COMPARISON BETWEEN COMMON SEISMIC CODES USED IN NEPAL AND EUROCODE 8: STUDY CASE ANALYSIS OF RC BUILDING

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Department of Structures for Engineering and Architecture

European Commission
ERASMUS
MUNDUS

A Thesis submitted in partial fulfillment of the requirements
For the Degree of Master in Structural Design of Sustainable Construction
European Erasmus Mundus Masters Course
Sustainable Construction Under Natural Hazards and Catastrophic Events

University of Naples “Federico II”
Date: 01.02.2018

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Dedication

I would like to dedicate this thesis to people who believe in me......

STATEMENT OF THESIS APPROVAL

This thesis is prepared by **Anup Shrestha** entitled '**Comparison between common seismic codes used in Nepal and Eurocode 8: Study case analysis of RC building'** is approved in partial fulfillment of the requirements for the degree of Master in Structural Design of sustainable construction by the following faculty members served as the supervisory committee chair and members.

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Date of Submission 01 February 2018

ACKNOWLEDGEMENTS

Firstly, I would like to acknowledge the European Commission for granting me this Erasmus Mundus scholarship to pursue this Master's program in Sustainable Construction Under Natural Hazards and Catastrophic Events (SUSCOS).

Most of all, I express my earnest gratitude and deep respect to my supervisor Professor Dr. Eng. Federico Massimo Mazzolani at UNINA for his inspirational guidance, untiring support and constant encouragement. All the time he devoted to our consultations, his patience and knowledge he shared with me throughout the semester without which this work wouldn't be possible. Working with him was truly an honor and experience of life time.

I am very grateful to Aggregate Professor PhD. Eng. Antonio Formisano, Department of Structures for Engineering and Architecture for his ever encouraging and motivating guidance and constant technical support during this thesis.

I am very thankful to Madame Isabelle Noirot International Relations, University of Liege and Barbora Skalova SUSCOS Course coordinator, Czech Technical University for their constant administrative supports right from the beginning.

I would also like to thank the whole SUSCOS team, all the coordinating universities and professors for all the efforts and dedication they put which makes SUSCOS such an amazing experience.

Finally, I would also like to express gratitude to my parents, sisters, brothers, friends and most importantly my wife Sangita Dandekhya for her constant encouragement and support, without which this work would have not been possible.

Anup Shrestha

ABSTRACT

KEYWORDS: *RC Building, Linear analysis, Response Spectrum Method, Lateral force Method, Pushover analysis, N2 method, ETABS, Indian Code, Nepal Code, Eurocode, ductility factor, response reduction factor.*

Earthquake risks and vulnerability to building structures have been identified by many countries and thus seismic analysis and design have become an integral part of their structural design process. Nepal has also recognized the necessity of seismic design following the past major earthquakes. It has developed the Nepal National Building Code (NBC) in 1994 AD but the implementation was very late. Most of this code was directly derived from the Indian code as the technology and construction practices in both the countries were similar. Due to this engineers mostly preferred to use Indian Code directly rather than Nepal code. However the code developed for Indian scenario and site condition may not be suitable for Nepalese context as Nepal is more prone to Earthquakes than India. This suggests a necessity to evaluate and compare both codes against much advance and developed code like Eurocode.

The aim of this thesis is to do a comparative study between the three seismic codes namely Nepal code (NBC 105, 1994), Indian Code (IS 1893-1, 2002) and Eurocode 8 (EN1998-1:, 2004) with a case study of a RC building located in Kathmandu, Nepal. The input parameters like materials, member size, soil type and ground motion were considered same for all three contexts in order to get fair results. In addition the effect of infill masonry walls in lateral load resisting capacity of the building was also checked in the building with these codes.

The research was carried out first by discussing the seismic analysis procedures (linear static and dynamic) outlined in the three codes. Then the analysis procedures introduced in the respective codes were compared and contrasted considering how they handle the major effects, characteristics of the structures and geotechnical considerations etc.

To get a better comparative view a RC building was analyzed and designed in “ETABS” software using linear static and dynamic procedures according to all three codes. The performance of the building under the parameters like base shear, storey displacement, inter-storey drift and reinforcement demands on the concrete members were compared for all three codes. A static nonlinear (pushover) analysis process was also carried out to get accurate performance level of the existing building.

The results showed that Eurocode has given highest base shear and drift values in many cases. It also made clear that the Indian and Nepal code lacks in addressing many issues like consideration of structural irregularity, infill walls, P-delta effects, non-linear analysis etc. The research showed that the study building was under performance in damage limitation and global behavior for Eurocode and the pushover analysis verified it. Thus a retrofitting intervention using all steel buckling restrained braces (BRB) was suggested for the study RC building after which a fair behavior factor close to code recommendation was achieved. A significant improvement in the ductility and strength of the structure was obtained using steel BRB solution.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABBREVIATIONS:

ADRS	Acceleration – Displacement Response Spectrum
AFD	Axial Force Diagram
ATC	Applied Technical Council
BMD	Bending Moment Diagram
BRB	Buckling Restrained Brace
CM	Center of Mass
CP	Collapse Prevention
CQC	Complete Quadratic Combination
CR	Center of Rigidity
CSI	Computers and Structures Inc.
CSM	Capacity Spectrum Method
DBE	Design Base Earthquake
DCH	Ductility Class High
DCM	Ductility Class Medium
DOF	Degree Of Freedom
EC	Eurocode
FEMA	Federal Emergency Management Agency
FRP	Fiber Reinforced Polymer
IO	Immediate Occupancy
IS	Indian Standard
LS	Life Safety
MCE	Maximum Considered Earthquake
MRF	Moment Resisting Frame
NBC	Nepal Building Code
NDT	Non Destructive Test
OL	Operation Level
PGA	Peak Ground Acceleration
RC	Reinforced Concrete
Sa	Spectral Acceleration
Sd	Spectral Displacement
SFD	Shear Force Diagram
SPT	Standard Penetration Test
SRSS	Square Root of Sum of Square

1.0 INTRODUCTION

1.1 Background

Nepal being located in a seismically active region has a long history of devastating earthquakes. The main source of earthquakes in Nepal and the Himalayan region is the subduction of the Indian plate underneath the Eurasian plate. The subduction of the Indian plate is at the rate of 25−30 cm/year, which causes contraction and stress concentration between the plate boundaries. Seismicity is considered to be high in this region based on the frequency and intensity of past earthquakes. Several major earthquakes were reported in 1255 AD, 1810 AD, 1866 AD, 1934 AD, 1980 AD, 1988 AD and the most recent one in 2015 AD in Nepal (Bilham & et al, 1995). Moreover the recent earthquake on 25 April 2015 with a magnitude of M_w 7.8 which hit central Nepal and its vicinity (USGS, 2015) caused 8,790 casualties and 22,300 injuries (CBS & GoN, 2015). Around 755,549 residential buildings, 4000 government offices, and 8200 school buildings were damaged due to this earthquake (CBS & GoN, 2015). The hypo-central depth was about 15 km and it was immediately followed by strong aftershock of M_w 6.7. The earthquake was located at Gorkha district of western Nepal near the Barpak village around 77 km NW of Kathmandu. A strong aftershock of MW 7.3 also jolted central Nepal on 12 May which further enhanced the damage and casualties.

The earthquake in 1988 prompted serious concern for the safety of the infrastructure. Following this major earthquake event, the Department of Urban Development and Building Construction (DUDBC) of the Ministry of Physical Planning and Works (MPPW) developed the Nepal National Building Code (NBC) in 1994 AD, with the assistance of the United Nations Development Program and United Nations Center for Human Settlement (UN-HABITAT). NBC was established when the Building Construction System Improvement Committee (established by the Building Act 1998) authorized MPPW to implement the code. Principally, the seismic design of structures in Nepal is based on NBC 105 (1994). However, most of the existing buildings in Nepal are designed based on the Indian standard code. This is because almost all of the engineering institutions' teachings in Nepal are based on Indian writer books, curriculums and their codes and also because Nepalese codes lack sufficient information to address the current design standards.

1.2 Scope of the study

Most of the buildings in rural Nepal are made of traditional adobe, stone/brick masonry and wooden framed structures. This comprises of about 80% of housings in the whole nation. In urban areas unreinforced brick masonry structures and Reinforced Cement Concrete (RCC) buildings are more common (Gautam, Rodrigues, & et al, 2016). This study has been limited to reinforced concrete building only because the RC construction in Nepal has been mushrooming and surpassed any other construction types in urban areas recently. It is replacing most of the traditional housing techniques of adobe and masonry constructions both in rural and urban areas of the country. Further very tall buildings are also not common in the country, the tallest building being less than 60 m till date. Therefore this research is more towards the multistory mid-height buildings that can be found in almost all major cities of the country. Moreover, majority of RC construction is covered by non- engineered to pre-engineered construction as owner built houses (Gautam, Rodrigues, & al, 2016). Pre-engineered construction here means the buildings built using mandatory rule of thumb given in Nepal Building Code (NBC 205). Only few percentages of buildings are well engineered using code provided analysis and design methods. This research is intended to those few engineered buildings mostly of which are based upon Indian code provision.

The main points of this research study can be pointed out as,

- To discuss and compare the seismic analysis procedures described in the Indian code (IS 1893 (part 1):2002), Nepal Building Code (NBC 105:1994) and the Eurocode 8 (EN 1998-1:2004).
- To demonstrate through case study of an existing RC building in Kathmandu how to apply the static and dynamic seismic analysis procedures described in selected codes to analyze buildings in Nepal.
- To compare the analysis and design results on the case study building and check its safety and performance against these codes.
- To perform a static non-linear analysis (pushover) and to recommend appropriate retrofitting intervention on the case study building if the existing building does not meet the standard required.

1.3 Limitation

In this study there were certain areas which were beyond the scope of this thesis intention. There were certain limitations of this study that are listed below.

- This study only deals with the multi-story RC buildings and is not meant to generalize for other types of building structures.
- The slab, staircase, being secondary structure elements: and foundation and retaining wall design checks and verifications being part of geo-technical part were not conducted in this study.
- For the pushover analysis although it is required to get the exact structure data of the existing building with Non Destructive Tests, we were forced to use the design data as all the construction drawings and initial design data were available. NDT was beyond this student’s reach.
- No experimental analyses were carried out during this study.
- For the retrofitting intervention the detail member and connection designs of the retrofitting elements were not carried out and only global behavior was evaluated.

1.4 Methodology

Firstly a thorough literature review on the above mentioned seismic codes i.e the Indian code (IS 1893-1:2002), the Nepal Code (NBC 105:1994) and the Eurocode, EC-8 (EN 1998-1:2004) was carried out. In this section, the analysis procedures that have been established in each of those codes were then outlined in step by step.

To demonstrate the analysis procedures established in above codes of practice, an existing reinforced concrete building located in Kathmandu, Nepal with 7 floors was selected and analyzed according to the guidelines provided in respective codes of practice. For fair comparison actual behavior of the building during performance check the materials, structural member sizes, construction techniques, soil site conditions and reference peak ground acceleration/zone factors have been taken from that of actual building location that is Kathmandu, Nepal.

Since it better represents the actual behavior of the structure, a three dimensional computer model of the building was made with elements of actual sizes, according to the guidelines provided in relevant sections of the particular codes of practice. For all the modeling and analysis purposes, computer software “ETABS 2016” version 16.0.3 has been used.

First linear static and dynamic seismic analysis methods were carried out. The analysis was carried out without considering infill masonry walls first. The analysis and design output results like drifts, base shear and reinforcement requirements (Demand/Capacity ratio) for different codes were studied and compared. The verification checks for columns and beams were carried out in ETABS and compared with each code. Same were done for the structure considering infill masonry walls.

Next, a non-linear static analysis (pushover) was carried out to check the performance of the existing building. In this analysis the building was modeled as close as possible to existing building considering the effects of infill masonry façade walls as an equivalent strut model. The global performance of the structure was evaluated with the pushover (Base Shear vs Displacement) curve and the actual structure behavior factor **(q)** was calculated based on (EN1998-1:, 2004) N2 Method.

Finally, if the verification was not satisfied an appropriate rehabilitation intervention was proposed on the case structure to improve its global behavior and response under given seismic action.

2.0 LITERATURE REVIEW

2.1 General

Seismic analysis of structures has become an essential part of the structural design process in almost all over the world lately. For this purpose some countries have developed their own codes of practice and they therefor analyze and design the structures accordingly. However for many countries like Nepal who do not have their own proper codes of practice have to depend upon some other countries codes which can be used for their purposes with appropriate adjustments. Mostly countries adopt code of practice from countries having similar nature of seismic activities and similar construction practices and materials used.

Nepal for instance although have developed Nepal National Building Code (NBC) in 1994, majority of clauses and provisions in its codes are directly derived from a much older and developed Indian Building Code. The seismic code of Nepal NBC 105:1994 is very superficial dealing only with the linear static method of analysis. There is no provision for retrofitting of existing buildings and also for seismic analysis of structures other than reinforced concrete like masonry and steel. There is also no provision for a response reduction factor (behavior factor) in the Nepalese seismic code (NBC105, 1994). However, the horizontal seismic coefficient is calculated by basic seismic coefficient, zone factor, important factor and structural performance factor. Likewise, there are also some drawbacks in IS codes too. In IS 1893:2002, the code does not address the effect of the load path, structural configuration and irregularities on the response reduction factor (Chaulagain & al, 2014). So a comparative study of these codes seems to be necessary.

Firstly the analysis procedures established in all three codes were outlined in brief, highlighting how those codes are used in analysis process. Then those codes of practice were compared considering how those codes have defined different parameters and how they have proposed values for them, which is very important to find out the advantages and disadvantages of adopting one code over the other.

2.2 Analysis procedure according to Indian Code (IS 1893-1, 2002)

2.2.1 General

The design approach adopted in this standard is to ensure that structures possess at least a minimum strength to withstand minor earthquakes (<Design Base Earthquake, DBE), which occurs frequently, without damages; resist moderate earthquakes (DBE) without significant structural damage though some non-structural damage may occur; and aims that structures withstand a major earthquake (Maximum Considered Earthquake, MCE) without collapse.

2.2.2 Horizontal elastic response spectra

The IS 1893 (part 1):2002 has defined the elastic response spectra, $\frac{s_a}{g}$ for 5 percent damping to be used in seismic analysis as follows.

$$0.00 \leq T \leq T_B \;, \frac{S_a}{g} = 1 + 15T$$

$$T_B \leq T \leq T_C \;, \frac{S_a}{g} = 2.5$$

$$T_C \leq T \leq 4.00 \;, \frac{S_a}{g} = S/T$$

Where,

$\frac{S_a}{g}$: 5 percent spectra

T : natural period of the structure

T_B : lower limit of the period of the constant spectral acceleration branch

T_C : upper limit of the period of the constant spectral acceleration branch

S : soil factor

The horizontal elastic response spectra are given for three types of soil classified based on the Standard Penetration Test value (N_{SPT}). For the soul classification and the corresponding parameters defining the elastic response spectra see Table 1.

Table 1: Soil Classification and Parameters defining horizontal elastic response spectra (IS 1893-1, 2002)

Soil Type	N_{SPT}	S	T_B	T_C
I (Rock)	>30	1	0.1	0.4
II (Medium)	10-30	1.36	0.1	0.55
III (Soft)	<10	1.67	0.1	0.67

2.2.3 Vertical Component of the seismic action

Vertical acceleration shall be considered in structures as described in Clause 6.1.1 of IS 1893 (Part 1): 2002, for structures with large spans, those in which stability is a criterion for design, or for overall stability analysis of structures. Reduction in gravity force due to vertical component of ground motions can be particularly detrimental in cases of pre-stressed horizontal members and of cantilevered members.

The design acceleration spectrum vertical motions, when require, may be taken as two-third of the design horizontal acceleration spectrum (see Clause 6.4.5) (IS 1893-1, 2002).

2.2.4 Design horizontal seismic coefficient

The design horizontal seismic coefficient, A_h has been defined as follows (IS 1893-1, 2002),

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

Where,

Z : Zone factor given in Table 2 (IS 1893-1, 2002), is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the Maximum

Considered Earthquake (MCE) zone factor to the factor for Design Basis Earthquake (DBE).

I : Importance factor, as defined in table 6 of (IS 1893-1, 2002), depending upon the functional use of the structures, characterized by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance.

R : Response reduction factor, as defined in table 7 of (IS 1893-1, 2002), depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However, the ratio (*I/R*) shall not be greater than 1.0

$\frac{S_a}{g}$: Average response acceleration coefficient.

Table 2: Zone factor, Z (Table 2 of (IS 1893-1, 2002))

Seismic Zone	**II**	**III**	**IV**	**V**
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z value	0.10	0.16	0.24	0.36

Table 3: Importance Factor, I (Table 6 of (IS 1893-1, 2002)

S.No.	**Structure**	**Importance Factor**
i)	Important service and community buildings, such as hospitals; schools; monumental structures; emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire station buildings; large community halls like cinemas, assembly halls and subway stations, power stations	1.5
ii)	All other buildings	1.0

Notes:

1. The design engineer may choose values of importance factor *I* greater than those mentioned above.
2. Buildings not covered in S.No. (i) and (ii) above may be designed for higher value of I, depending on economy, strategy considerations like multi-storey buildings having several residential units.
3. This does not apply to temporary structures.

Table 4: Response reduction factor[1], R (Table 7 of (IS 1893-1, 2002))

S No.	Lateral Load resisting system	R
	Building Frame Systems	
i)	Ordinary RC moment resisting frame (OMRF)[2]	3.0
ii)	Special RC moment resisting frame (SMRF)[3]	5.0
iii)	Steel frame with	
	a) Concentric braces	4.0
	b) Eccentric braces	5.0
iv)	Steel moment resisting frame designed as per SP 6 (6)	5.0
	Building with shear walls[4]	
v)	Load bearing masonry wall buildings[5]	
	a) Unreinforced	1.5
	b) Reinforced with horizontal RC bands	2.5
	c) Reinforced with horizontal RC bands and vertical bars at corners of rooms and jambs of openings.	3.0
vi)	Ordinary reinforced concrete shear walls[6]	3.0
vii)	Ductile shear walls[7]	4.0
	Building with dual systems[8]	
viii)	Ordinary shear wall with OMRF	3.0
ix)	Ordinary shear wall with SMRF	4.0
x)	Ductile shear wall with OMRF	4.5
xi)	Ductile shear wall with SMRF	5.0

(Note: Refer Table 7 of IS 1893 (Part 1): 2002 for full details, which are described by superscripts 1 to 8)

2.2.5 Seismic analysis of buildings

2.2.5.1 Seismic weight of the building

The seismic weight of a building shall be calculated as per Clause 7.43 of IS 1893 (Part 1): 2002. The seismic weight of the whole building is the sum of the seismic weights of all the floors. The seismic weight of each floor is its full dead load plus an appropriate amount of imposed loads as given in table 8 of IS 1893 (Part 1): 2002.

Table 5: Percentage of imposed load to be considered in seismic weight calculation in (Table 8 of (IS 1893-1, 2002))

Imposed uniformity distributed floor loads (KN/m^2)	Percentage of imposed load
Upto and including 3.0	25
Above 3.0	50

2.2.5.2 Structural Irregularity in Plan

A building shall be categorized as irregular, if at least one of the conditions described in table 4 and 5 of IS 1893-1:2002 are applicable (Refer clause 7.1 of (IS 1893-1, 2002))

A building shall be considered as irregular in plan, if at least one of the conditions described below is applicable (Refer Table 4 of IS 1893-1:2002).

- ✓ Torsional irregularity:

  Torsional irregularity to be considered to exist when the maximum storey drift, computed with design eccentricity, at one end of the structures transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structures.

- ✓ Re-entrant corners:

  Plan configuration of a structure and its lateral force resisting system contain re-entrant corners, where both projections of the structure beyond the re-entrant corner are greater than 15 percent of its plan dimension in the given direction.

- ✓ Diaphragm discontinuity:

  Diaphragm with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next.

- ✓ Out-of-Plane offsets:

  Discontinuities in a lateral force resistance path, such as out-of-plane offsets of vertical elements.

- ✓ Non-parallel System:

  The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes or the lateral force resisting elements.

2.2.5.3 Vertical irregularity

A building shall be considered as vertically irregular, if at least one of the conditions described below is applicable (Refer Table 5 of IS 1893-1:2002).

- ✓ Stiffness irregularity

  (a) Soft storey:

  A soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above.

  (b) Extreme soft storey:

  An extreme soft storey is one in which the lateral stiffness is less than 60 percent of that in the storey above or less than 70 percent of the average stiffness of the three storeys above.

- ✓ Mass irregularity:
  Mass irregularity shall be considered to exist where the seismic weight of any storey is more than 200 percent of that of its adjacent storeys. The irregularity need not be considered case of roofs.
- ✓ Vertical geometric irregularity:
  Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any storey is more than 150 percent of that in its adjacent storey.
- ✓ In-Plane Discontinuity in vertical elements resisting lateral force:
  An in-plane offset of the lateral force resisting elements greater than the length of those elements.
- ✓ Discontinuity in capacity – Weak storey:
  A weak storey is one in which the storey lateral strength is less than 80 percent of that in the storey above.

