slope Ke and post-yield slope γ. line segments on the idealized force-displacement curve shall be located using an iterative graphical procedure that approximately balances the area below and above the curve. The effective lateral stiffness, Ke, shall be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure. Thus, Vy is the intersection of initial and effective stiffness.

3.5.2 Previous studies on Calculation of Ductility Reduction Factors of Existing Buildings

Newmark and Hall [18]

Define the ductility factor is the ratio of maximum deformation to the yield deformation and proposed the following equations for the determination of ductility reduction factor (R_μ).

$R_\mu = 1.0$ (T < 0.03 second) (8a)

$R_\mu = \sqrt{2\mu - 1}$ (0.12 < T < 0.03 second) (9a)

$R_\mu = \mu$ (T > 1.0 second) (10a)

T. Paulay and M. J. N. Priestley [48]

Divides the time period of the structure for calculating ductility reduction factor.

$R_\mu = 1.0$ for zero-period structures (8a)

$R_\mu = \sqrt{2\mu - 1}$ for short-period structure (9a)

$R_\mu = \mu$ for long period structure (10a)

$R_\mu = 1 + (\mu - 1)\, T/0.70$ (0.70 s < T < 0.3) (11)

Miranda and Bertero [19]

Introduced the equation for reduction factor by considering 124 ground motions recorded on a wide range of soil conditions. The soil conditions were classified as rock, alluvium and very soft sites characterized by low shear wave velocity. A 5% of critical damping was assumed. The ductility factor was give by

$R_\mu = \frac{\mu - 1}{\phi} + 1$ (12)

Values of Φ are calculated by using the relations:

$\phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T}\exp(-2(\ln(T) - 0.2)^2)$ for alluvium site (13)

$\phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T}\exp(-1.5(\ln(T) - 0.6)^2)$ for rock site (14a)

$$\phi = 1+\frac{T1}{3T}-\frac{3T1}{4T}\exp\left(-3\left(\ln\left(\frac{T}{T1}\right)-0.25\right)^{2}\right) \text{ for soft site} \quad (14b)$$

Lai and Biggs [20]

In this study design inelastic response spectra were based on mean inelastic spectra computed for 20 artificial ground motions. Analyses carried on for periods equally spaced between 0.1s and 10s with 50 natural periods. The ductility reduction factors corresponding to the proposed coefficients are given by

$$R\mu = \alpha + \beta(\log T)$$

Table 3.1: α & β coefficients proposed by authors Lai & Biggs

Table 2.1: α & β coefficients proposed by authors Lai & Biggs [13]

Period Range	Coefficient	μ=2	μ=3	μ=4	μ=5
0.1 < T < 0.5	α β	1.6791 0.3291	2.2296 0.7296	2.6587 1.0587	3.1107 1.4307
0.5 < T < 0.7	α β	2.0332 1.5055	2.7722 2.5320	3.3700 3.4217	3.8336 3.8323
0.7 < T < 4.0	α β	1.8409 0.2642	2.4823 0.6605	2.9853 0.9380	3.4180 1.1493

M. Mahmoudi [21]

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

Ioana Olteanu, Ioan-Petru Clongradi, Mihaela Anechitei and M. Budescu[22]

Presents the characteristics of the ductility concept for the structural system. The reduction of the seismic forces is realized based on the ductility, redundancy and the strength excess of the structure. Among these, the most significant reduction of the design forces is based on the ductility of the structure that depends on the chosen structural type and material characteristics.

3.5.3 Ductility of unconfined beam sections.

Calculation of moment and curvature [25].

a) At just prior to cracking of the concrete

b) At first yield of the tension steel.

c) When the concrete reaches an extreme fibre strain of 0.004

a) Before cracking (elastic behaviors)

M_{crack} = fr *I/ y $_{bottom}$ (15)

.

$\phi_{crack} = \frac{Fr/Ec}{ybottom}$ (16)

b) At first yield of the tension steel

M_y = As f_y jd (17)

$k = [(\rho+\rho')^2 n^2 + 2(\rho+\rho'd'/d)\, n\,]^{1/2} - (\rho+\rho')\, n$ (18)

$\phi_y = \frac{Fy/Es}{d(1-k)}$ (19)

a = (As fy - As' fy)/ 0.85 fc' b (20)

Mu = 0.85 fc' a b (d-a/2) + As' fy (d-d') (21)

c) When the concrete reaches the extreme fibre strain

$\phi_u = \varepsilon_c / c$ (22)

n = Es / Ec (23)

ρ = As/ bd (24)

ρ' = As'/ bd (25)

The same concept is followed by SAP 2000 [33] to develop moment curvature relation.

Plastic hinge length

Various empirical expressions have been proposed by investigators for the equivalent length of plastic hinge l_p and the maximum concrete strain ε_c at ultimate curvature.

BAKER:

a) for members with unconfined concrete.

$l_p = k_1\ k_2\ k_3\ (\ z/d\ )^{1/4}$ (26)

b) for members confined by transverse steel

$l_p = 0.80\ k_1\ k_3\ (\ z/d)\ c$ (27)

CORLEY:

For simply supported beam

$l_p = 0.50\ d + 0.20\ \sqrt{d}\ (z/d)\ ;\ \varepsilon_c = 0.003 + 0.02\ b/z + (\rho_s\ fy/2$ (28)

Mattok suggested

$l_p = 0.50\ d + .05d$ (29)

$\varepsilon_c = 0.003+0.02\ b/z + 0.20\ \rho_s$ (30)

Sawyer: $l_p = (0.25d+0.075z)$

Where,

k_1 = 0.70 for mild steel or 0.90 for cold worked steel.

$K_2 = 1+ 0.50\ P_u/P_o$, where P_u = axial compressive force in member and P_o is axial compressive strength of member without bending moment.

K3 = 0.60 when fc' = 35.2 N/mm^2 or 0.90 when fc' =11.7 N/mm^2; assuming fc' = 0.85 * cube strength of concrete.

z = distance of critical section to the point of contraflexure

d = effective depth of member.

A good estimate of the effective plastic hinge length may be obtained from the expression.

$l_p = 0.08\ l + 0.022\ d_b\ fy$

Where

d_b = nominal diameter of bar in mm.

For user-defined hinge properties, the procedure used by Park and Paulay [25] is used to determine moment –rotation relationships of members from the moment-curvature relationships. In this procedure, the moment is assumed to vary linearly along the beam and columns with a contraflexure point at the middle of the members. With this assumption, the relations are

$\theta y = L.\ \phi y\ /6$ (31)

Plastic hinge rotation capacity of members is estimated using the following equations proposed by ATC-40 [26] and value at ultimate moment is obtained by adding plastic rotations to the yield rotation.

$$\theta p = (\phi ult - \phi y)\ lp \quad (32)$$

ATC-40[26] suggests that plastic hinge length equals to half of the section depth in the direction of loading. This technique was adapted to calculate plastic hinge length in this study.

3.6 Redundancy Factor

Redundant is usually defined as: exceeding what is necessary or naturally excessive. Building should have a high degree of redundancy for lateral load resistance [16]. More redundancy in the structure leads to increased level of energy dissipation and more overstrength. In a nonredundant system the failure of a member is equivalent to the failure of the entire structure however in a redundant system failure will occur if more than one member fails. Thus, the reliability of a system will be a function of the system's redundancy meaning that the reliability depends on whether the system is redundant or nonredundant[16].

Table 3.2: Redundancy factor (R_R) was taken from ATC

Lines of vertical seismic framing	Drift redundancy factor
2	0.71
3	0.86
4	1.0

Overstrength, redundancy and ductility together contribute to the fact that an earthquake resistant structure can be designed for much lower force than is implied by the strong shaking.

3.7 Codal provisions for reduction factors:

In design codes the considered seismic force used to dimensioning the structural elements is multiplied by several coefficients, in order to simplify the design process. One of them is the reduction factor. The behavior factor of the response is computed as a product of three factors

$R = R_s\ R_\mu\ R_{\xi,}$

3.7.1 Response Reduction Factor as per ATC-19

In the ATC-19[27] meeting from 1995 damping reduction factor was replaced by the redundancy factor RR. $R = (R_s\ R_\mu)\ R_R$

Table 3.3: The strength reduction factor for Reinforced Concrete Structures

Structural type	R_s
Rc structures medium and high elevation	1.6………………………….4.6
Rc structures with irregularities in elevation	2.0…………………………..3.0

3.7.2 Response Reduction Factor as per IBC, 2003

The seismic force at the bottom of the building, according to the American design code, is computed with the following relationship (IBC, 2003) [28]:

$$V = \frac{12SDS}{R} W$$

Table 3.4: Reduction factor According to IBC 2003

Structural type	R
Special Rc frames	8.0
Intermediate Rc frames	5.0
Ordinary Rc frames	3.0

3.7.3 Response Reduction Factor as per New Zeeland design norm

According to the New Zeeland design norm[30], the coefficient C_μ from the seismic force relation is determined by taking into consideration the fundamental oscillation period, T_1, the ductility displacement, μ_Δ, and soil type,

$F_{tot} = C_\mu R Z Wt$

The value of Cμ coefficient ranges between 0.40 and 0.040, for ductile Rc structures with the ductility displacement, μ, between 4 and 6. The values of the ductility displacement, μ_Δ, are listed in table

Table 3.5: Ductility displacement values, μ_Δ, according to New Zeeland design norms

Structures	RC	Prestressed concrete
Structures with elastic behaviors	1.25	1.00
Structures with limited ductility	-	-
Frames	3.00	2.00
Coupled walls	2.00	-
Ductile structures	-	-
Moment resisting frames	6.00	5.00
Walls	5.00	-

3.7.4 Response Reduction Factor in Japanese design code

According to Japanese design code [30] the seismic force at the bottom of the structure is computed using the following relation (Building standard Law of Japan, 2004)

$Vu_{n,i} = D_{s,i} Fe_{s,i} Vi.$

The coefficient Ds,i depends on the structural type and it represents the inverted value of the behaviour factor from the European norm. This factor is influenced by the material used. In table are presented only the values for reinforced concrete structures depending on the structural type and ductility class.

Table 3.6: Response Reduction Factor According to BSL J

Ductility	Moment-resisting frame	Other frames	Frames with bracing
Excellent	0.30	0.35	0.40
Good	0.35	0.40	0.45
Normal	0.40	0.45	0.50
Low	0.45	0.50	0.55

3.7.5 Response Reduction Factor in IS 1893 (part1):2002

According to IS 1893[16] (part1):2002, Criteria for Earthquake Resistant Design of Structures, design seismic base shear can be computed as

$V_B = A_h W$

$$A_h = \frac{ZISa}{2Rg}$$

Where, R is the response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. The values of R of the buildings are given in the table.

Table 3.7: Response reduction factor R for Buildings Systems

Lateral load resisting system	R
Building frame system	
Ordinary RC moment-resisting frame (OMRF)	3.0
Special RC moment-resisting frame (SMRF)	5.0
Unreinforced masonry wall building	1.50

3.7.6 Response Reduction Factor in NBC 105:1994

Nepal National Building Code, NBC 105:1994 [32], establish the following relation for Seismic Design of Buildings in Nepal.

The design horizontal seismic force coefficient, Cd shall be taken as

Cd = CZIK

Where, K is the structural performance factor. The structural type may be different in each of two directions in a building and in that case the appropriate value for K shall be selected for each direction. When more than one structural type is used in the structure for the direction under consideration, the structural performance factor for the element providing the majority of the seismic load resistance shall be applied provided that the elements of the other structural types have the ability to accept the resulting deformations.

Table 3.8: Structural performance factor

Item	Structural type	Minimum detailing requirement	Structural performance
1.a	Ductile moment resisting frame	Fulfill the ductility requirement of IS 4326 and for steel frames, additional requirements of NBC 111-94	1.0
1.b	Frame as in 1.a. with reinforcement concrete shear wall	For frame : as for 1.a RC shear walls must comply with appropriate detailing for ductility requirement	1.0
2.a	Frame as in 1.a with either steel bracing members detailed for		1.5

	ductility or reinforced concrete infill panels		
2.b	Frame as in 1.a with masonry infill	Must comply with detailing requirements of : IS 4326	2.0
3	Diagonally- braced steel frame with ductile bracing acting in tension only	Must comply with the detailing for ductility requirements of Nepal Steel Construction Standard	2.0
4.	Cable –stayed chimneys	Appropriate materials standard	3.0
5.	Structures of minimal ductility including reinforced concrete frames not covered by 1 or 2 above, and masonry bearing wall structures	Appropriate materials standard	4.0

Building codes allow for an elastic structural analysis based on applied forces reduced accounting for the presumed ductility supplied by the structure. For elastic analysis, use of reduced forces will result in a significant underestimate of displacement demands. Therefore, the displacements from the reduced-force elastic analysis must be multiplied by the ductility factor to produce the true "inelastic" displacements.

3.8 Formulation used for this study

For the determination of Overstrength factor (Ω) concept of FEMA 451 is used, which gives

$\Omega = Vu / Vy \ X \ Vy/Vd = \Omega o \ X \ R_R$

$\Omega = Vu/Vd$ (6)

The expression of equation (6) is same as the indication given by IS 1893-2002.

For the determination of displacement ductility following expression is used

$\mu = \Delta u / \Delta y$ (7)

For determination of ductility reduction factor, equation (11) is used

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

For the determination of Response Reduction Factor (R), the main concept given by ATC-19 is used, which is given in equation (4)

$R = \Omega \times R\mu \times R_R$

But in our case, Overstrength and redundancy factor is taken as single term i.e overstrength factor and the IS 1893-2002 gives the value of Force Reduction Factor = (2R), same concept is used to determine Response Reduction Factor of the study structures.

$2R = \Omega \times R\mu$

$R = \Omega \times R\mu/2$

4 REVIEW ON NON-LINEAR METHODS OF ANALYSIS

4.1 Introduction

Researcher formulates the different techniques for the study of non-linear behaviors of the structure.

4.1.1 Previous Study on Non-Linear Analysis

Krawinkler H. and Seneviratha [39]

Conducted a detailed study on pushover analysis. The accuracy of pushover predictions were evaluated on a 4-story steel perimeter framed in 1994 Northridge earthquake. The comparison of pushover and nonlinear dynamic analysis results showed that pushover analysis provides good predictions of seismic demands for low-rise structures having uniform distribution of inelastic behavior over the height.

Mwafy A.M. and Elnashai [40]

Performed a series of pushover analysis and incremental dynamic collapse analysis to investigate and the applicability of pushover analysis. Twelve RC buildings with different structural system were studied. The results showed that triangular load pattern outcomes were in good correlation with dynamic analysis results. It was also noted that pushover analysis is more appropriate for low-rise and short period structures and triangular loading is adequate to predict the response of such structures.

Chopra A.K and Goel R.K [41]

Developed an improved pushover analysis procedure named as Modal Pushover Analysis (MPA) which is based on structural dynamic theory. Firstly, the procedure was applied on to linearly elastic buildings and it was shown that the procedure is equivalent to the well known response spectrum analysis. Then, the procedure was extended to estimate the seismic demands of inelastic systems. The MPA was more accurate than all pushover analysis is estimating floor displacements, storey drifts, plastic hinge rotations and plastic hinge locations.

4.1.2 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 nos of steps. Each steps of load will be applied to the structure. The steps is 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.

Virote Boonyapinyo[1], Norathape Choopool[2] and Pennung Warnitchai[3.] [42]

The performances of reinforced-concrete buildings evaluated by nonlinear static pushover analysis and nonlinear time history analysis were compared. The results show that the nonlinear static pushover analysis is accurate enough for practical applications in seismic performance evaluation when compared with nonlinear dynamic analysis of MDOF system.

A.Kadid and A. Boumrkik. [43]

Use a non linear 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.

Gergely, P., R.N. White, and K.M. Mosalam, [44]

Use static nonlinear pushover analysis for evaluation and modeling of infilled frames buildings. Conclusions are made that elastic seismic analysis methods are inadequate for the estimation of the internal force and displacement distributions.

4.1.3 Static Pushover Analysis Procedure

Pushover analysis can be performed as either force-controlled or displacement- controlled depending on the physical nature of the load and the behavior expected from the structure. Force-controlled option is useful when the load is known (such as gravity loading where structure is loaded gravity load plus 25 % of live load) and the structure is expected to be able to support the load. Displacement- controlled procedure is used when specified drifts are sought (such as in seismic loading), where the magnitude of the applied load is not known in advance, or when the structure can be expected to lose strength or become unstable. The first mode response of the structure was assigned as the load pattern for the lateral push applied to the structure.

Nonlinear version of finite element package SAP2000 [33] can model nonlinear behavior and perform pushover analysis directly to obtain capacity curve for three dimensional models of the structure. A displacement-controlled pushover analysis is basically composed of the following steps:

- ❖ Developing a three dimensional bare frame model of existing RC buildings.
- ❖ Application of gravity loads and live loads.
- ❖ Application of 10% static lateral load induced due to earthquake, at CG of the building
- ❖ Developing M-θ relationship for critical regions (Plastic hinging zone) of beam and column element with shear strength confirming and non confirming.
- ❖ Pushing the structure using the load patterns of static lateral loads, up to displacements larger than those associated with target displacement using static pushover analysis
- ❖ Developing hinge progressing sequence in different steps of the loading.
- ❖ Developing tables of roof displacement vs. base shear or pushover curve.

The earthquake forces are estimate as per IS 1893-2002 (part-2) [16]. Moment – rotation and Axial load- Bending moment (P-M2-M3) relationships for flexural and compression members have been developed using SAP 2000 software. Above relationships are also analytically calculated by the methods suggested by R. Park and T. Paulay [25].

4.1.4 Default Vs User-Defined Hinge Properties for Concrete Sections

The built-in default hinge characteristics of concrete sections are based on ATC-40[26] and FEMA-273[14] criteria which consider basic parameters controlling the behavior. Based on these parameters, in this study, default moment hinges assigned to all beams have same plastic rotation capacities (M3) and default PMM hinges assigned to all columns have same plastic rotation capacities regardless of the section dimensions. Slope between points B and C is taken as 10% total strain hardening for steel and yield rotation is taken as zero for default concrete moment and PMM hinges and then user defined hinge properties is assign to the elements.

For user-defined hinge properties, the procedure used by Park and Paulay [25] was utilized to determine moment-rotation relationships of members from the Moment-curvature relationships. In this procedure, the moment is assumed to vary linearly along the beams and columns with a contra- flexure point at the middle of the members. Based on this assumption, the relationship between curvature and rotation at yield is obtained.

In this study user defined plastic hinge is generated only on beam element. In Numerical model, there is only option to put the reinforcement of column element. Moment curvature relation of beam element according to the detailing of beam section is established which gives ultimate moment, yield moment, ultimate curvature, yield curvature. Plastic hinge length is taken as 0.50 d (ATC-40). From these data, scale factor, rotation of various segments of plastic hinge is obtained. (Detail in annex)

Pushover analyses were performed using both default and user-defined hinge properties and the effect of hinge properties were illustrated on pushover curves as shown in figure.

Pushover analyses with default and user-defined hinge properties yield differences in sequence of plastic hinging and hinge pattern. The rotation value at the yield point of hinges is not needed for pushover analyses performed by SAP2000 because the program uses cross-sectional dimensions in the elastic range.

Default hinge properties based on ATC-40[26] and FEMA-273[14] criteria are generally preferred to perform pushover analysis by SAP2000 because determination of cross-sectional characteristics of all members of a structure, especially for a three dimensional structure, and inputting these sectional properties into the program make the pushover analysis impractical. Thus, the results of a pushover analysis with default hinge properties should be interpreted with caution since default hinges could not simulate the exact nonlinear behavior of the structure.

4.1.5 Force-Displacement Relationships

When the structure is analyzed with three loading conditions (GRAV, EQX and EQY), pushover curve of the structure is obtained. The curve is the base shear vs. deformation curve.

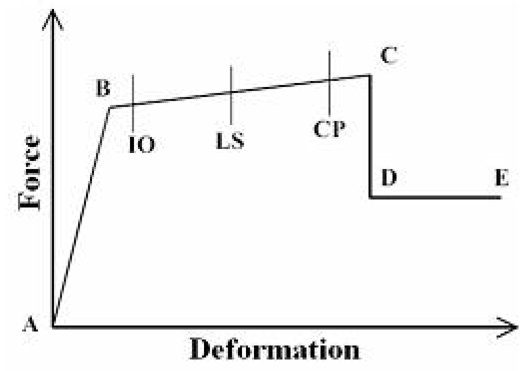


Figure 4.1. Component Force-Deformation Curve

A generic component behavior curve is represented in figurer. The points marked on the curve are expressed by the software vender [33] as follows:

- Point A is the origin
- Point B represents yielding. No deformation occurs in the hinge up to point B, regardless of the deformation value specified for point B, the deformation (rotation) at point B will be subtracted from the deformations at points C, D, and E. Only the plastic deformation beyond point B will be exhibited by the hinge.
- Point C represents the ultimate capacity for pushover analysis. However, a positive slope from C to D may be specified for other purposes.
- Point D represents a residual strength for pushover analysis. However, a positive slope from C to D or D to E may be specified for other purposes.
- Point E represents total failure. Beyond point E on the horizontal axis, if it is not desired that the hinge to fail this way, a large value for the deformation at point D may be specified.

4.1.6 Capacity

Capacity is a representation of the structures ability to resist the seismic demand. The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure [36]. In order to determine the capacities beyond the elastic limit, non linear analysis is required.

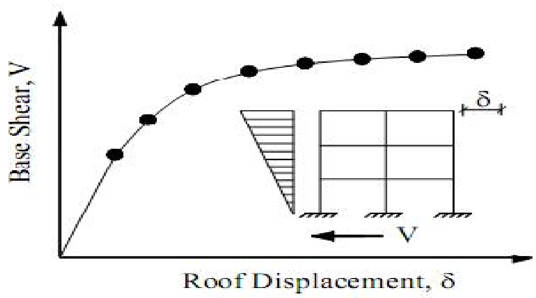


Figure 4.2. Global capacity (Pushover curve) of a structure.

Capacity curve is the fundamental for the determination of response reduction factor.

Maximum displacement, yield displacement, yield shear force and maximum shear force, initial stiffness and effective stiffness can be obtained from the capacity curve. The health of the structure is judged by the capacity curve.

Idealization of Capacity Curve

The capacity curve presents the primary data for the evaluation of the response reduction factor for structures, but first of all it must be idealized in order to extract the relevant information from the plot. The intension is to obtain the overstrength and the ductility reduction factor by studying the pushover curve.

For this purpose a bi-linear curve is fitted to the capacity curve, such as the first segment starts from the origin, intersects with the second segment at the significant yield point and the second segment starting from the intersection ends at the ultimate point. The slope of the first segment is found by tracing the individual changes in slopes of the plot increments; the mean slope of the all increments are calculated from each step and compared with the latter, searching for a dramatic change. First segment, referred to as elastic portion, is then obtained with a mean slope of the successive parts of the curve until a remarkable change occurs. The second segment, referred to as post-elastic portion, is plotted by acquiring the significant yield point by means of equal energy concept in which the area under the capacity curve and the area under the bi-linear curve is kept equal. Graphical method or auto cad program is developed to read and plot the pushover data then fit the bi-linear curve by utilizing the above mentioned methodology.

This method is an improved version of the one, proposed by FEMA 273[14] which offers a visual trial & error process and suggests that the first segment intersects the original curve at 60% of the significant yield strength.

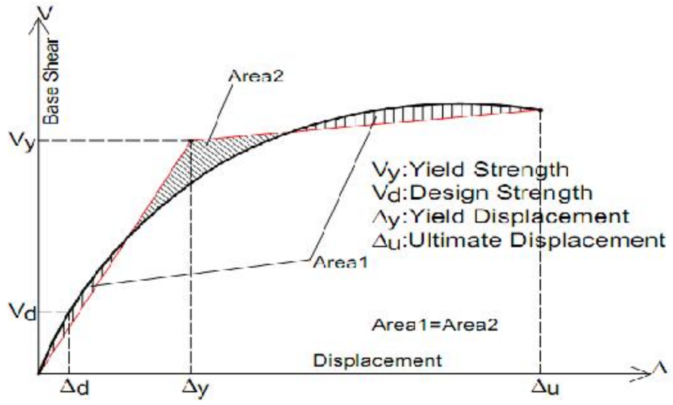


Figure 4.3. Bi-linear Idealization of a Generic Capacity Curve

Bi-linear idealization provides the essential components, which are significant strength and the significant yield displacement as well as the predetermined design strength and the ultimate displacement. With the help of these data, the overstrength factor which is calculated as the ratio of the yield strength to the design strength. Moreover, ductility ratio can be calculated as the ratio of maximum displacement to the yield displacement which is the key element in calculation of the ductility reduction factor.