2.2.5.4 Structural Analysis

IS 1893 (Part 1): 2002 describes two types of linear-elastic analysis as:

I. Lateral force method of analysis also called *Seismic Coefficient Method* (Static)

II. Modal Response Spectrum analysis (Dynamic)

a) The use of above two methods of analysis shall be decided based on the structural characteristics of the buildings.

b) For the consequences of structural regularity on the structural analysis method, refer Table – 6 (Clause 7.8.1 of IS (IS 1893-1, 2002))

Table 6: Consequences of structural regularity on structural model and the analysis method

Regularity	Building Height (m)	Zone	Analysis Method
Regular	>40m	IV, V	Dynamic Analysis
	>90m	II, III	Dynamic Analysis
	All other buildings		Lateral Force Method
Irregular	>12m	IV, V	Dynamic Analysis
	>40m	II, III	Dynamic Analysis
	All other buildings		Lateral Force Method

Note-

For irregular buildings, lesser than 40m height on zones II and III, dynamic analysis, even though not mandatory, is recommended in IS 1893 (Part 1) : 2002.

2.2.5.5 Static Lateral force method of analysis

The total design lateral force or design seismic base shear (V_B) along any principal direction shall be determined by the following expression (Refer Clause 7.5.3 of (IS 1893-1, 2002)),

$V_B = A_h W$

Where,

A_h: Design horizontal acceleration spectrum value using the fundamental natural period T_a in the considered direction of vibration.

W: Seismic weight of the building.

2.2.5.6 Fundamental natural period

The approximate fundamental natural periods of vibration (T_a), in seconds for different types of buildings have been defined as follows (Refer Clause 7.6.1 of (IS 1893-1, 2002));

- ✓ For a moment-resisting frame building without brick infill panels may be estimated as,

  $T_a = 0.075\, h^{0.75}$ for RC frame building

  $= 0.085\, h^{0.75}$ for steel frame building and

- ✓ For all other buildings,

  $$T_a = \frac{0.09h}{\sqrt{d}}$$

  Where,

  H = Height of the building in m and

  d = Base dimension of the building at the plinth level in m, along the considered direction of the lateral force.

2.2.5.7 Distribution of design force

The design base shear (V_B) shall be distributed along the height of the building as per the following expression (Refer Clause 7.7.1 of (IS 1893-1, 2002));

$$Q_i = V_B \frac{w_i}{\sum_{j=1}^{n} w_j h_j^2} h_i^2$$

Where

Q_i: Design lateral force at floor *i*,

W: Seismic weight of the floor *i*,

h_i: Height of floor i measured from base,

n: Number of stories in the building is the number of levels at which masses are located

2.2.5.8 Dynamic analysis – Response Spectrum Method

This type of analysis is generally recommended to use for any building. The following are the important aspects that should be considered in the analysis procedure in accordance with the code.

a) When the design bases shear (V_B), obtained by response spectrum analysis is lesser than 80% the base shear ($\overline{V_B}$) (CSI Knowledge base, America, 2012), calculated using a fundamental period T_a, where T_a is as per section 7.6 of IS 1893 (Part 1): 2002, all the response quantities shall be multiplied by $0.8x\overline{V_B}/V_B$.

b) The number of modes to be used in the analysis should be such that the sum total of modal masses of all modes considered is at least 90 percent of the total seismic mass correction beyond 33 percent. If modes with natural frequency beyond 33 Hz are to be considered, modal combination shall be carried out only for modes up to 33 Hz. The effect of higher modes shall be included by considering missing mass correction following well established procedures (Refer Clause 7.8.4.2 of (IS 1893-1, 2002)).

c) Combination of modal responses is an important step in the modal response spectrum analysis. The Clause 7.8.4.4 of IS 1893 (Part 1): 2002 recommends the "Complete Quadratic Combination" CQC rule as an accurate procedure for this. For buildings with regular or normally irregular plan configurations, the code IS 1893-1:2002 allows to use a model as a system of masses lumped at the floor levels with each mass having one degree of freedom, that of lateral displacement in the direction under consideration (Refer Clause 7.8.4.5 of IS 1893-1:2002).

d) IS 1893-1:2002 recommends the accidental torsional effects to be taken into account in the seismic analysis whenever a spatial model is used.

2.2.5.9 Torsional effects

Provision shall be made in all buildings for increase in shear forces on the lateral force resisting elements resulting from the horizontal torsional moment arising due to eccentricity between the center of mass and center of rigidity as described in Clause 7.9 of IS 1893 (Part 1) : 2002. The design forces calculated are to be applied at the center of mass appropriately displaced so as to cause design eccentricity between the displace center of mass and center of rigidity. However, negative torsional shear shall be neglected.

The design eccentricity, e_{di} to be used at floor *i* shall be taken as:

$e_{di} = \{1.5\ e_{si} + 0.05\ b_i\}$

or $\{e_{si} - 0.05\ b_i\}$

Whichever of these gives the more severe affect in the shear of any frame where,

e_{si} = Static eccentricity at floor i defined as the distance between center of mass and center of rigidity.

b_i = Floor plan dimension of floor *i*, perpendicular to the direction of force.

2.2.5.10 Storey drift limitation

The storey drifts in any storey due to the minimum specified design lateral force, with partial safety factor of 1.0, shall not exceed 0.004 times the storey height (Refer Clause 7.11.1 of (IS 1893-1, 2002)). For the purpose of displacement requirements only, it is permissible to use seismic force obtained from the computed fundamental period (T) of the building without the lower bound limit on design seismic force specified in Clause 7.8.2 of IS 1893 (Part 1) : 2002.

There shall be no drift limit for single storey building which has been designed to accommodate storey drift.

2.3 Analysis procedure according to Eurocode 8, (EN1998-1:, 2004)

2.3.1 Design seismic action

The structures shall be designed to fulfill the two fundamental requirements; no-collapse requirement and damage limitation requirement, as stated in EN 1998-1:2004 (EC 8). The proposed peak ground acceleration values will represent the seismic action for no-collapse requirement and they will be different for buildings of different importance classes.

Table 7: Classification of buildings into important classes (EN1998-1:, 2004)

Importance Level	Classification	Examples
I	Buildings of minor importance for safety of public and other property	Agricultural buildings, isolated structures, domestic structures
II	Buildings of low-moderate importance for safety of public and other properties	Hotels, offices, apartment buildings of less than 10 storeys high, Factories up to 4 storeys high Car parking buildings, Shopping centres less than 10,000 m^2 gross area, Public assembly buildings for fewer than 100 persons Emergency medical and other emergency facilities not designated as post-disaster,
III	Building of significant importance for safety of pubic and other properties	Hotels, offices, apartment buildings over 10 storeys high, Factories and heavy machinery plants over 4 storeys high Shopping centres of over 10000m^2 gross area excluding parking. Public assembly buildings for more than 100 persons Airport Terminals, principal railway stations
IV	Buildings of greater importance with post disaster functions for civil protection	Pre-schools, Schools, colleges, universities, Major infrastructure facilities eg. Power stations, substations Medical facilities for surgery and emergency treatment, Hospitals, Fire and police stations, Ambulance facilities Buildings housing toxic or explosive substances in sufficient quantities to be dangerous to the public if released Extreme hazard facilities (Dams etc.)

The structures shall be classified into four categories (Table 7). The importance class I includes the structures which does not require an explicit seismic consideration in the design process. The importance class II, III and IV include the structures identified as important during an earthquake event considering their function, the consequences of failure and the economic aspects. Therefore, importance class II, III and IV buildings shall be designed for seismic actions having 475, 1500 and 2500 year return periods respectively (Prasanna, 2016).

The design peak ground acceleration value for each category of buildings shall be calculated as

$$a_g = \gamma_I . a_{g,475}$$

Where,

a_g : Design peak ground acceleration

γ_1 : Importance factor (Refer Table-2)

$a_{g,475}$: Peak ground acceleration for 475 years return period seismic action (Refer Table 2: Note)

Table 8: Importance Factor

Importance Class	γ_1
I	--
II	1
III	1.5
IV	1.8
Note: For Kathmandu, the (reference) peak ground acceleration for 475 year return period shall be taken as 0.25g. (Wijeyewickrema & et al, 2011)	

2.3.2 Horizontal elastic response spectra

Eurocode 8 (EN1998-1:, 2004) defines horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ by the following expressions

$$0 \leq T \leq T_B: S_e(T) = a_g . S . [1 + \frac{T}{T_B} . (\eta . 2,5 - 1)]$$

$$T_B \leq T \leq T_C: S_e(T) = a_g . S . \eta . 2,5$$

$$T_C \leq T \leq T_D: S_e(T) = a_g . S . \eta . 2,5[\frac{T_C}{T}]$$

$$T_D \leq T \leq 4s: S_e(T) = a_g . S . \eta . 2,5[\frac{T_C T_D}{T^2}]$$

Where

$S_e(T)$ is the elastic response spectrum;

T is the vibration period of a linear single-degree-of-freedom system;

a_g is the design ground acceleration on type A ground ($a_g = \gamma_I.a_{gR}$);

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor with a reference value of η = 1 for 5% viscous damping,

The values of the periods T_B, T_C and T_D and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type.

Table 9: Values of parameters describing the recommended Type 1 elastic spectra (EN1998-1:, 2004)

Ground Type	S	T_B(s)	T_C(s)	T_D(s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

2.3.3 Horizontal design response spectra

The design response spectrum for the seismic analysis of buildings shall be obtained by reducing the elastic response spectra by the value of behavior factor (q) as recommended in EC 8 and are given in the specific section of the code. The design response spectra shall be then given as

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

$$T_B \leq T \leq T_C: S_d(T) = a_g.S.\frac{2,5}{q}$$

$$T_C \leq T \leq T_D: S_d(T)\begin{cases} = a_g.S.\frac{2,5}{q}[\frac{T_C}{T}] \\ \geq \ \beta.a_g \end{cases}$$

$$T_D \leq T: S_d(T) = \begin{cases} a_g.S.\frac{2,5}{q}[\frac{T_C T_D}{T^2}] \\ \geq \ \beta.a_g \end{cases}$$

Where

a_g, S, T_C and T_D are as defined in section 2.2.2 above;

$S_d(T)$ is the design spectrum;

q is the behavior factor;

β is the lower bound factor for the horizontal design spectrum.

In selecting the behavior factors, the buildings of importance class II, III and IV shall be considered as ductility class medium (DCM) or high (DCH).

The behavior factor (q) used in the reinforced concrete structures as given in EN 1998-1/5.2.2.2 is given by

$$q = q_0 k_w \geq 1.5$$

Where

q: behavior factor

q_0: basic value of the behavior factor (Refer Table 10)

k_w: factor reflecting the prevailing failure mode in structural systems with walls (Refer Table 11)

Table 10: Basic value of the behavior factor (q0) for systems regular in elevation (EN 1998-1:2004/5.2.2.2 (Table 5.1))

STRUCTURAL TYPE	DCM	DCH
Frame system dual system, coupled wall system	3,0 α_u/α_1	4,5 α_u/α_1
Uncoupled wall system	3,0	4,0 α_u/α_1
Torsionally flexible system	2,0	3,0
Inverted pendulum system	1,5	2,0

α_u and α_1 are defined in EN 1998-1/5.2.2.2 (4) as

α_1 is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant;

α_u is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor α_u may be obtained from a nonlinear static (pushover) global analysis.)

In the absence of the calculated value of the multiplication factor $\alpha_{u/}$ α_1 as above, EN 1998-1/5.2.2.2 (5) gives approximate values for buildings regular in plan (Refer Table 12).

Table 11: Factor k_w reflecting the prevailing failure mode (EN 199801:2004/5.2.2.2 (11)P)

Structural Type[1]	k_w
Frame and frame-equivalent dual systems	1.00
Wall, wall-equivalent and Torsionally flexible systems	$0.5 \leq \frac{1+\alpha_0}{3} \leq 1$

1. For definitions of structural types refer EN 1998-1/5.2.2.1
2. α_0 is the prevailing aspect ratio of the walls of the structural system and if the aspect ratios h_{wi}/l_{wi} of all walls *i* of a structural system do not significantly differ, the prevailing aspect ratio shall be determined as (EN 1998-1/5.2.2.2 (12))

$$\alpha_0 = \sum h_{wi} / \sum l_{wi}$$

Where

h_{wi}: height of the wall *i*

l_{wi}: length of the section of wall *i*

Table 12: Approximate values for multiplication factor α_u/α_1 for buildings regular in plan (EN 1998-1;2004/5.2.2.2 (5))

Structural Type	α_u/α_1
Frames or frame equivalent dual systems	
One-storey buildings	1,1
Multistory, one bay frames	1,2
Multistory, multi bay frames or frame equivalent dual systems	1,3
Wall or wall-equivalent dual systems	
Wall systems with only two uncoupled walls per horizontal direction	1.0
Other uncoupled wall systems	1.1
Wall-equivalent dual, or coupled wall systems	1.2

2.3.4 Vertical component of the seismic action

EN 1998-1: 2004/4.3.3.5.2 states that if a_{vg} is greater than 0,25g (2,5 m/s^2) the vertical component of the seismic action should be taken into account in the cases listed below.

- For horizontal or nearly horizontal structures members spanning 20m or more;
- For horizontal or nearly horizontal cantilever components longer than 5m;
- For horizontal or nearly horizontal pre-stressed components;
- For beams supporting columns;
- In base-isolation systems;

The vertical component of the seismic action shall be represented by an elastic response spectrum given by the expressions in section 3.2.2.3 of (EN1998-1:, 2004).

2.3.5 Seismic analysis of buildings

2.3.5.1 Seismic mass of the building

EN 1998-1: 2004/3.2.4 states that seismic mass of the building which is taken into account in evaluating the inertial effects of the design seismic action is in the following combination of actions.

$\Sigma G_{k,j}$ "+" $\Sigma\psi_{E,i} \cdot Q_{k,i}$

Where

$G_{k,j}$: permanent load

$Q_{k,i}$: variable load

$\Psi_{E,i}$ $= \psi_{2,i}\varphi$ (EN 1998-1:4.2.4)

$\psi_{2,i}$: factor representing the quasi permanent value of the variable action (EN 1990:2002 – Table 13)

φ : (EN 1998-1: Table 4.2 – Refer Table 14)

2.3.5.2 Seismic load combination

The seismic load combination to be used in the analysis and design of buildings shall be taken as the load combination given in EN 1990: Basis for designs

$$\sum G_{k,j} + A_{E,d} + \sum \psi_{2,i} Q_{k,i}$$

Where,

G : permanent actions (self-weight and other dead loads)

A : design seismic action

Q : variable actions (live loads)

$\psi_{2,i}$: factor representing the quasi permanent value of the variable action (EN 1990:2002 – Table 13)

Table 13: Recommended values of ψ factors in EN 1990/Table A1.1

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

Table 14: Values of φ factors

Type of available action	Storey	φ
Categories A-C	Roof	1,0
	Storeys with correlated occupancies	0,8
	Independently occupied storeys	0,5
Categories D-F and archives		1,0

2.3.5.3 Structural Regularity in plan

The buildings shall be categorized as regular or irregular according to provisions given in EN 1998-1: 2004/4.2.3.

The criteria for regularity in plan are described in EN 1998-1:2004/4.2.3.2. The following conditions shall be checked in order to categorize the selected structure is regular in plan.

- Lateral stiffness and the mass distribution shall be approximately symmetrical in plan with respect to two orthogonal axes
- The plan configuration shall be compact
- The slenderness $\lambda = L_{max}/L_{min}$ of the building in plan shall not be greater than 4.
- The structural eccentricity e_{c0} and the torsional radius, r (at each level and for each direction of analysis) shall be

X-direction; $e_{0x} \leq 0.3r_x$

$r_x \geq l_s$

Y-direction; $e_{0y} \leq 0.3r_y$

$r_y \geq l_s$

For definitions of the center of stiffness and of the torsional radius in multi storey buildings refer “Manual for the seismic design of steel and concrete buildings to Eurocode 8” or refer EN 1998-1:2004 Section 4.2.3.2 (7)(8)(9).

2.3.5.4 Structural regularity in elevation

A building must satisfy all the requirements given in Clause 4.2.3.3 of EN 1998-1:2004 to be classified as regular in elevation. The requirements are briefed here as follows.

- All the vertical load resisting elements shall continue uninterrupted from foundation level to the top of the building or where set backs are present to the top of the setback.
- Mass and stiffness shall either remain constant with height or reduce only gradually without abrupt changes.
- In buildings with moment-resisting frames, the lateral resistance of each storey (i.e. the seismic shear initiating failure within that storey, for the code-specified distribution of seismic loads) shall not vary ‘disproportionately’ between storeys.

- Buildings with setbacks (i.e. where the plan area suddenly reduces between successive storeys) are generally irregular, but may be classified as regular if additional condition defined in the EC 8 are satisfied.

2.3.5.5 Structural Analysis

Section 4.3.3 of EN 1998-1:2004 describes two types of linear-elastic analysis as

I. Lateral force method of Analysis (Static)

II. Modal Response Spectrum analysis (Dynamic)

a) The use of above two methods of analysis shall be decided based on the structural characteristics of the building.

b) For the consequences of structural regularity on the structural analysis method refer Table – 15 (EN 1998-1:2004/ Table 4.1)

c) The criteria given in EN 1998-1:2004/ 4.3.1 shall be considered in the structural model used in the analysis

Table 15: Consequences of structural regularity on structural model and the analysis method (EN1998-1:, 2004)

Regularity		Allowed simplification		Behavior factor
Plan	Elevation	Model	Linear-elastic analysis	(for linear analysis)
Yes	Yes	Planar	Lateral Force	Reference value
Yes	No	Planar	Modal	Decreased value
No	Yes	Spatial	Lateral Force	Reference value
No	No	Spatial	Modal	Decreased Value

2.3.5.6 Static lateral force method of analysis

a) The static lateral force method of analysis is used for buildings only which satisfy the requirements given in EN 1998-1:2004/4.3.3.2.1 (2).

b) The total seismic base shear of the building shall be determined by the following expression (See EN 1998-1:2004/eq. 4.5).

$F_b = S_d\,(T_1)\cdot m\cdot\lambda$

Where

$S_d(T_1)$: the spectral acceleration obtained from the design response spectrum for the fundamental period of vibration T_1.

m : the seismic mass of the building (Refer Section 3.2.4 of EN 1998-1:2004)

λ : correction factor as given in EN 1998-1:2004/4.3.3.2.2

T_1 : fundamental period of vibration of the building as given in EN 198-1:2004/4.3.3.2.1 (2), (3), (4) and (5).

c) The total horizontal load shall then be distributed over the height of the building. Normally the distribution of lateral loads shall be done by making simple assumption on the mode shape, that is, for regular buildings, the mode shape is a straight line of which the displacement is directly proportional to the height (fundamental mode of vibration). With this assumption, the force at storey level F_k shall be determined as (EN 1998-1:2004/eq.4.10)

$$F_k = F_b \frac{z_i m_i}{\sum z_j m_j}$$

Where, z_i and z_j represent the heights of the masses m_i and m_j above the level of application of the seismic action.

2.3.5.7 Modal response spectrum analysis

a) This type of analysis is generally recommended to use for any building. The followings are the important aspects that should be considered in the analysis procedure in accordance with the code.

b) The response of all modes of vibration contribution significantly to the global response shall be considered. The code specifies that, this requirement is taken to be satisfied if

- The sum of the effective modal masses for modes taken into analysis amounts to 90% of the total mass of the structure
- All modes with effective modal masses greater than 5% of the total mass are taken

c) Combination of modal responses is an important step in the modal response spectrum analysis. EN 1998-1:2004/4.3.3.3.2 recommends the “Complete Quadratic Combination” (CQC) rule as an accurate procedure for this. The results of the modal analysis in each direction are then combined by the recommended methods as described in EN 1998-1:2004/4.3.3.5.1.

d) EC 8 recommends the accidental torsional effects to be taken into account in the seismic analysis whenever a spatial model is used.

2.3.5.8 Accidental torsional effects

In order to account for uncertainties in the location of masses and in the special variation of the seismic motion as described in EN 1998-1:2004/4.3.2, the calculated centre of mass at each floor level *i* shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity:

$e_{ai} = \pm 0.05.\ L_i$

where

e_{ai} is the accidental eccentricity of storey mass i from its nominal location, applied in the same direction at all floors;

L_i is the floor dimension perpendicular to the direction of the seismic action.

Whenever a spatial model is used for analysis, as described in section 4.3.3.3.3 of (EN1998-1:, 2004), the accidental torsional effects may be determined as the envelope of the effects resulting from the application of static loadings, consisting of sets of torsional moments M_{ai} about the vertical axis of each storey *i*:

$M_{ai} = e_{ai}.F_i$

2.3.5.9 Displacement

As described in EN 1998-1:2004/4.3.4, in the case of a linear analysis the displacement of a point of the structural system induced by the design seismic action is calculated by the product of displacement behavior factor and the displacement of the same point of the structural system as determined from the linear analysis.

$$d_s = q_d d_e$$

2.3.5.10 Inter-storey drift

EN 1998-1:2004/4.4.2.2 (2) defines the design inter-storey drift *(d_r)* as the difference of the average lateral displacements (d_s) at the top and bottom of the storey under consideration.

According to clauses 4.4.3.1 and 4.4.3.2 of EN 1998-1:2004, the inter-storey drift (d_r) should be limited in order to verify the damage limitation requirement given by the following expression.

$$d_r \upsilon \leq (\alpha).h.$$

Where, reduction factor υ, accounts for the lower return period to be considered in damage limitation requirement and it is 0.4 for the buildings of importance class III and IV and 0.5 for the buildings of important class I and II (section 4.4.3.2 (2) of (EN1998-1:, 2004)). The value of α has three different figures, 0.005, 0.0075 and 0.01 depending on the type of non-structural elements in the building. The ‘h’ is the height of the storey.

2.3.5.11 P-Δ effects

The clause 4.4.2.2 (2) of EN 1998-1:2004 recommends that P-Δ effects need not be taken into account if the value of inter storey drift sensitivity coefficient is less than 0.1. The inter storey drift sensitivity coefficient, θ is given by the expression below.

$$\theta = \frac{P_{tot}.d_r}{V_{tot}.h} \leq 0.10$$

Where d_r is inter-storey drift, h is the storey height, V_{tot} is the total seismic storey shear and P_{tot} is the total gravity load at and above the storey considered in the seismic design situation.

For the values of inter-storey drift sensitivity coefficient between 0.1 and 0.2, the code advices to multiply the seismic action effects obtained from the analysis by a factor equal to 1/(1-θ). However, the inter-storey drift sensitivity coefficient shall not exceed 0.3.

2.4 Analysis Procedure as described in Nepal National Building Code (NBC 105, 1994)

2.4.1 General

The NBC 105:1994 provides minimum requirements for the seismic design of structures. This codes requires to be applied in conjunction with IS 4326-1993 Code of practice for Earthquake Resistant Design and Construction of Buildings.

2.4.2 Design Spectra and Lateral Force Coefficients

2.4.2.1 Design Horizontal Seismic Coefficient for the Seismic Coefficient method

As per section 8.1.1 of (NBC 105, 1994) the design horizontal seismic force coefficient, C_d shall be taken as:

$$C_d = CZIK$$

Where

C : is the basic seismic coefficient for the fundamental translational period in the direction under consideration.

Z : is the seismic zoning factor as obtained from Figure 8.2 of (NBC 105, 1994) for appropriate location

I : is the importance factor for the structure as obtained from Table 8.1 of (NBC 105, 1994)

K : is the structure performance factor as given in Table 8.2 of (NBC 105, 1994)

2.4.2.2 Design Spectrum for the Modal Response Spectrum Method

Similarly the design spectrum, $C_d(T_i)$, as described in section 8.1.2 of (NBC 105, 1994) shall be taken as :

$$C_d(T_i) = C(T_i)ZIK$$

Where $C(T_i)$ is the ordinate of the basic response spectrum for translational period, T_i.

2.4.2.3 Basic Response Spectrum / Seismic Coefficient

The basic response spectrum as per section 8.1.4 of (NBC 105, 1994) is given by $C(T_i)$ and shall be determined from Figure 1 for the appropriate site subsoil category, and period, T_i. The same spectrum is used as a basic seismic coefficient, C as per section 8.1.3 of NBC 105:1994. The spectrum can be given as

$$0{,}00 \leq T \leq T_{B,}\ C = 0.08$$

$$T_B \leq T \leq 3.0\ C = \frac{S}{T}$$

where

C : Basic seismic coefficient or response spectrum coefficient

T : natural period of the structure

T_B : limit of the period of the constant spectral coefficient/acceleration branch

S : Soil factor

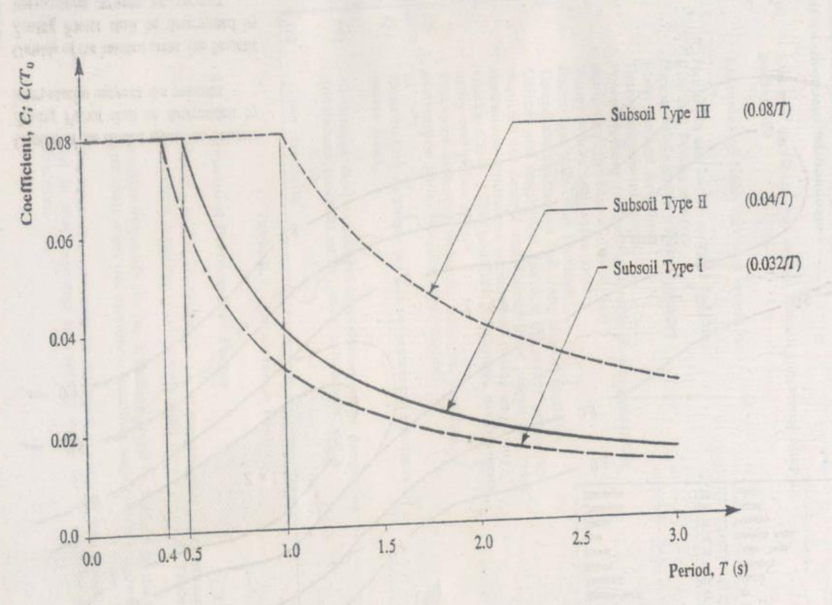


Figure 1: Basic Seismic Coefficient, C
Basic Response Spectrum, C(Ti)
(Fig. 8.1 of (NBC 105, 1994))

Table 16: Soil Type and Parameters defining basic response spectra

Soil Type	N_{SPT}	S	T_B
I (Rock)	>30	0.032	0.4
II (Medium)	10-30	0.04	0.5
III (Soft)	<10	0.08	1.0

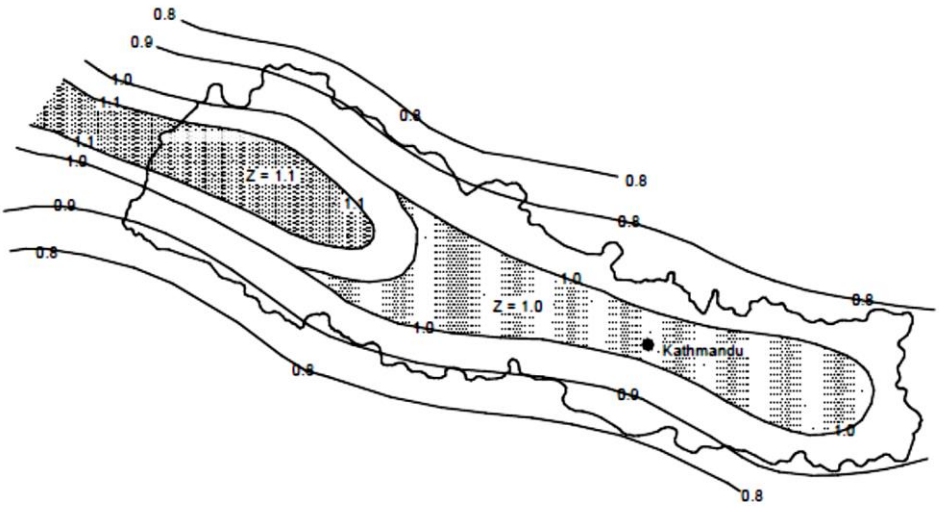


Figure 2: Seismic Zoning Factor, Z of Nepal
(Fig. 8.2 of (NBC 105, 1994))

Table 17: Importance Factor I (Table 8.1 (NBC 105, 1994))

	Type of Building	Importance Factor *I*
(a)	Monumental Buildings	1.5
(b)	Essential facilities that should remain functional after an earthquake Examples of these facilities would be : Hospitals and other medical facilities Fire and Police stations Emergency vehicle shelters/garages Food storage structures Emergency relief stores Power stations (including standby power-generating equipment for essential facilities) Water works and water towers Radio and television facilities Telephone exchanges and transmission facilities Offices and residential quarters for senior personnel required for central and district-level rescue and relief operations (ministers, secretaries, police and army chiefs; CDO, LDO and DDC chairmen, district-level army and police chiefs Places of assembly (schools, colleges, cinemas, convention halls, temples, dharmsalas).	1.5
(c)	Distribution facilities for gas or petroleum products in urban areas.	2.0
(d)	Structures for the support or containment of dangerous substances (such as acids, toxic substances, etc.).	2.0
(e)	Other structures	1.0

Table 18: Structural Performance Factor K and other Design Requirements for Horizontal Load-Resisting Systems of Buildings and other Structures (Table 8.2: (NBC 105, 1994))

Item	Structural Type	Minimum Detailing Requirements	Structural Performance Factor *K*
1.(a)	Ductile moment-resisting frame	Must comply with the detailing for ductility requirements of IS 4326 and for steel frames, the additional requirements of NBC 111-94	1.0
(b)	Frame as in 1(a) with reinforced concrete shear walls	For frames: as for 1(a). Reinforced concrete shear walls must comply with appropriate detailing for ductility requirements.	1.0[1]
2.(a)	Frame as in 1(a) with either steel bracing members detailed for ductility or reinforced concrete infill panels	For frames: as for 1(a). Steel bracing members must comply with the detailing for ductility requirements NBC 111-94. Reinforced concrete infill panels must comply with the detailing requirements of NBC 109-94	1.5[1,2]
(b)	Frame as in 1(a) with masonry infills	Must comply with the detailing for ductility requirements of IS 4326	2.0[1,2]
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

Notes:

1 *These factors shall apply only if the steel bracing members, the shear walls and/or the infill panels are taken into consideration in both the stiffness and lateral strength calculations.*

2 *These factors shall apply only if the frame acting alone is capable of resisting at least 25 percent of the design seismic forces.*

2.4.2.4 Vertical Seismic Forces

The effect of the vertical components of seismic motion need not be considered in design of a structure except as specified in section 12 of (NBC 105, 1994). Where consideration of vertical seismic forces is required, the design vertical seismic coefficient shall be taken as one half of the horizontal seismic coefficient given in section 8.1.1 of (NBC 105, 1994).

2.4.3 Methods of Analysis

2.4.3.1 General

Analysis for the design earthquake actions shall be in accordance with one of the following methods as per section 5.1 of NBC 105:1994.

(a) The Seismic Coefficient Method

(b) The Modal Response Spectrum Method

For structures of up to 40 m in height the Seismic Coefficient Method may be used. For all other structures the Modal Response Spectrum Method shall be used, section 5.2 (NBC 105, 1994).

The Modal Spectrum Method should be used for (NBC 105, 1994):

(a) Buildings with irregular configurations

(b) Buildings with abrupt changes in lateral resistance

(c) Buildings with abrupt changes in lateral stiffness with height

(d) Buildings with unusual shape, size or importance.

2.4.3.2 Seismic Weight

The seismic weight at each level, W_i, shall be taken as the sum of the dead loads and the seismic live loads between the mid-heights of adjacent storeys.

The seismic live load shall be taken as a percentage of the design live load as given in Table (NBC 105, 1994).

The seismic weight for roofs shall include allowance for ice if appropriate.

Table 19: Design Live Load percentage for seismic weight calculation, Table 6.1, (NBC 105, 1994)

Design Live Load	Percentage of Design Live Load
Up to 3 kPa	25
Above 3 kPa and for vehicle garages	50
For Roofs	NIL

2.4.3.3 Period of Vibration

For the purpose of initial member sizing, the following approximate formulae for fundamental time period T_1 may be used, section 7.3 (NBC 105, 1994):

(a) For framed structures with no rigid elements limiting the deflection:

$T_1 = 0.085\ H^{3/4}$ for steel frames

$T_1 = 0.06\ H^{3/4}$ for concrete frames

(b) For other structures:

$$T_1 = \frac{0.09\ H}{\sqrt{D'}}$$

2.4.3.4 Design Eccentricity

The design eccentricity, e_d, shall be determined as follows (section 8.2.2 (NBC 105, 1994)):

(a) If e_c is less than 0.1 b and the building is 4 storeys or less in height:

e_d may be taken as equal to 0.

(b) If e_c is less than 0.3 b and (a) does not apply;

$e_d = e_c + 0.1\ b$ or $e_d = e_c - 0.1\ b$

Whichever is the most severe for the element under consideration.

(c) If e_c is greater than 0.3 b, the structure should be analyzed using a three-dimensional modal response spectrum analysis with the mass at each level displaced by ± 0.1 b, whichever is the most severe for the element under consideration.

2.4.3.5 Seismic Coefficient Method

2.4.3.5.1 Horizontal Seismic Base Shear

The horizontal seismic shear force acting at the base of the structure, in the direction being considered, shall be:

$V = C_d W_i$

Where C_d is as defined in section 8.1.1 of (NBC 105, 1994).

2.4.3.5.2 Horizontal Seismic Forces

The horizontal seismic force at each level i shall be taken as:

$$F_i = VW_i h_i / \sum W_i h_i$$

Provided that:

(a) Where the height to width ratio of the horizontal load resisting system is equal to or greater than 3, then 0.1 V shall be considered as concentrated at the top storey and the remaining 0.9 V shall be distributed in accordance with the equation above.

(b) For chimneys and smoke-stacks resting on the ground, 0.2 V shall be considered as concentrated at the top and the remaining 0.8 V shall be distributed in accordance with the equation above.

(c) For elevated tanks, the force F_i is equal to V and acts through the centre of gravity of the total weight of the structure and contents.

The set of equivalent static forces specified in this section shall be assumed to act simultaneously at each level in the direction being considered and shall be applied through points eccentric to the centre of rigidity as specified in section **8.2.2 of** (NBC 105, 1994).

2.4.3.6 Modal Response Spectrum Analysis

2.4.3.6.1 Design Spectrum

The design spectrum used for the Modal Response Spectrum Method shall be as given in section 8.1.2 of (NBC 105, 1994). The relative response of each contributing mode *i* shall be determined by multiplying the mode response by the value of *C(Ti)* from section 8.1.2 of (NBC 105, 1994).

A sufficient number of modes shall be considered to ensure that at least 90% of the mass is participating in the direction under consideration.

2.4.3.6.2 Combination of Modal Effects

An established method shall be used for the combination of modal effects. The combination method shall take into account the effect of closely spaced modes. Modes shall be considered to be closely spaced if their frequencies are within 15%. The combined modal effects shall be scaled by the modal combination factor, S, where:

$$S = \frac{0.9C_d W_t}{\sum combined\ modal\ base\ shears\ in\ the\ direction\ under\ consideration}$$

Provided that S shall not be taken as less than 1.0.

2.4.3.7 Inter-Storey Deflections

The ratio of the inter-storey deflection to the corresponding storey height shall not exceed 0.010 nor shall the inter-storey deflection exceed 60 mm.

2.5 Comparison of analysis procedures as described in the Indian Code, the Eurocode and the Nepal Building Code

2.5.1 General

The sections 2.2, 2.3 and 2.4 have described the analysis procedure according to Indian code, Eurocode and Nepal Code respectively. This section has been used to discuss and compare the analysis procedures, which have been described in those codes of practice, the advantages and disadvantages between them, how those codes have defined different parameters and their proposed values for them and how those codes have considered different structural effects in their analysis etc.

2.5.2 Sub-soil conditions

In defining the elastic response spectra, the Eurocode have defined for five sub-soil conditions whereas the Indian and Nepal Code has defined the spectra only for three sub-soil conditions. The latter is simpler and does not require sophisticated soil tests.

2.5.3 Structural regularity

For the purpose of seismic design, building structures are categorized into being regular or non-regular. However, the regularity has been considered in seismic design process by different codes of practice in different ways.

The Eurocode has considered the effect of a building being irregular in various situations. The code recommends using a reduced value for basic behavior factor q_0 for buildings which are not regular in elevation and requiring dynamic three dimensional modal analyses for plan irregular buildings.

The Indian and Nepal code seems to address the irregularities by just requiring dynamic analysis.

2.5.4 Seismic hazard factor

According to the Eurocode, the design seismic actions have to be evaluated based upon Maximum Considered Earthquake (MCE), whereas the Indian code recommends to use a reduced zone factor (Z/2) in evaluating seismic actions representing the Design Base Earthquake (DBE) situation which consequently gives lower response values compared to Eurocode. The Nepal Seismic Code also recommends Zone factor like Indian code but with different values ranging from 0.8 – 1.1.

2.5.5 Period of Vibration

The fundamental period of vibration as per Eurocode and Indian code for preliminary approximation is given as:

$T_1 = 0.085H^{3/4}$ (for moment resisting steel frames)
$= 0.075\ H^{3/4}$ (for moment resisting concrete frames and eccentrically braced steel frames)
$= 0.05\ H^{3/4}$ (for all other structures).

In Nepal code the same is given as:

$T_1 = 0.085H^{3/4}$ (for moment resisting steel frames)
$= 0.06\ H^{3/4}$ (for moment resisting concrete frames and eccentrically braced steel frames)
$= 0.09\ H/\sqrt{D'}$ (for all other structures).

2.5.6 Seismic Weight/ Mass Source

The seismic weight of the building consists of full dead load plus appropriate portion or percentage of imposed/live load as specified in all seismic codes considered. However the percentage of imposed load to consider varies with codes.

The Indian code and Nepal seismic code agrees on a common percentage of imposed load to consider for mass source i.e. for LL upto and including 3.0 KN/m^2 →25% and above 3.0 KN/m^2 →50%. These codes also states that for calculation of design seismic forces of buildings, imposed load on roof need not be considered. But, weights of equipment and other permanently fixed facilities should be considered (IS 1893-1, 2002). In regions of severe snow loads and sand storms exceeding intensity of 1.5 KN/m^2, 20% of design snow load or sand load, respectively shall be included in the estimation of seismic weight (IS 1893-1, 2002).

The Eurocode on the other hand have a different approach to consider the proportion of imposed loads to calculate seismic mass of building which is described in section 3.2.4 of (EN1998-1:, 2004).

2.5.7 Behavior Factor or Response Reduction Factor

The earthquake loads imposed in the structure are typically greater than the loads considered in design. Most of the seismic design codes today include the nonlinear response of a structure through a response reduction factor. Different codes and guidelines specify the response reduction factor to scale down the elastic response reduction of a structure. The factor is termed as "behavior factor (q)" in Eurocode 8 and "response reduction factor (R)" in IS 1893. There is no provision for reduction factor in Nepal code NBC 105. Different codes specify different values of 'R' or 'q' factors depending on the type of structural system and ductility class of the structures. For example Eurocode specifies 'q' factor value for a DCH multi-bay, multi-storey RC MRF equals to 5.85 whereas for similar RC special Moment Resisting Frame (SMRF) Indian code specifies 'R' factor equals 5. The IS code does not address the effect of the load path, structural configuration and irregularities on the response reduction factor whereas the Eurocode 8 seems to be clearer regarding the effects of these parameters.

2.5.8 Design base shear force

Design base shear force can be determined either by static method or dynamic method of analysis, according to three of the codes considered. As per the Eurocode the design base shear forces can be determined by two of above methods independently. However, the Indian and Nepal code has defined a lower bound value for design base shear force. As per these codes, when the design base shear (V_B), obtained by response spectrum analysis is lesser than the base shear ($\overline{V_B}$), calculated using static method of analysis, then all the response quantities shall be multiplied by $0.8x\overline{V_B}/V_B$.

2.5.9 Accidental Torsional effect

In order to account for accidental torsional effect, the Eurocode and the Indian code recommend applying the earthquake loads at a position 0.05b from the nominal center of mass where as in Nepal code there is no provision for accidental torsional effect.

2.5.10 Provision for Eccentricity

Eccentricity in Eurocode is described as the distance between the center of mass and center of rigidity. The lateral loads are supposed to be applied at this static eccentricity point. But in Indian code and Nepal code this static difference between mass center and rigidity center is slightly modified to calculate design eccentricity.

Indian code gives design eccentricity as:

$e_{di} = \{1.5\ e_{si} + 0.05\ b_i\}$
or $\{e_{si} - 0.05\ b_i\}$

And Nepal Code gives design eccentricity as section 8.2.2 of (NBC 105, 1994):

$$e_d = e_c + 0.1\ b \text{ or } e_d = e_c - 0.1\ b$$

2.5.11 P-delta effects

The Eurocode has described the way to determine the *P*-delta effects in calculation based upon θ, the inter-storey sensitivity coefficient. However, the Indian code and Nepal code does not provide such provisions and methods to determine the *P*-delta effects in seismic design calculations and usually they are neglected.

2.6 Literature Review on Pushover Analysis

2.6.1 General

The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. A plot of the total base shear versus top displacement in a structure is obtained by this analysis that would indicate any premature failure or weakness. The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity on a building frame, and plastic rotation is monitored, and lateral inelastic forces versus displacement response for the complete structure is analytically computed. This type of analysis enables weakness in the structure to be identified. The decision to retrofit can be taken in such studies.

Federal Emergency Management Agency (FEMA) and Applied Technical Council (ATC) are the two agencies which formulated and suggested the Non-linear Static Analysis or Pushover Analysis under seismic rehabilitation programs and guidelines. This included documents FEMA-356, FEMA-273 and ATC-40.