4.1.7 Demand

Demand is the representation of the earthquake induced ground by ground motion. Ground motions during an earthquake produce complex horizontal displacement patterns in the structures that may vary with time [36]. For a given structure and ground motion, the displacement demand is an estimate of the maximum expected response of the building during the ground motion.

Procedure to determine demand [ATC-40]

1. Construct a bilinear representation of the capacity curve.

- Draw the post-elastic stiffness K_s by judgment to represent an average stiffness in the range in which the structure strength has leveled off.

- Draw the effective elastic stiffness K_e by constructing a secant line passing through the point on the capacity curve corresponding to a base shear of $0.60V_y$, where V_y is defined by the intersection of the K_e and K_s.

2. Calculate effective fundamental period.
3. Calculate the target displacement by displacement coefficient method.

Alternately, the displacement demand during earthquake is obtained by considering FEMA 451. Geometrically, the ratio of Ve /Vy is numerically equal to the ratio of Δu/Δy. so in this study for the calculation of ductility supply the ratio of Δu/Δy is used. For the prediction of ductility demand, total elastic force is calculated without considering reduction factor. Stiffness of the structure is determined by using K = Ve/Δe (FEMA 356).

For example, modal 3 have the design base shear 379.6 KN (calculation is based on IS 1893-2002). If force reduction factor is not considered, the total elastic force demand is 3796, which is 10 times more than design force.

From capacity curve, the initial stiffness of the structure is 46402 KN/m, then displacement ductility demand is calculated as (µd) =Ve/K, where Ve is the elastic base shear demand and K is the initial stiffness which represent the slope when the structure is in fully elastic.

Thus, µd = 3796/46402 =0.082 m

4.1.8 Performance

The performance is dependent on the manner that the capacity is able to handle the demand [29].

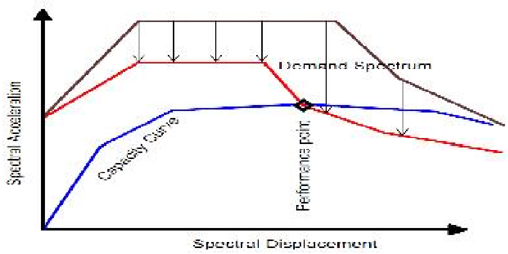


Figure 4.4. Performance point in a structure

The NEHRP Guidelines for Seismic Rehabilitation of Buildings, FEMA 273 and Provision for seismic Regulations for New buildings and other structures, FEMA 302 define three discrete Structural Performance Levels namely Immediate Occupancy Level (IO), Life Safety (LS), and Collapse Prevention (CP)

Immediate Occupancy (IO)

It is the post earthquake damage state where only minor structural damage has occurred with no substantial reduction in building strength. So, the building is safe to occupy but possibly not useful until repaired.

Life Safety (LS)

It is the post earthquake damage state in which significant damage to the structure has occurred, but some margin against the partial or total collapse remain. In this stage, the building is safe during the earthquake but probably

Collapse Prevention (CP)

It is the post earthquake damage state in which the structure is on the verge of experiencing either local or total collapse.

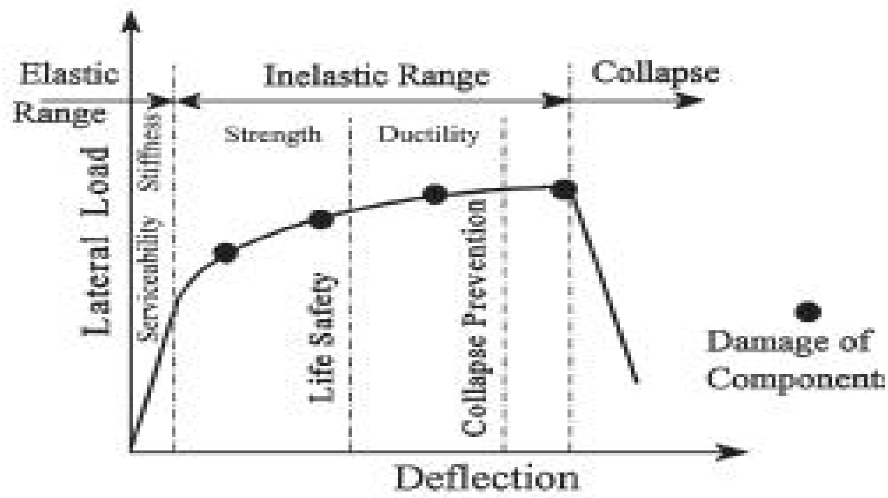


Figure 4.5. Ranges of pushover curve

(Source: ELSEVIER, ENGINEERING STRUCTURE JORUNAL)

4.1.9 Equal displacement rule

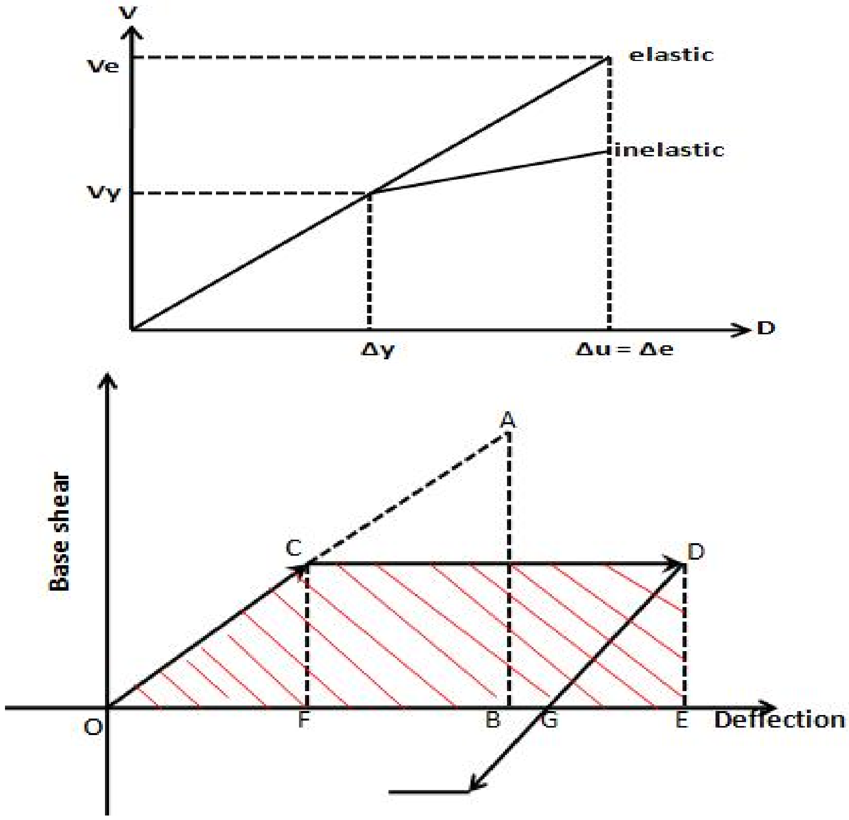


Figure 4.6. Elasto-Plastic Response of Structure

In figure [4.6], "The relationship between elastic displacement B and inelastic displacement E depends on the natural period of the structure. If the period is greater than 0.7 s, analyses have shown that E is approximately equal to B (i.e., the deflection of the equivalent elastic structure is approximately equal to that of the elasto-plastic structure, $\Delta u = \Delta e$). This is referred to as the *Equal Displacement Principle"* [45].

Structure which have time period greater than 0.7 s, ductility reduction factor is calculated by using the equation:

$R\mu = \mu$ ($T > 0.70$ s) ………………………………………… (33)

Where $R\mu$ = ductility reduction factor and μ = displacement ductility [48].

4.1.10 Equal Energy Rule

Low period structure tends to display significant residual deformations. So, in low period structure equal energy concept is used. [9]

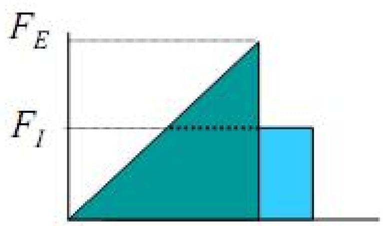


Figure 4.7. Concept of Equal Energy Rule

$F_E/F_I = R\mu = \sqrt{2\mu - 1}$ (T < 0.3 s) [48]........................34

NBC [45], indicates that for period less than 0.3 s, analyses have shown that Equal Energy Principal applies. That is, the area OAB is equal to OCDE [figure 4.6].

A gradual variation in R is found to occur between structural period of 0.3 s and 0.7s.

$R\mu = 1 + (\mu - 1)\, T/0.70$[48]..35

Equation 33 is consistent with assuming that the deflection of the elastic and elasto-plastic systems is the same [49].

Equation (34) is consistent with assuming that the potential energy stored at the maximum deflection is same for the elastic and elasto-plastic systems [49].

4.2 Procedure for seismic analysis of RC Building as per IS 1893 (Part 1): 2002

4.2.1 Equivalent static lateral force method

Total design lateral force or design seismic base shear V_b along any principal direction shall be determined by

$V_b = A_h W$

$T = 0.075\ h^{0.75}$

Determination of Design Base Shear

Design base shear, $V_B = A_h W$

$A_h = Z/2*I/R*S_a/g$

$Q_i = V_b.\frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2}$

4.2.2 Response Spectrum Method

Procedure for calculating design base shear without considering the stiffness of infill.

a) Determination of Eigen values and Eigenvectors

Mass matrices and stiffness matrices of the frame lumped mass model are,

M = [M] and K = [K]

For the above stiffness and mass matrices, eigenvalues and eigenvectors are worked out as follows

$|K-\omega^2 m| = 0$

By solving the above equation, natural frequencies (eigenvalues) of various modes are calculated. The quantity of ω_i^2, is called the ith eigenvalue of a matrix $[-M\ \omega_i^2+K]\ \phi i$. Each natural frequency (ω_i) of the system has a corresponding eigenvector (mode shape), which is denoted by ϕi. The mode shape corresponding to each natural frequency is determined from the equations

$[-M\ \omega_i^2+K]\phi_1 = 0$

$[-M\ \omega_i^2+K]\phi_2 = 0$

$[-M\ \omega_i^2+K]\phi_3 = 0$

$[-M\ \omega_i^2+K]\phi_n = 0$

Solving the above equation, modal vector (eigenvectors), mode shape and natural period under different modes are found

$\{\phi\} = \{\phi_1\ \phi_2\ \phi_3\ \phi_4 \ldots\ldots\ldots\ldots\phi_n\}$

Determination of Modal Participation Factors

The modal participation factor (Pk) of mode k is,

$$P_k = \frac{\sum_{i=1}^{n} W_i \varphi_{ik}}{\sum_{i=1}^{n} W_i (\varphi_{ik})^2}$$

Determination of Modal Mass

$$M_k = \frac{\left[\sum_{i=1}^{n} W_i \varphi_{ik}\right]^2}{g \sum_{i-1}^{n} W_i (\varphi_{ik})^2}$$

Determination of Lateral Force at Each Floor in Each Mode

The design lateral force (Qik) at floor i in mode k is given by

$Q_{ik} = A_k\ \phi_{ik}\ P_k\ W_i$

Determination of Storey Shear Forces in Each Mode

The peak force is given by,

$$V_{ik} = \sum_{j=i+1}^{n} Q_{ik}$$

Determination of Storey Shear Force due to All Modes can be obtained as;

The peak storey shear force (V_i) in storey i due to all modes considered is obtained by combining those due to each mode in accordance with modal combination i.e. SRSS (square root if sum of squares) or (complete quadratic combination) methods.

Square roof of sum of squares (SRSS)

If the building does not have closely spaced modes, the peak response quantity (λ) due to all modes considered shall be obtained as,

$$\lambda = \sqrt{\sum_{k=l}^{r} (\lambda_k)^2}$$

The base shear to the study buildings is vertically distributed by using equivalent static lateral force method, but response spectrum method is used in one building compare the differences in design base shear.

4.3 Displacement Coefficient Method (FEMA-273)

The Displacement Coefficient Method described in FEMA-273 estimate the structural performance in terms of a target displacement representing the maximum expected top displacement representing the maximum expected top displacement. It combines the pushover analysis with a modified version of the equal displacement approximation, according to which the linear elastic spectral displacement or spectral acceleration corresponding to the effective period and damping of the equivalent SDOF system, is corrected by some factors. These factors were obtained for regular frame buildings. The factors for buildings with vertical mass, stiffness and strength irregularities are being examined by Karawinkler and SeneViratna [22]. Among its advantages is that the DCM provides a direct numerical procedure to define displacement demand and need to conversion in spectral format.

For this analysis bilinear representation of capacity curve is required to be used in the procedure. The procedure described is for bilinear representation. After the construction of bilinear curve, effective fundamental period (Te) of the structure is calculated using Equation

$Te = Ti\sqrt{Ki/Ke}$

The target displacement δ_t in FEMA-273 is given by

$\delta t = C_O C_1 C_2 C_3 Sa \ g\ Te^2/4\Pi^2$

Where,

C0: modification factor to relate spectral displacement and likely roof displacement of the structure. The first modal participation factor at the roof level is used.

C1: modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C2: modification factor to represent the effect of hysteresis shape on the maximum displacement response. In this study, C2 was taken as 1.1 for both elastic and inelastic deformation levels. As the estimates of Displacement Coefficient Method (FEMA-356) depend on the coefficient C2, the coefficient C2 should be taken as unity in the elastic range and should take the specified value for the considered performance level in the inelastic range for seismic performance evaluation purposes.

C3: modification factor to represent increased displacements due to second-order effects. Sa: response spectrum acceleration at the effective fundamental period of the structure.

Te: effective fundamental period of the structure.

In this method, different target displacements can be obtained for different seismic performance levels. In this study, target displacements for each ground motion record were calculated for life safety performance level.

Table 3.9: Values for modification factor C_0

Number of stories	1	2	3	5	10
Modification factor	1.0	1.2	1.3	1.4	1.5

Factor C1 is the modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response:

C1 = 1.0 for Te < ToC1 = 1/R [1.0+ (R-1) To/Te] for T > To

The coefficient R is expressed in terms of base shear at yield strength Vby as $R = \frac{Sa/g}{CoVby/W}$ Where,

W is the total dead load and expected live load. Vby is determined using pushover analysis where the pushover curve is defined by a bilinear relation as shown in

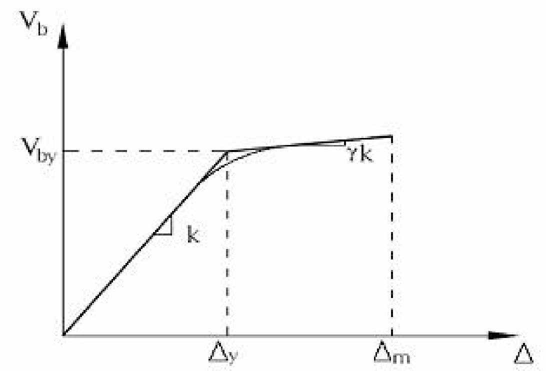


Figure 4.8. Bilinear idealization of pushover curve.

Factor C2 is the modification factor represent the effect of hysteresis shape on the maximum displacement response. Values for C2 may be taken as follows.

Table 3.10: Values for C2

	T = 0.1 second		T ≥ To second	
Structural performance Level	**Framing type 1**	**Framing type 2**	**Framing type 1**	**Framing type 2**
IO	1.0	1.0	1.0	1.0
LS	1.3	1.0	1.1	1.0
CP	1.5	1.0	1.2	1.0

Factor C3 is the modification factor to represent increased P-Δ effects. For building with positive post yield stiffness, C3 may be set equal to 1.0. for buildings with negative post yield stiffness value , C3 is calculated using the following relations

$C3 = 1.0 + |\gamma|\ (R-1)^{3/2} / Te$

Where α is the ration of post yield stiffness to effective elastic stiffness for the bilinear pushover curve idealization.

4.4 Selection of appropriate method of analysis for the study

Due to simplicity and different researcher also used non-linear static pushover analysis with sufficient accuracy [section 4.1.2] to study non-linear behaviors of the structure, is the main clues for the selection the Static Non-linear Pushover Analysis in this study.

5 ANALYSIS AND INTERPRETATION OF RESULTS:

5.1 Distribution of Lateral Force

Lateral force in the study buildings is calculated by seismic coefficient method, given by IS 1893-2002. The results of lateral forces, design base shear and total weight of the study buildings are tabulated in table 5.1.

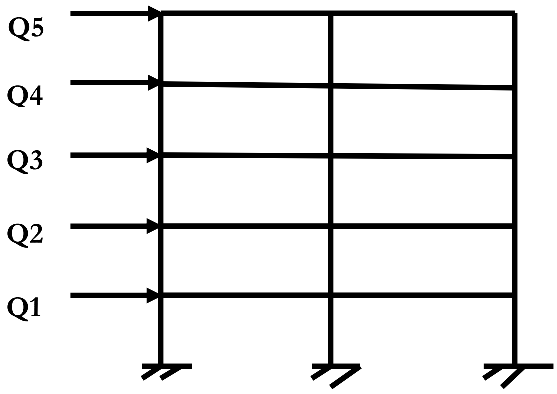
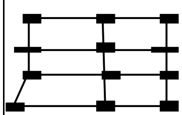
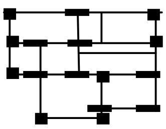
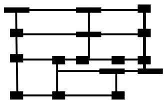
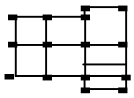
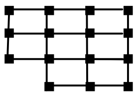
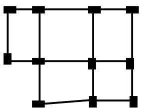
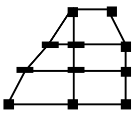
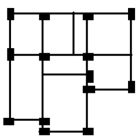
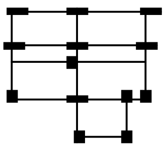
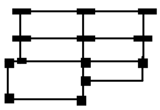
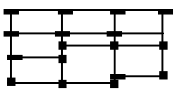
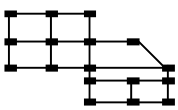


Figure 5.1. Distribution of lateral force

Table 5.1: Distribution of lateral forces:

Model	Plan	Q1(KN)	Q2 (KN)	Q3 (KN)	Q4 (KN)	Q5 (KN)	Vb (KN)	W (KN)
1		35.35	142.20		62.88	-	435.97	4844.08
2		60.37	191.50	205.85	-	-	457.72	5085.82

Model	Plan	Q1(KN)	Q2 (KN)	Q3 (KN)	Q4 (KN)	Q5 (KN)	Vb (KN)	W (KN)
3		49.14	167.09	163.33	-	-	379.56	4217.36
4		61.66	159	92.64	-	-	313.29	3481
5		22.24	88.98	200.20	298.73	188.65	798.80	8875.72
6		23.88	95.51	189.65	146.13	-	455.17	5057.50
7		13.68	54.73	123.13	144.40	54.36	390.55	4392.09
8		48.52	160.36	142.43	-	-	350	3903.7
9		35.52	122.72	116.26	-	-	274.50	3050
10		55.88	163.01	136.31	-	-	355.20	3946.64
11		31.37	125.48	214.98	131.34	-	503.63	5590
12		15.76	63.10	141.85	252.18	197.15	670	7435

5.2 Natural Structural Period of the Structure

Time period of study is based on Numerical analysis in SAP 2000 and IS 1893 (Part 1): 2002. Time Period of structure obtained from Numerical analysis is indicated as T (Numerical Model) where as from code, $T = 0.075\ h^{0.75}$ is tabulated as T (code). Code based procedure gives higher time period than time period of structure obtained from model analysis. The results of natural period of the study structure is tabulated in Table 5.2

Table 5.2: Time period and number of stories of study buildings

Model No	No. of Storey	T (Numerical Model)	T (Code)
1	3	0.36	0.375
2	3	0.35	0.375
3	3	0.34	0.375
4	3	0.3	0.375
5	5	0.55	0.55
6	4	0.43	0.465
7	5	0.46	0.55
8	3	0.31	0.375
9	3	0.3	0.375
10	3	0.31	0.375
11	4	0.4	0.465
12	5	0.43	0.55

5.3 Ductility reduction factor (Rμ)

Ductility of a structure, or its members, is the capacity to undergo large inelastic deformations without significant loss of strength or stiffness. Displacement ductility factor is the ratio of ultimate deformation to yield deformation (FEMA 451). It is represented by the symbol μ. $\mu = \Delta u / \Delta y$. Ductility reduction factor is calculated using the equation (11).

$R_{\mu} = 1 + (\mu - 1)\, T/0.70$

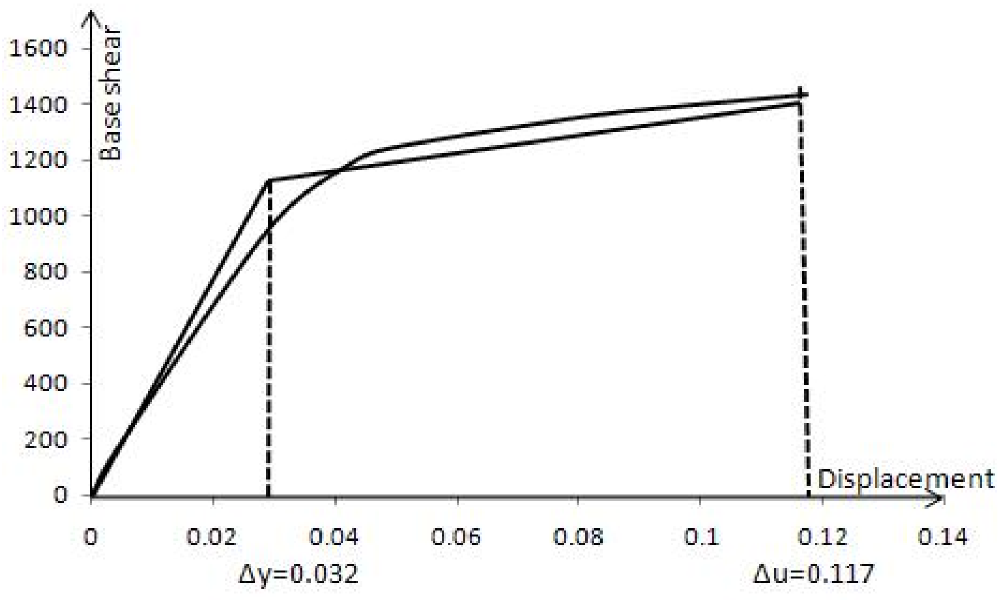


Figure 5.2. Representation of displacement ductility

For an example: Model 2 (EQY) has Δu equal to 0.117m and Δy= 0.032. In this case, displacement ductility factor $\mu = 0.117 / 0.032 = 3.7$. Similarly, for the calculation of ductility demand, firstly elastic base shear is determined. In this case, elastic base shear is equal to 4577.2 KN. Initial stiffness (K) equal to 31913. Elastic displacement demand is calculated as:

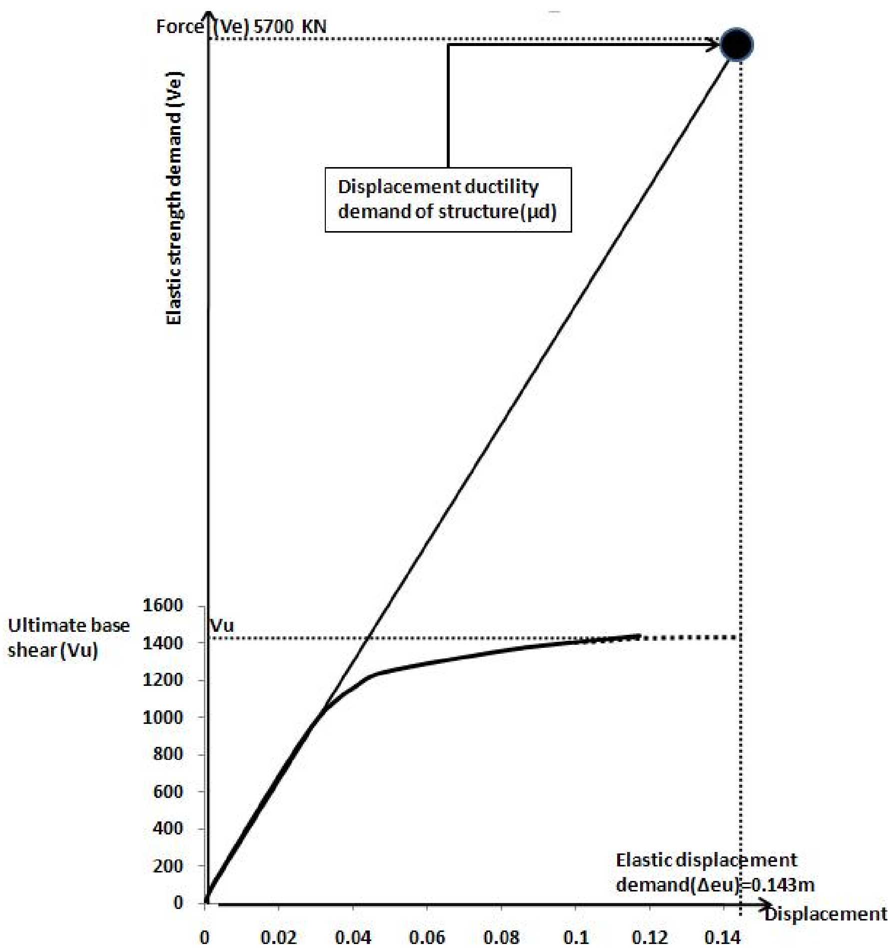


Figure 5.3. Representation of ductility demand of the structure

Δeu= Ve / K = 4577.2 / 31913 = 0.143

Displacement ductility demand of the structure is computed as (μd) = Δeu / Δy

= 0.143 / 0.032 = 4.5

Ductility reduction factor (Rμ) =1+(3.7-1) 0.30/0.70 = 2.22

EQX and EQY are the conditions of Pushing X and Y direction to the structure.