2.6.2 Types of Pushover Analysis

2.6.2.1 General

Presently, there are two non-linear static analysis procedures available, one termed as the Displacement Coefficient Method (DCM), documented FEMA-356 and other the Capacity Spectrum Method (CSM) documented in ATC-40. Both methods depend on lateral load-deformation variation obtained by non-linear static analysis under the gravity loading and idealized lateral loading due to the seismic action. This analysis is called Pushover Analysis. The Eurocode (EN1998-1:, 2004) also documents an approach of pushover analysis called N2 Method which is also a kind of displacement coefficient method.

2.6.2.2 Capacity Spectrum Method

Capacity Spectrum Method is a non-linear static analysis procedure which provides a graphical representation of the expected seismic performance of the structure by intersecting the structure’s capacity spectrum with the response spectrum (demand spectrum) of the earthquake. The intersection point is called as the performance point, and the displacement coordinate dp of the performance point is the estimated displacement demand on the structure for the specified level of seismic hazard.

2.6.2.3 Displacement Coefficient Method

Displacement Coefficient Method is a non-linear static analysis procedure which provides a numerical process for estimating the displacement demand on the structure, by using a bilinear representation of the capacity curve and a series of modification factors or coefficients to calculate a target displacement. The point on the capacity curve at the target displacement is the equivalent of the performance point in the capacity spectrum method.

2.6.3 Performance Point

It is the point where the capacity spectrum intersects the appropriate demand spectrum. To have the desired performance in the structure it should be designed by considering these points of forces.

2.6.4 Building Performance Level

2.6.4.1 General

Building performance is the combined performance of both structural and non-structural components of the building. Different performance levels are used to describe the building performance using the pushover analyses, which are described below.

2.6.4.2 Operational level (OL):

As per this performance level building are expected to sustain no permanent damages. Structure retains original strength and stiffness. Major cracking is seen in partition walls and ceilings as well as in the structural elements.

2.6.4.3 Immediate occupancy level (IO):

Buildings meting this performance level are expected to sustain no drift and structure retains original strength and stiffness. Minor cracking in partition walls and structural elements is observed. Elevators can be restarted. Fire protection is operable.

2.6.4.4 Life Safety Level (LS):

This level is indicated when some residual strength and stiffness is left available in the structure. Gravity load bearing elements function, no out of plane failure of walls and tripping of parapet is seen. Some drift can be observed with some failure to the partition walls and the building is beyond economical repair. Among the non-structural elements failing hazard mitigates but many architectural and mechanical and mechanical systems get damaged.

2.6.4.5 Collapse Prevention Level (CP):

Buildings meeting this performance level are expected to have little residual strength and stiffness, but the load bearing structural elements function such as load bearing walls and columns. Building is expected to sustain large permanent drifts, failure of partitions infill and parapets and extensive damage to non-structural elements. At this level the building remains in collapse level.

2.6.5 Plastic Hinge

Location of inelastic action of the structural member is called as plastic hinge. The maximum moments caused by the earthquake occur near the ends of the beams and columns, the plastic hinges are likely to form there and most ductility requirements apply to section near the junction.

2.6.6 Assignment of Hinges for Pushover Analysis (ETABS, 2016)

For non-linear static analysis we may simulate post-yield behavior by assigning concentrated plastic hinges to frame and tendon objects. Elastic behavior occurs over member length, and then deformation beyond the elastic limit occurs entirely within hinges, which are modeled in discrete locations.

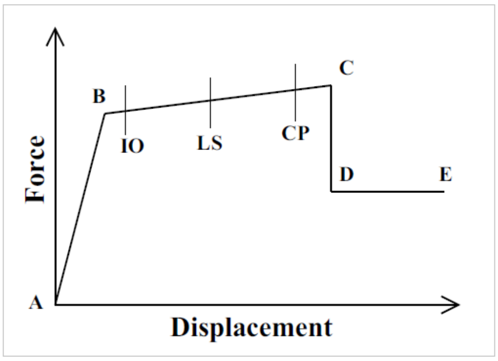


Figure 3: Force - Displacement curve of a Hinge.

Inelastic behavior is obtained through integration of the plastic strain and plastic curvature which occurs within a user-defined hinge length, typically on the order of member depth (FEMA 356, 2000). To capture plasticity distributed along member length, a series of hinges may be modeled. Multiple hinges may also coincide at the same location.

Plasticity may be associated with force-displacement behaviors (axial and shear) or moment-rotation (torsion and bending). Hinges may be assigned (uncoupled) to any of the six DOF. Post-yield behavior is described by the general backbone relationship shown to the right. The modeling of strength loss is discouraged, to mitigate load redistribution (which may lead to progressive collapse) and to ensure numerical convergence.

CSI Software automatically limits negative slope to 10% of elastic stiffness, though overwrite options are available. For informational purposes, additional limit states (IO, LS, CP) may be specified which are reported in analysis, but do not affect results. Unloading from the point of plastic deformation follows the slope of initial stiffness.

Both P-M2-M3 hinges and fiber hinges are available to capture coupled axial and biaxial-bending behavior. The P-M2-M3 hinge is best suited for nonlinear static pushover, whereas the fiber hinge is best for hysteretic dynamics (ETABS, 2016).

Hinge properties are used to define nonlinear force-displacement or moment-rotation behavior that can be assigned to discrete locations along the length of frame (line) objects or to the mid-height of wall objects. These nonlinear hinges are used during static nonlinear analysis, fast nonlinear analysis (FNA) modal time history analysis, and nonlinear direct integration time history analysis. For all other types of analysis, the hinges are rigid and have no effect on the behavior of the member. The number of hinges not only affects computation time, but also the ease in which model behavior and results may be interpreted. Therefore, it is strongly recommended that hinges be assigned only at locations where the occurrence of nonlinear behavior is highly probable.

Note: It is important that frame and wall objects be designed, e.g. reinforcement should be defined for concrete frames and walls, prior to running a nonlinear analysis utilizing hinges. This is done by defining frame sections with section designer in ETABS.

2.6.7 Capacity

2.6.7.1 General

It is defined as the expected ultimate strength (in flexure, shear and axial loading) of the structural components excluding the reduction factors commonly used in the design of concrete members. The capacity generally refers to the strength at the yield point of the element or structure's capacity curve. For deformation controlled component's, capacity beyond the elastic limit generally includes the effect of strain hardening.

2.6.7.2 Capacity Curve:

The plot between base shear and roof displacement is referred as capacity curve. Also, mentioned as pushover curve.

2.6.7.3 Capacity Spectrum

The capacity curve transformed from base shear v/s roof displacement (V v/s d) to spectral acceleration v/s spectral displacement (Sa v/s Sd) is referred as capacity spectrum.

2.6.7.4 Capacity Spectrum Method:

A nonlinear static procedure that produce a graphical representation of the expected seismic performance of the building by intersecting the structure's capacity curve with a response spectrum representation of earthquake's displacement demand on the structure, the intersecting point is called performance point and the displacement coordinate dp of the performance point is the estimated displacement demand on the structure for the specified level of hazard.

2.6.8 Demand Spectrum

Demand is represented by an estimation of the displacement or deformation that the structure is expected to undergo. This is in contrast to conventional, linear elastic analysis procedures in which demand is represented by prescribed lateral forces applied to the structure.
Demand Spectrum is the plot between average spectral acceleration versus time period. It represents the earthquake ground motion in capacity spectrum method.

2.6.9 N2 Method (EN1998-1:, 2004)

N2 is simple non-linear method described in Eurocode 8 (EN 1998-1:2004) used for calculation of structures during earthquakes. It combines multi degree pushover analysis with spectrum analysis of equivalent single degree of freedom (SDOF) system. It is formulated in acceleration-displacement format, which is very suitable for visual overview of basic variables that account for seismic response of the structure. N2 method can be considered as combination of pushover analysis and spectrum analysis. Inelastic demanded spectrum is obtained from elastic spectrum. Results obtained are accurate enough if structure has dominant first mode of oscillation. Seismic load (demand) in N2 method is defined in the shape of elastic acceleration spectrum. For better visualization seismic demand in N2 method is defined as elastic spectrum in acceleration-displacement (ADRS) format. (Mestrovic, Cizmar, & Pende, 2008)

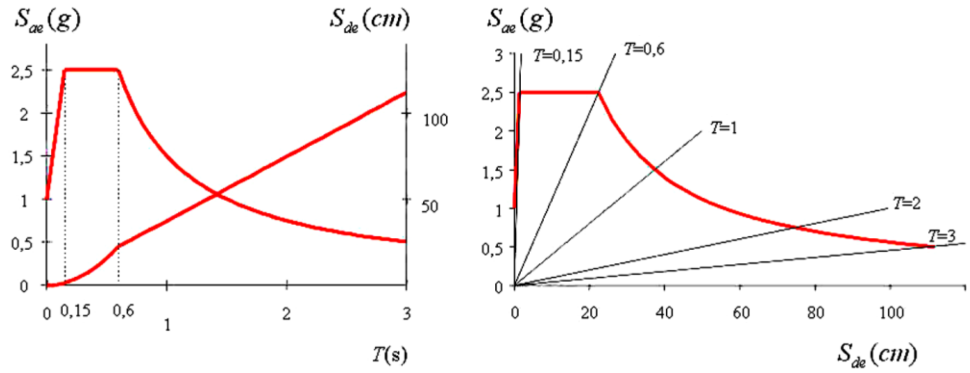


Figure 4: Acceleration Spectrum and ADRS Curve (N2 Method)

As this method is nonlinear, inelastic spectrum must be defined. Only two factors are needed: ductility factor and reduction factor. This kind of inelastic spectrum in acceleration-displacement format is called demand spectrum. Typical demand spectrum is shown in figure 4.

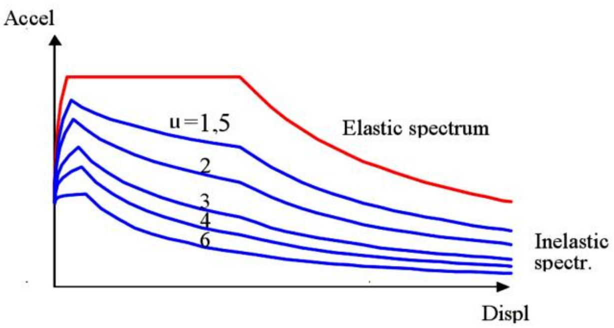
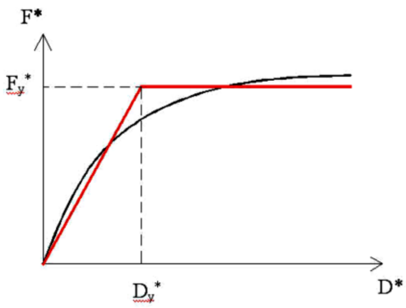


Figure 5: Inelastic Spectrum

Structure is modeled as 3D spatial model with multiple degrees of freedom (MDOF) model. With pushover method characteristic nonlinear force-displacement relation for MDOF can be calculated (usually base shear and displacement in highest point are used). Using transformation factor Γ transfer to equivalent single degree of freedom (SDOF) is made. Nonlinear force – displacement relation is simplified using ideal elastic – plastic relation as shown in figure 6.



$$D^* = \frac{D_t}{\Gamma}$$

$$F^* = \frac{V}{\Gamma}$$

Figure 6: Idealization of Force - Displacement relation

As the final result capacity diagram in acceleration – displacement format is obtained (see figure 7). Demand spectrum and capacity diagram are always on the same figure. Intersection of radial line which corresponds to elastic period T* of idealized bilinear system with elastic demand spectrum (μ=1) defines demand elastic displacement S_{de}. Inelastic demand, related to acceleration Say and displacement S_d corresponds to intersection of capacity diagram with demand spectrum (with demanded ductility μ). For medium and short range periods structures ($T^*\geq T_C$) rule of equal displacement can be applied (demand inelastic displacement S_d equals to demand elastic displacement S_{de}).

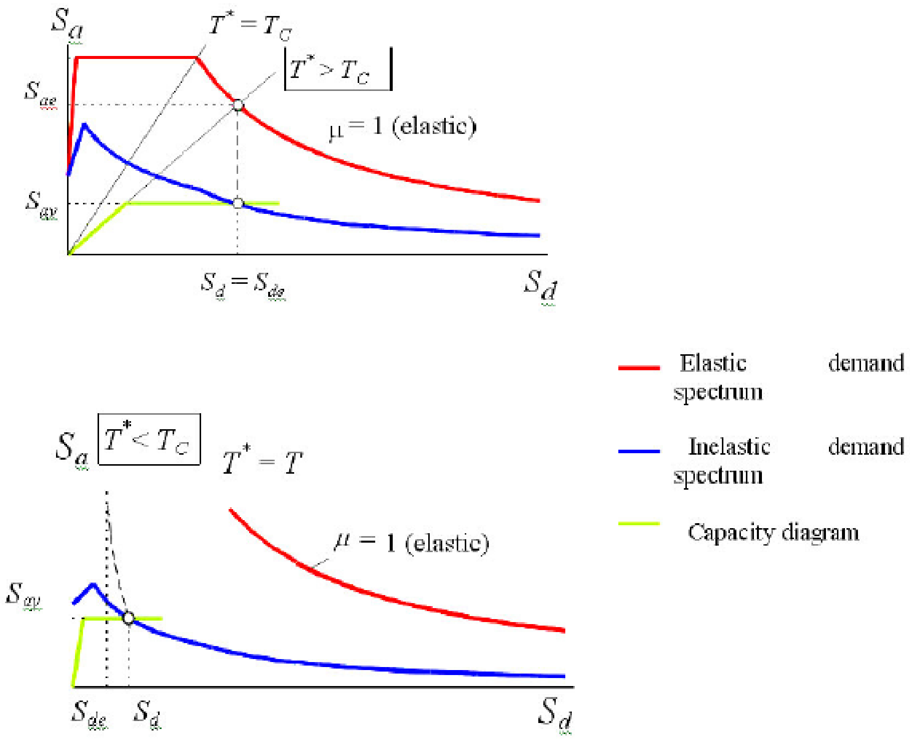


Figure 7: Elastic and demand spectrum in relation with capacity diagram T*≥TC picture above and for T*<TC picture below) (Mestrovic, Cizmar, & Pende, 2008)

2.7 Review over previous research studies

When going through the literature, it has been found that a number of researches have been carried out in the similar area of study in different parts of the world. This section briefly presents some of those important studies, explaining the objectives, the methodology they have adopted and major findings through the results obtained etc.

In their research, **(Singh, Namdev, & Lang, 2012)** intended to compare the code provisions for seismic analysis and design of ductile RC frame buildings. All current seismic design codes are based on a prescriptive Forced-Based Design approach. In this approach, a linear elastic analysis is performed and inelastic energy dissipation is considered indirectly through a response reduction factor (or a behavior factor). Building codes define different ductile classes and specify different response reduction factors based on the material, configuration and detailing. Codes also differ specifying the effective stiffness of RC members, procedures to estimate drift and allowable limits on drift. This research paper presents a comparative study of different ductility classes and corresponding response reduction factors, reinforcement detailing provisions and a case study of seismic performance of a ductile RC frame building designed using four major codes ASCE7 (United States), EN 1998-1 (Euro), NZS 1170.5 (New Zealand) and IS 1893 (India).

Based upon the results, as a conclusion, it states that the comparison of broad ductility classes suggests significant variation in different codes. It also concludes that, it is not possible to directly compare the response reduction factors for various ductility classes due to the variation in provisions for reinforcement detailing and capacity design provisions. It further states that the most of codes combine the effect of overstrength and ductility in a single reduction factor, except for NZS 1170.5, which considers the overstrength separately through a “structural performance factor”.

This study also confirms that NZS 1170.5 results in the highest design base shear for a given period, for almost all the cases considered in the study. The design base shear as per Eurocode 8 has become close to that of NZS 1170.5, while IS 1893 has resulted in lowest design base shear force for a given hazard. Based upon the seismic performance of an eight storied RC frame building, it has been noted that the inter storey drift ratio was greater than 2.5% for DBE and, equal or greater 4% for MCE for most of the codes.

In the research conducted by **(Bhavsar & al, 2014),** a comparative study has been done based upon a seismic analysis performed for a RC building according to Indian standard and Euro standard. The paper highlights the importance of doing such a study, because there is a possibility that the International Standards may have more parameters that are not included in Indian Standards. It further mentions the importance of Eurocode in developing country like India, because most of the Gulf countries, which are having remarkable infrastructures, also follow Eurocode.

In making the comparison, it has considered most of important criteria such as response reduction factor, ductility classes, maximum story displacements, drift limitations, base shear, reactions and axial loads etc.

The paper concludes that the design base shear force obtained with IS 1893 was lower than the design base shear force calculated using the Eurocode, because of the high response reduction factor, which has been used in analysis with Indian code.

In another research paper, **(Neupane & Shrestha, 2015)** has studied comparative analysis of seismic codes of Nepal and India for RC buildings. In this paper the authors have tried to challenge the existence of one of the widespread belief in Nepal that Indian seismic codes design for greater seismic forces in the RC frames than Nepal codes and are therefore more conservative. The paper focuses that the results from one analysis cannot be concluded as general and that both the codes could be conservative depending upon various parameters and conditions.

Based on their comparative analysis, the authors have concluded that the results of seismic analysis depends on three major factors - the location of site, the soil type at site and the number of story of the building. They suggest that no any code can be interpreted as being faulty; rather both codes have their own design principles and assumptions which considerably differ the seismic capacity of the building being designed.

Moreover the researchers highlights that the findings outline the lack of harmony between the two codes and it is a challenge to urgently stipulate unambiguous rules and coherent code provision in a seismically active nation like Nepal.

In a conference paper by **(Mestrovic, Cizmar, & Pende, 2008)**, the necessity of nonlinear methods in determining the performance level of the building is coined. The paper also describes in brief the step by step procedure of pushover analysis using N2 method from Eurocode 8 (EN1998-1:, 2004). N2 is simple non-linear method used for calculation of structures during earthquake. The paper gives numerical example of N2 method. The authors conclude in the paper that inelastic structural response is crucial in earthquake engineering and modern methods, supported with usage of computers and strict design codes ensure better understanding of structural response during earthquakes and at the same time seismic resistant structures.

3.0 GENERAL DESCRIPTION OF CASE STUDY

3.1 Description of the selected Building

The building considered in this study is an existing 7-story reinforced concrete structure. The building has 4 bays in X direction and 4 bays in Y direction. The bay widths are variable.

3.2 Project Data

- Location of Building → Kathmandu, Nepal
- Built Year → 2012 AD
- Total number of storeys → Basement + 6 Floors
- Total coverage area of building → 495.26 m^2 (Basement),
- Total construction area →2233.15 m^2
- Typical Floor Area → 350.55 m^2
- Total Building Height from ground to roof covering → 21.946 m
- Typical Floor height →3.66 m
- The Dimension of Basement Floor → 24.66m X 21.33m
  and for typical floor above → 22.43m X 21.33m
- Basic Seismic Force Resisting System – Moment Resisting RC Frame system (Rigid Diaphragms in the form of concrete slab, RC Frames and Columns
- Foundation – Raft, Reinforced concrete retaining wall at the basement level.

Figure 8: Architectural Plan of Basement and Ground Floor

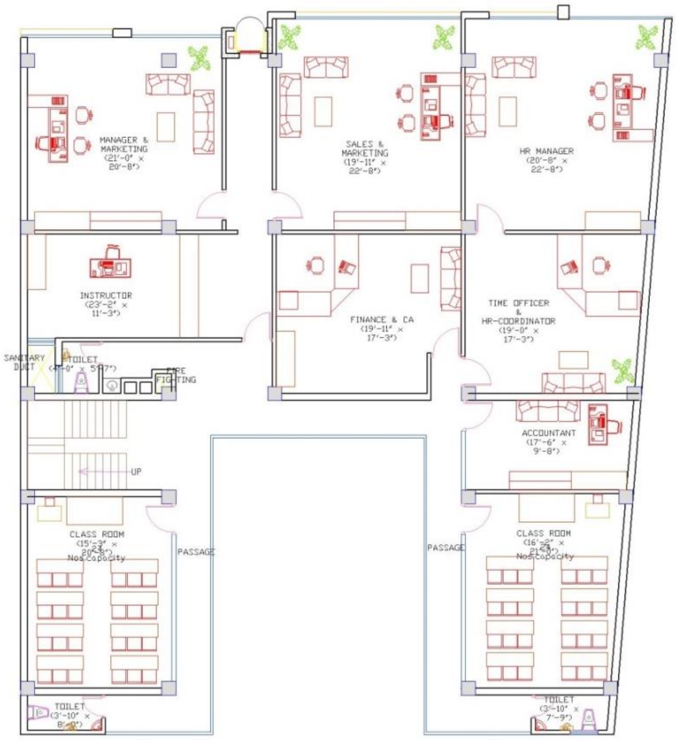


Figure 9: Architectural Plan of Typical Floor

Figure 10: 3D Rendered View Front and Back

3.3 Materials Data

- Characteristic Strength of concrete used
  - For Beam, Slab and Raft foundation → M20 or C20/25 (f_{ck} = 20 N/mm^2)
    Corresponding modulus of elasticity, E = 22360 N/mm^2 and Poisson's Ratio, v = 0.2
  - For Column → M25 or C25/30 (f_{ck} = 25 N/mm^2)
    Corresponding modulus of elasticity, E = 25000 N/mm^2 and Poisson's Ratio, v = 0.2
    Unit Weight of Concrete = 25 KN/m^3
- Fe500 steel used for Reinforcement bars.

Density of Steel = 7850 kg/m^3
Modulus of Elasticity; E = 2x10^5 N/mm^2
Minimum Yield Strength; f_y = 500 N/mm^2
Minimum Tensile Strength; f_u = 545 N/mm^2
Expected Yield Strength; f_{ye} = 550 N/mm^2
Expected Tensile Strength; f_{ue} = 599.5 N/mm^2

- Fe415 steel used for Confinement bars (Ties).

Density of Steel = 7850 kg/m^3
Modulus of Elasticity; E = 2x10^5 N/mm^2
Minimum Yield Strength; f_y = 415 N/mm^2
Minimum Tensile Strength; f_u = 485 N/mm^2
Expected Yield Strength; f_{ye} = 456.5 N/mm^2
Expected Tensile Strength; f_{ue} = 533.5 N/mm^2

- External and Internal Brick Walls

Density of Brick Masonry Wall = 18.85 KN/m^3 (1920 Kg/m^3)
Density of Cement Plaster in Wall = 20.40 KN/m^3

3.4 Cross Sections for Structural Components

The structure is designed as a reinforced concrete bare frame model. The structural system consists of special moment resisting frame system confirming to ductile detailing as per (IS 13920, 1993). For the initial designing process the infill walls effects in lateral load resistance were not considered. The columns are RC square and beams are T- beams monolithic with RC slabs. The lift wall is made of RC shear wall. The dimensions of the structural components are given in Table 20.

Table 20: Dimensions of Structural Members

Member type	Dimension (mm)
1. Columns	450X450, 300X300
2. Main Beams	300X450
3. Secondary Beams	230X380
4. Slab Thickness	150
4. Lift wall	175
5. Retaining Wall	230

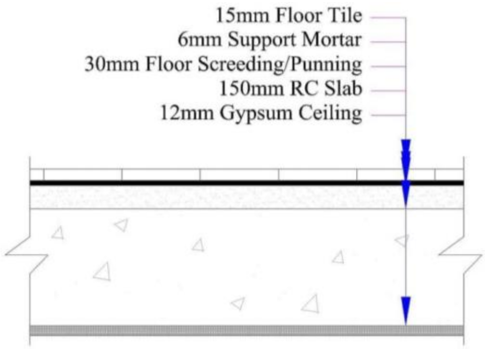


Figure 11: Typical Floor Slab Section

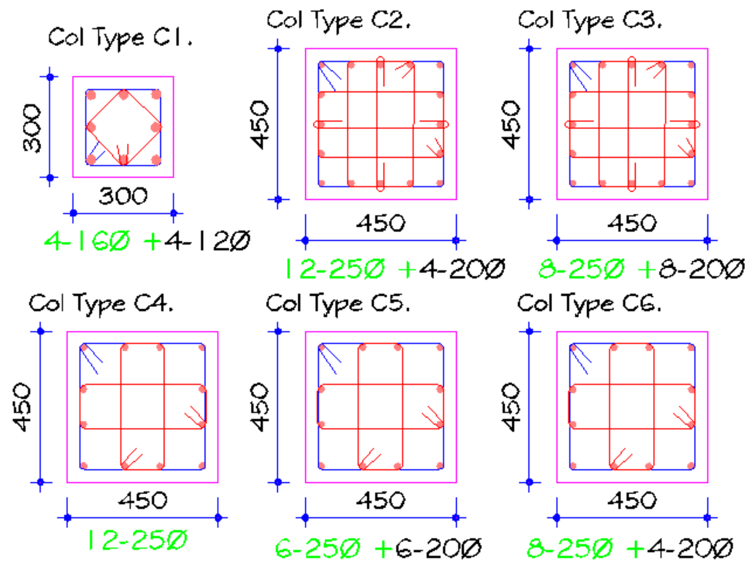


Figure 12: Details of Columns at Basement and Ground Floor

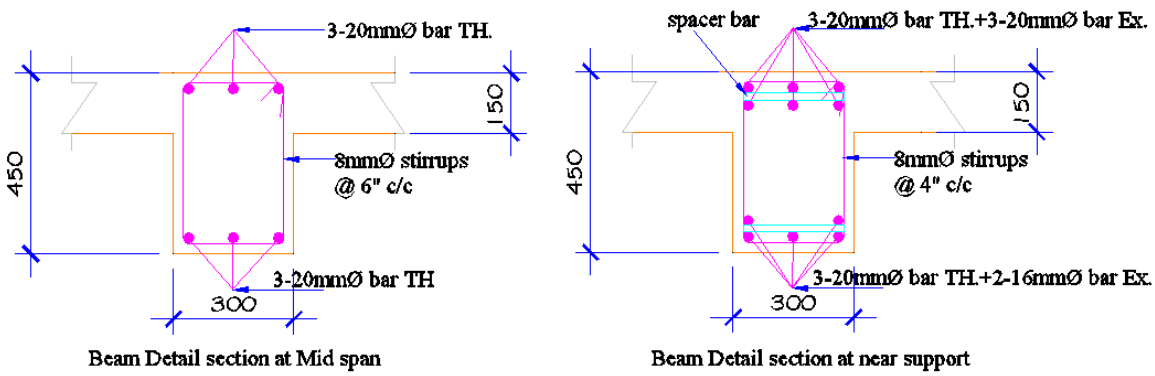


Figure 13: Details of Typical Beam section

Please refer the Annex section for detailed structural drawings of the considered building.

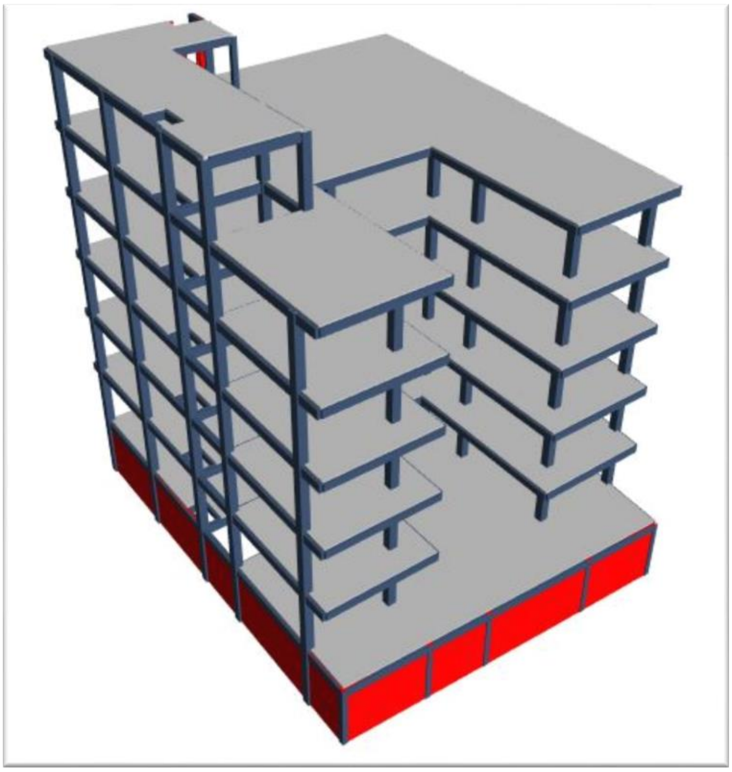


Figure 14: Rendered 3D view of Structural Frames

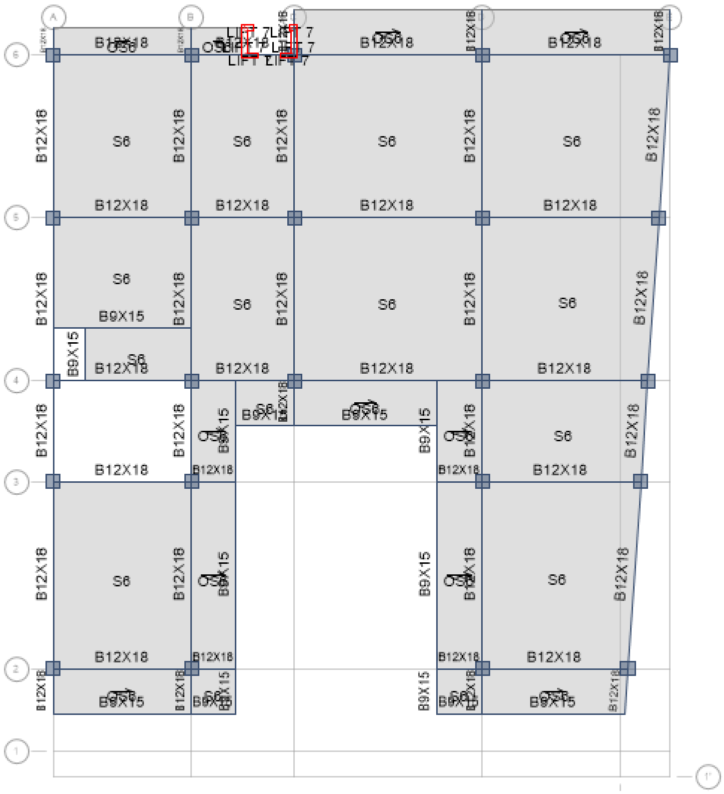


Figure 15: Model of Typical Floor in ETABS 2016

3.5 Actions

3.5.1 Permanent Loads (G)

The permanent load is composed by the self-weight of the structure and the over imposed dead weight like ceiling, floor finishing, wall loads etc and any other permanent loads. The self-weight load will be calculated automatically by the software ETABS by taking into account the geometrical properties and the weight per unit volume which is 2500Kgf/m3 for concrete and 7850Kgf/m3 for steel.

3.5.2 Finishing Loads

Finishing loads are considered according to the type of finishing used for different occupancy purpose rooms. Please refer the typical slab section detail in figure (8) for finishing items and thickness considered.

Table 21: Flooring and Ceiling Loads (Typical Rooms)

S.N.	Descriptions	Thickness (mm)	Unit wt (KN/m^3)	Total Wt. (KN/m^2)
1	Floor Tile	15	26.7	0.401
2	Support Mortar	6	20.4	0.122
3	Floor Screeding/Punning	30	20.4	0.612
4	Gypsum Ceiling	12	7.848	0.094
			Total Permanent Load	**1.2 KN/m^2**

Table 22: Flooring and Ceiling Loads at Toilets

S.N.	Descriptions	Thickness (mm)	Unit wt (KN/m^3)	Total Wt. (KN/m^2)
1	Floor Tile	15	26.7	0.401
2	Support Mortar	6	20.4	0.122
3	Floor Screeding/punning	30	20.4	0.612
4	Light weight Filling works	175	8	1.400
			Total Permanent Load	**2.5 KN/m^2**

Table 23: Flooring and Ceiling Loads at Terrace Roof Garden

S.N.	Descriptions	Thickness (mm)	Unit wt (KN/m^3)	Total Wt. (KN/m^2)
1	Soil Layer for plants	200	19	3.800
2	Water Proofing Layer	2	20.4	0.041
3	Floor Screeding/punning	50	20.4	1.020
4	Water Proofing Layer	2	20.4	0.041
5	Gypsum Ceiling	12	8.00	0.096
			Total Permanent Load	**5.0 KN/m^2**

The slab deck is modeled as a shell element in the software ETABS which acts as a horizontal rigid diaphragm. Thickness of the concrete slab is taken as 150mm.

The slabs and beams are monolithic cast in situ concrete. Apart from this the perimeter beams are loaded for façade/claddings. On the front and back sides there are glass façades and at the adjacent sides which are fused with property lines there are full brick (23 cm thick) solid masonry walls as cladding. The internal partition walls are composed of light weight partitions like sandwich panels, aluminum partitions and at some places made of half brick thick brick masonry walls. The brick masonry walls are also calculated as permanent loadings on the building structure.

Table 24: Self Weight of Structure per floor

Storey	Column (M25)	Beams (M20)	Lift walls (M25)	Slab (M20)	Retaining walls (M20)	Total
	KN	KN	KN	KN	KN	KN
6TH	180.47	195.12	52.02	306.27	-	**733.87**
5TH	439.49	725.48	52.02	1474.56	-	**2691.55**
4TH	439.49	717.91	52.02	1474.56	-	**2683.98**
3RD	439.49	717.91	52.02	1474.56	-	**2683.98**
2ND	439.49	717.91	52.02	1474.56	-	**2683.98**
1ST	439.49	717.91	52.02	1474.56	-	**2683.98**
GROUND	498.94	823.02	52.02	1778.33	1942.31	**5094.61**
SUM	**2876.85**	**4615.26**	**364.12**	**9457.41**	**1942.31**	**19255.94**

Table 25: Dead Load due to the walls on the beam of the Building:

S.No	Building Components	Dimensions		Finishing Load	Self-Weight	Total Unit Weight	Units	Remarks
		Width (B) mm	Depth (D) or Height (H) mm					
A	outer wall without opening	230	3200	1.63	13.88	15.52	KN/m	
B	outer wall with opening	230	3200	1.39	11.80	13.19	KN/m	15% opening wall
C	inner wall with opening	230	3200	1.14	9.72	10.86	KN/m	30% opening
D	inner wall with opening	230	3200	0.98	8.33	9.31	KN/m	40% opening wall
E	inner wall without opening	115	3200	1.63	6.04	7.67	KN/m	
F	Inner wall with opening	115	3200	1.47	5.13	6.60	KN/m	15% opening wall
G	Inner wall with opening	115	3200	1.14	4.23	5.37	KN/m	30% opening wall
H	parapet wall	115	1000	0.51	1.89	2.40	KN/m	

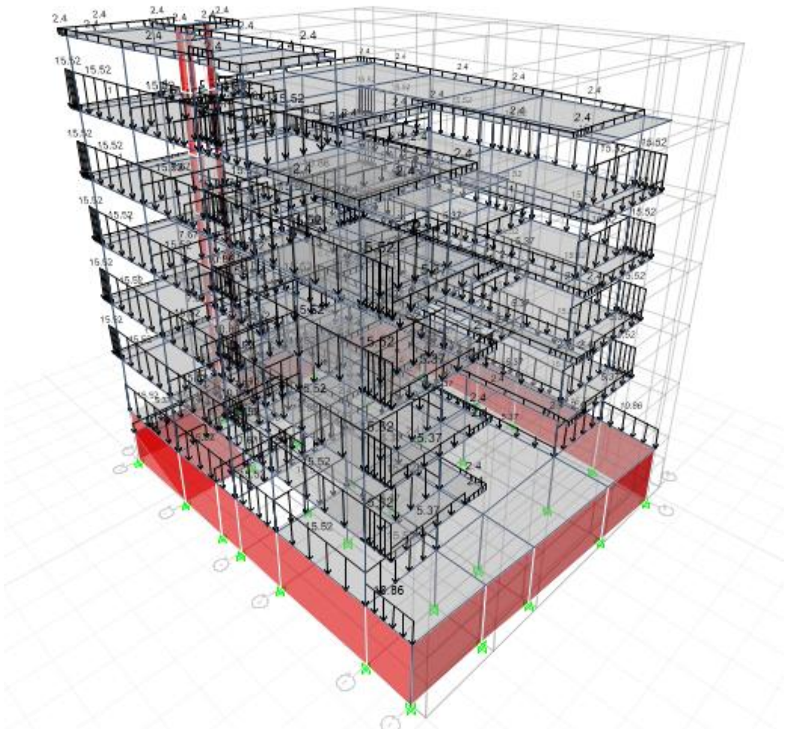


Figure 16: Wall Load Applied on ETABS Model

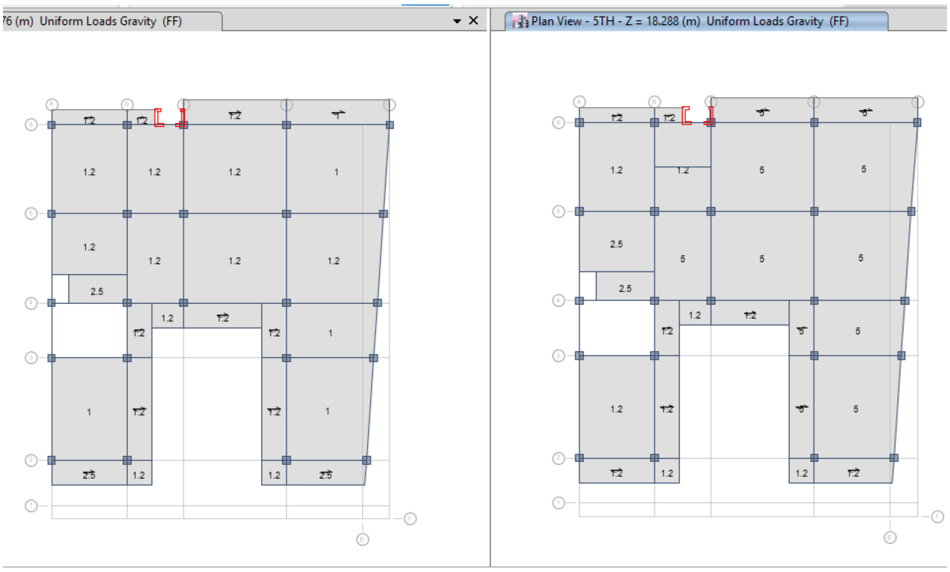


Figure 17: Floor Finishing Load applied on ETABS Model

3.5.3 Live Loads

This case study building category of use is B where the major uses are Office areas. The imposed loads on story floors in buildings are applied from (IS 875-2, 1998) which also satisfies the requirements in (EN 1991-1-1, 2002). The imposed live load was assigned as area loads on the floor slabs. The imposed loads applied to occupancy specific floors were given below:

Table 26: Imposed Live Load according to occupancy

Floor occupancy type	Imposed Load applied (KN/m^2)
Toilets and Bathrooms	2.0
Individual person rooms	2.0
All other rooms	3.0
Staircase, Corridors and Balconies	4.0
Accessible Roof	2.0 & 3.0
Overhead water tank	5.0
Gym Hall and Lift machine	5.0

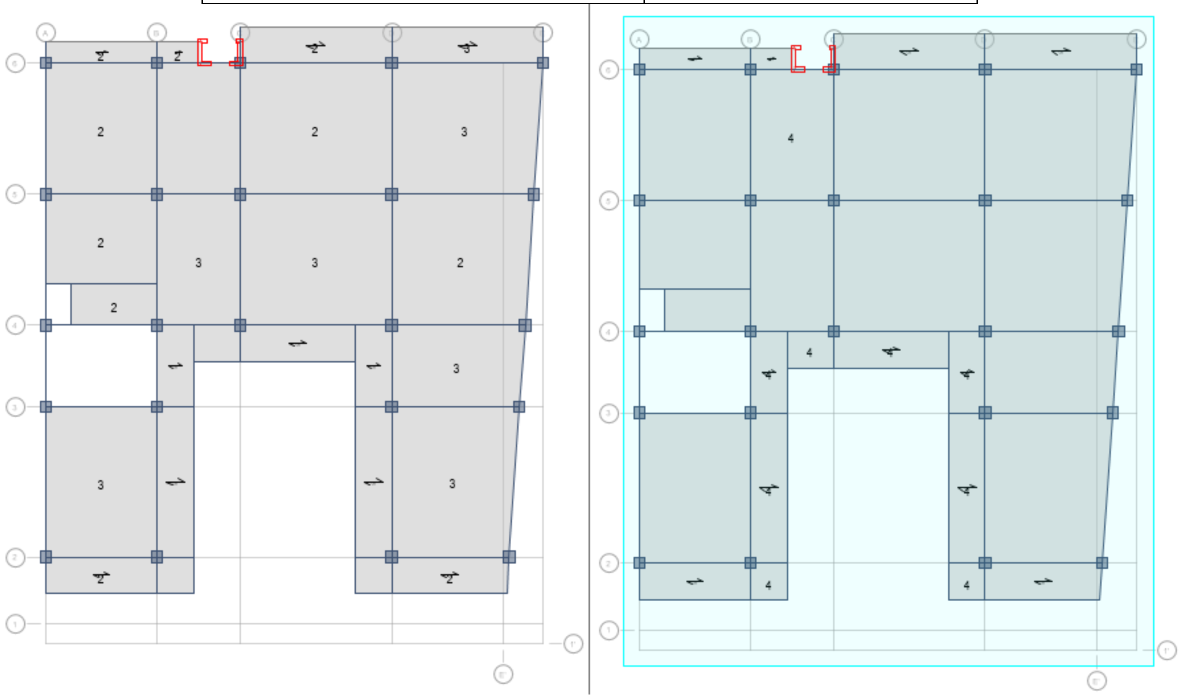


Figure 18: Imposed Load Assignment in ETABS 2016

3.5.4 Snow Loads

Since the building location is in Kathmandu where there is no or very little snow fall in centuries the snow load is not considered in this case study building.