The overall results of ultimate displacement Δu, yield displacement Δy, displacement ductility supply μ, Initial stiffness of structure K, elastic base shear demand Velastic, elastic displacement Δeu and ductility demand μd is presented in Table 5.3, 5.4, and 5.5.

Table 5.3: Level of displacement ductility in the study buildings

Model	EQ Push	Δu	Δy	μ	K	Velastic	Δeu	μd
1	EQX	0.165	0.029	5.7	32437	4359.7	0.134	4.6
	EQY	0.149	0.035	4.3	28444	4359.7	0.153	4.4
2	EQX	0.111	0.022	5	47632	4577.2	0.096	4.4
	EQY	0.117	0.032	3.7	31913	4577.2	0.143	4.5
3	EQX	0.076	0.024	3.2	46402	3795.6	0.082	3.4
	EQY	0.062	0.027	2.3	41803	3795.6	0.091	3.4
4	EQX	0.2	0.026	7.7	33703	3132.9	0.093	3.6
	EQY	0.196	0.036	5.5	34731	3132.9	0.090	2.5
5	EQX	0.139	0.045	3.1	18987	7988	0.421	9.5
	EQY	0.087	0.029	3	21202	7988	0.377	13
6	EQX	0.198	0.038	5.2	22430	4551.7	0.203	5.3
	EQY	0.199	0.043	4.6	18199	4551.7	0.250	5.8
7	EQX	0.223	0.044	5.1	13177	3905.5	0.296	6.7
	EQY	0.199	0.04	5	19990	3905.5	0.195	4.9
8	EQX	0.10	0.04	2.5	23333	3500	0.15	3.75
	EQY	0.058	0.025	2.25	45000	3500	0.08	3.11
9	EQX	0.061	0.017	3.6	19520	2745	0.141	8.3
	EQY	0.049	0.015	3.3	22639	2745	0.121	8.1
10	EQX	0.16	0.039	4.1	26471	3552	0.134	3.4
	EQY	0.164	0.029	5.6	37718	3552	0.094	3.2
11	EQX	0.201	0.055	3.7	14515	5031.7	0.347	6.3
	EQY	0.119	0.046	2.6	16511	5031.7	0.305	6.6
12	EQX	0.094	0.028	3.4	25232	6700	0.266	9.3
	EQY	0.12	0.028	4.3	27356	6700	0.245	8.57

Table 5.4: Comparison of displacement ductility (user defined hinge result)

Model	EQ Push	μ	μd
1	EQX	5.7	4.63
	EQY	4.26	4.38
2	EQX	5.04	4.37
	EQY	3.67	4.48
3	EQX	3.18	3.41
	EQY	2.31	3.36
4	EQX	7.67	3.58
	EQY	5.51	2.54
5	EQX	3.13	9.45
	EQY	2.99	12.99
6	EQX	5.2	5.34
	EQY	4.62	5.82
7	EQX	5.08	6.74
	EQY	4.97	4.88
8	EQX	2.5	3.75
	EQY	2.25	3.11
9	EQX	3.59	8.27
	EQY	3.27	8.08
10	EQX	4.11	3.44
	EQY	5.65	3.25
11	EQX	3.66	6.3
	EQY	2.58	6.62
12	EQX	3.36	9.29
	EQY	4.29	8.57

Table 5.5: Displacement ductility (default hinge)

Model	EQ Push	Δu	Δy	μ
1	EQX	0.2	0.049	4.05
	EQY	0.186	0.046	4.5
2	EQX	0.185	0.032	5.63
	EQY	0.28	0.035	8
3	EQX	0.309	0.03	6
	EQY	0.231	0.038	4.8
4	EQX	0.106	0.024	4.26
	EQY	0.162	0.037	4.29
5	EQX	0.185	0.076	7.89
	EQY	0.2	0.061	3.28
6	EQX	0.19	0.041	4.61
	EQY	0.225	0.046	4.78
7	EQX	0.251	0.047	5.32
	EQY	0.211	0.065	3.23
8	EQX	0.18	0.07	2.71
	EQY	0.15	0.065	2.31
9	EQX	0.085	0.027	2.96
	EQY	0.106	0.016	4.5
10	EQX	0.2	0.03	6.67
	EQY	0.19	0.027	7.04
11	EQX	0.251	0.044	5.68
	EQY	0.251	0.031	5
12	EQX	0.1	0.041	2.42
	EQY	0.12	0.034	3.43

5.4 Overstrength factor (Ω)

The structure has finally reached it strength and deformation capacity. The additional strength beyond the design strength is called the overstrength. Numerically,

Overstrength factor (Ω) = apparent strength/design strength

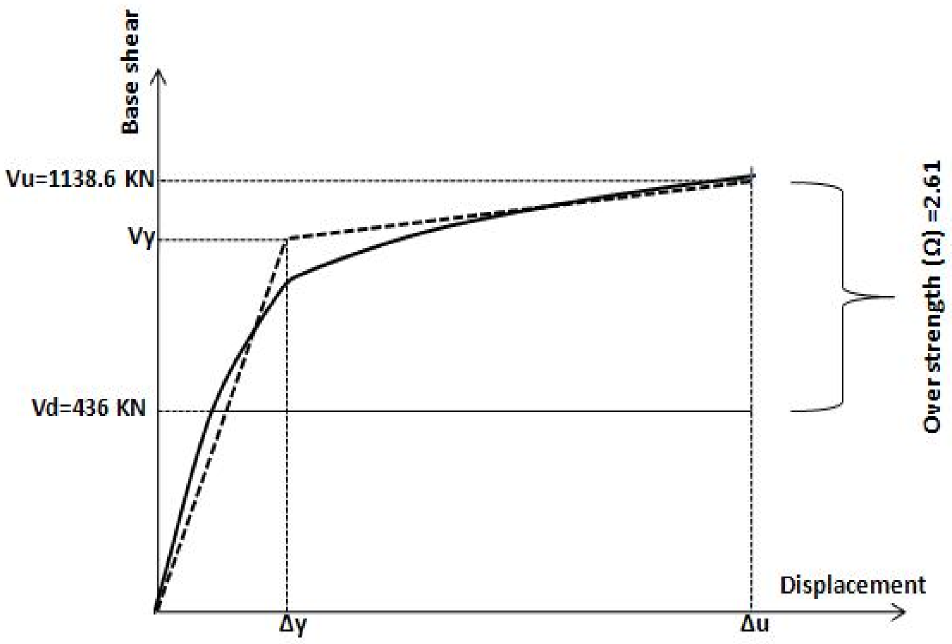


Figure 5.4. Calculation of over strength factor

From above figure, it is clear that, $\Omega = (V_u/V_y) \times (V_y/V_d)$

Vu/Vy represents the redundancy factor and Vy/Vd represent overstrength factor.

If both factor (overstrength and redundancy) considered at once as a overstrength factor then, $\Omega = V_u/V_d$. This concept is used to calculate overstrength factor in whole study. In model 1, ultimate base shear is 1138.60 KN and design base shear is 436 KN. Overstrength factor (Ω) = 1138.60 /436 = 2.61. Ultimate base shear Vu, design base shear Vd and overstrength of the study buildings are presented in Table 5.6

Table 5.6: Overstrength factor based on default hinge

Model	EQ Push	Vu	Vd	Ω
1	EQX	915.54	436	2.1
	EQY	1007.09	436	2.31
2	EQX	915.44	457.7	2
	EQY	735.3	457.7	1.61
3	EQX	654.56	379.6	1.72
	EQY	730.1	379.6	1.92
4	EQX	439.82	313.3	1.4
	EQY	451.3	313.3	1.44
5	EQX	1428.58	798.8	1.79
	EQY	865.43	798.8	1.08
6	EQX	864.82	455.2	1.9
	EQY	819.31	455.2	1.8
7	EQX	425.6	390.6	1.09
	EQY	677.56	390.6	1.73
8	EQX	505.7	266.2	1.9
	EQY	479.09	266.2	1.8
9	EQX	281.31	274.5	1.02
	EQY	334.7	274.5	1.22
10	EQX	488	355.2	1.37
	EQY	499	355.2	1.4
11	EQX	607.46	503.2	1.21
	EQY	601.11	503.2	1.19
12	EQX	991.6	670	1.48
	EQY	1300	670	1.94

5.5 Response reduction factor (R)

Response reduction is used to scale down the elastic response of the structure. Numerically, R = Overstrength factor × Redundancy factor × Ductility factor

But, in this study, overstrength and redundancy is considered as overstrength factor.

Finally, Force reduction factor (2R) = Overstrength factor × ductility reduction factor

$R = (\Omega \times R\mu)/2$

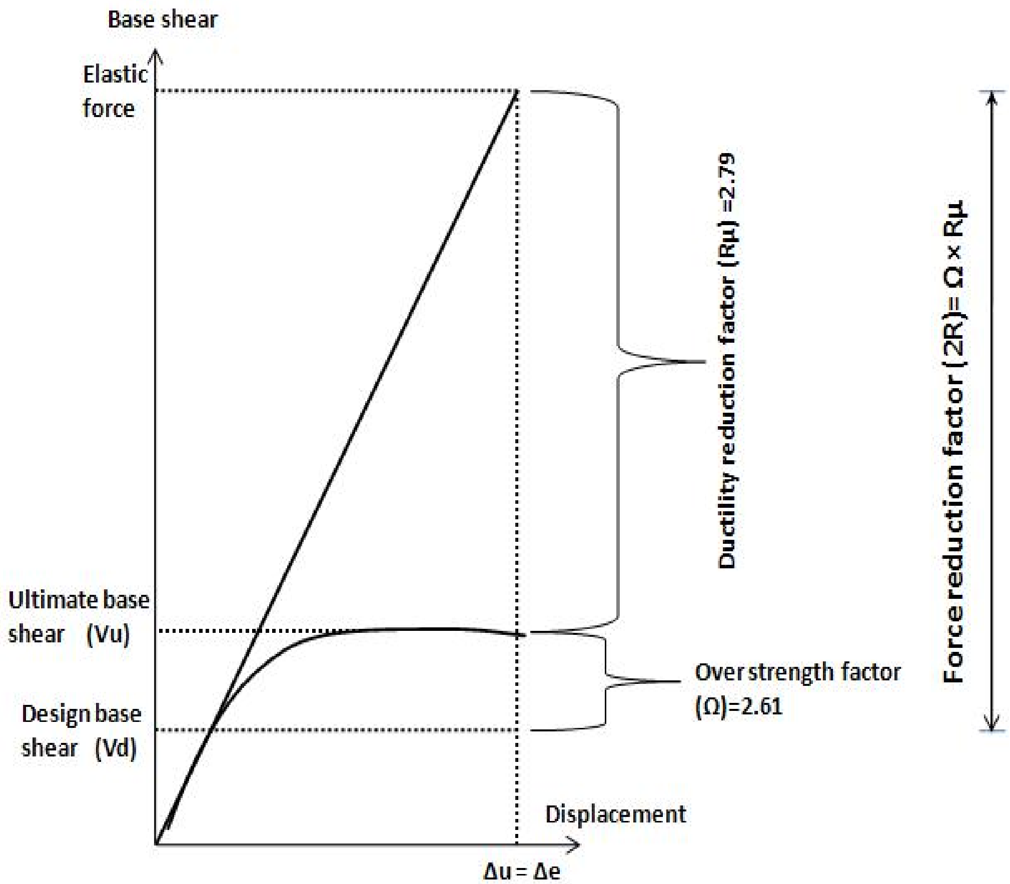


Figure 5.5. Representation of Force Reduction Factor

For an example, model 1 have overstrength factor (Ω) = 2.61 and ductility factor (Rμ) = 2.79 (in EQX condition). In this case, response reduction factor:
R = (2.61X 2.79) /2 = 3.64. All other calculations are presented in the tabular form.

The response reduction factor of the study buildings based on user defined hinges is presented in Table 5.7. Response reduction factor based on default hinge results are presented in Table 5.8. Comparisons of Rμ, Ω, and R, based on default and user-defined hinges are presented in Table 5.9 and final R value based on minimum R among two loading condition (EQX and EQY) is presented in Table 5.10.

Relation of R value with column beam capacity ratio (C/B) and load path is shown in Table 5.11. Similarly, relation of R with satisfied C/B capacity ration and complete load path is presented in Table 5.12. Relation of R with C/B ratio satisfied condition and incomplete load path is presented in Table 5.13.Relation of C/B capacity ratio, Rμ, overstrength factor Ω, and response reduction factor is shown in Table 5.14. Comparison of high Rμ and less Ω and response reduction factor >= 5 is presented in Table 5.15. Comparison of R with less Rμ and high overstrength is shown in Table 5.16. Finally, performance of the study buildings is given in Table 5.17

Table 5.7: Response reduction factor based on user defined hinge

Modal	EQ Push	Ω	Rμ	2R	R
1	EQX	2.61	2.79	7.28	3.64
	EQY	3.26	2.54	8.29	4.14
2	EQX	3.48	2.68	9.33	4.66
	EQY	3.15	2.22	6.99	3.50
3	EQX	3.62	2.05	7.44	3.72
	EQY	3.6	1.63	5.87	2.94
4	EQX	4.17	2.11	8.78	4.39
	EQY	4.14	1.66	6.85	3.43
5	EQX	1.7	2.51	4.26	2.13
	EQY	1.08	2.57	2.78	1.39
6	EQX	2.4	3.46	8.30	4.15
	EQY	2.3	3.10	7.13	3.57
7	EQX	2.1	3.63	7.62	3.81
	EQY	2.36	3.46	8.18	4.09
8	EQX	3.24	1.66	5.39	2.70
	EQY	3.8	1.55	5.90	2.95
9	EQX	1.56	2.11	3.29	1.65
	EQY	1.44	1.97	2.84	1.42
10	EQX	3.74	2.37	8.87	4.44
	EQY	4.28	1.99	8.53	4.26
11	EQX	1.85	2.51	4.65	2.33
	EQY	1.8	1.79	3.23	1.61
12	EQX	2.06	2.45	5.05	2.52
	EQY	1.94	3.02	5.86	2.93

Table 5.8: Response reduction factor based on default hinge

Model	EQ Push	Ω	Rμ	2R	R
1	EQX	2.1	2.57	5.39	2.70
	EQY	2.31	2.80	6.47	3.23
2	EQX	2	3.32	6.63	3.32
	EQY	1.61	4.50	7.25	3.62
3	EQX	1.72	3.43	5.90	2.95
	EQY	1.92	2.85	5.46	2.73
4	EQX	1.4	2.40	3.36	1.68
	EQY	1.44	2.41	3.47	1.74
5	EQX	1.79	6.41	11.48	5.74
	EQY	1.08	2.79	3.01	1.51
6	EQX	1.9	3.22	6.11	3.06
	EQY	1.8	3.32	5.98	2.99
7	EQX	1.09	3.84	4.18	2.09
	EQY	1.73	2.47	4.27	2.13
8	EQX	1.9	1.76	3.34	1.67
	EQY	1.8	1.58	2.84	1.42
9	EQX	1.02	1.84	1.88	0.94
	EQY	1.22	2.50	3.05	1.53
10	EQX	1.37	3.51	4.81	2.41
	EQY	1.4	3.67	5.14	2.57
11	EQX	1.21	3.67	4.45	2.22
	EQY	1.19	3.29	3.91	1.96
12	EQX	1.48	1.87	2.77	1.39
	EQY	1.75	2.49	4.36	2.18

Table 5.9: Comparison of Rμ, Ω and R of Study Buildings

Modal	EQ Push	Rμ		Ω		R	
		default	user	default	user	default	user hinge
1	EQX	2.57	2.79	2.1	2.61	2.7	3.64
	EQY	2.8	2.54	2.31	3.26	3.23	4.14
2	EQX	3.32	2.68	2	3.48	3.32	4.66
	EQY	4.5	2.22	1.61	3.15	3.62	3.5
3	EQX	3.43	2.05	1.72	3.62	2.95	3.72
	EQY	2.85	1.63	1.92	3.6	2.73	2.94
4	EQX	2.4	2.11	1.4	4.17	1.68	4.39
	EQY	2.41	1.66	1.44	4.14	1.74	3.43
5	EQX	6.41	2.51	1.79	1.7	5.74	2.13
	EQY	2.79	2.57	1.08	1.08	1.51	1.39
6	EQX	3.22	3.46	1.9	2.4	3.06	4.15
	EQY	3.32	3.1	1.8	2.3	2.99	3.57
7	EQX	3.84	3.63	1.09	2.1	2.09	3.81
	EQY	2.47	3.46	1.73	2.36	2.13	4.09
8	EQX	1.76	1.66	1.9	3.24	1.67	2.7
	EQY	1.58	1.55	1.8	3.8	1.42	2.95
9	EQX	1.84	2.11	1.02	1.56	0.94	1.65
	EQY	2.5	1.97	1.22	1.44	1.53	1.42
10	EQX	3.51	2.37	1.37	3.74	2.41	4.44
	EQY	3.67	1.99	1.4	4.28	2.57	4.26
11	EQX	3.67	2.51	1.21	1.85	2.22	2.33
	EQY	3.29	1.79	1.19	1.8	1.96	1.61
12	EQX	1.87	2.45	1.48	2.06	1.39	2.52
	EQY	2.49	3.02	1.75	1.94	2.18	2.93

Conclusion

In most of the cases, calculation of R-value using default hinge properties has higher value than default hinge properties.

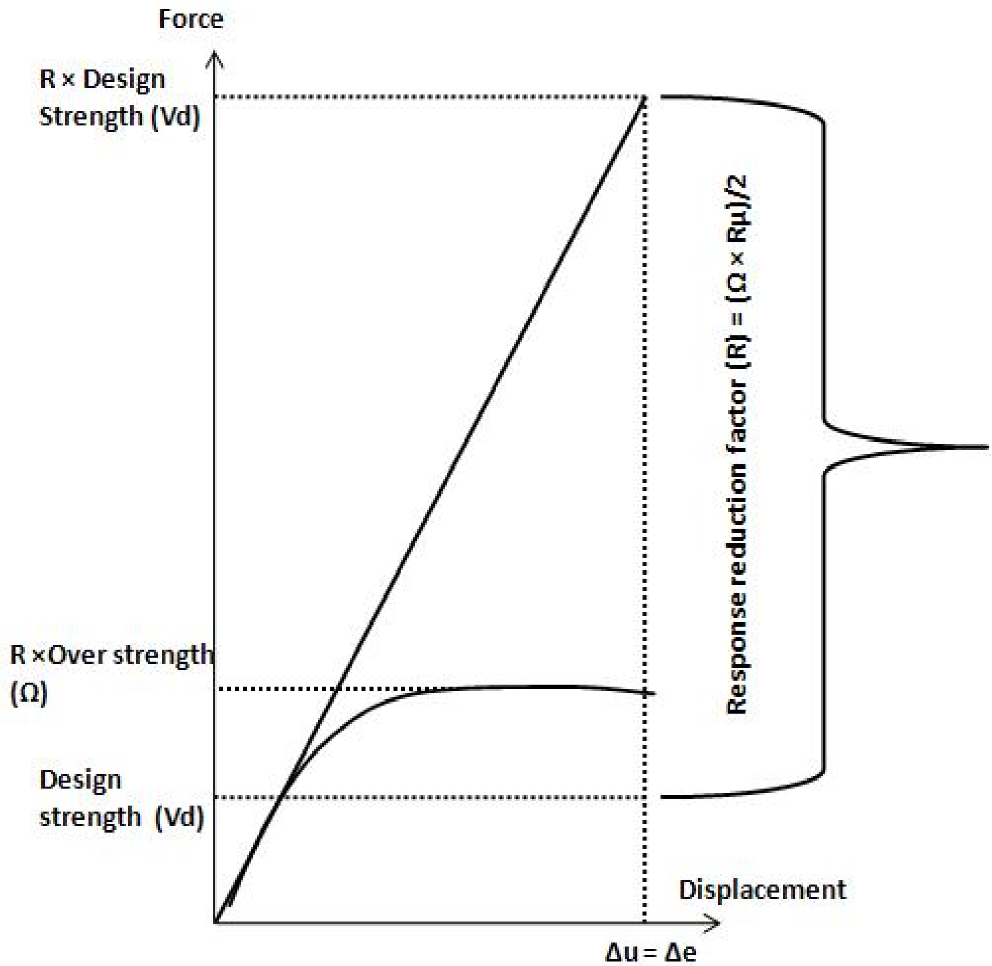


Figure 5.6. Design strength, over strength and response reduction factor

Table 5.10: Final R value of Study Buildings

Model	**R** *max*	**R** *min*
1	4.14	3.64
2	4.66	3.5
3	3.72	2.94
4	4.39	3.43
5	2.13	1.39
6	4.15	3.57
7	4.09	3.81
8	2.95	2.7
9	1.65	1.42
10	4.44	4.26
11	2.33	1.61
12	2.93	2.52

Conclusion

Average R-value of study building is 3.18

Table 5.11: Relation of R with C/B ratio and load path

Model	C/B capacity ratio		Load path		R	
	C/B>1.1	C/B<1.1	Complete	Incomplete	**R** *max*	**R** *min*
1	√		√		4.14	3.64
2	√		√		4.66	3.5
3	√			√	3.72	2.94
4	√		√		4.39	3.43
5		√	√		2.13	1.39
6	√		√		4.15	3.57
7	√		√		4.09	3.81
8	√			√	2.95	2.7
9		√		√	1.65	1.42
10	√		√		4.44	4.26
11		√	√		2.33	1.61
12		√	√		2.93	2.52

Table 5.12: Relation of R with satisfied C/B ratio and complete load path.

Model	C/B capacity	Load path	R	
	C/B>1.1	Complete	**R** *max*	**R** *min*
1	√	√	4.14	3.64
2	√	√	4.66	3.5
4	√	√	4.39	3.43
6	√	√	4.15	3.57
7	√	√	4.09	3.81
10	√	√	4.44	4.26

Table 5.13: Relation of R with C/B ratio and load path

Model	C/B capacity	Load path	**R**	
	C/B>1.1	Incomplete	**R** *max*	**R** *min*
3	√	√	3.72	2.94
8	√	√	2.95	2.7

Conclusion

Building with Column/beam capacity ratio is satisfied and complete load path have higher R value than building with incomplete load path.

Table 5.14: Relation of C/B ratio, µ, Rµ and R

Model	C/B > 1.1	C/B <1.1	ductility Supply(µ)	ductility Demand (µ)	comparison of ductility	R	
						R *max*	**R** *min*
1	√		5.7	4.63	µs > µd	4.14	3.64
2	√		5.04	4.37	µs > µd	4.66	3.5
3	√		3.18	3.41	µs~µd	3.72	2.94
4	√		7.67	3.58	µs > µd	4.39	3.43
5		√	3.13	9.45	µs < µd	2.13	1.39
6	√		5.2	5.34	µs~µd	4.15	3.57
7	√		4.97	4.88	µs > µd	4.09	3.81
8	√		2.5	3.75	µs < µd	2.95	2.7
9		√	3.59	8.27	µs < µd	1.65	1.42
10	√		4.11	3.44	µs > µd	4.44	4.26
11		√	3.66	6.3	µs < µd	2.33	1.61
12		√	3.36	9.29	µs < µd	2.93	2.52

Conclusion

Building with sufficient Column/ beam capacity ratio satisfies displacement ductility supply ≥ displacement ductility demand.

Table 5.15: Comparison with high Rμ and less Ω

Model	Loading	Rμ	Ω	R
6	EQX	3.46	2.4	4.15
	EQY	3.1	2.3	3.57
7	EQX	3.63	2.1	3.81
	EQY	3.46	2.36	4.09

Table 5.16: Comparison with less Rμ and high Ω

Model	Push	Rμ	Ω	R
2	EQX	2.68	3.48	4.66
	EQY	2.22	3.15	3.5
3	EQX	2.05	3.62	3.72
	EQY	1.63	3.6	3.5
4	EQX	2.11	4.17	4.39
	EQY	1.66	4.14	3.43
10	EQX	2.37	3.74	4.44
	EQY	1.99	4.28	4.26

Conclusion

Making structure stronger than design value we can reduce the ductility demand of the structure.

Table 5.17: Performance of study buildings

Modal	EQ	Δu	Δy	Δp	Vd	Vy	Vp	Vu
1	EQX	0.165	0.029	0.015	436.0	840	497.85	1138.6
1	EQY	0.149	0.035	0.019	436.0	1180	569.75	1421.0
2	EQX	0.111	0.022	0.011	457.7	1200	534.87	1593.8
2	EQY	0.117	0.032	0.016	457.7	1236	555.02	1441.4
3	EQX	0.076	0.024	0.014	379.6	1100	640.33	1372.8
3	EQY	0.062	0.027	0.016	379.6	1090	656.46	1367.6
4	EQX	0.200	0.026	0.012	313.3	927	423.14	1307.7
4	EQY	0.196	0.036	0.012	313.3	1077	409.14	1297.6
5	EQX	0.139	0.045	0.019	798.8	855	776.93	1354.9
5	EQY	0.087	0.029	0.046	798.8	633	688.51	863.8
6	EQX	0.198	0.038	0.029	455.2	880	574.37	1091.9
6	EQY	0.199	0.043	0.034	455.2	800	556.40	1048.4
7	EQX	0.223	0.044	0.040	390.6	600	442.33	818.5
7	EQY	0.199	0.040	0.033	390.6	693	490.47	920.3
8	EQX	0.10	0.04	0.006	350	450	571.25	1134
8	EQY	0.058	0.025	0.006	350	500	549.28	1359
9	EQX	0.061	0.017	0.018	274.5	383	338.79	429.3
9	EQY	0.049	0.015	0.015	274.5	360	339.36	395.1
10	EQX	0.160	0.039	0.018	355.2	1031	474.48	1328.1
10	EQY	0.164	0.029	0.013	355.2	1240	491.74	1519.0
11	EQX	0.201	0.043	0.048	503.2	722	567.64	929.3
11	EQY	0.119	0.041	0.042	503.2	800	621.22	903.2
12	EQX	0.094	0.028	0.033	670.0	1120	975.00	1380.0
12	EQY	0.120	0.036	0.044	670.0	980	626.00	1300.0

5.6 Re-detailing the model 5

When Model 5 is detailed as per ductile requirement, then the structure meet the requirement of Strong column weak beam philosophy (annex 4.1). The results and the comparison of various parameters are presented in Table 5.18

Table 5.18: Comparison of Model 5 with Ductile Detailing

Model5	C/B < 1.1		C/B>1.1	
EQ push	EQX	EQY	EQX	EQY
K	18987	21202	20000	22000
Vu	1354.9	863.8	1800	1700
Vd	798.8	798.8	798.8	798.8
Velastic	7988	7988	7988	7988
Δu	0.139	0.087	0.62	0.376
Δy	0.045	0.029	0.08	0.076
Δe	0.42	0.37	0.4	0.36
µ	3.1	3	7.75	4.947368421
µd	9.33	12.75	5	4.736842105
Ductility	µ < µd	µ < µd	µ > µd	µ > µd
Rµ	2.65	2.58	4.15	3.94
Ω	1.696169254	1.081372058	2.25	2.128192288
2R	4.494848523	2.78993991	9.3375	8.385077615
R	2.247424262	1.394969955	4.66875	4.192538807

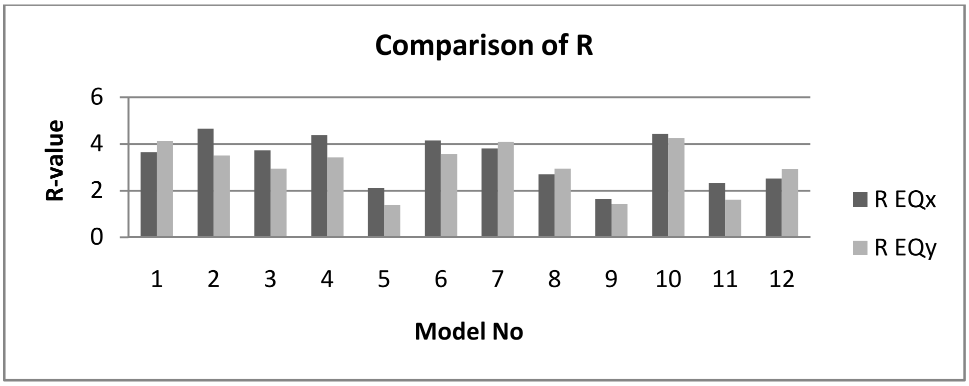


Figure 5.7. Comparison of R value in X and Y directions

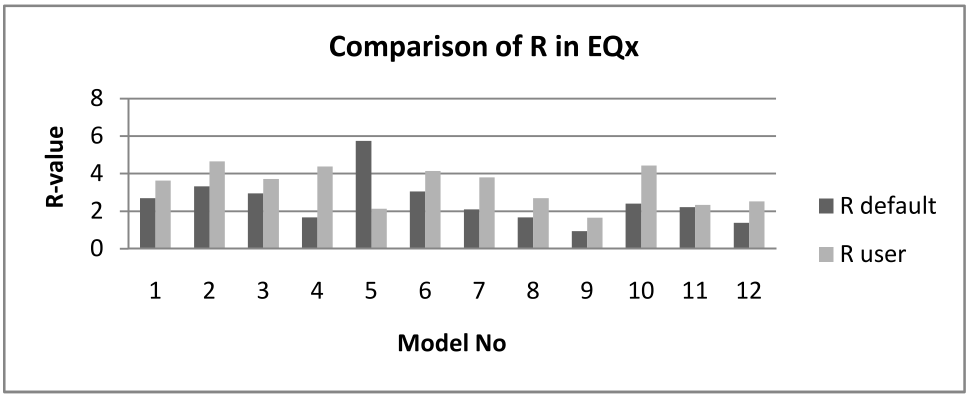


Figure 5.8. Comparison of R in EQX loading

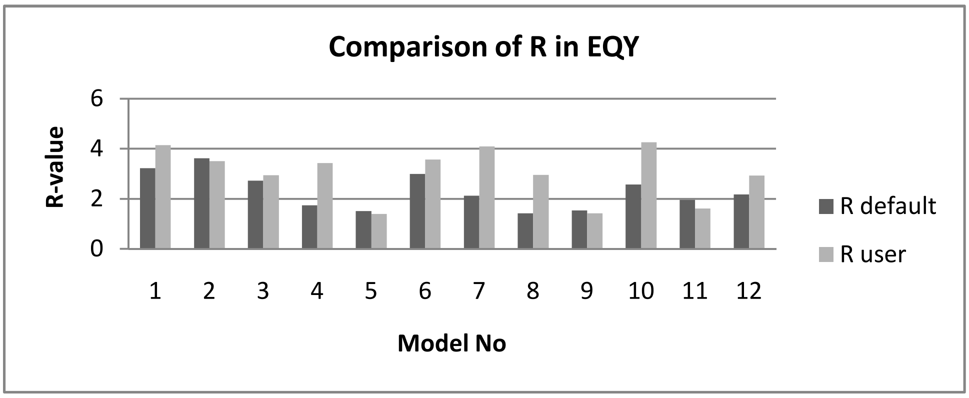


Figure 5.9. Comparison of R in EQY loading

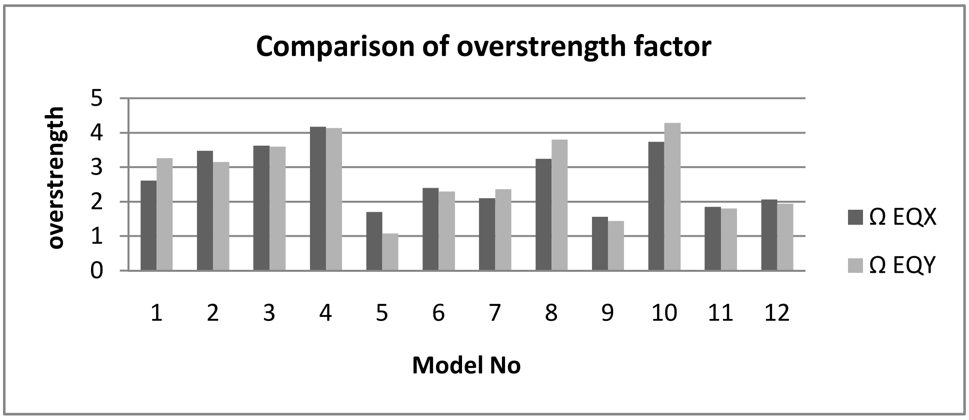


Figure 5.10. Comparison of overstrength factor

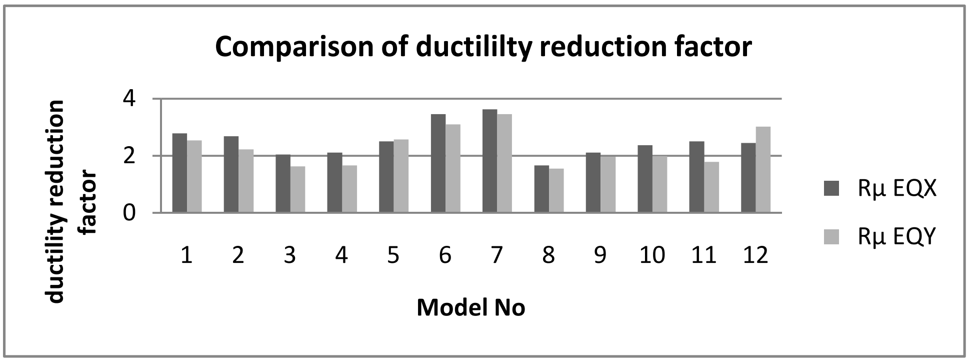


Figure 5.11. Comparison of ductility reduction factor

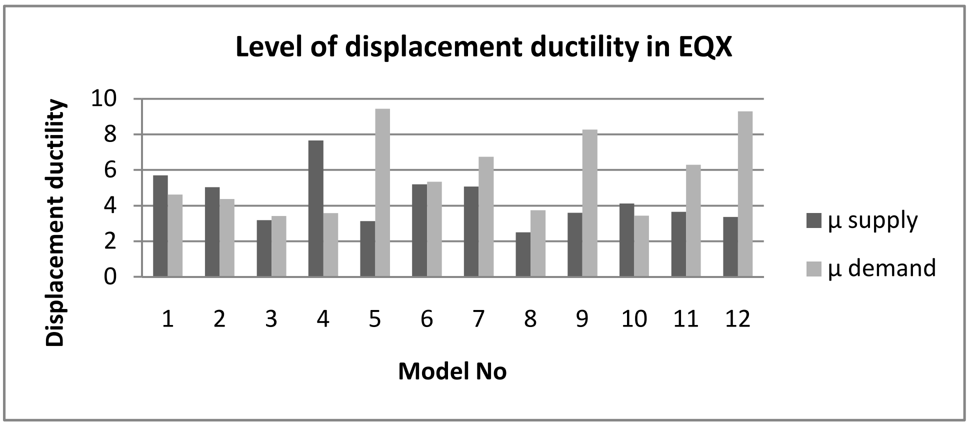


Figure 5.12. Comparison of displacement ductility in EQX loading

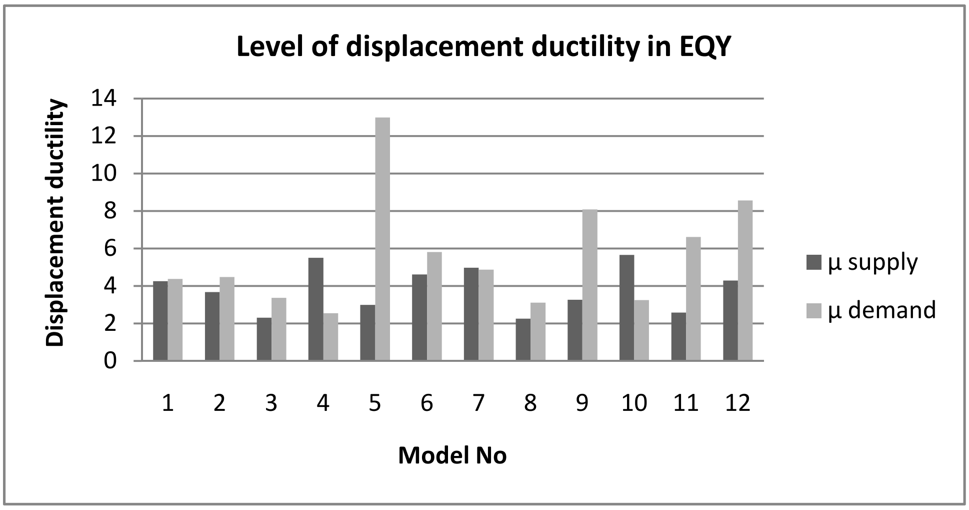


Figure 5.13. Comparison of displacement ductility in EQY loading

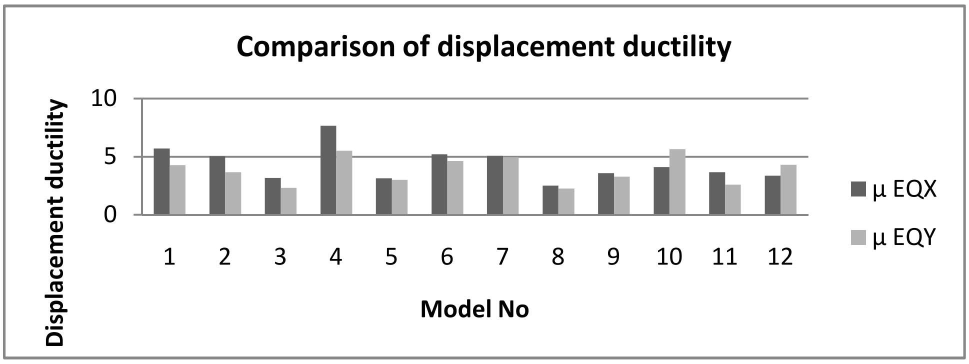


Figure 5.14. Comparison of displacement ductility

5.7 Average R-value on different structural conditions

- When column/ beam capacity ratio is satisfied and load path is complete, R = 4.0
- When column/ beam capacity ratio is satisfied and load path is incomplete, R = 3.08
- When column/ beam capacity ratio is not satisfied and load path is complete, R = 2.16
- When column/ beam capacity ratio is not satisfied and load path is incomplete, R = 1.54

6 CONCLUSION AND RECOMMENDATION

6.1 Conclusions

Calculation of response reduction factor using non-linear pushover analysis as described in methodology section of this dissertation has been done for 12 typical RC buildings in Kathmandu Valley. The obtained Response Reduction Factors are further analyzed and compared with different structural parameters of the buildings. Followings are the conclusions arrived at from the analysis and interpretation of the results.

- R Value of all buildings are less than 5
- Frames which do not meet the criteria of “Strong column-Weak beam” do not meet the high ductility demand required by special moment resisting frame. The response reduction factor obtained from non-linear pushover analysis was too less than assumed in those types of buildings.
- In the Buildings with column/beam capacity ratio is satisfied and having complete load path, got higher R value than buildings with incomplete load path.
- Buildings having ductility supply ≥ ductility demand satisfies column/beam capacity ratio..
- If the over strength factor is more the total response reduction factor can be achieved even the ductility factor is less.
- If the Building having C/B ratio < 1.1 has changed to meet the condition of C/B > 1.1. The R value is significantly increased which indicates that the major factor for Response Reduction Factor is C/B ratio.

6.2 Limitations of the Study

- Only 12 buildings were randomly selected for the study, most of the buildings are irregular in plan.
- Only nonlinear static pushover analysis was done
- All the selected buildings did not satisfy the requirement of strong column weak beam philosophy
- Only the effect of horizontal force were considered in analysis
- Soil structure interaction was not considered
- Only bare frame analysis was done
- User defined hinge was used only in beam elements

6.3 Recommendations

To achieve higher value of R, Buildings must have to meet Strong column weak beam philosophy and complete load path. Therefore, it is recommended to meet strong-column weak –beam principle to consider the RC building as special moment resisting frame building.

6.4 Recommendations for further study

- Determination of R-value of the buildings by using non-linear time history analysis
- Non-linear analysis of the building considering the effect of infill wall
- Effect of soil structure interaction on response reduction

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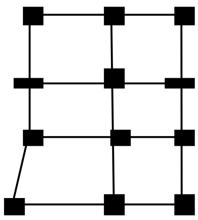
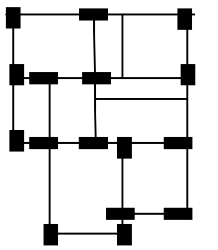
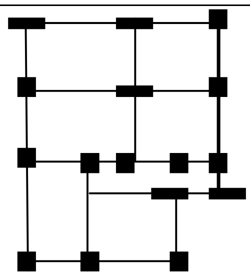
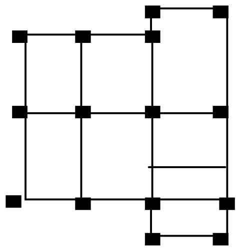
ANNEXES

Annex 1.1. Plan of Study Buildings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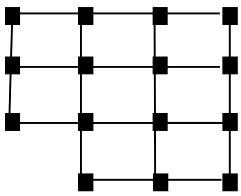	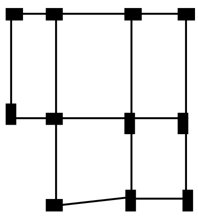	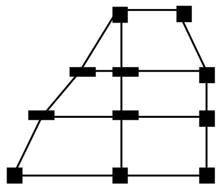	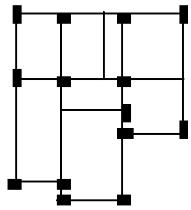
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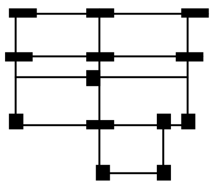
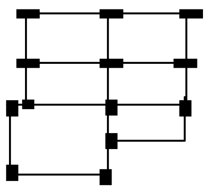
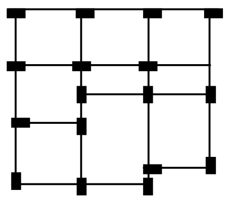
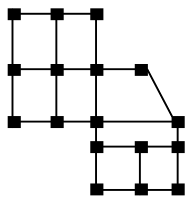
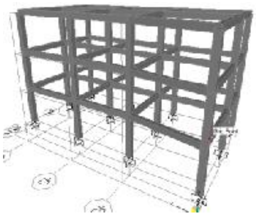
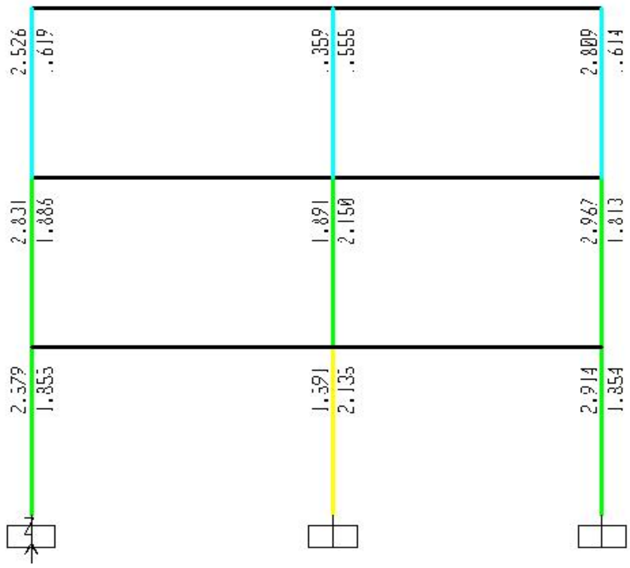
			
Modal 9	Modal 10	Modal 11	Modal 12

Annex 1.2. Area of Study Building in different Floor

Area (m^2)					
Modal	GF	FF	SF	TF	TF
1	109.8	105.8	-	-	102.3
2	121.6	120.6	-	-	76.5
3	116.0	111.9	-	-	89.3
4	108.0	104.1	-	-	28.9
5	155.5	148.1	148.1	148.1	89.0
6	90.0	90.0	90.0		69.7
7	60.4	65.0	65.0	65.0	13.4
8	112.8	97.3	-	-	46.9
9	95.5	81.8	-	-	25.3
10	99.5	96.7	-	-	54.5
11	124.1	124.1	122.2	-	48.7
12	107.0	107.0	107.0	107.0	70.0

Annex 1.3. Capacity check

Model	Column/beam capacity ratio>1.1
	 Check the ratio
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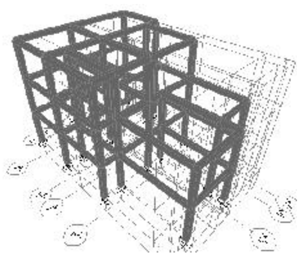
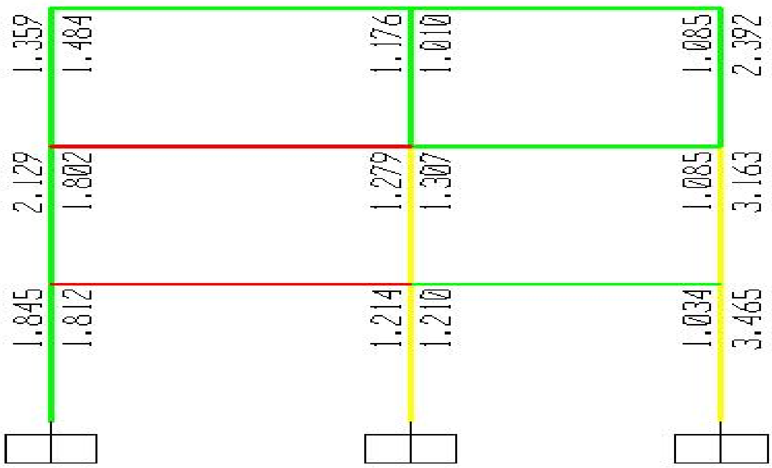
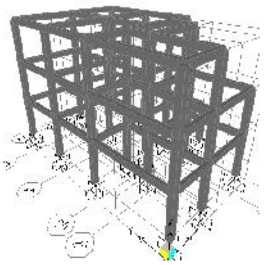
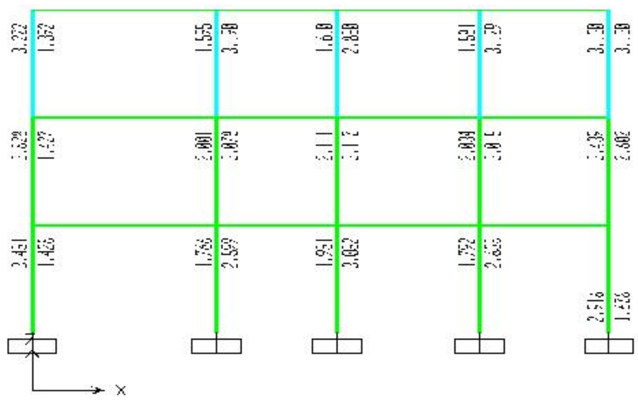
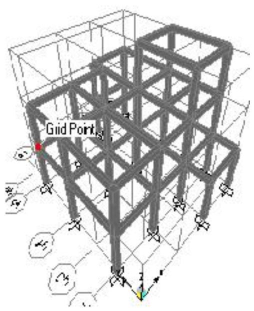
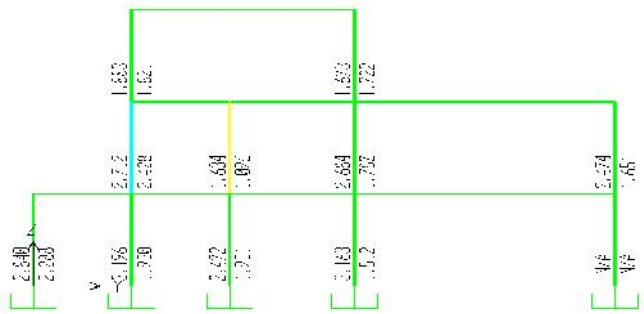
	