3.5.5 Wind Loads

The wind action and earthquake actions are not considered to interact with each other together. "Earthquake is not likely to occur simultaneously with high wind or maximum flood" – section 6.2 of (IS 1893-1, 2002). So in this case study for seismic the action due to wind load is not considered.

3.5.6 Seismic Loads

Seismic loads vary with the type of seismic code used for analysis. This is dealt separately in the specific code based seismic analysis in next chapters.

4.0 SEISMIC ANALYSIS ACCORDING TO INDIAN CODE IS 1893 (Part 1) : 2002

4.1 Design Seismic action

4.1.1 Zone factor, Z

The zone factor, Z for different zones in India is given in table 2 of (IS 1893-1, 2002). Nepal is considered to fall under zone V with very severe seismic intensity. Thus the **Z** factor is taken as **0.36**.

4.1.2 Importance factor, I

This is an office purpose commercial building having 7 storeys. Therefore the importance factor according to table 6 of (IS 1893-1, 2002) has been selected as **1.0.**

4.1.3 Response Reduction factor, R

Considering that the structure consists of special moment resisting frames confirming to the ductile detailing requirement as per (IS 13920, 1993), the value of **R** was selected as **5.0** according to table 7 of (IS 1893-1, 2002).

4.1.4 Average response acceleration coefficient, Sa/g

According to the soil test reports of the project the ground type is considered to be medium type of soil with the deposited material at the top and silty clay of low plasticity mixed with coarse and fine sands at the bottom. Thus, the **Average Response Acceleration Coefficient** is taken for Soil Type-2 and 5% damping from figure 2 of (IS 1893-1, 2002). For seismic response calculation the natural period of vibration of the structure is used as the building is existing one.

Therefore

$$\frac{S_a}{g} = \frac{1.36}{T}$$

4.1.5 Structural Regularity

Clause 7.1 of (IS 1893-1, 2002) defines the criteria to be satisfied in order a building to be considered as regular. Accordingly, a building shall be considered irregular, if any of the conditions given in table 4 and 5 of (IS 1893-1, 2002) are not satisfied. In case of this investigated building there are both plan irregularity *(vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes)* and vertical geometry irregularity. Therefore, the building was considered as irregular.

4.2 Method of analysis

4.2.1 Structural Model

Since the selected building is irregular and its height is more than 12m and located in zone V, the method of analysis to perform is dynamic analysis as per section 7.8.1 of (IS 1893-1, 2002). However, a static lateral force method of analysis has also been performed since the base shear force obtained by dynamic analysis has to be compared against that of calculated by static lateral force method.

A three dimensional mathematical model was used in this analysis since it can depict the special distribution of the mass and the stiffness of the structure adequately.

In this study, two models were considered for analyses.

4.2.2 Model Without Infill

4.2.2.1 General

At first the building was considered to have no significant structural effect from the masonry infill walls on its behavior when subjected to seismic load. The reinforced concrete frame wall system was considered as the only lateral load resisting system in the building and therefore, the presence of masonry infill walls were not considered in making the model. However, their weight was considered in the calculation of seismic weight of the building. The following assumptions and basic characteristics were considered to model the study building in ETABS software.

- ✓ Column and beam elements are modeled as line elements whereas the floor slabs and concrete walls are modeled as shell elements.
- ✓ The elements were modeled with the actual sizes and reinforcement such that they adequately represent the distribution of stiffness and mass of the building.
- ✓ The influence of cracked sections is not considered in the model as it is not specifically discussed in IS 1893 (part 1): 2002.
- ✓ All the joints (Beam- Column, Column-Foundation etc) are considered to be rigid joints.
- ✓ Frames are connected by means of rigid diaphrams in horizontal plane at each floor level.
- ✓ The reinforced concrete staircase is not considered in the model. However, its weight is calculated and applied to the beams as line loads. The stiffness contributions due to staircases are ignored in this model and study.
- ✓ All floor loads were applied to the deck which distributes uniformly the load to the beams and all wall loads are applied to beams as uniformly distributed line loads.
- ✓ Masses have been considered as lumped into a selected master-joint at each floor, because the floor diaphragms are rigid in their planes.
- ✓ The accidental torsional effects were considered by applying design eccentricity directly to the model while defining the lateral load system. The design eccentricity is calculated as per Clause 7.9 of IS 1893 (Part 1): 2002.

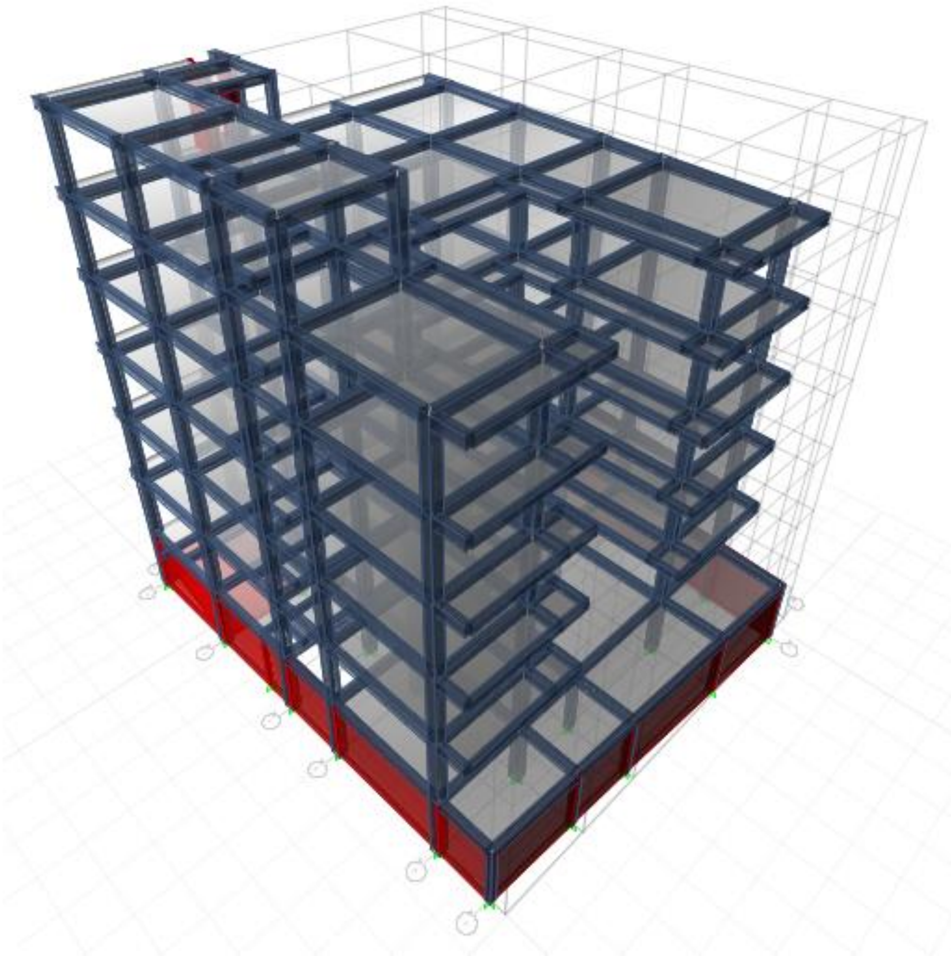


Figure 19: Three dimensional (spatial) model of building in ETABS

4.2.2.2 Seismic coefficient method (Linear Static analysis)

Analysis according to seismic coefficient method also known as lateral force method is carried out in three main steps as follows.

I. Estimating the self-weight and seismic weight of the building
II. Calculating the seismic base shear in relevant directions
III. Distribution of lateral forces at each floor level.

4.2.2.3 Seismic weight of the building

The seismic weight of the building was calculated considering all the gravity loads and appropriate percentage of imposed live loads as described in section 2.2.4.1 (Clause 7.43 of (IS 1893-1, 2002)).

Table 27: Load Patterns considered in ETABS

Name	Type	Self-Weight Multiplier	Auto Load
DEAD	Dead	1	
LIVE1	Live	0	
WALL	Superimposed Dead	0	
FF	Superimposed Dead	0	
EQX	Seismic	0	IS1893 2002
EQY	Seismic	0	IS1893 2002
STAIR	Superimposed Dead	0	
LIVE2	Live	0	
LIVEROOF	Roof Live	0	

Table 28: Mass Source IS 1893-1:2002

Load Pattern	Seismic Multiplier
Dead	1
Live 1 ($\leq$ 3 KN/m^2)	0.25
Wall load	1
Floor finish	1
Stair Dead	1
Live 2 (> 3 KN/m^2)	0.5
Roof Live	0

Table 29: Seismic Mass Summary by Story

Story	UX (kg)	UY (kg)	UZ (kg)
6TH	96443.16	96443.16	0
5TH	503425.5	503425.5	0
4TH	521123.97	521123.97	0
3RD	549701.31	549701.31	0
2ND	549701.31	549701.31	0
1ST	549701.31	549701.31	0
TOTAL SEISMIC MASS	2770096.56	2770096.56	0

Table 30: Center of Mass, Rigidity and eccentricity (Indian Code)

Story	Diaphragm	XCM	YCM	XCR	YCR	Static Ecc - X	Static Ecc - Y	Design Ecc - X	Design Ecc - Y
		m	m	m	m	m	m	m	m
6TH	D1	3.0781	18.6059	7.7272	18.3218	4.6491	-0.2841	7.43465	0.38135
5TH	D1	9.7956	15.2972	9.4304	17.6658	-0.3652	2.3686	0.4757	4.7404
4TH	D1	9.5964	14.162	9.4423	17.9441	-0.1541	3.7821	0.79235	6.86065
3RD	D1	9.7341	14.1593	9.3774	18.2387	-0.3567	4.0794	0.48845	7.3066
2ND	D1	9.7341	14.1593	9.2265	18.5176	-0.5076	4.3583	0.2621	7.72495
1ST	D1	9.7341	14.1593	8.8458	19.2364	-0.8883	5.0771	-0.30895	8.80315
GROUND	D1	9.6685	12.878	9.7961	9.4776	0.1276	-3.4004	1.2079	-3.8876

4.2.2.4 Design seismic base shear

The total design seismic base shear (V_B) for each horizontal direction has been determined by the expression given in clause 7.5.3 of (IS 1893-1, 2002) as,

$$V_B = A_h W, \quad A_h = \frac{ZIS_a}{2Rg}$$

Where,

A_h: Design horizontal acceleration spectrum value using the fundamental natural period T_a in the considered direction of vibration, given by

W: Seismic weight of the building.

The fundamental natural period of vibration (T_a) has been obtained by model analysis performed on the three dimensional computer model of the building.

The design base shear force acting in each horizontal direction is shown in table 31 below.

Table 31: Design Seismic base shear by static lateral force method (without infill) (Indian Code)

Soil Type	Fundamental period, T_a (s)		Z	I	R	S_a/g		Seismic Weight, W (KN)	V_B (KN)	
	X	Y				X	Y			
Medium	1.395	1.282	0.36	1.0	5.0	0.975	1.061	27165.368	953.49	1037.16

4.2.2.5 Distribution of lateral forces

The design base shear (V_B) was then distributed along the height of the building as per expression in Clause 7.7.1 of (IS 1893-1, 2002)).

Table 32: Distribution of design seismic base shear at each storey level (without infill) (Indian Code)

Storey Level	W_i (KN)	h_i (m)	$W_i h_i^2$	Q_i (KN)	
				Q_{ix}	Q_{iy}
6^{TH}	945.770	21.9456	455491.684	103.1581	112.2098
5^{TH}	4936.842	18.288	1651131.467	373.9425	406.7542
4^{TH}	5110.402	14.6304	1093874.415	247.7368	269.4746
3^{RD}	5390.646	10.9728	649046.392	146.9937	159.8918
2^{ND}	5390.646	7.3152	288465.063	65.3305	71.063
1^{ST}	5390.646	3.6576	72116.266	16.3326	17.7658
TOTAL			**4210125.29**	**953.49**	**1037.16**

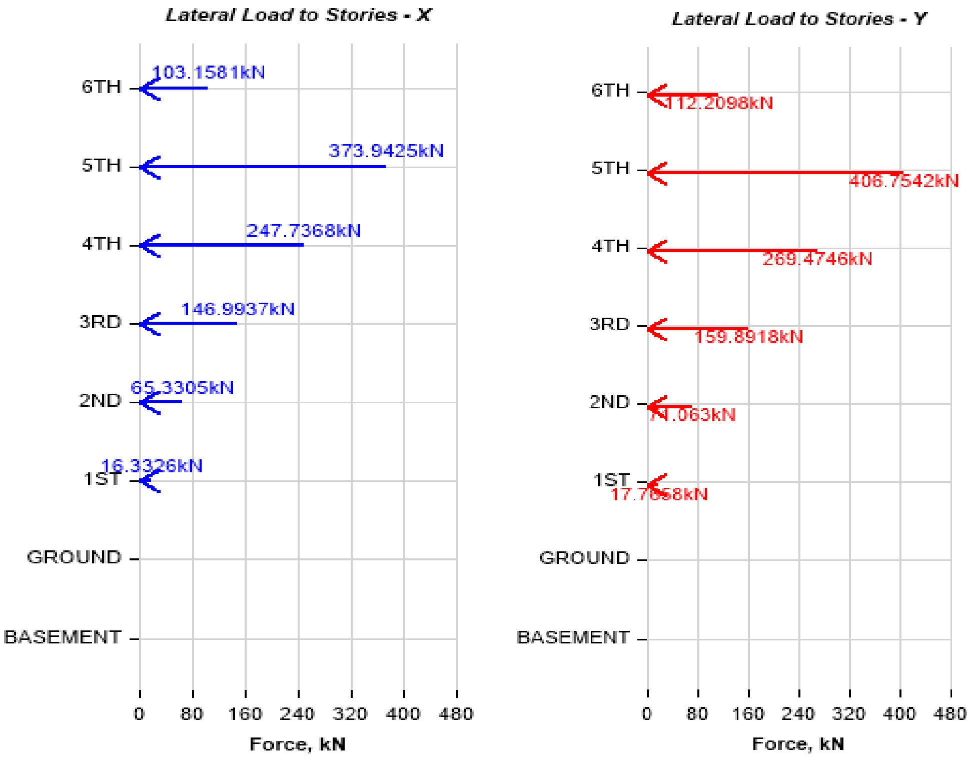


Figure 20: Lateral Load distribution without infill (Indian Code)

4.2.2.6 Model Response Spectrum Method (Dynamic Analysis)

4.2.2.6.1 General rules

The general rules recommended for this type of analysis were followed in case of the study building and are as follows.

- ✓ Modal response spectrum analysis has been performed independently for the ground excitation in two horizontal directions. The excitation in vertical direction was not considered since the structure does not have large span beams, pre-stress components or cantilever projections.

- ✓ The acceleration spectrum defined in Clause 6.4.2 of (IS 1893-1, 2002) was used for the test building.
- ✓ For the combination of different modes, the "Complete Quadratic Combination (CQC) rule was used.
- ✓ The results of the modal analysis in both directions were combined by SRSS rule.
- ✓ The accidental torsional effect was considered by overwriting the eccentricity value in the response spectrum load case which then will be considered in the analysis to calculate additional torsional moments to the buildings.

4.2.2.6.2 Response Spectrum Functions

The response spectrum functions defined in ETABS model was considered as per (IS 1893-1, 2002).

Table 33: Response Spectrum Function - IS 1893:2002

Name	Period sec	Acceleration	Damping	Z	Soil Type
IS RS	0	0.36	5	0.36	II
IS RS	0.1	0.9			
IS RS	0.55	0.9			
IS RS	0.8	0.612			
IS RS	1	0.4896			
IS RS	1.2	0.408			
IS RS	1.4	0.349714			
IS RS	1.6	0.306			
IS RS	1.8	0.272			
IS RS	2	0.2448			
IS RS	2.5	0.19584			
IS RS	3	0.1632			
IS RS	3.5	0.139886			
IS RS	4	0.1224			
IS RS	4.5	0.1224			
IS RS	5	0.1224			
IS RS	5.5	0.1224			
IS RS	6	0.1224			
IS RS	6.5	0.1224			
IS RS	7	0.1224			
IS RS	7.5	0.1224			
IS RS	8	0.1224			
IS RS	8.5	0.1224			
IS RS	9	0.1224			
IS RS	9.5	0.1224			
IS RS	10	0.1224			

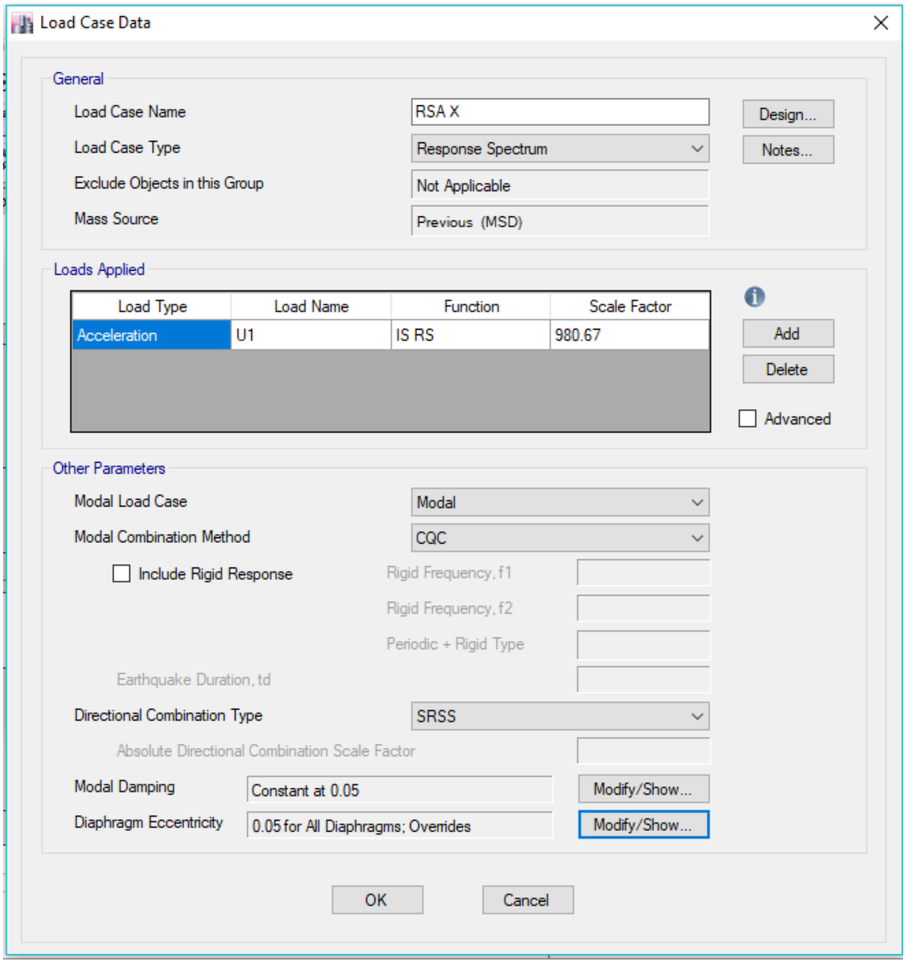


Figure 21: Response Spectrum Load Case Definition in ETABS (IS 1893:2002)

4.2.2.6.3 Periods and effective masses

In the modal response spectrum analysis, adequate numbers of modes of vibration were taken into account as the sum of the modal masses in each horizontal direction to exceed 90% of the total mass of the structure as given in Clause 7.8.4.2 of (IS 1893-1, 2002). In this modal analysis 24 numbers of modes were considered.

The modal analysis results are shown in the tables below.