Model 2	

	
Model 3	

Model	Column/Beam capacity ratio >1.1
	
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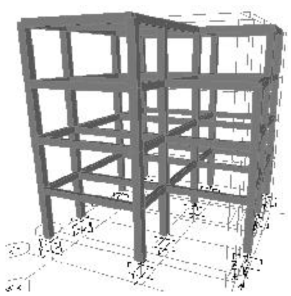
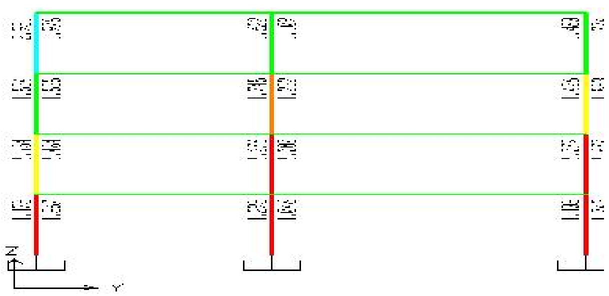
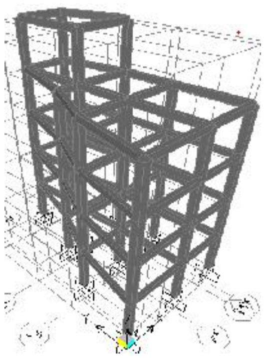
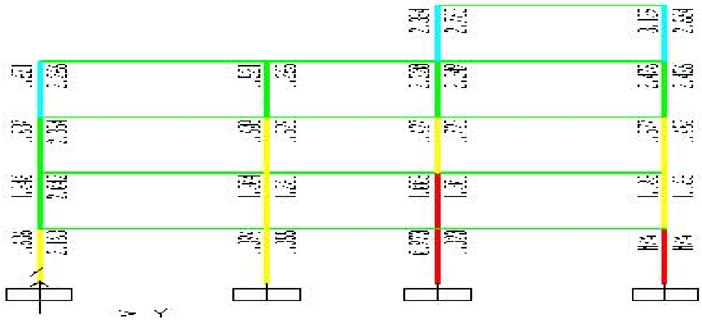
	
Model 6	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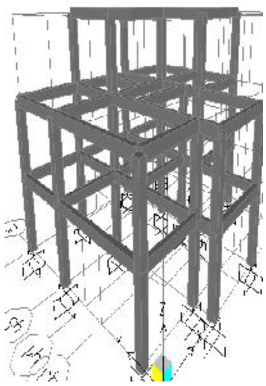
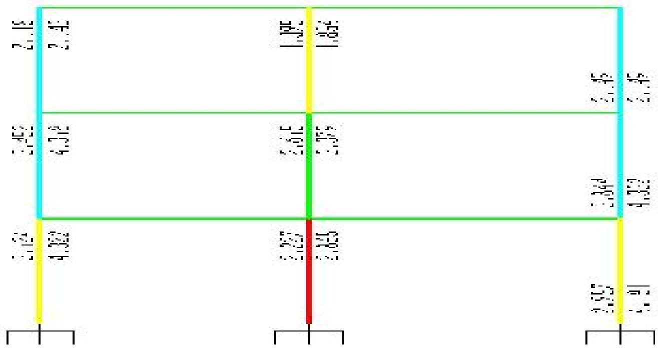
Model	Column/Beam capacity >1.1
	
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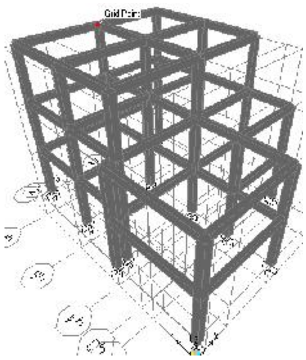	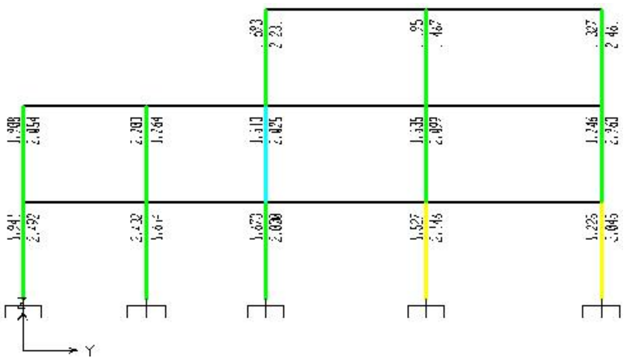
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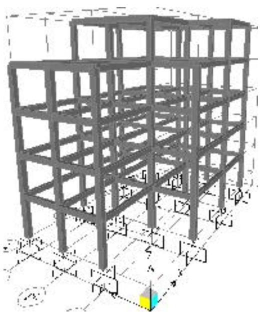
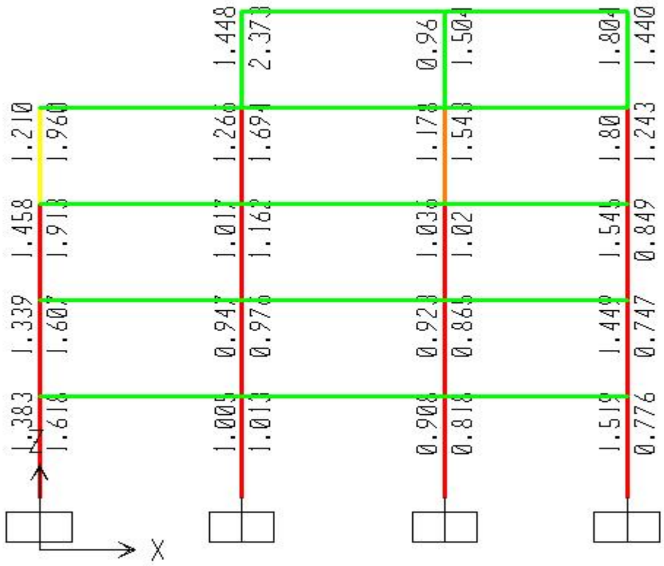
Model	Column/Beam capacity < 1.1
	
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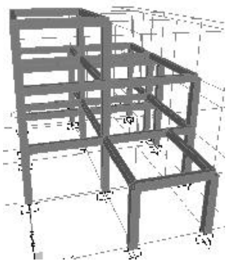	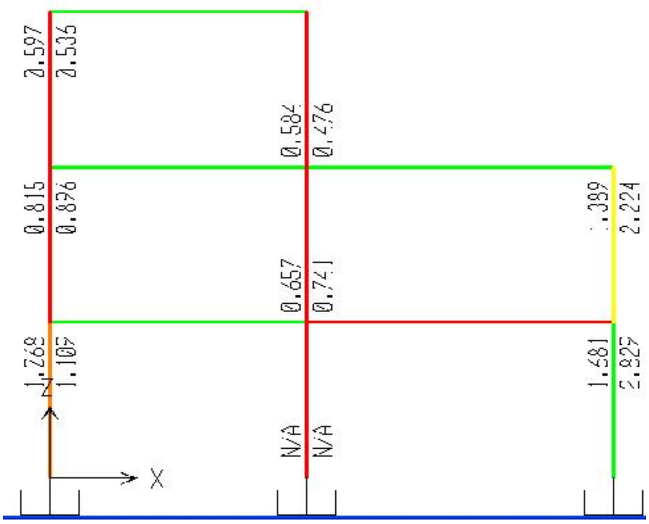
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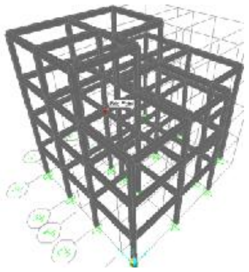	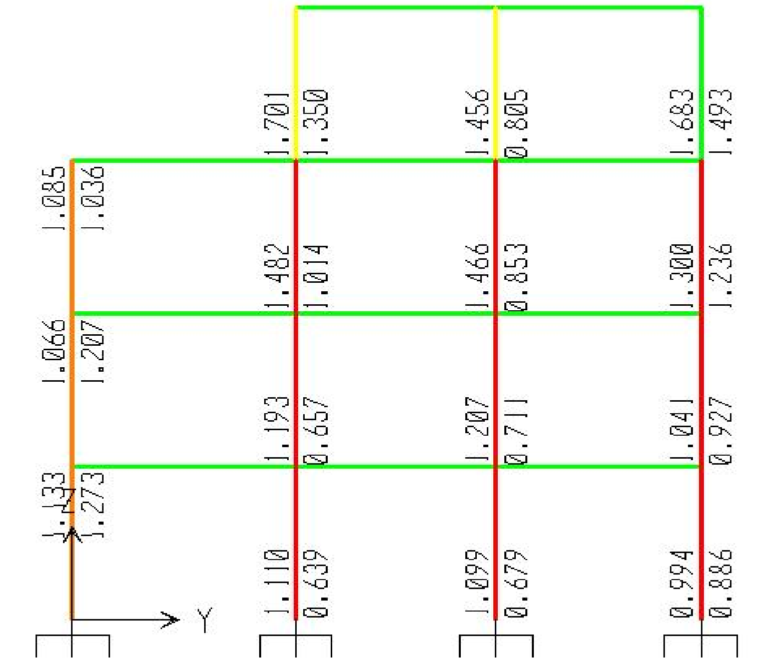
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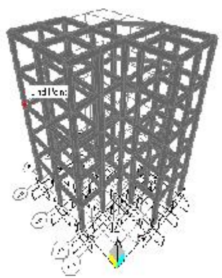
	
Model 12	

Annex 2.1. Results from Static Pushover Analysis

(USER DEFINED PLASTIC HINGE RESULT)

Model 1X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0.00015	0	174	0	0	0	0	0	0	0	174
1	0.00288	92.99	173	1	0	0	0	0	0	0	174
2	0.01666	539.6	136	38	0	0	0	0	0	0	174
3	0.03471	835.4	124	36	14	0	0	0	0	0	174
4	0.04188	887.3	110	43	21	0	0	0	0	0	174
5	0.07369	990.0	99	32	37	5	1	0	0	0	174
6	0.10987	1061.	90	31	29	12	12	0	0	0	174
7	0.14053	1107.	86	32	20	6	27	3	0	0	174
8	0.16539	1138.	86	31	15	2	32	8	0	0	174

Model 1Y

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0	0	174	0	0	0	0	0	0	0	174
1	0.00698	209.9	173	1	0	0	0	0	0	0	174
2	0.02479	716.5	132	42	0	0	0	0	0	0	174
3	0.04067	963.0	117	45	12	0	0	0	0	0	174
4	0.07194	1210.	103	43	28	0	0	0	0	0	174
5	0.08126	1254.	99	45	27	3	0	0	0	0	174
6	0.11433	1340.	86	41	38	1	8	0	0	0	174

7	0.14433	1412.	82	41	30	11	8	2	0	0	174
8	0.14876	1420.	82	40	30	8	7	7	0	0	174
9	0.14913	1421.	82	40	30	8	7	7	0	0	174
10	0.14913	1421.	82	40	30	8	7	7	0	0	174

Model 2X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	-0.0008	0	216	0	0	0	0	0	0	0	216
1	-0.0007	0.531	215	1	0	0	0	0	0	0	216
3	0.02115	1007.	156	56	4	0	0	0	0	0	216
5	0.02876	1196.	139	72	5	0	0	0	0	0	216
7	0.05021	1411.	115	68	32	0	1	0	0	0	216
9	0.06554	1485.	96	69	48	2	1	0	0	0	216
11	0.09100	1552.	90	62	55	6	3	0	0	0	216
13	0.11079	1593.	85	58	56	9	5	3	0	0	216

Model 2 Y

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0.00007	0	216	0	0	0	0	0	0	0	216
1	0.00317	131.7	215	1	0	0	0	0	0	0	216
2	0.02898	955.4	154	54	8	0	0	0	0	0	216
3	0.04197	1186.	138	59	19	0	0	0	0	0	216
4	0.04895	1247.	129	67	20	0	0	0	0	0	216
5	0.07494	1342.	123	45	48	0	0	0	0	0	216
6	0.08651	1377.	120	37	56	3	0	0	0	0	216

7	0.08651	1377.	120	37	56	3	0	0	0	0	216
8	0.10195	1410.	117	27	59	12	1	0	0	0	216
9	0.10195	1410.	117	27	59	12	1	0	0	0	216
10	0.10766	1422.	117	25	59	13	2	0	0	0	216
11	0.10766	1422.	117	25	59	13	2	0	0	0	216
12	0.11732	1441.	116	24	58	14	4	0	0	0	216
13	0.11732	1441.	116	24	58	14	4	0	0	0	216

Model 3X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0.000051	0	230	0	0	0	0	0	0	0	230
1	0.000704	37.341	229	1	0	0	0	0	0	0	230
3	0.017899	835.22	178	52	0	0	0	0	0	0	230
5	0.036041	1178.7	153	64	13	0	0	0	0	0	230
7	0.048616	1282.7	139	59	32	0	0	0	0	0	230
9	0.063312	1342.3	125	64	41	0	0	0	0	0	230
11	0.076355	1372.7	123	54	50	3	0	0	0	0	230

Model 3Y

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0.00017	0	230	0	0	0	0	0	0	0	230
1	0.00051	15.93	229	1	0	0	0	0	0	0	230
3	0.01884	782.0	176	54	0	0	0	0	0	0	230
5	0.03936	1190.	151	55	24	0	0	0	0	0	230
7	0.04496	1261.	141	61	28	0	0	0	0	0	230

9	0.05710	1344	132	66	32	0	0	0	0	0	230
11	0.06242	1367.	126	71	33	0	0	0	0	0	230

Model 4X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0	0	162	0	0	0	0	0	0	0	162
1	0.00097	52.79	162	0	0	0	0	0	0	0	162
2	0.01773	620.0	124	38	0	0	0	0	0	0	162
3	0.03508	897.0	114	42	6	0	0	0	0	0	162
4	0.05750	1049.	110	26	26	0	0	0	0	0	162
5	0.06708	1090.	97	27	38	0	0	0	0	0	162
6	0.09448	1143.	93	25	40	4	0	0	0	0	162
7	0.12167	1189.	90	14	42	12	4	0	0	0	162
9	0.14543	1230.	87	14	32	16	13	0	0	0	162
11	0.16945	1263.	86	15	24	13	24	0	0	0	162
15	0.19953	1307.	85	12	25	4	36	0	0	0	162

Model 4Y

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0.000122	0	162	0	0	0	0	0	0	0	162
1	0.000542	15.435	161	1	0	0	0	0	0	0	162
3	0.035569	844.68	104	50	8	0	0	0	0	0	162
4	0.056205	1022.2	86	50	26	0	0	0	0	0	162
5	0.076301	1123.1	76	53	31	2	0	0	0	0	162
6	0.100715	1176.0	70	50	35	5	2	0	0	0	162

7	0.120315	1206.3	69	43	39	6	5	0	0	0	162
8	0.120315	1206.3	69	43	39	6	5	0	0	0	162
9	0.148132	1244.6	62	37	41	13	9	0	0	0	162
10	0.180141	1281.6	60	34	38	9	21	0	0	0	162
11	0.180142	1281.7	60	34	38	9	21	0	0	0	162
12	0.18016	1281.7	60	34	38	9	21	0	0	0	162
13	0.19504	1296.5	60	33	36	7	25	1	0	0	162
14	0.195972	1297.5	59	34	36	7	24	2	0	0	162

Model 5X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0.00003	0	348	0	0	0	0	0	0	0	348
1	0.01334	257.1	348	0	0	0	0	0	0	0	348
2	0.02870	548.7	301	47	0	0	0	0	0	0	348
3	0.04451	729.8	269	78	1	0	0	0	0	0	348
4	0.06172	896.8	263	59	26	0	0	0	0	0	348
5	0.07791	1043.	251	64	33	0	0	0	0	0	348
6	0.09332	1148.	236	79	33	0	0	0	0	0	348
7	0.11239	1250.	232	80	36	0	0	0	0	0	348
8	0.12777	1319.	225	69	39	15	0	0	0	0	348
9	0.13925	1354.	220	73	35	20	0	0	0	0	348
10	0.15003	1376.	216	73	37	16	6	0	0	0	348

Model 5Y

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									

0	0.00019	0	348	0	0	0	0	0	0	0	348
1	0.00410	86.21	347	1	0	0	0	0	0	0	348
3	0.01931	408.6	286	62	0	0	0	0	0	0	348
5	0.03488	607.5	264	82	2	0	0	0	0	0	348
7	0.04992	712.5	257	75	16	0	0	0	0	0	348
9	0.06599	787.0	255	56	37	0	0	0	0	0	348
11	0.08127	846.0	251	47	50	0	0	0	0	0	348
13	0.08662	863.7	249	42	57	0	0	0	0	0	348

Model 6X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0.00007	0	202	0	0	0	0	0	0	0	202
1	0.00130	27.90	201	1	0	0	0	0	0	0	202
2	0.02193	490.6	166	36	0	0	0	0	0	0	202
3	0.04330	763.0	146	47	9	0	0	0	0	0	202
5	0.07188	912.7	131	49	22	0	0	0	0	0	202
6	0.09195	954.7	125	40	37	0	0	0	0	0	202
8	0.14696	1038.	115	30	37	12	8	0	0	0	202
10	0.18793	1084.	111	30	27	10	22	2	0	0	202
11	0.18937	1085.	111	30	25	10	22	4	0	0	202
13	0.1924	1088.	111	29	25	10	23	4	0	0	202
14	0.1976	1091.	111	29	22	12	23	5	0	0	202

Model 6Y

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0	0	202	0	0	0	0	0	0	0	202

1	0.00406	77.25	201	1	0	0	0	0	0	0	202
2	0.02435	446.4	171	31	0	0	0	0	0	0	202
3	0.04525	680.1	153	40	9	0	0	0	0	0	202
4	0.06600	833.8	141	44	17	0	0	0	0	0	202
5	0.07476	868.7	137	46	19	0	0	0	0	0	202
6	0.09620	915.0	130	40	31	1	0	0	0	0	202
7	0.11626	952.9	126	31	37	8	0	0	0	0	202
8	0.13706	985.0	116	40	32	8	6	0	0	0	202
9	0.15763	1008.	113	37	38	1	13	0	0	0	202
10	0.17885	1031.	110	39	31	8	14	0	0	0	202
11	0.19879	1048.	108	37	23	17	16	1	0	0	202

Model 7X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0.00014	0	232	0	0	0	0	0	0	0	232
1	0.00117	14.26	232	0	0	0	0	0	0	0	232
2	0.02715	356.6	187	45	0	0	0	0	0	0	232
3	0.05280	528.9	159	70	3	0	0	0	0	0	232
4	0.07988	635.3	144	66	22	0	0	0	0	0	232
5	0.10297	695.4	130	69	33	0	0	0	0	0	232
6	0.13144	739.2	122	71	39	0	0	0	0	0	232
8	0.15389	763.7	112	67	49	4	0	0	0	0	232
10	0.15389	763.8	112	67	49	4	0	0	0	0	232
12	0.18892	794.9	100	70	49	9	4	0	0	0	232
14	0.22343	818.4	95	68	44	17	8	0	0	0	232

Model 7Y

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	-7.27E-	0	232	0	0	0	0	0	0	0	232
1	0.00013	2.565	232	0	0	0	0	0	0	0	232
2	0.02351	399.7	172	60	0	0	0	0	0	0	232
3	0.04987	642.9	159	56	17	0	0	0	0	0	232
4	0.06921	752.8	147	57	28	0	0	0	0	0	232
5	0.07791	778.5	140	61	31	0	0	0	0	0	232
6	0.10317	817.7	135	51	46	0	0	0	0	0	232
9	0.13104	852.9	127	41	53	11	0	0	0	0	232
11	0.15690	882.5	121	41	52	15	3	0	0	0	232
13	0.18536	909.3	116	45	46	9	16	0	0	0	232
15	0.19864	920.2	114	46	34	20	18	0	0	0	232

Model 8X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0	0	174	0	0	0	0	0	0	0	174
1	0.00186	54.14	174	0	0	0	0	0	0	0	174
2	0.02195	506.3	157	17	0	0	0	0	0	0	174
3	0.03391	737.2	139	35	0	0	0	0	0	0	174
4	0.05281	861.7	117	52	5	0	0	0	0	0	174
5	0.05281	861.7	117	52	5	0	0	0	0	0	174
6	0.05970	903.2	113	45	16	0	0	0	0	0	174
7	0.05970	903.2	113	45	16	0	0	0	0	0	174
9	0.07037	968.2	107	40	27	0	0	0	0	0	174
11	0.07037	968.2	107	40	27	0	0	0	0	0	174

13	0.09159	1085.	93	52	27	2	0	0	0	0	174
15	0.10417	1134.	88	52	32	2	0	0	0	0	174

Model 8Y

Step	**Displacement**	**BaseForce**	**AtoB**	**BtoIO**	**IOtoLS**	**LStoCP**	**CPtoC**	**CtoD**	**DtoE**	**BeyondE**	**Total**
	m	KN									
0	0	0	174	0	0	0	0	0	0	0	174
1	0.00013	28.92	174	0	0	0	0	0	0	0	174
2	0.02021	870.0	150	24	0	0	0	0	0	0	174
3	0.02619	1031.	130	44	0	0	0	0	0	0	174
4	0.03345	1146.	120	49	5	0	0	0	0	0	174
6	0.03345	1146.	120	49	5	0	0	0	0	0	174
8	0.03666	1191.	117	51	6	0	0	0	0	0	174
10	0.03666	1191.	117	51	6	0	0	0	0	0	174
12	0.05566	1357.	104	46	23	0	0	1	0	0	174
14	0.05596	1359.	103	46	24	0	0	1	0	0	174

Model 9X

Step	**Displacement**	**BaseForce**	**AtoB**	**BtoIO**	**IOtoLS**	**LStoCP**	**CPtoC**	**CtoD**	**DtoE**	**BeyondE**	**Total**
	m	KN									
0	0.00E+0	0	115	3	0	0	0	0	0	0	118
1	0.00229	50.23	114	4	0	0	0	0	0	0	118
2	0.01676	323.5	87	30	1	0	0	0	0	0	118
3	0.02158	374.5	83	33	2	0	0	0	0	0	118
4	0.02668	401.8	78	38	2	0	0	0	0	0	118
5	0.03098	406.6	77	39	2	0	0	0	0	0	118
6	0.05271	422.9	75	33	10	0	0	0	0	0	118

12	0.05595	424.9	74	33	11	0	0	0	0	0	118
13	0.05595	424.9	74	33	11	0	0	0	0	0	118
14	0.05595	424.9	74	33	11	0	0	0	0	0	118
18	0.06694	434.3	72	34	7	5	0	0	0	0	118
19	0.07076	436.9	72	34	5	7	0	0	0	0	118
20	0.07076	436.9	72	34	5	7	0	0	0	0	118
21	0.07271	437.9	72	34	5	6	0	1	0	0	118
22	0.07446	439.4	72	34	4	6	0	2	0	0	118
23	0.07452	439.4	72	34	4	6	0	2	0	0	118

Model 9y

Step	**Displacement**	**BaseForce**	**AtoB**	**BtoIO**	**IOtoLS**	**LStoCP**	**CPtoC**	**CtoD**	**DtoE**	**BeyondE**	**Total**
	m	KN									
0	0	0	118	0	0	0	0	0	0	0	118
1	0.01375	311.2	116	2	0	0	0	0	0	0	118
2	0.01689	368.3	111	7	0	0	0	0	0	0	118
3	0.01739	372.8	107	11	0	0	0	0	0	0	118
4	0.01857	376.2	106	12	0	0	0	0	0	0	118
5	0.02952	379.5	104	11	3	0	0	0	0	0	118
6	0.03039	380.4	103	10	5	0	0	0	0	0	118
7	0.03135	380.9	103	9	6	0	0	0	0	0	118
8	0.03425	382.8	102	8	8	0	0	0	0	0	118
9	0.03524	383.1	102	8	8	0	0	0	0	0	118
10	0.03876	385.7	101	9	8	0	0	0	0	0	118
11	0.03900	385.8	101	9	8	0	0	0	0	0	118
12	0.04890	395.0	101	9	8	0	0	0	0	0	118
13	0.04898	395.0	101	9	8	0	0	0	0	0	118
14	0.05085	397.0	101	9	8	0	0	0	0	0	118