Table 34: Modal Periods and Frequencies (without infill) (Indian Code)

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad²/sec²
Modal	1	1.395	0.717	4.5044	20.29
Modal	2	1.282	0.78	4.8997	24.0069
Modal	3	1.039	0.962	6.0447	36.5379
Modal	4	0.439	2.28	14.3265	205.2497
Modal	5	0.39	2.562	16.0945	259.0341
Modal	6	0.313	3.199	20.102	404.092
Modal	7	0.242	4.135	25.9806	674.9899
Modal	8	0.203	4.93	30.9773	959.5951
Modal	9	0.175	5.718	35.9255	1290.6446
Modal	10	0.161	6.209	39.0106	1521.8274
Modal	11	0.143	6.992	43.9343	1930.2264
Modal	12	0.125	7.997	50.2475	2524.8078
Modal	13	0.115	8.675	54.5086	2971.183
Modal	14	0.105	9.479	59.5576	3547.102
Modal	15	0.087	11.496	72.2318	5217.426
Modal	16	0.077	12.91	81.1169	6579.9514
Modal	17	0.068	14.762	92.7538	8603.2668
Modal	18	0.059	17.074	107.282	11509.4272
Modal	19	0.038	26.447	166.1704	27612.5965
Modal	20	0.03	32.967	207.1383	42906.2919
Modal	21	0.023	44.24	277.9708	77267.7674
Modal	22	0.003	297.728	1870.6806	3499446.0164
Modal	23	0.003	324.524	2039.0457	4157707.1764
Modal	24	0.002	436.868	2744.9231	7534603.0643

Table 35: Modal Participating Mass Ratios (Part 1 of 2) (without infill) (Indian Code)

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	1	1.395	0.3837	0.011	0	0.3837	0.011	0
Modal	2	1.282	0.0112	0.6258	0	0.3949	0.6368	0
Modal	3	1.039	0.2467	0.001	0	0.6416	0.6378	0
Modal	4	0.439	0.0525	0.0005	0	0.6941	0.6384	0
Modal	5	0.39	0.0006	0.0902	0	0.6947	0.7285	0
Modal	6	0.313	0.0447	0.0006	0	0.7394	0.7291	0
Modal	7	0.242	0.0192	0.0002	0	0.7586	0.7293	0
Modal	8	0.203	0.001	0.0361	0	0.7596	0.7654	0
Modal	9	0.175	7.579E-06	0.0006	0	0.7596	0.766	0
Modal	10	0.161	0.0248	0.0019	0	0.7845	0.7679	0
Modal	11	0.143	0.0017	0.0016	0	0.7861	0.7695	0
Modal	12	0.125	0.0005	0.0173	0	0.7867	0.7868	0
Modal	13	0.115	0.0027	9.296E-06	0	0.7894	0.7868	0

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	14	0.105	0.0098	0.0015	0	0.7992	0.7883	0
Modal	15	0.087	3.045E-05	0.0112	0	0.7993	0.7995	0
Modal	16	0.077	0.0084	2.48E-05	0	0.8076	0.7995	0
Modal	17	0.068	0.0006	0.004	0	0.8082	0.8035	0
Modal	18	0.059	0.0069	0.001	0	0.8151	0.8045	0
Modal	19	0.038	0.1791	0.0004	0	0.9943	0.8049	0
Modal	20	0.03	0.0003	0.195	0	0.9946	0.9999	0
Modal	21	0.023	0.0054	0.0001	0	1	1	0
Modal	22	0.003	0	0	0	1	1	0
Modal	23	0.003	0	0	0	1	1	0
Modal	24	0.002	0	0	0	1	1	0

Table 36: Modal Participating Mass Ratios (Part 2 of 2) (without infill) (Indian Code)

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	0.0137	0.2125	0.2309	0.0137	0.2125	0.2309
Modal	2	0.4156	0.0057	0.0014	0.4292	0.2183	0.2323
Modal	3	0.0001	0.2095	0.3953	0.4293	0.4277	0.6277
Modal	4	0.0006	0.1169	0.0301	0.4299	0.5446	0.6578
Modal	5	0.1855	0.0015	6.334E-06	0.6155	0.5461	0.6578
Modal	6	0.0005	0.0762	0.0612	0.6159	0.6223	0.719
Modal	7	0.0001	0.0128	0.0113	0.616	0.6352	0.7303
Modal	8	0.0227	0.0011	4.937E-05	0.6387	0.6363	0.7304
Modal	9	0.0005	0.001	0.0143	0.6392	0.6373	0.7447
Modal	10	0.0022	0.0236	0.0074	0.6414	0.6608	0.7521
Modal	11	0.0025	0.0026	0.007	0.6439	0.6634	0.759
Modal	12	0.0224	0.0006	0.0027	0.6663	0.664	0.7618
Modal	13	6.87E-07	0.0033	0.0009	0.6663	0.6673	0.7627
Modal	14	0.0017	0.0137	0.0083	0.668	0.681	0.7709
Modal	15	0.0131	0.0001	0.0007	0.6812	0.6811	0.7717
Modal	16	1.355E-05	0.01	0.0057	0.6812	0.6911	0.7773
Modal	17	0.005	0.0008	2.021E-05	0.6862	0.6919	0.7774
Modal	18	0.0013	0.0097	0.0032	0.6875	0.7016	0.7806
Modal	19	0.0007	0.2897	0.0001	0.6882	0.9912	0.7807
Modal	20	0.3117	0.0005	5.6E-06	0.9999	0.9918	0.7807
Modal	21	0.0001	0.0082	0.2193	1	1	1
Modal	22	0	0	0	1	1	1
Modal	23	0	0	0	1	1	1
Modal	24	0	0	0	1	1	1

Table 37: Modal Direction Factors (without infill) (Indian Code)

Case	Mode	Period sec	UX	UY	UZ	RZ
Modal	1	1.395	0.574	0.035	0	0.391
Modal	2	1.282	0.016	0.978	0	0.005
Modal	3	1.039	0.432	0.022	0	0.546
Modal	4	0.439	0.549	0.035	0	0.416
Modal	5	0.39	0.006	0.983	0	0.011
Modal	6	0.313	0.467	0.031	0	0.502
Modal	7	0.242	0.515	0.061	0	0.424
Modal	8	0.203	0.02	0.926	0	0.053
Modal	9	0.175	0.548	0.03	0	0.422
Modal	10	0.161	0.407	0.132	0	0.461
Modal	11	0.143	0.485	0.067	0	0.448
Modal	12	0.125	0.047	0.779	0	0.173
Modal	13	0.115	0.502	0.031	0	0.467
Modal	14	0.105	0.444	0.128	0	0.428
Modal	15	0.087	0.033	0.803	0	0.164
Modal	16	0.077	0.463	0.036	0	0.501
Modal	17	0.068	0.12	0.807	0	0.073
Modal	18	0.059	0.377	0.115	0	0.508
Modal	19	0.038	0.968	0.002	0	0.03
Modal	20	0.03	0.002	0.997	0	0.001
Modal	21	0.023	0.023	0	0	0.977
Modal	22	0.003	0	0	0	1
Modal	23	0.003	0	0	0	1
Modal	24	0.002	0	0	0	1

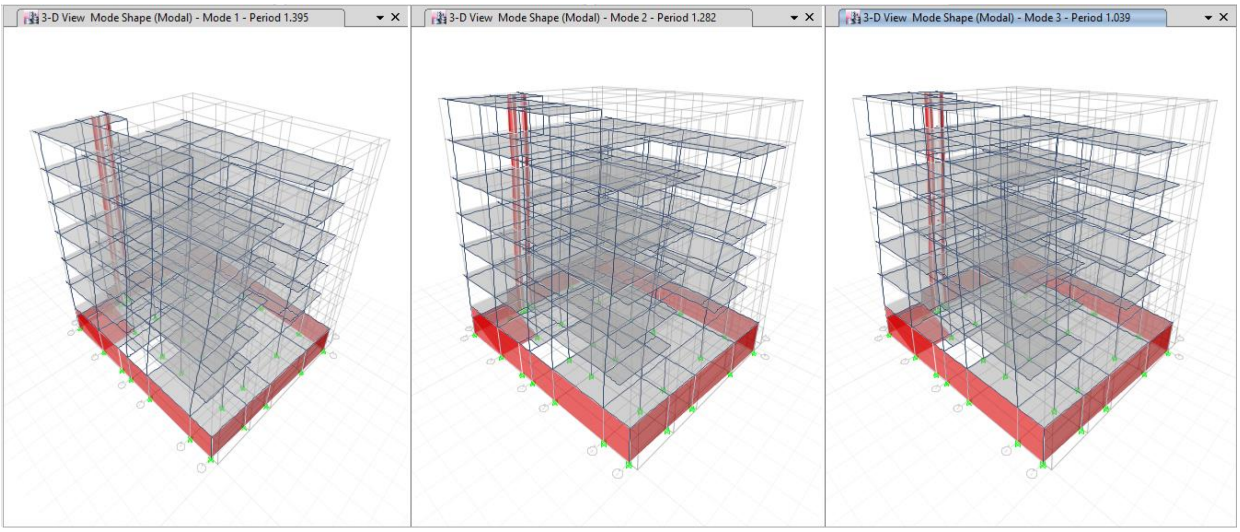


Figure 22: Three Fundamental Mode Shapes (without infill)

4.2.2.6.4 Torsional Effects

The accidental eccentricity was taken as 5% of the floor dimension in all the storeys. In addition the calculated design eccentricities were overwritten to the ETABS model as shown in figure below to shift the centre of mass by the design eccentricity so that the seismic action produces an additional torsional moment while analyzing and designing the structure.

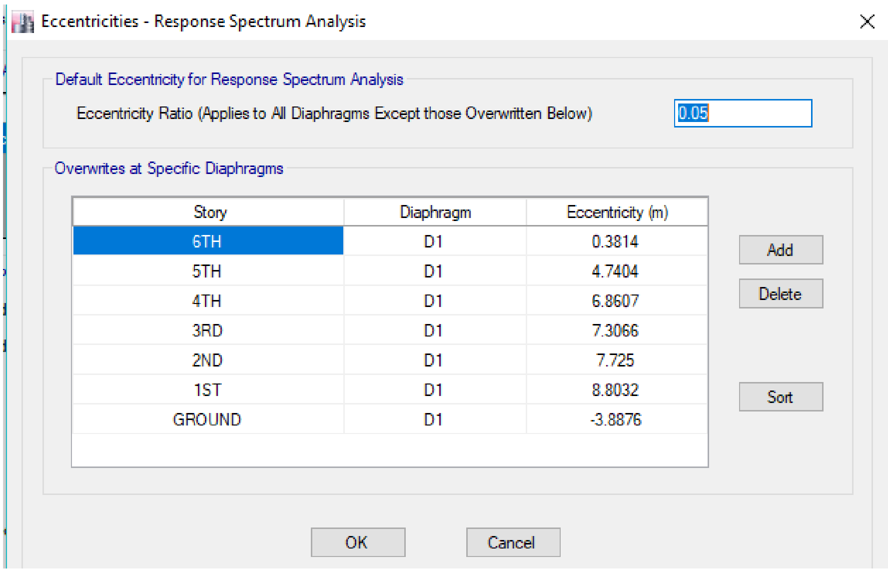


Figure 23: Eccentricities Overwrites in ETABS for torsion

4.2.2.6.5 Storey shear forces by modal response spectrum analysis method

When the design base shear (V_B), obtained by response spectrum analysis is lesser than 80% the base shear ($\overline{V_B}$), obtained by static method, then as per section 7.6 of IS 1893 (part 1) : 2002, the response quantities are scaled up by the factor $0.80x\overline{V_B}/V_B$. In our analysis the response spectrum base shear is greater than 80% of the static base shear, hence no up scaling was needed.

Table 38: Storey shear forces by modal response spectrum analysis method (without infill) (Indian Code)

Storey	Storey shear force (KN)	
	X	Y
6^{TH}	68.4147	94.7785
5^{TH}	286.766	372.9517
4^{TH}	429.2946	551.8057
3^{RD}	538.6204	691.0898
2^{ND}	632.2181	812.1035
1^{ST}	700.6141	892.0537
GROUND	787.3648	962.3147

Table 39: Summary of Base Shear Forces (without infill) (Indian Code)

Direction	Base Shear Force (KN)		
	Static (V_B)	Dynamic ($\overline{V_B}$)	$\overline{V_B}/V_B$
X	953.494	787.365	82.58%
Y	1037.16	962.315	92.78%

4.2.2.7 Storey displacement and drift

The displacement of the center of mass (CM) of each floor level of the building was obtained by both static and dynamic analyses. The drift (d_r) at each floor levels of the structure was evaluated considering the difference of the deflections (d) in the center of mass (CM) at the top and bottom of the storey. The inter-storey drift (d_r) at each floor levels were checked against the maximum allowable value for damage limitation requirement, given as 0.004 times the storey height (h) according to clause 7.11.1 of (IS 1893-1, 2002). The displacement values were adjusted by multiplying by **2R** to obtain the displacement values at ultimate limit state at Maximum Considered Earthquake (MCE) situation (elastic).

Table 40: Storey Displacement (Without Infill) (Indian Code)

Storey	Storey displacement, d in (mm)			
	Static Analysis		Dynamic Analysis	
	X	Y	X	Y
6TH	26.5	35.3	15.5	26.1
5TH	27.1	28.4	15.8	20.2
4TH	24.8	23.8	14.9	17.3
3RD	18.9	17.9	11.8	13.5
2ND	11.9	10.9	7.9	8.6
1ST	4.9	4.2	3.4	3.4
GROUND	0.1	0.1	0.1	0.1

Table 41: Storey Drift criteria for damage limitation (Without Infill) (Indian Code)

Storey	Storey drift, (d_r),				Storey height (h), m	Drift limit. 0.004
	Static Analysis		Dynamic Analysis			
	X	Y	X	Y		
6TH	0.00104	0.001112	0.00065	0.000716	3.6576	0.004
5TH	0.00173	0.001483	0.001046	0.001168	3.6576	0.004
4TH	0.002668	0.001961	0.001662	0.001564	3.6576	0.004
3RD	0.003293	0.002229	0.002202	0.001844	3.6576	0.004
2ND	0.0034	0.002147	0.002514	0.001909	3.6576	0.004
1ST	0.002222	0.001275	0.001743	0.001255	3.6576	0.004
GROUND	3.00E-05	2.10E-05	2.40E-05	1.90E-05	3.6576	0.004
Here all the storey drifts values are less than the code limit of 0.004.						

Table 42: Elastic storey displacement (Without Infill) (Indian Code)

Storey	Multiplier to obtain displacements at ULT at MCE situation (2R)	Storey displacement at ULT at MCE situation, d in (mm)			
		Static Analysis		Dynamic Analysis	
		X	Y	X	Y
6TH	10	265	353	155	261
5TH	10	271	284	158	202
4TH	10	248	238	149	173
3RD	10	189	179	118	135
2ND	10	119	109	79	86
1ST	10	49	42	34	34
GROUND	10	01	01	01	01

4.2.3 Model With Infill

In the second model the lateral load resisting contribution from masonry infill walls were also considered. This was taken into account by modeling the external masonry wall cladding as an equivalent diagonal strut having modulus of elasticity and thickness equivalent to that of wall. To calculate the equivalent width and other properties for equivalent diagonal strut Mainstone and Weeks (1970) model as prescribed in (FEMA 306, 1998), was used. The same were also suggested by Panagiotakos and Fardis Model (1996).

The equivalent strut is represented by the actual infill thickness that is in contact with the frame (t_{inf}) and the diagonal length (L_{inf}) and an equivalent width, a, given by:

$$a = 0175\ (\lambda_1 h_{col})^{-0.4}\ r_{inf}$$

where,

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4E_{fe} I_{col} h_{inf}}\right]^{\frac{1}{4}}$$

And, h_{col} = column height between centerlines of beams

h_{inf} = height of infill panel,

E_{fe} = expected modulus of elasticity of frame material

E_{me} = expected modulus of elasticity of infill material

I_{col} = moment of inertia of column

L_{inf} = diagonal length of infill panel

t_{inf} = thickness of infill panel and equivalent strut

θ = angle whose tangent is the infill height-to-length aspect ratio.

$$\theta = tan^{-1}\left(\frac{h_{inf}}{L_{inf}}\right)$$

L_{inf} = length of infill panel

Only the masonry walls in full contact with the frame elements were need to be considered when computing in-plane stiffness. That means in this study building the external wall claddings of east and west sides parallel along Y-axis were taken into account.

Unreinforced Infill walls parameters for Modeling in Etabs as Equivalent Diagonal struts (FEMA 306, 1998)

Modulus of elasticity, E_m=550 f_m	= 1096.67	N/mm^2
Compressive strength of masonry, $f_m = 0.433 f_b^{0.64} f_{mo}^{0.36}$	= 2.0	N/mm^2
compressive strength of brick, f_b	= 3.5	N/mm^2
compressive strength of mortar (1:4) mix ratio, f_{mo}	= 7.5	N/mm^2
Poisson ratio of brick wall	= 0.17	
Modulus of elasticity of concrete frame, E_f	= 25000	N/mm^2
Moment of inertia of adjoining column, I_c	=3.64E+09	mm^4
Thickness of Equivalent strut, t_s	= 230	mm

Table 43: Parameters for modelling Masonry Infill walls Equivalent diagonal strut (Mainstone and weeks (1970))

Span Name	thickness of wall, t (mm)	length of wall, L (mm)	height of wall, h (mm)	diagonal length, L_{inf} (mm)	angle, θ (rad)	λ_1	width of strut, a (mm)
Grid A(2-3)	230	5843	3200	6662	0.501	2.091	868
Grid A(3-4)	230	2947	3200	4351	0.827	2.181	557
Grid A(4-5)	230	5029	3200	5961	0.567	2.130	771
Grid A(5-6)	230	5029	3200	5961	0.567	2.130	771
Grid E(2-3)	230	5843	3200	6662	0.501	2.091	868
Grid E(3-4)	230	2947	3200	4351	0.827	2.181	557
Grid E(4-5)	230	5029	3200	5961	0.567	2.130	771
Grid E(5-6)	230	5029	3200	5961	0.567	2.130	771

The infill walls were assigned to the ETABS model as an equivalent diagonal strut element with the parameters obtained above. The connection of strut to the concrete frame members were considered as pinned connection.

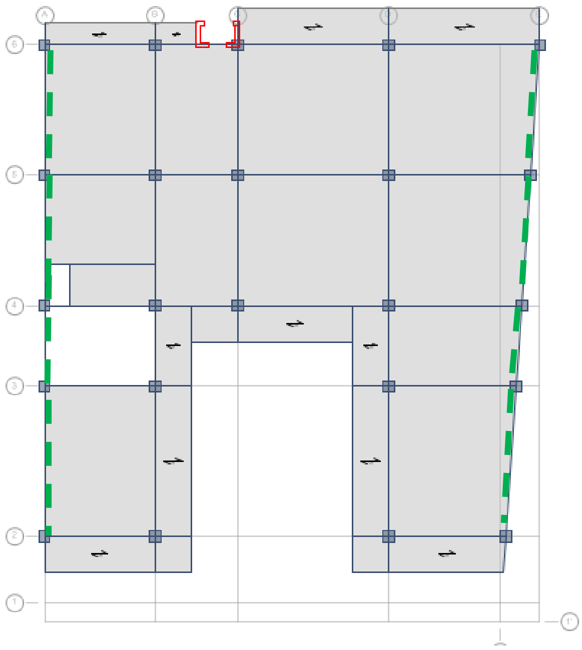


Figure 24: Plan showing infill wall position

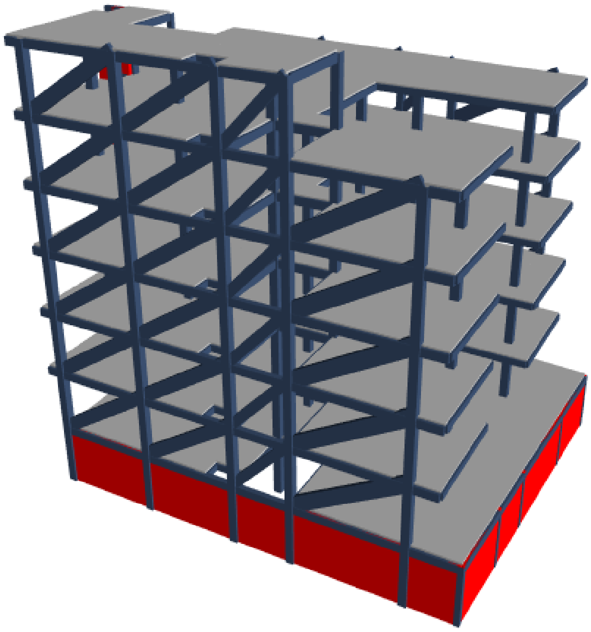


Figure 25: ETABS 3D model with infill walls as equivalent struts

The same analysis procedures as earlier model were performed for this model with infill too i.e. Linear Static and Dynamic analysis methods. The design base shear, lateral force distribution, storey shear forces, model analysis results and displacement – drifts are shown in the tables below for this model.