15	0.05275	398.7	101	9	8	0	0	0	0	0	118

Model 10X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0.00016	0	170	0	0	0	0	0	0	0	170
1	0.00079	16.54	170	0	0	0	0	0	0	0	170
2	0.02107	553.3	142	28	0	0	0	0	0	0	170
3	0.04180	925.6	112	54	4	0	0	0	0	0	170
4	0.05821	1066.	97	61	12	0	0	0	0	0	170
6	0.08009	1164.	91	56	23	0	0	0	0	0	170
8	0.10574	1230.	80	56	32	2	0	0	0	0	170
10	0.12919	1277.	77	48	39	4	2	0	0	0	170
12	0.14434	1306.	72	46	40	8	4	0	0	0	170
15	0.16024	1328.	72	42	39	9	6	2	0	0	170

Model 10Y

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0	0	170	0	0	0	0	0	0	0	170
1	0.00062	26.74	169	1	0	0	0	0	0	0	170
2	0.02144	811.9	141	29	0	0	0	0	0	0	170
3	0.03428	1106.	114	55	1	0	0	0	0	0	170
4	0.05315	1276.	98	54	18	0	0	0	0	0	170
5	0.07545	1355.	92	39	38	1	0	0	0	0	170
6	0.09806	1411.	84	35	47	3	1	0	0	0	170
9	0.11852	1445.	83	30	42	11	4	0	0	0	170

11	0.13899	1479.	82	27	42	11	8	0	0	0	170
13	0.16041	1514.	80	27	27	19	15	2	0	0	170
15	0.16377	1519.	77	30	23	21	15	4	0	0	170

Model 11X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0.00144	0	271	1	2	0	0	0	0	0	274
1	0.00144	2.96E	271	1	2	0	0	0	0	0	274
2	0.02184	296.1	231	41	2	0	0	0	0	0	274
3	0.04564	549.6	207	62	5	0	0	0	0	0	274
4	0.06314	684.6	196	59	19	0	0	0	0	0	274
5	0.08746	756.0	175	65	34	0	0	0	0	0	274
6	0.10853	799.4	169	54	51	0	0	0	0	0	274
9	0.12912	837.9	161	50	56	7	0	0	0	0	274
11	0.15081	870.7	151	55	54	9	5	0	0	0	274
13	0.171	896.0	142	60	48	15	9	0	0	0	274
15	0.19257	919.6	139	57	49	9	20	0	0	0	274
17	0.20144	929.2	137	58	46	9	24	0	0	0	274

Model 11Y

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0.00062	0	271	1	2	0	0	0	0	0	274
1	0.00075	2.1	270	2	2	0	0	0	0	0	274
2	0.01644	261.1	241	31	2	0	0	0	0	0	274
3	0.03172	503.9	223	49	2	0	0	0	0	0	274

4	0.04687	684.3	202	70	2	0	0	0	0	0	274
5	0.05739	760.8	173	92	9	0	0	0	0	0	274
6	0.07342	821.0	155	104	15	0	0	0	0	0	274
7	0.08963	856.1	142	107	25	0	0	0	0	0	274
8	0.09906	874.2	135	94	45	0	0	0	0	0	274
9	0.09906	874.3	135	94	45	0	0	0	0	0	274
10	0.11503	896.6	133	84	55	2	0	0	0	0	274
11	0.11794	902.2	132	83	57	2	0	0	0	0	274
12	0.11794	902.2	132	83	57	2	0	0	0	0	274
13	0.11852	903.1	131	84	56	3	0	0	0	0	274

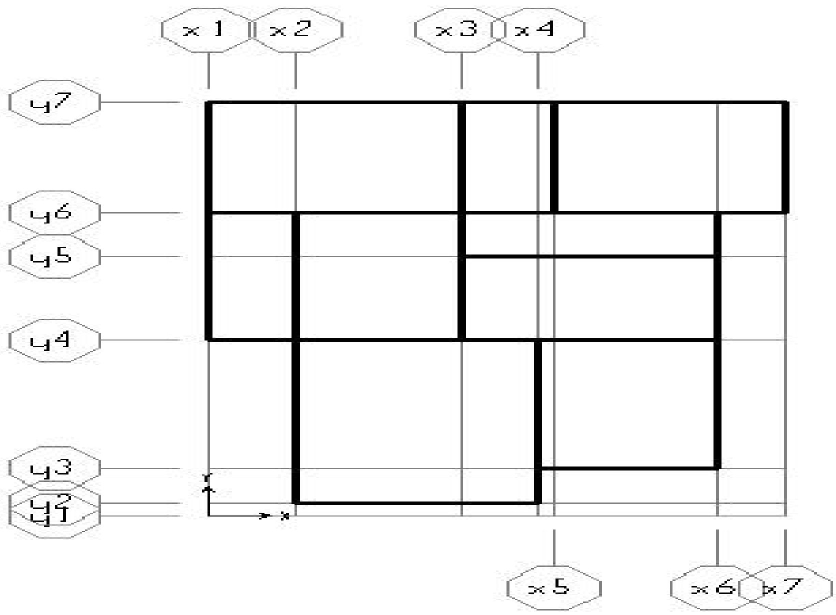
Model 12X

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0.00003	0	390	0	0	0	0	0	0	0	390
1	0.00386	124.3	389	1	0	0	0	0	0	0	390
2	0.00967	290.3	358	32	0	0	0	0	0	0	390
3	0.01445	362.2	335	55	0	0	0	0	0	0	390
4	0.03978	496.3	297	93	0	0	0	0	0	0	390
5	0.07201	606.6	292	98	0	0	0	0	0	0	390
6	0.09965	697.9	287	93	10	0	0	0	0	0	390
7	0.12505	775.0	265	77	48	0	0	0	0	0	390
8	0.15779	828.2	258	62	70	0	0	0	0	0	390
9	0.18615	874.0	252	55	83	0	0	0	0	0	390
11	0.21168	911.2	243	53	76	18	0	0	0	0	390
13	0.23744	944.1	224	59	74	33	0	0	0	0	390
15	0.25003	956.5	219	60	52	57	0	2	0	0	390

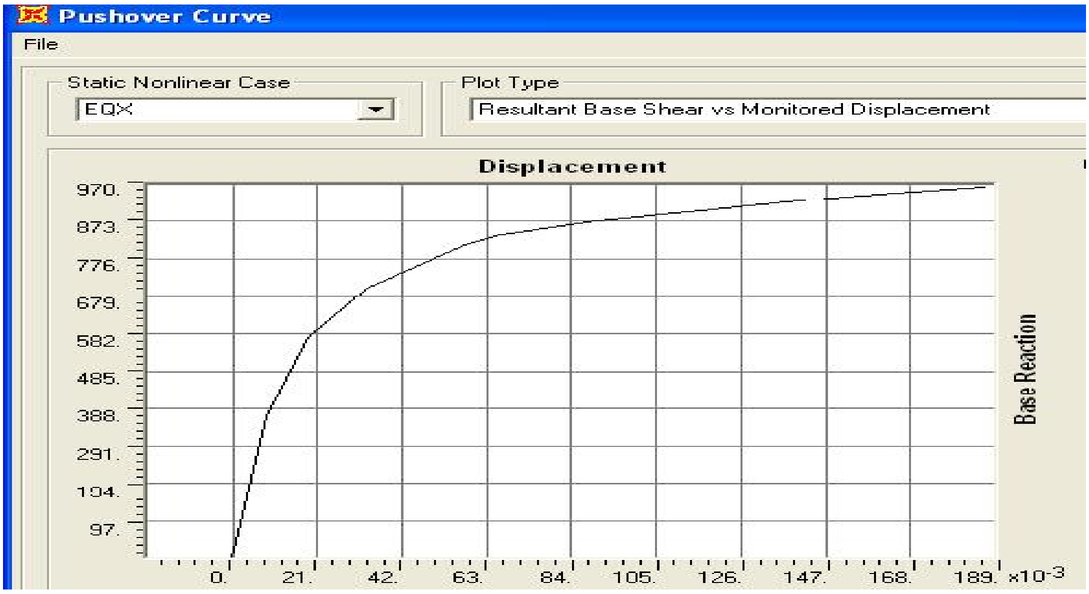
Model 12Y

Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0.00032	0	390	0	0	0	0	0	0	0	390
1	0.00459	105.4	389	1	0	0	0	0	0	0	390
2	0.02439	380.8	337	53	0	0	0	0	0	0	390
3	0.04761	542.4	315	75	0	0	0	0	0	0	390
4	0.06769	650.5	306	84	0	0	0	0	0	0	390
5	0.08905	750.2	300	88	2	0	0	0	0	0	390
6	0.10922	839.0	289	86	15	0	0	0	0	0	390
8	0.12575	895.4	270	88	32	0	0	0	0	0	390
10	0.14781	939.9	257	80	53	0	0	0	0	0	390
12	0.16781	977.6	257	71	62	0	0	0	0	0	390
14	0.19023	1019.	252	57	80	1	0	0	0	0	390
16	0.20032	1035.	251	55	79	5	0	0	0	0	390

Annex 2.2. Stages for formation of plastic hinge



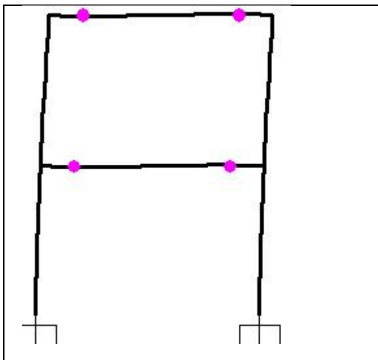
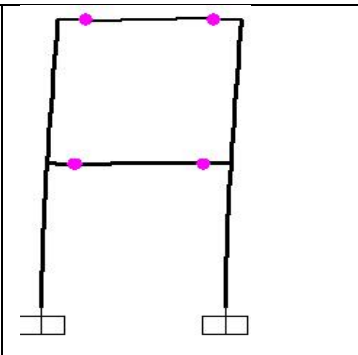
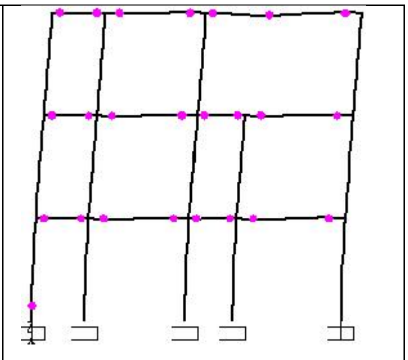
Data from static non-linear pushover analysis

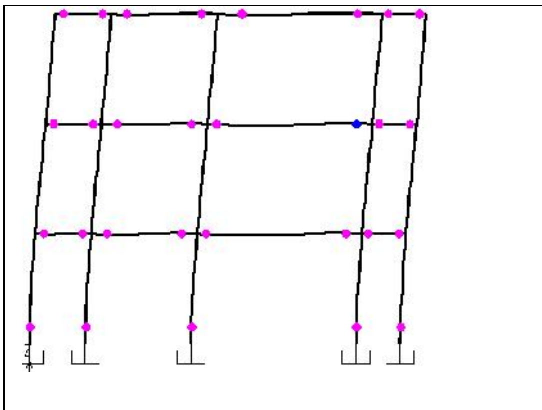
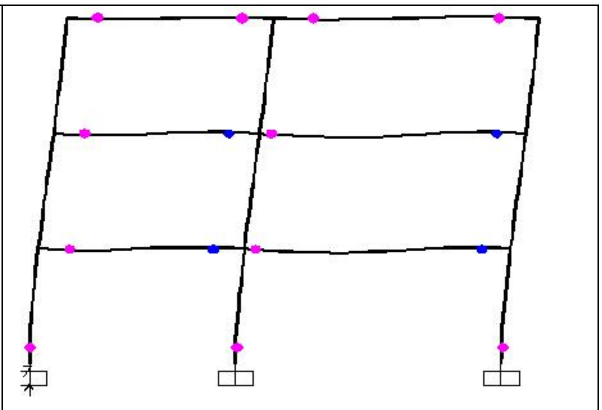


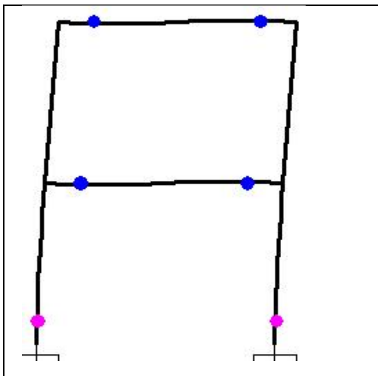
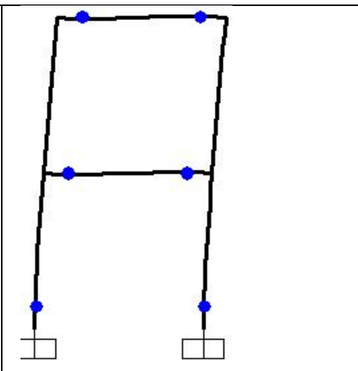
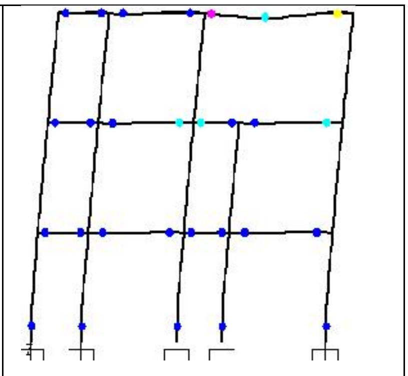
When earthquake force acts in X-direction.

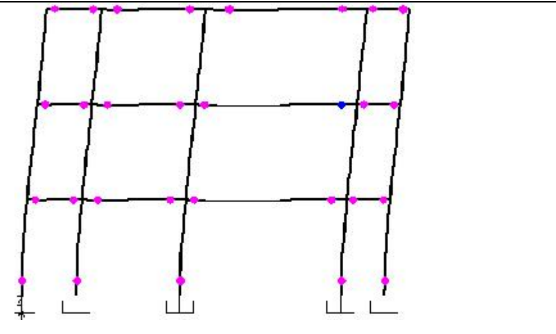
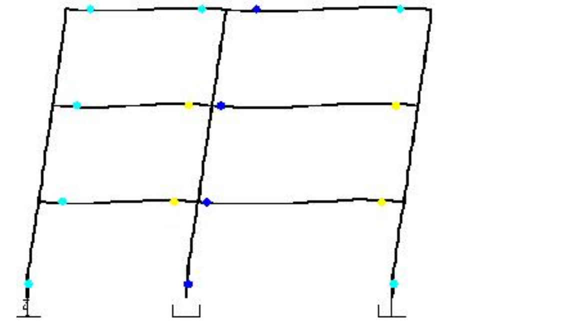
Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	0	0	211	15	0	0	0	0	0	0	226
1	0.00066	42.43	210	16	0	0	0	0	0	0	226
5	0.05671	810.2	127	94	5	0	0	0	0	0	226
6	0.06457	833.6	121	97	8	0	0	0	0	0	226
7	0.08740	872.9	118	64	44	0	0	0	0	0	226
8	0.11879	905.8	116	32	73	5	0	0	0	0	226
9	0.14002	925.7	115	29	73	6	0	3	0	0	226
10	0.16007	943.0	115	26	50	29	0	6	0	0	226
12	0.18157	960.6	113	26	31	40	0	16	0	0	226
14	0.18694	963.1	113	26	25	41	0	21	0	0	226
15	0.18694	963.1	113	26	25	41	0	21	0	0	226

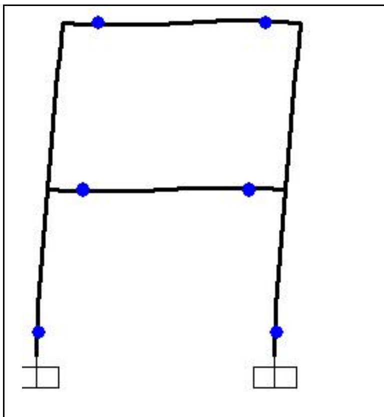
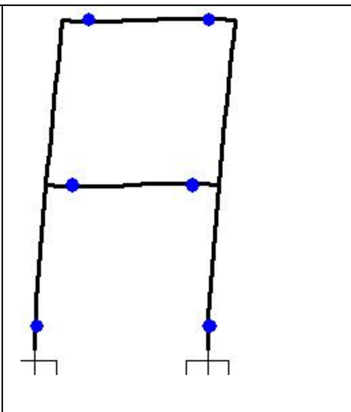
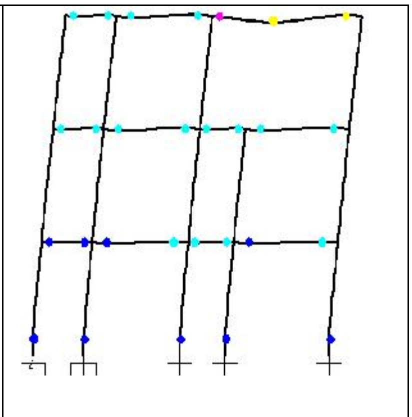
Formation of plastic hinge when earthquake force is applied in X direction.

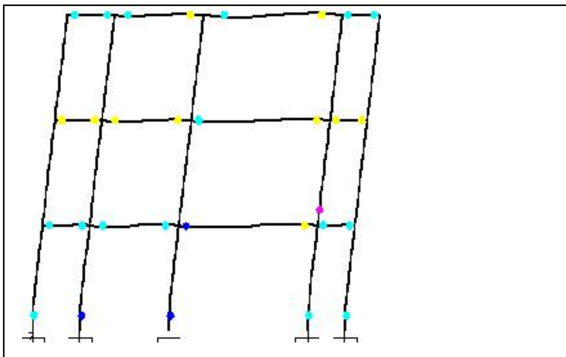
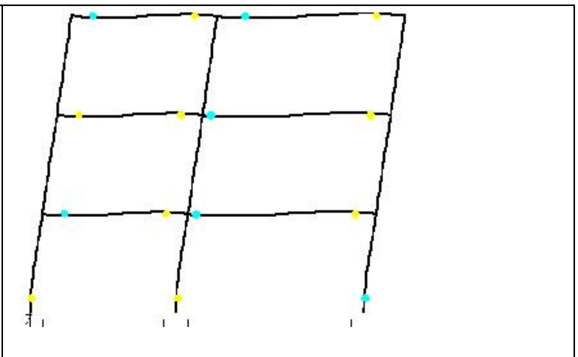
		
Section at y2(step-5)	Section at y3 (step-5)	Section at y4(step-5)

	
Section at y6(step-5)	Section at y7(step-5)

		
Section at y2(step-10)	Section at y3(step-10)	Section at y4(step-10)

	
Section at y6(step-10)	Section at y7(step-10)

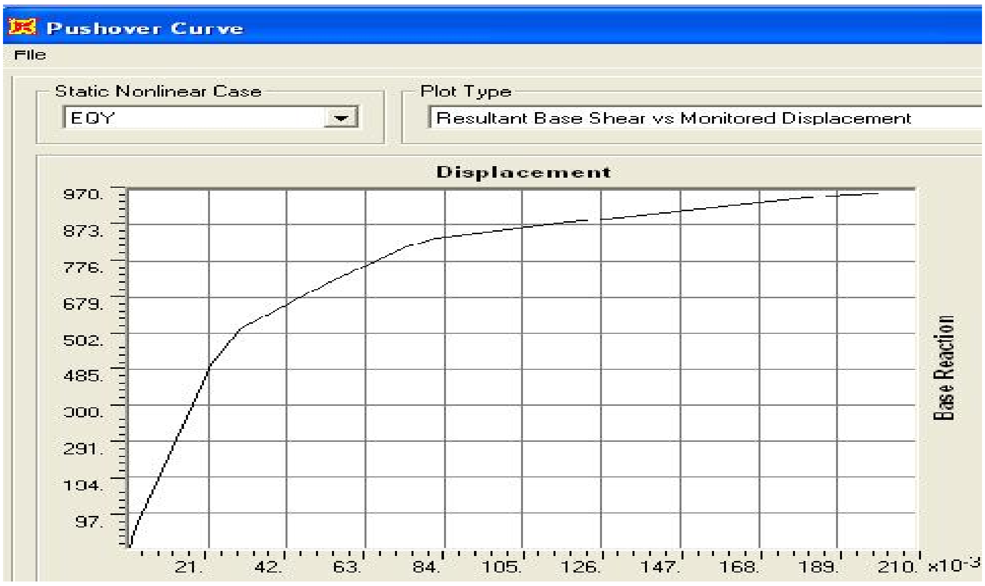
		
Section at y2(step-15)	Section at y3(step-15)	Section at y4(step-15)

	
Section at y6(step-15)	Section at y7(step-15)

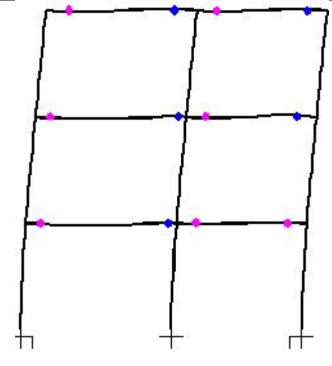
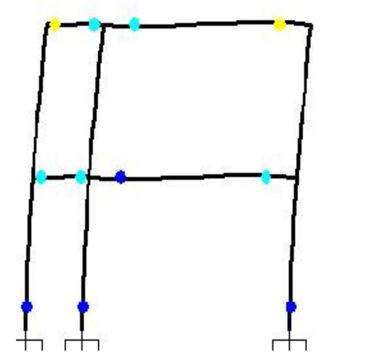
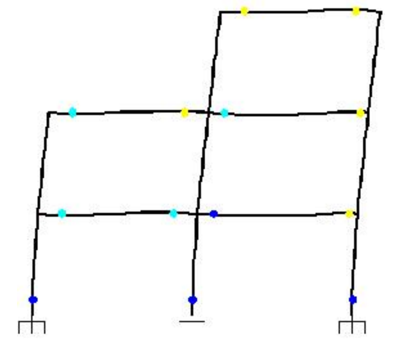
Data from static non-linear analysis when earthquake force acts on Y-direction (EQY)

Step	**Displacement**	**BaseForce**	**AtoB**	**BtoIO**	**IOtoLS**	**LStoCP**	**CPtoC**	**CtoD**	**DtoE**	**BeyondE**	**Total**
	m	KN									
0	0.000154	0	211	15	0	0	0	0	0	0	226
1	0.001275	44.612	210	16	0	0	0	0	0	0	226
2	0.021528	495.99	178	48	0	0	0	0	0	0	226
3	0.029981	596.40	162	64	0	0	0	0	0	0	226
4	0.051849	714.93	157	69	0	0	0	0	0	0	226
5	0.073123	811.10	146	62	18	0	0	0	0	0	226
6	0.082163	839.64	140	58	28	0	0	0	0	0	226
7	0.11427	876.83	138	32	56	0	0	0	0	0	226
8	0.136949	899.55	137	12	73	4	0	0	0	0	226
10	0.167526	935.17	137	10	47	30	0	2	0	0	226
12	0.187871	953.59	137	10	30	34	0	15	0	0	226
14	0.191472	955.92	137	10	29	32	0	18	0	0	226
15	0.200154	960.48	137	10	27	29	0	23	0	0	226

Pushover curve in Y-direction



Plastic hinge formation at Step 5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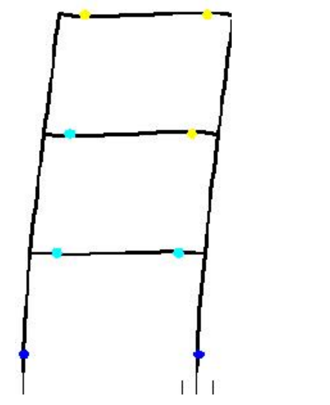
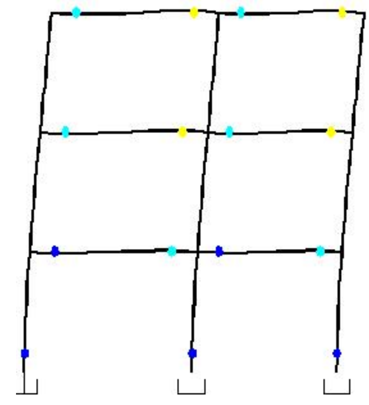
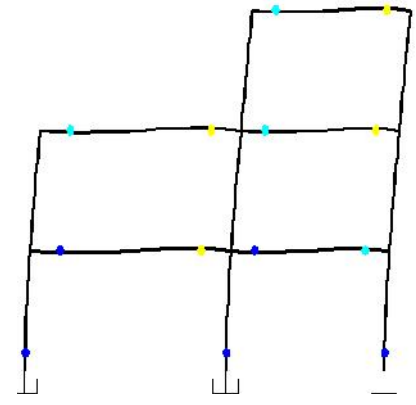
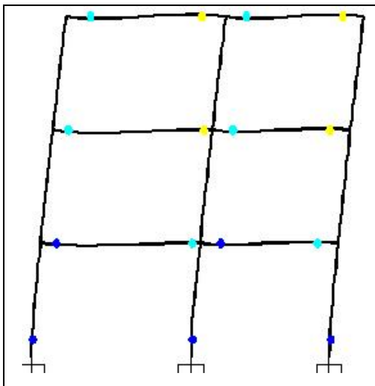
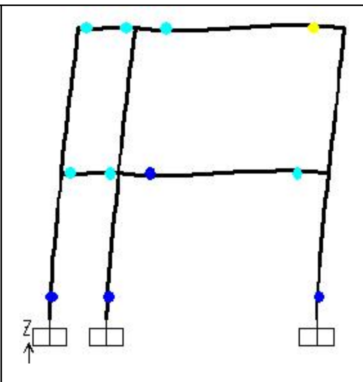
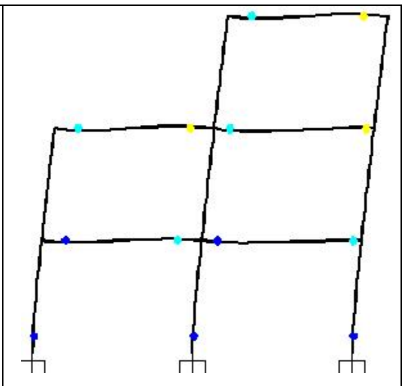
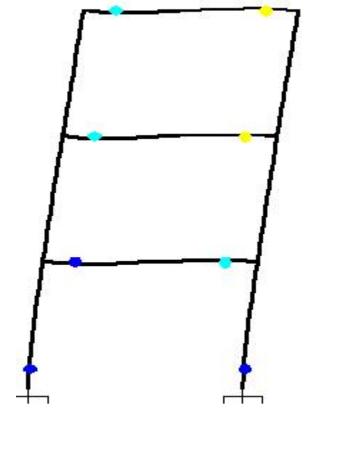
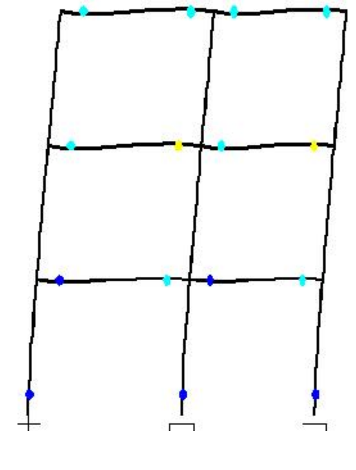
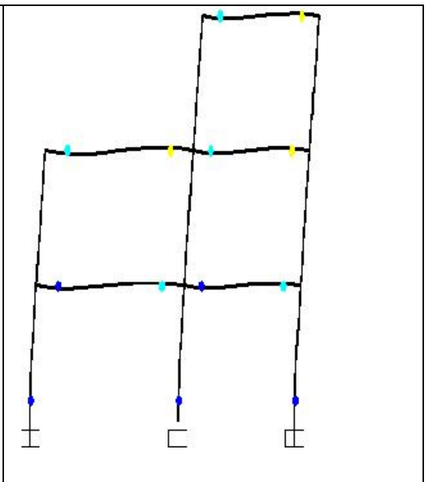
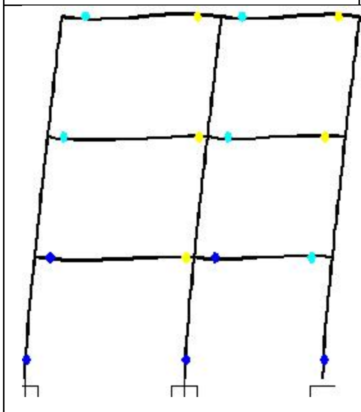
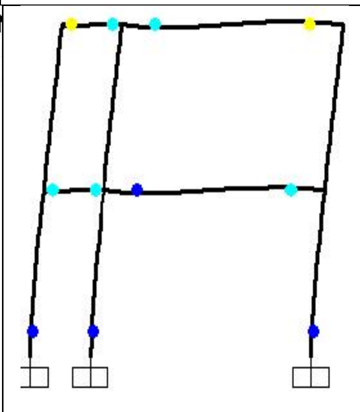
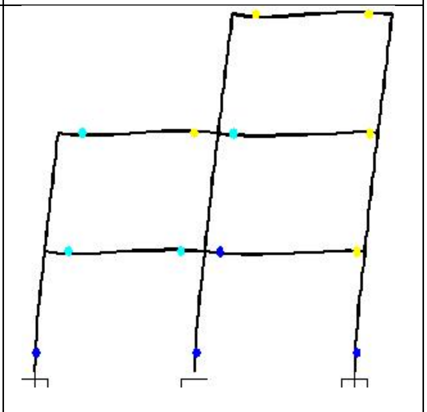
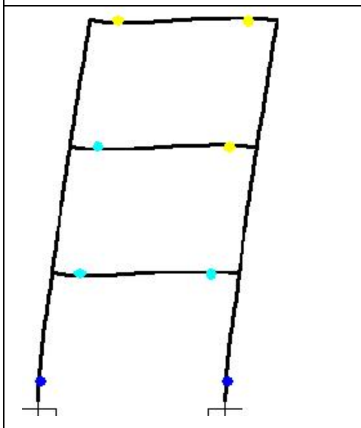
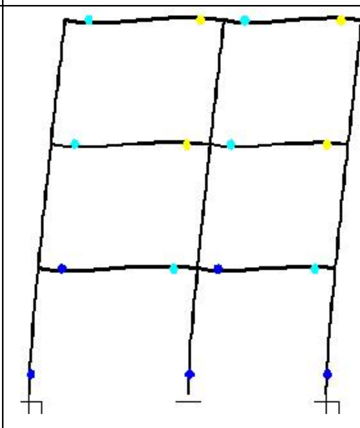
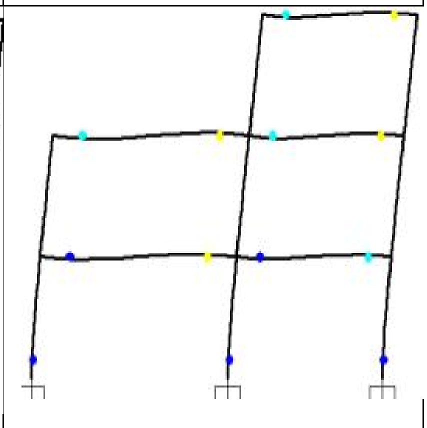
		
Section at x5(step-5)	Section at x6(step-5)	Section at x7(step-5)

		
Section at x1(step-10)	Section at x2(step-10)	Section at x3(step-10)

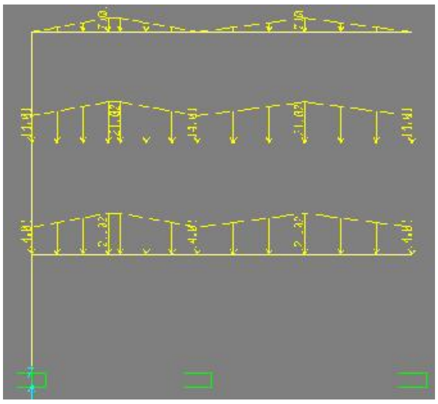
		
Section at x5(step-10)	Section at x6(step-10)	Section at x7(step-10)
		
Section at x1(step-15)	Section at x2(step-15)	Section at x3(step-15)
		
Section at x5(step-15)	Section at x6(step-15)	Section at x7(step-15)

Annex 3.1. Sample Calculation of R-Value of Study Building

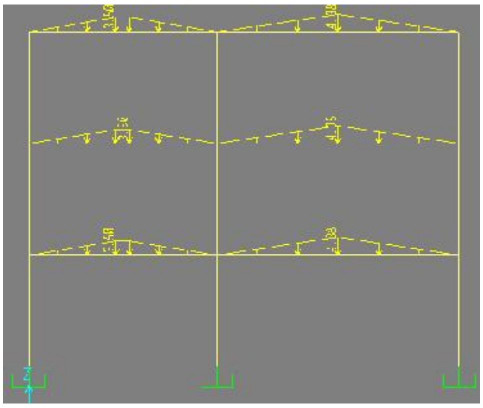
Step 1. Determination of total weight of the building by considering DL and LL.

Step 2. The dead load and live load of slab is taken as trapezoidal distribution whereas dead load of wall is uniformly distributed.

Step 3. Assigning the calculated load on beam.



Assigning dead load



Assigning live load

Step 4.Calculation of design base shear and vertically distribute the design base shear as per IS 1893-2002.

Sample calculation (Modal 7)

Vertical distribution of base shear to different floor as per IS 1893 (Part 1):2002

Equivalent static lateral force method:

Design seismic base shear V_b along any principal direction shall be determined by

$V_b = A_h W$

Time Period of the structure is determined by;

$T = 0.075 h^{0.75}$

$= 0.075 X (14.25)^{0.75}$

=0.55 sec.

From IS 1893:2002, for medium soil

Sa/g =2.47, Z=0.36,I = 1, and R = 5 is taken

$A_h = Z/2*I/R*S_a/g$

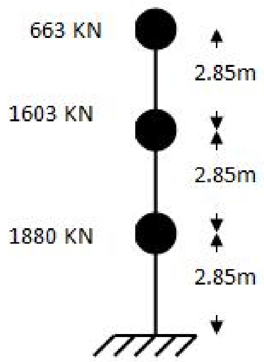
Ah = 0.08892

Design base shear, $V_B = A_h W = 0.08892 * 4392.09 = 390.55$ KN

Vertical distribution of base shear

$$Q_i = V_b . \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2}$$

Q1 = 13.68KN, Q2 = 54.73KN, Q3 = 123.13KN, Q4= 144.40KN, and Q5 = 54.36KN



Response Spectrum Method

Modal 2

Sample calculation of Target displacement in Pushover analysis

Column sizes are 0.23m * 0.31m

$$\text{Stiffness } [K] = \begin{vmatrix} k1 \\ k2 \\ k3 \end{vmatrix} = \begin{vmatrix} 79781 \\ 79781 \\ 58591 \end{vmatrix}$$

$$[M] = \begin{vmatrix} 188 & 0 & 0 \\ 0 & 160.3 & 0 \\ 0 & 0 & 66.3 \end{vmatrix}, \qquad [K] = \begin{vmatrix} k1 & -k2 & 0 \\ -k2 & k2+k3 & -k3 \\ 0 & -k3 & k3 \end{vmatrix}$$

$$[K] = \begin{vmatrix} 159262 & -79781 & 0 \\ -79781 & 138372 & -58591 \\ 0 & -58591 & 58591 \end{vmatrix}$$

For the above stiffness and mass matrices, eigenvalues and eigenvectors are worked out as follows

$|K-\omega^2 m| = 0$

Solving,

$\omega_1^2 = 1489$; $T_1 = 2\pi/\sqrt{\omega_1^2}$

$\omega_2^2 = 134$; $T_2 = 2\pi/\sqrt{\omega_2^2}$

$\omega_3^2 = 930$; $T_3 = 2\pi/\sqrt{\omega_3^2}$

From above calculation T1 = 0.16 second, T2 = 0.54 second and T3 = 0.20 second

Eigen values $[\omega^2] = \begin{vmatrix} 1489 & 0 & 0 \\ 0 & 134 & 0 \\ 0 & 0 & 930 \end{vmatrix}$

The mode shape corresponding to each natural frequency is determined from the equations.

$[-Mw_1^2+K]\phi_1 = 0$

$[-Mw_2^2+K]\phi_2 = 0$

$[-Mw_3^2+K]\phi_3 = 0$

When $\omega_1^2 = 1489$; $\begin{vmatrix} 120370 & -798781 & 0 \\ -79781 & -100315 & -58591 \\ 0 & -58591 & -40130 \end{vmatrix} \begin{vmatrix} \phi 11 \\ \phi 21 \\ \phi 31 \end{vmatrix} = 0$

Which gives, $\begin{vmatrix} \phi 11 \\ \phi 21 \\ \phi 31 \end{vmatrix} = [\phi T\ m\ \phi] = \begin{vmatrix} 0.011 \\ -0.057 \\ 0.083 \end{vmatrix}$

Similarly when $\omega_2^2 = 134$;

$\begin{vmatrix} \phi 12 \\ \phi 22 \\ \phi 32 \end{vmatrix} = [\phi T\ m\ \phi] = \begin{vmatrix} 0.034 \\ 0.056 \\ 0.066 \end{vmatrix}$ Similary when $\omega_3^2 = 930$;

$\begin{vmatrix} \phi 13 \\ \phi 23 \\ \phi 33 \end{vmatrix} = [\phi T\ m\ \phi] = \begin{vmatrix} 0.056 \\ 0.004 \\ -0.08 \end{vmatrix}$

Eigenvectors can be computed as $\{\phi\} = \{\phi 1 \quad \phi 2 \quad \phi 3\} = \begin{vmatrix} 0.011 & -.057 & .083 \\ 0.034 & 0.056 & 0.066 \\ 0.056 & 0.004 & -0.08 \end{vmatrix}$

Natural period; $[T] = \begin{vmatrix} 0.16 & 0 & 0 \\ 0 & 0.54 & 0 \\ 0 & 0 & 0.20 \end{vmatrix}$

Determination of modal participation factor

$P1 = \frac{188 \times 0011 - 1603 \times 0057 + 663 \times 0083}{(188 \times (0011)^2 + 1603 \times (0057)^2 + 6630 \times (0083)^2} = -1.56$

Similarly, P2 = 19.74; P3 = 5.76

Total mass of the sample building is 414.6

m_1 = 0.59%, m2 = 91.41%, m3 = 7.83%

Ah = Z/2 * I/R * Sa/g; then, Ah1 = 0.09, Ah2 = 0.09 and Ah3 = 0.09

$$\begin{vmatrix} Q11\\ Q21\\ Q31\end{vmatrix} = \begin{vmatrix}0.09 & -1.56 & 0.011 & 1880\\ 0.09 & -1.56 & -0.057 & 1603\\ 0.09 & -1.56 & 0.083 & 663\end{vmatrix} = \begin{vmatrix}-2.90\\ 12.83\\ -7.73\end{vmatrix}$$

Similarly, $\begin{vmatrix} Q12\\ Q22\\ Q32\end{vmatrix} = \begin{vmatrix}112.5\\ 158.03\\ 77.03\end{vmatrix}$ and $\begin{vmatrix} Q13\\ Q23\\ Q33\end{vmatrix} = \begin{vmatrix}54.58\\ 3.32\\ -27.49\end{vmatrix}$

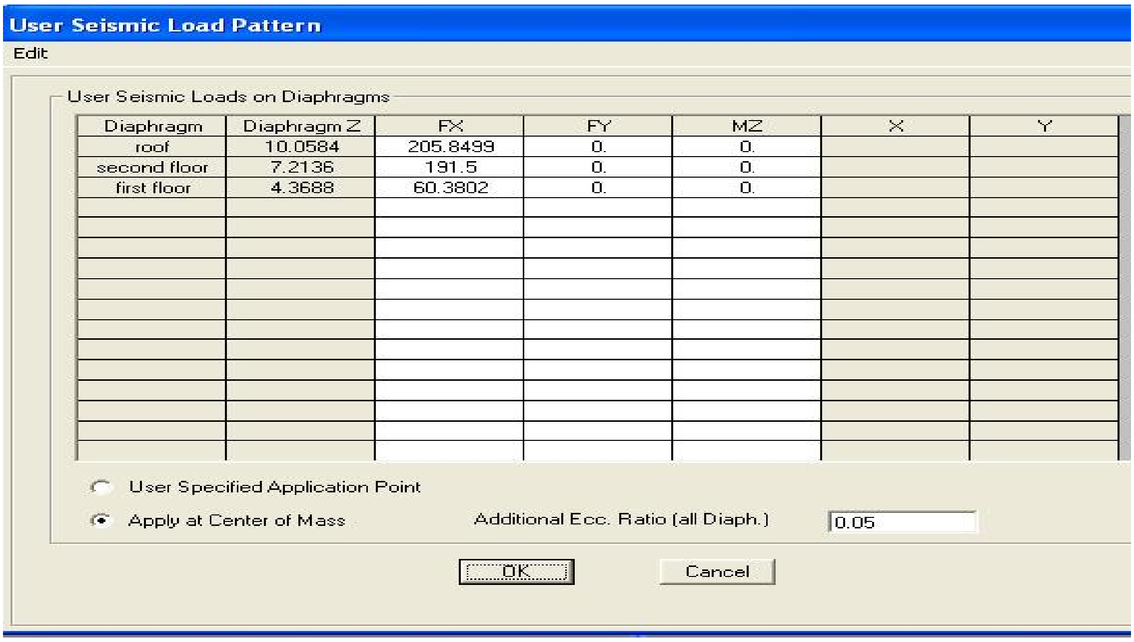
$$\begin{vmatrix} V11\\ V21\\ V31\end{vmatrix} = \begin{vmatrix}2.2\\ 5.1\\ -7.73\end{vmatrix};\ \begin{vmatrix} V12\\ V22\\ V32\end{vmatrix} = \begin{vmatrix}347.56\\ 235.06\\ 77.03\end{vmatrix} \text{ And } \begin{vmatrix} V13\\ V23\\ V33\end{vmatrix} = \begin{vmatrix}30.41\\ -24.17\\ -27.49\end{vmatrix}$$

$V1 = (2.2^2+347.56^2+30.41^2)^{1/2} = 348.895$ KN

$V2 = (5.1^2+235.06^2+24.17^2)^{1/2} = 236.35$ KN

$V3 = (7.73^2+77.03^2+27.49^2)^{1/2} = 82.15$ KN

Step5. Assigning the lateral load to the C.G of each diaphragm.



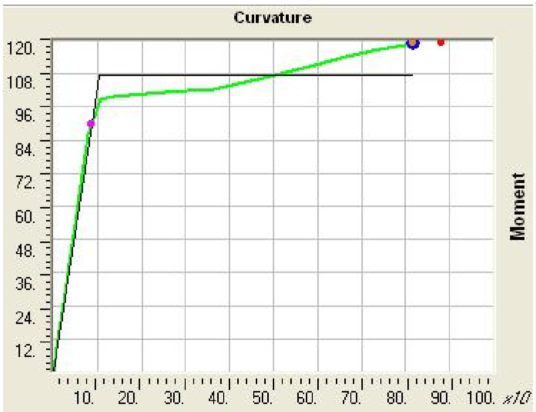
Step 6. Defining the static non-linear load cases. The load cases are defined as GRAV, EQX (Push in X-direction), EQY (Push in Y-direction). In GRAV load condition the load case is DL+0.25 LL is used whereas In EQX and EQY user defined load is used.

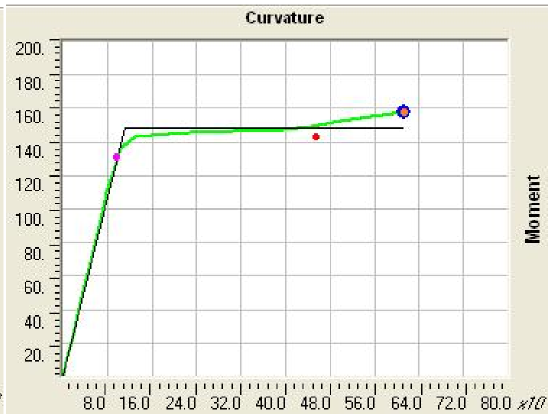
Step 7. Assign default plastic hinge in beam and column element. In beam default M3 hinge (FEMA 356) is used whereas in column P-M2-M3 hinge is used. User defined hinge for beam (M3) is obtained as:

The stress-strain diagram of steel and concrete use in analysis is:

Stress-strain diagram of concrete stress-strain diagram of steel


Beam size is 15”X 9” ,Top- 6-16mmϕ bars and ottom-4-16mmϕ bars are used in Normal case. In (–ve) cycle (reverse case), the reinforcement is just altered and the resulting moment curvature is obtained as follows:

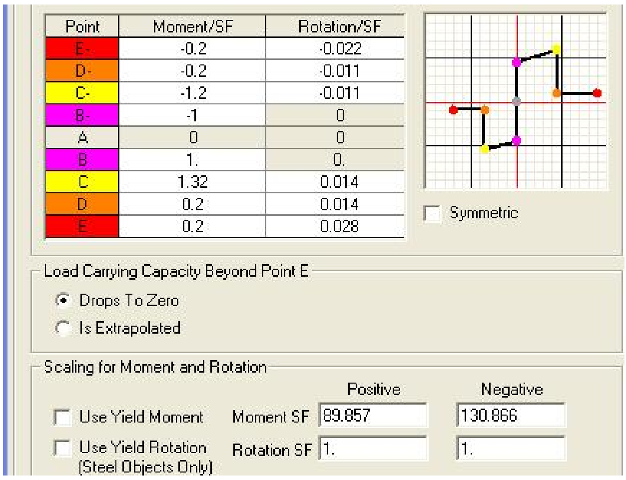


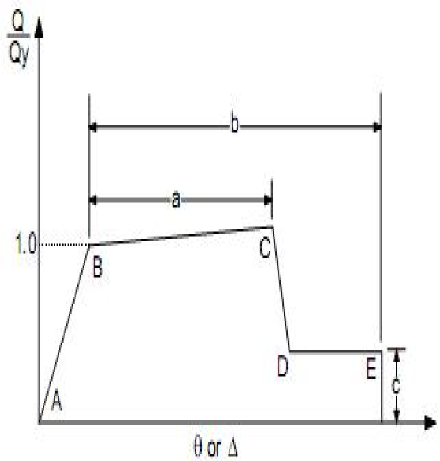


Normal condition moment curvature Reinforcement reverse condition

Sagging moment capacity	Hogging moment capacity
Maximum moment (Mu) =119.045 KN-M Maximum curvature (ϕmax) = 0.082 Yield moment (My) =89.857 KN-m	Maximum moment (Mu) =157.349 KN-M Yield moment (My) =130.866 KN-m Maximum curvature (ϕmax) = 0.062

Yield curvature (ϕyield) = 0.0087 Idealized curvature (ϕidealized) =0.010 Rotation at C = 0.082 X 0.172 = 0.014 Rotation at D = 0.014 Rotation at E = 2 X 0.014 = 0.028 Scale factor (S.F) = 119.045 / 89.857 = 1.32 d= (0.381-0.038) = 0.343m Plastic hinge length = 0.50 X d	Yield curvature (ϕyield) = 0.0097 Idealized curvature (ϕidealized) =0.011 Rotation at C = Curvature X plastic hinge length = 0.062 X 0.172 = 0.011 Rotation at D = 0.011 Rotation at E = 2 X rotation at D = 0.022 Scale factor (S.F) = 157.349 /130.866 = 1.20 d= (0.381-0.038) = 0.343m





Step 8. The force is controlled in Gravity load cases and for Push X and Push Y displacement controlled concept is used. Target displacement is calculated as per FEMA 273 and the program is launched to that displacement.

Calculation of target displacement.

The target displacement δ_t in FEMA-273 is given by

$\delta t = C_O C_1 C_2 C_3 Sa\ g\ Te^2/4\Pi^2$

Where,

C0 = 1.30, C1 = 1, C2 = 1.2

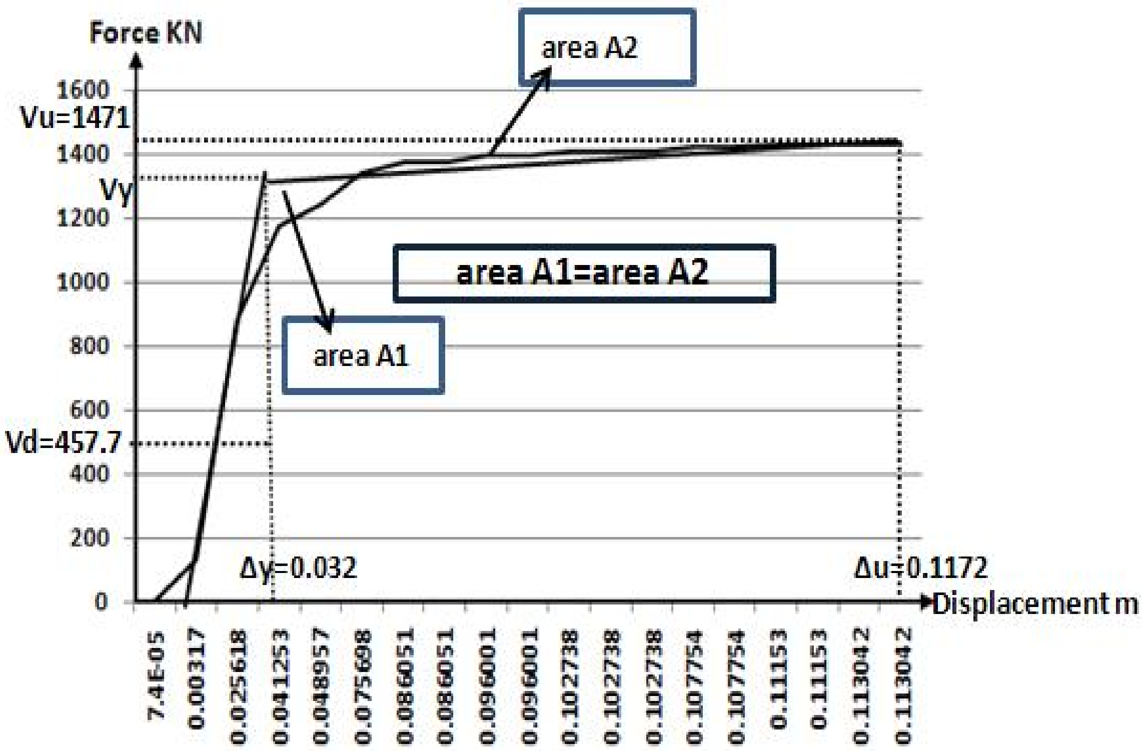
R = 2.5*5085.82/ (755*1.30) =13

$C3 = 1.0 + |\gamma|\ (R\text{-}1)^{3/2} / Te = 1.0 + |-1.6|\ (13\text{-}1)^{3/2} / 0.28337 = 235.71$

δt = 0.6238 m

Step9. After run analysis capacity curve is obtained. With the help of capacity curve ultimate deformation, yield deformation, ultimate shear, and yield shear is obtained. (Yield deformation is obtained after bilinear idealization of capacity curve).

Step10. Determination overstrength factor, by the ratio of ultimate shear to design base shear.For Push Y (EQY consideration)



Bilinear representation of capacity curve

Overstrength factor (Ω) = Vu/Vd = 1471/457.7 =3.20

Step11. Determination of ductility factor, by the ratio of ultimate deformation to the yield deformation (it is also called ductility supply by the structure)

Displacement ductility factor (μ) = Δu/Δy = 0.1172/0.032 = 3.67

Ductility reduction factor (Rμ) = 1+ (μ-1)T/0.70 = 3.09

Step12. Determination of elastic deformation (ratio of elastic force to initial stiffness). Elastic force = 10*457.7 = 4577 KN

Initial stiffness = 31913 KN/m

Elastic deformation demand (Δe) =4577/31913 =0.14m

Step 13. Determination of elastic ductility demand (ration of elastic deformation to yield deformation.

Elastic ductility demand (μd) = 0.14/0.032 = 4.48

In this case ductility demand > ductility supply

Step 14. Determination of response reduction factor R by using the relation

2R = overstrength factor * ductility factor

= 3.20*3.09 =9.888

R = 4.94

Step 15. Repeatation the same procedure except step7 for user defined hinge.

Annex 4.1. Modal 5 (when proper detailing)

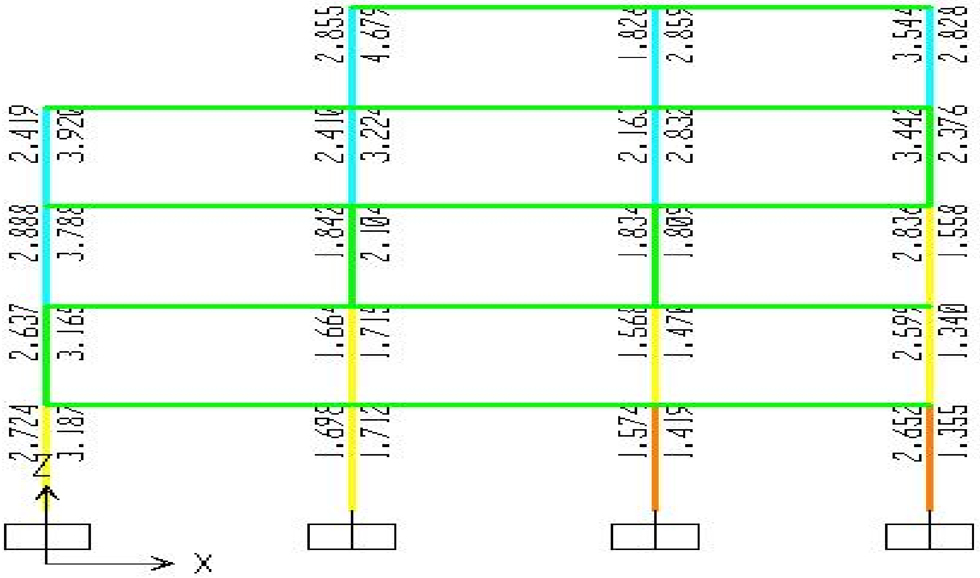
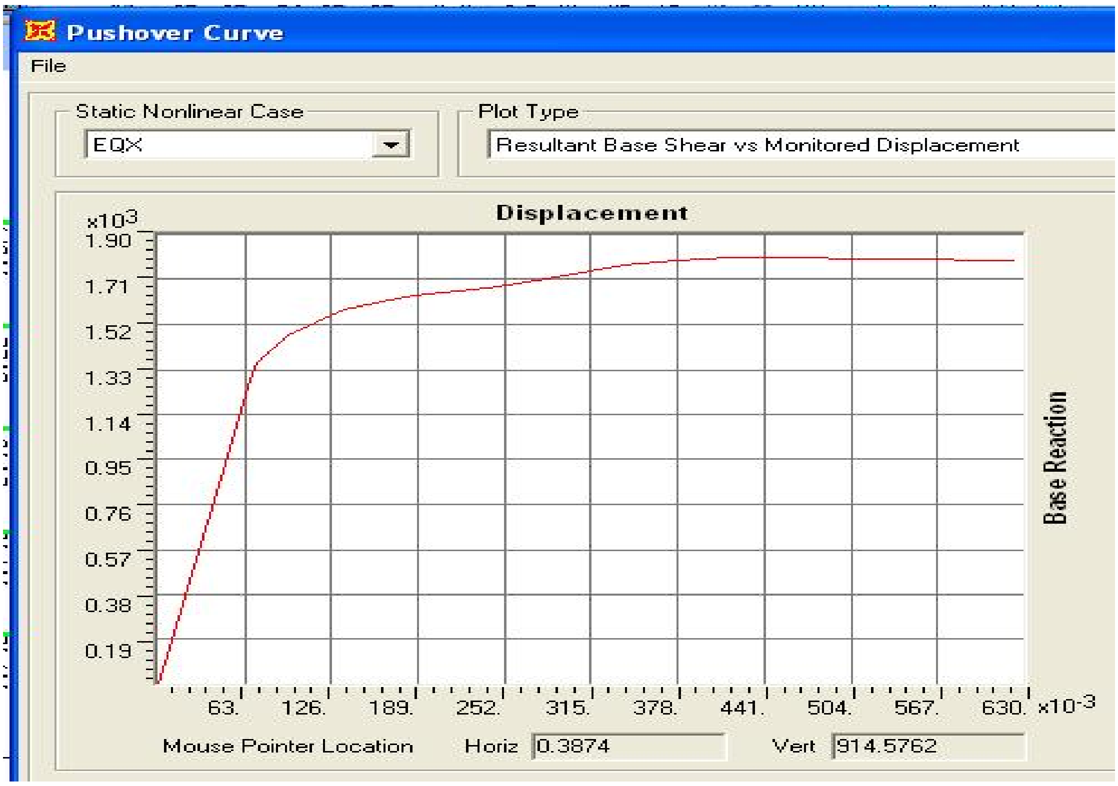


Figure capacity ratio of column and beam.



When modal 5 is detailed according to the ductile detailing as per IS Code. The capacity ratio of column and beam is shown in figure. The capacity curve is shown in figure. The maximum deformation is 0.62 m; maximum base shear is 1800 KN. the design base shear is 798.88 KN. The initial stiffness of the capacity curve is 20000. By bilinear representation of the capacity curve, the yield deformation is 0.08m. The elastic deformation is 0.40m.

Ductility supply> Ductility demand

Displacement ductility μ =5

Ductility reduction factor $R\mu$ = 1+ (μ-1)T/0.70 = 4.45

Overstrength reduction factor Ω = 2.25

Response reduction factor R = 4.45 *2.25/2 = 5.00

Similarly for EQ Y case, the maximum deformation is 0.37m, yield deformation is 0.08m. Maximum base shear is 1700 KN; design base shear is 798.8 KNIn this case also, ductility supply is > ductility demand

Displacement ductility μ = 0.376/0.08 = 4.7

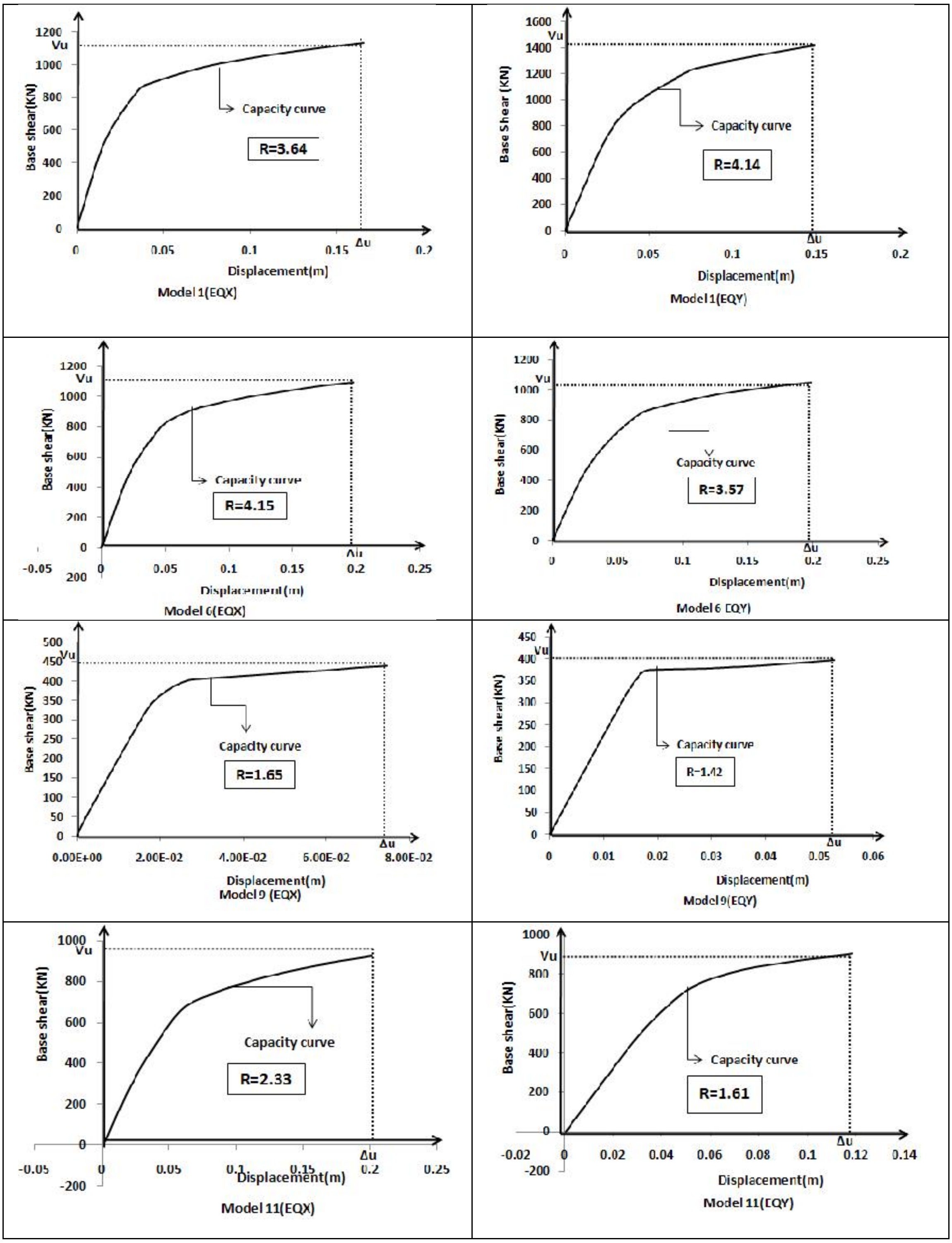
Overstrength factor Ω = 1700/798.80 = 2.13

Ductility reduction factor $R\mu$ = 1+ (μ-1)T/0.70 = 3.9

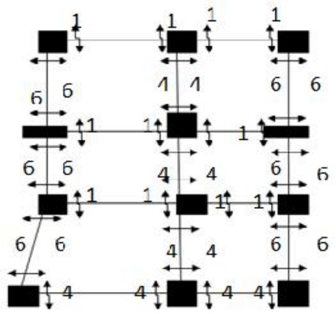
Response reduction factor R = 3.9 * 2.13/2 = 4.16

Thus from above, it is clear that in a structure, when strong column weak beam condition is not matched then, the ductility reduction factor and overstrength decrease which results to decrease response reduction factor. If design and detailing is done appropriately, then it results to increase the value of response reduction factor.

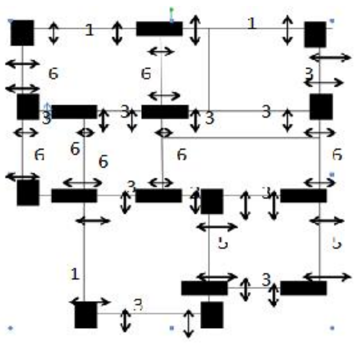
Annex 5.1. Capacity curve of some sample models



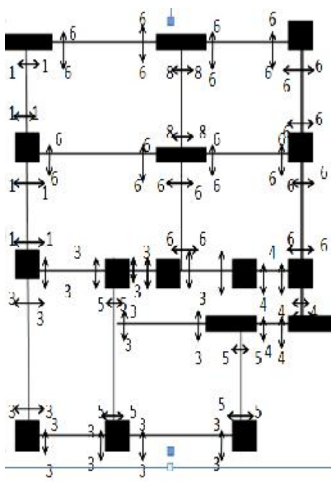
Annex 6.1. Detailing of study buildings



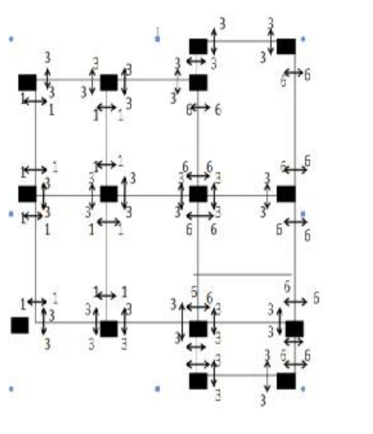
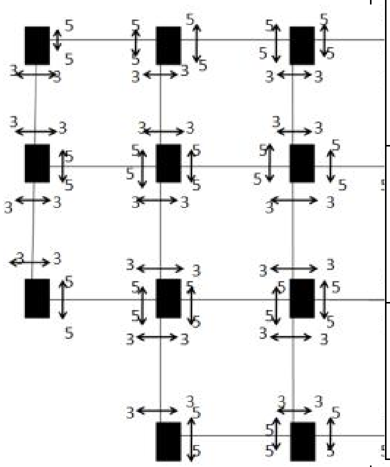
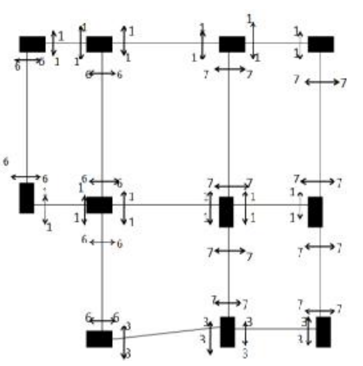
Section	Top reinforcement	Bottom reinforcement
1/1	2-12mmϕ +2-16mmϕ	2-12mmϕ+1-16mmϕ
4/4	6-16mmϕ	4-16mmϕ
6/6	6-16mmϕ	4-16mmϕ

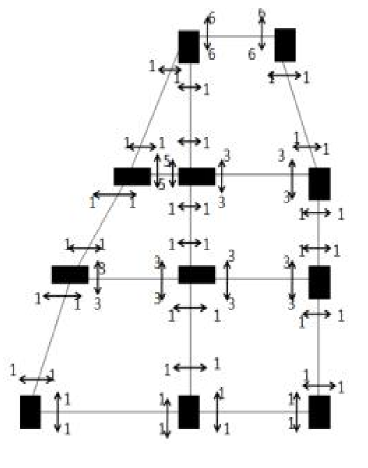


Section	Top reinforcement	Bottom reinforcement
1/1	6-16mmϕ	4-16mmϕ
3/3	4-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ
5/5	2-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ
6/6	4-12mmϕ+2-16mmϕ	2-12mmϕ+1-16mmϕ

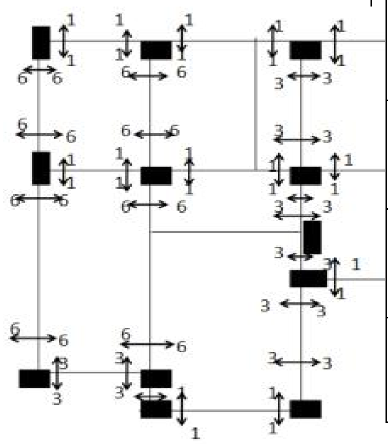


Section	Top reinforcement	Bottom reinforcement
1/1	2-12mmϕ+4-16mmϕ	2-12mmϕ+1-16mmϕ
3/3	3-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ
4/4	2-12mmϕ+1-16mmϕ	4-16mmϕ
5/5	2-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ
6/6	5-16mmϕ	3-16mmϕ

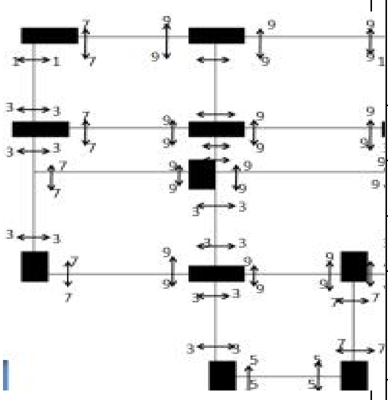
	8/8	3-16mmϕ	3-16mmϕ
	Section	Top reinforcement	Bottom reinforcement
	1/1	1-12mmϕ+4-16mmϕ	1-12mmϕ+2-16mmϕ
	3/3	4-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ
	6/6	2-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ
	Section	Top reinforcement	Bottom reinforcement
	3/3	5-12mmϕ	4-12mmϕ
	5/5	3-16mmϕ+2-20mmϕ	2-16mmϕ+1-20mmϕ
	Section	Top reinforcement	Bottom reinforcement
	1/1	4-16mmϕ	3-16mmϕ
	6/6	5-12mmϕ	4-12mmϕ
	7/7	3-16mmϕ+2-20mmϕ	2-16mmϕ+1-20mmϕ



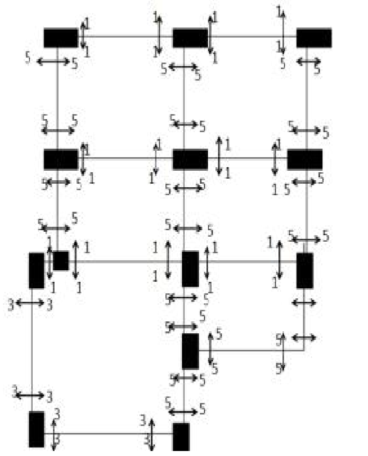
Section	Top reinforcement	Bottom reinforcement
1/1	2-12mmϕ+3-16mmϕ	2-12mmϕ+1-16mmϕ
3/3	4-16mmϕ	3-16mmϕ
6/6	2-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ



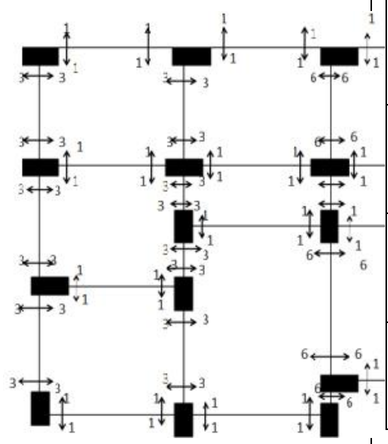
Section	Top reinforcement	Bottom reinforcement
1/1	2-12mmϕ+3-16mmϕ	3-16mmϕ
3/3	3-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ
6/6	6-16mmϕ	4-16mmϕ



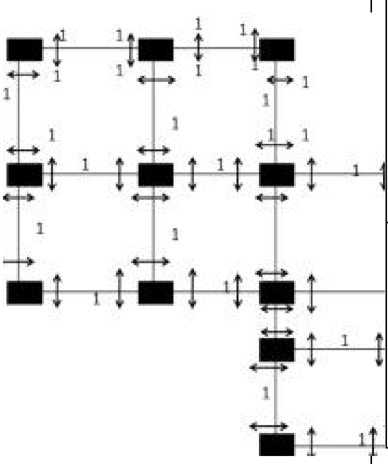
Section	Top reinforcement	Bottom reinforcement
1/1	4-16mmϕ	3-16mmϕ
3/3	6-16mmϕ	3-16mmϕ
5/5	4-12mmϕ	3-12mmϕ
7/7	3-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ
9/9	3-12mmϕ+2-16mmϕ	2-12mmϕ+1-16mmϕ



Section	Top reinforcement	Bottom reinforcement
1/1	5-16mmϕ	4-16mmϕ
3/3	6-16mmϕ	4-16mmϕ
5/5	4-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ



Section	Top reinforcement	Bottom reinforcement
1/1	3-12mmϕ+2-16mmϕ	2-12mmϕ+1-16mmϕ
3/3	3-12mmϕ+1-16mmϕ	2-12mmϕ+1-16mmϕ
6/6	6-16mmϕ	4-16mmϕ



Section	Top reinforcement	Bottom reinforcement
1/1	4-16mmϕ	3-16mmϕ

Model No	Beam size	Column size	C1	C2	C3	C4
1	15”*9”	12”*12”	10-16mmϕ	4-16mmϕ +6-12mmϕ	6-16mmϕ+ 4-12mmϕ	_
2	15”*9”	15”*12”	10-16mmϕ	4-16mmϕ+ 4-20mmϕ	6-16mmϕ+ 4-20mmϕ	8-20mmϕ
3	15”*9”	15”*12”	8-20mmϕ	4-20mmϕ+ 4-16mmϕ	_	_
4	15”*9”	14”*14”	8-16mmϕ	10-12mmϕ	8-16mmϕ+ 2-12mmϕ	_
5	15”*9”	12”*12”	6-16mmϕ+ 4-20mmϕ	10-16mmϕ	4-20mmϕ+ 8-16mmϕ	_
6	15”*9”	12”*12”	10-16mmϕ	8-16mmϕ+ 2-12mmϕ	6-16mmϕ+ 4-20mmϕ	_
7	15”*9”	12”*9”	8-16mmϕ	6-16mmϕ+ 4-20mmϕ	4-16mmϕ+ 4-20mmϕ	_
8	15”*9”	14”*14”	10-16mmϕ	8-16mmϕ+ 2-12mmϕ	4-16mmϕ+ 6-12mmϕ	_
9	15”*9”	12”*9”	10-12mmϕ	4-16mmϕ+ 6-12mmϕ	2-16mmϕ+ 8-12mmϕ	10-16mmϕ
10	15”*9”	14”*14”	4-20mmϕ+ 4-16mmϕ	8-16mmϕ	4-16mmϕ+ 4-12mmϕ	_
11	15”*9”	12”*9”	2-20mmϕ+ 8-16mmϕ	4-16mmϕ+ 4-20mmϕ	6-16mmϕ+ 4-12mmϕ	_
12	15”*9”	14”*14”	8-20mmϕ	4-16mmϕ+ 4-20mmϕ	_	_