Table 44: Design Seismic base shear by static lateral force method (with infill) (Indian Code)

Soil Type	Fundamental period, T_a (s)		Z	I	R	S_a/g		Seismic Weight, W (KN)	V_B (KN)	
	X	Y				X	Y			
Medium	1.248	0.930	0.36	1.0	5.0	1.09	1.462	27165.368	1065.487	1430.256

Table 45: Distribution of design seismic base shear at each storey level (with infill) (Indian Code)

Storey Level	W_i (KN)	h_i (m)	$W_ih_i^2$	Q_i (KN)	
				Q_{ix}	Q_{iy}
6TH	945.770	21.9456	455491.684	115.2746	154.7388
5TH	4936.842	18.288	1651131.467	417.8638	560.9195
4TH	5110.402	14.6304	1093874.415	276.8348	371.6091
3RD	5390.646	10.9728	649046.392	164.2589	220.4929
2ND	5390.646	7.3152	288465.063	73.0039	97.9968
1ST	5390.646	3.6576	72116.266	18.251	24.4992
TOTAL			**4210125.29**	**1065.487**	**1430.256**

Table 46: Modal Periods and Frequencies (with infill) (Indian Code)

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad²/sec²
Modal	1	1.248	0.801	5.0335	25.3362
Modal	2	0.93	1.075	6.7567	45.6534
Modal	3	0.845	1.183	7.4347	55.2749
Modal	4	0.388	2.577	16.1933	262.2239
Modal	5	0.3	3.335	20.9525	439.0084
Modal	6	0.272	3.682	23.1332	535.1443
Modal	7	0.214	4.673	29.3587	861.9312
Modal	8	0.17	5.899	37.0646	1373.783
Modal	9	0.162	6.155	38.6702	1495.3829
Modal	10	0.145	6.874	43.1918	1865.5287
Modal	11	0.131	7.628	47.9289	2297.1755
Modal	12	0.113	8.825	55.4506	3074.7653
Modal	13	0.106	9.428	59.2351	3508.8026
Modal	14	0.1	9.982	62.7191	3933.689
Modal	15	0.082	12.164	76.4309	5841.6796
Modal	16	0.075	13.362	83.9536	7048.2118
Modal	17	0.065	15.435	96.9822	9405.5551
Modal	18	0.057	17.558	110.3209	12170.692
Modal	19	0.038	26.454	166.213	27626.7661
Modal	20	0.03	33.067	207.7637	43165.7553
Modal	21	0.023	44.304	278.3695	77489.582
Modal	22	0.003	297.728	1870.6814	3499448.7183
Modal	23	0.003	324.525	2039.0477	4157715.6744
Modal	24	0.002	436.868	2744.9235	7534604.8674

Table 47: Modal Participating Mass Ratios (Part 1 of 2) (with infill) (Indian Code)

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	1	1.248	0.6085	0.0028	0	0.6085	0.0028	0
Modal	2	0.93	0.0082	0.6231	0	0.6167	0.6258	0
Modal	3	0.845	0.0233	0.0301	0	0.64	0.6559	0
Modal	4	0.388	0.0896	0.0007	0	0.7296	0.6566	0
Modal	5	0.3	0.0011	0.0808	0	0.7307	0.7375	0
Modal	6	0.272	0.0065	0.0022	0	0.7373	0.7396	0
Modal	7	0.214	0.0315	0.0002	0	0.7688	0.7399	0
Modal	8	0.17	0.0001	0.0281	0	0.7689	0.7679	0
Modal	9	0.162	0.0015	0.0041	0	0.7704	0.772	0
Modal	10	0.145	0.0153	0.0007	0	0.7857	0.7727	0
Modal	11	0.131	0.0049	0.0002	0	0.7905	0.7729	0
Modal	12	0.113	0.0004	0.017	0	0.791	0.7899	0
Modal	13	0.106	0.0018	0.0003	0	0.7928	0.7901	0

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	14	0.1	0.0081	0.0003	0	0.8008	0.7904	0
Modal	15	0.082	0.0002	0.0109	0	0.8011	0.8013	0
Modal	16	0.075	0.0069	0.0005	0	0.808	0.8018	0
Modal	17	0.065	0.0011	0.0037	0	0.8091	0.8056	0
Modal	18	0.057	0.0063	0.0012	0	0.8154	0.8068	0
Modal	19	0.038	0.179	0.0004	0	0.9943	0.8073	0
Modal	20	0.03	0.0003	0.1927	0	0.9946	0.9999	0
Modal	21	0.023	0.0054	0.0001	0	1	1	0
Modal	22	0.003	0	0	0	1	1	0
Modal	23	0.003	0	0	0	1	1	0
Modal	24	0.002	0	0	0	1	1	0

Table 48: Modal Participating Mass Ratios (Part 2 of 2) (with infill) (Indian Code)

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	0.003	0.38	0.0264	0.003	0.38	0.0264
Modal	2	0.3968	0.0069	0.0256	0.3999	0.3869	0.052
Modal	3	0.009	0.0451	0.5922	0.4089	0.432	0.6442
Modal	4	0.0007	0.1723	0.0017	0.4096	0.6043	0.6459
Modal	5	0.2075	0.0025	0.0017	0.6171	0.6068	0.6475
Modal	6	0.0044	0.013	0.0809	0.6215	0.6198	0.7285
Modal	7	0.0001	0.0233	0.0005	0.6216	0.6431	0.7289
Modal	8	0.019	8.81E-07	0.0029	0.6407	0.6431	0.7318
Modal	9	0.0019	0.0041	0.0098	0.6426	0.6473	0.7417
Modal	10	0.0013	0.0154	0.0144	0.644	0.6627	0.7561
Modal	11	6.378E-06	0.0065	0.0045	0.644	0.6692	0.7606
Modal	12	0.0242	0.0005	0.0003	0.6682	0.6697	0.7609
Modal	13	0.0006	0.0021	0.0013	0.6688	0.6718	0.7622
Modal	14	0.0002	0.0112	0.0099	0.669	0.683	0.7721
Modal	15	0.0138	0.0002	1.667E-05	0.6828	0.6832	0.7721
Modal	16	0.0005	0.0084	0.0064	0.6833	0.6916	0.7785
Modal	17	0.0047	0.0015	0.0002	0.6879	0.6931	0.7786
Modal	18	0.0017	0.0089	0.0031	0.6896	0.7019	0.7818
Modal	19	0.0007	0.2894	0.0001	0.6903	0.9913	0.7819
Modal	20	0.3096	0.0005	5.257E-06	0.9999	0.9918	0.7819
Modal	21	0.0001	0.0082	0.2181	1	1	1
Modal	22	0	0	0	1	1	1
Modal	23	0	0	0	1	1	1
Modal	24	0	0	0	1	1	1

Table 49: Modal Direction Factors (with infill) (Indian Code)

Case	Mode	Period sec	UX	UY	UZ	RZ
Modal	1	1.248	0.932	0.007	0	0.061
Modal	2	0.93	0.014	0.957	0	0.029
Modal	3	0.845	0.084	0.068	0	0.849
Modal	4	0.388	0.854	0.015	0	0.131
Modal	5	0.3	0.012	0.971	0	0.017
Modal	6	0.272	0.197	0.049	0	0.754
Modal	7	0.214	0.7	0.034	0	0.265
Modal	8	0.17	0.067	0.829	0	0.105
Modal	9	0.162	0.553	0.127	0	0.32
Modal	10	0.145	0.244	0.091	0	0.665
Modal	11	0.131	0.594	0.02	0	0.386
Modal	12	0.113	0.027	0.858	0	0.114
Modal	13	0.106	0.564	0.054	0	0.382
Modal	14	0.1	0.323	0.069	0	0.608
Modal	15	0.082	0.053	0.835	0	0.112
Modal	16	0.075	0.36	0.064	0	0.576
Modal	17	0.065	0.127	0.789	0	0.084
Modal	18	0.057	0.303	0.165	0	0.533
Modal	19	0.038	0.967	0.002	0	0.031
Modal	20	0.03	0.002	0.997	0	0.001
Modal	21	0.023	0.023	0	0	0.977
Modal	22	0.003	0	0	0	1
Modal	23	0.003	0	0	0	1
Modal	24	0.002	0	0	0	1

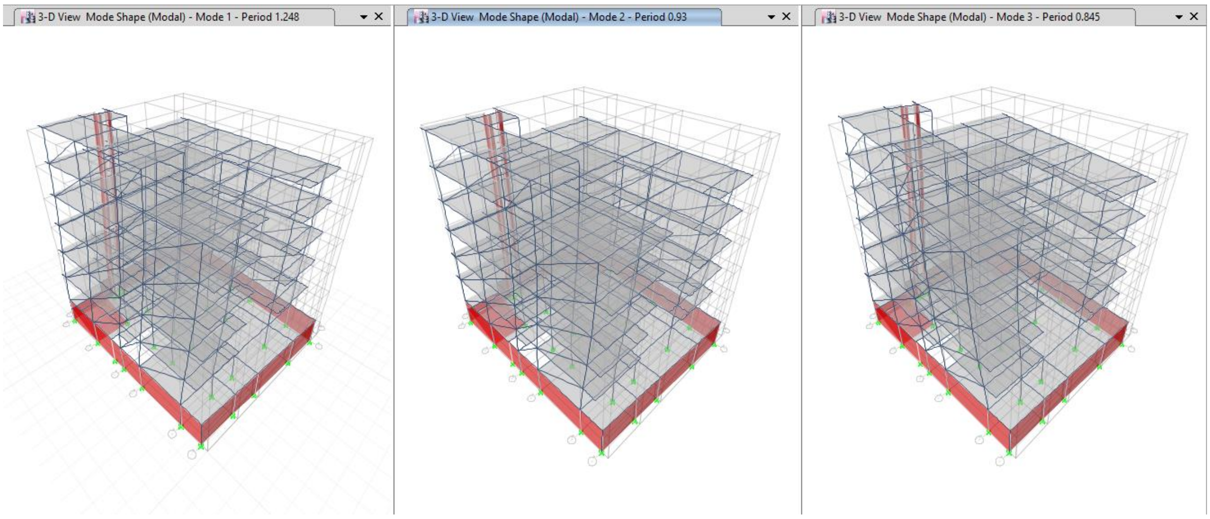


Figure 26: Three Fundamental Mode Shapes (with infill)

Table 50: Storey shear forces by modal response spectrum analysis method (with infill) (Indian Code)

Storey	Storey shear force (KN)	
	X	Y
6^{TH}	71.8404	103.1984
5^{TH}	346.4205	454.9841
4^{TH}	535.7241	727.6962
3^{RD}	678.8408	949.5285
2^{ND}	801.425	1116.194
1^{ST}	886.7441	1208.3985
GROUND	957.3826	1261.5417

Table 51: Summary of Base Shear Forces (with infill) (Indian Code)

Direction	Base Shear Force (KN)		
	Static (V_B)	Dynamic ($\overline{V_B}$)	$\overline{V_B}/V_B$
X	1065.49	957.38	89.85%
Y	1430.26	1261.54	88.20%

Table 52: Storey Displacement (With Infill) (Indian Code)

Storey	Storey displacement, d in (mm)			
	Static Analysis		Dynamic Analysis	
	X	Y	X	Y
6^{TH}	29.3	25.1	19	18.8
5^{TH}	24.8	19.7	18.9	14.2
4^{TH}	20.4	17	17.2	12.5
3^{RD}	15.1	12.9	13.5	9.9
2^{ND}	9.3	8.2	8.9	6.5
1^{ST}	3.7	3.4	3.8	2.8
GROUND	0.1	0.1	0.1	0.1

Table 53: Storey Drift criteria for damage limitation (With Infill) (Indian Code)

Storey	Storey drift, (d_r),				Storey height (h), m	Drift limit. 0.004
	Static Analysis		Dynamic Analysis			
	X	Y	X	Y		
6^{TH}	0.001076	0.000671	0.000592	0.000411	3.6576	0.004
5^{TH}	0.001311	0.000988	0.000936	0.000728	3.6576	0.004
4^{TH}	0.00168	0.001358	0.001436	0.001017	3.6576	0.004
3^{RD}	0.00186	0.001583	0.001855	0.001245	3.6576	0.004
2^{ND}	0.001834	0.001622	0.002118	0.001358	3.6576	0.004
1^{ST}	0.001094	0.001072	0.001589	0.000933	3.6576	0.004
GROUND	3.30E-05	2.50E-05	2.60E-05	2.20E-05	3.6576	0.004
Here all the storey drifts values are less than the code limit of 0.004.						

5.0 SEISMIC ANALYSIS ACCORDING TO EUROCODE (EN 1998-1:2004)

5.1 Design Seismic action

5.1.1 Classification of building

The building is an ordinary office building thus, it is categorized as importance class II Table 8. (EN1998-1:, 2004)

5.1.2 Design peak ground acceleration

The peak ground acceleration for Kathmandu Valley for 475 year return period is considered to be 0.25g (Wijeyewickrema & et al, 2011).

$a_g = (1.0 \text{ x } 0.25g) = 0.25g$

5.1.3 Behavior factor (q)

This building has been designed for Ductility Class High (DCH) conditions as the reinforcement detailing of the structure corresponds to the requirements suggested in (EN1998-1:, 2004), and EN1992. The behavior factor (q) used in the reinforced concrete structures as given in EN 1998-1/5.2.2.2 is given by

$q = q_0 k_w \geq 1.5$

$q_0 = 4{,}5\ \alpha_u/\alpha_1$ (for Multistory, multi bay frames $\alpha_u/\alpha_1 = 1{,}3$)

$k_w = 1.0$

Since the selected building is irregular in elevation, 80% of the q_0 has to be used in calculations, as described in (EN1998-1:, 2004).

Therefore, q = 1.0x4.5x1.3x0.8 = 4.68

Soil Type C (Medium Soil)

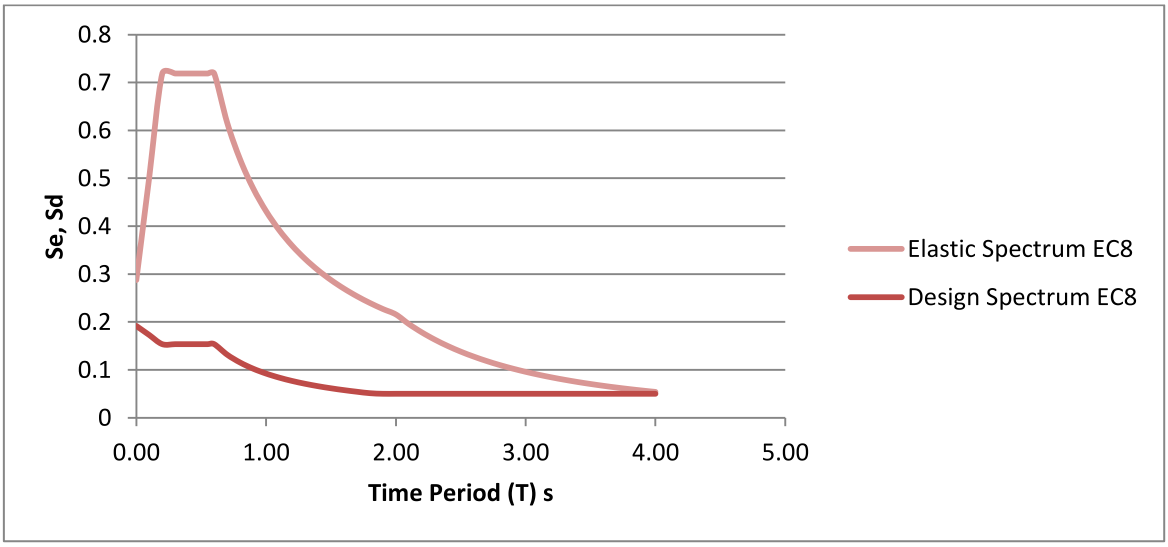


Figure 27: Elastic response spectrum and design response spectrum

5.2 Method of analysis

5.2.1 Structural Model

Both model response spectrum and static lateral force method of analyses were performed on a three dimensional structural model of the building (same as of Indian code section). All the computer analyses were performed with the ETABS software (ETABS, 2016).

The (EN1998-1:, 2004) recommends using a spatial model as the preference method for all type of buildings (Clause 4.3.1 of (EN1998-1:, 2004)). On account of that, for the study building a three dimensional model was developed. In this study also, two models were considered for analyses. One without infill wall and another with infill wall contribution.

5.2.2 Model Without Infill

5.2.2.1 General

At first the building was considered to have no significant structural effect from the masonry infill walls on its behavior when subjected to seismic load. The reinforced concrete frame wall system was considered as the only lateral load resisting system in the building and therefore, the presence of masonry infill walls were not considered in making the model. However, their weight was considered in the calculation of seismic weight of the building. The assumptions and basic characteristics considered to model the building in ETABS software were same as that of Indian code approach in section 4.2.2.1.

5.2.2.2 Lateral force method of analysis

Analysis according to seismic coefficient method also known as lateral force method is carried out in three main steps as follows.

I. Estimating the self-weight and seismic weight of the building

II. Calculating the seismic base shear in relevant directions

III. Distribution of lateral forces at each floor level.

5.2.2.3 Estimating of seismic mass of the building

The seismic mass of the building was taken as the following combination of dead load and the variable loads as stated in section 3.2.4 of (EN1998-1:, 2004),

$\Sigma G_{k,j}$ "+" $\Sigma \psi_{E,i} \cdot Q_{k,i}$

$G_{k,j}$: permanent load

$Q_{k,i}$: variable load

$\Psi_{E,i}$ = $\psi_{2,i}\varphi$ (EN 1998-1:4.2.4)

$\psi_{2,i}$: factor representing the quasi permanent value of the variable action (EN 1990:2002 – Table 13)

φ : (EN 1998-1: Tabe 4.2 – Refer Table 14)

Table 54: Mass Source EN 1998-1:2004

Load Pattern	$\psi 2i$	φ	ψEi	Multiplier
Dead (G)				1
Live residential	0.3	0.5	0.15	0.15
Wall load (G)				1
Floor finish (G)				1
Stair Dead (G)				1
Stair live	0.8	0.5	0.4	0.4
Roof Live	0.3	1.0	0.3	0.3

Table 55: Seismic Mass Summary by Story (EN1998-1:2004)

Story	UX (kg)	UY (kg)	UZ (kg)
6TH	91092.74	91092.74	0
5TH	527655.54	527655.54	0
4TH	508449.95	508449.95	0
3RD	531572.14	531572.14	0
2ND	531572.14	531572.14	0
1ST	531572.14	531572.14	0
TOTAL SEISMIC MASS	2721914.65	2721914.65	0

5.2.2.4 Calculation of seismic base shear

As described earlier in Eurocode design review, the seismic base shear force for each horizontal direction was determined by the following equation,

$$F_b = S_d\,(T_1)\cdot m\cdot\lambda$$

Where

$S_d(T_1)$: the spectral acceleration obtained from the design response spectrum for the fundamental period of vibration T_1.

m : the seismic mass of the building

λ : correction factor as given in EN 1998-1:2004/4.3.3.2.2 = 1.0

T_1 : fundamental period of vibration of the building

The fundamental natural period of vibration (T_1) has been obtained by model analysis performed on the three dimensional computer model of the building.

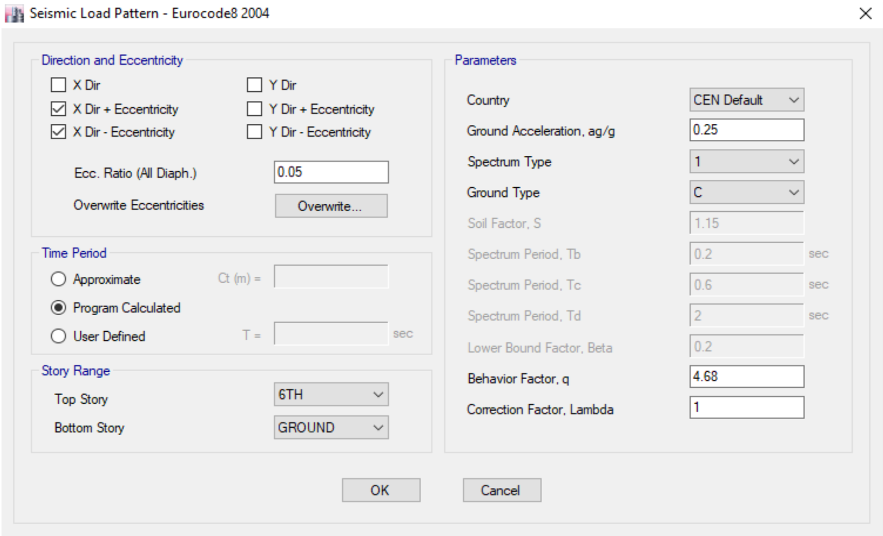


Figure 28: Seismic Load Application Parameters in ETABS

The design base shear force acting in each horizontal direction is shown in table 55 below.

Table 56: Design Seismic base shear by lateral force method (without infill) (Eurocode)

Soil Type	Fundamental period, T_1 (s)		λ	q	$S_d(T_1)$		Seismic Weight, W (KN)	F_B (KN)	
	X	Y			X	Y		X	Y
Medium	1.395	1.282	1.0	4.68	0.0661	0.0719	26692.8649	1764.64	1919.39

5.2.2.5 Distribution of lateral forces

The seismic base shear (F_b) was distributed at each storey level by using the following expression as (EN 1998-1:2004/eq.4.10)

$$F_k = F_b \frac{z_i m_i}{\sum z_j m_j}$$

Table 57: Distribution of design seismic base shear at each storey level (without infill) (Eurocode)

Storey Level	m_i (t)	Height z_i (m)	$m_i z_i$	Q_i (KN)	
				Q_{ix}	Q_{iy}
6TH	91.09274	21.9456	1999.084835	114.7083	124.7675
5TH	527.65554	18.288	9649.764516	553.7075	602.264
4TH	508.44995	14.6304	7438.826148	426.843	464.2743
3RD	531.57214	10.9728	5832.834778	334.6905	364.0407
2ND	531.57214	7.3152	3888.556519	223.127	242.6938
1ST	531.57214	3.6576	1944.278259	111.5635	121.3469
TOTAL			**30753.34505**	**1764.64**	**1919.39**

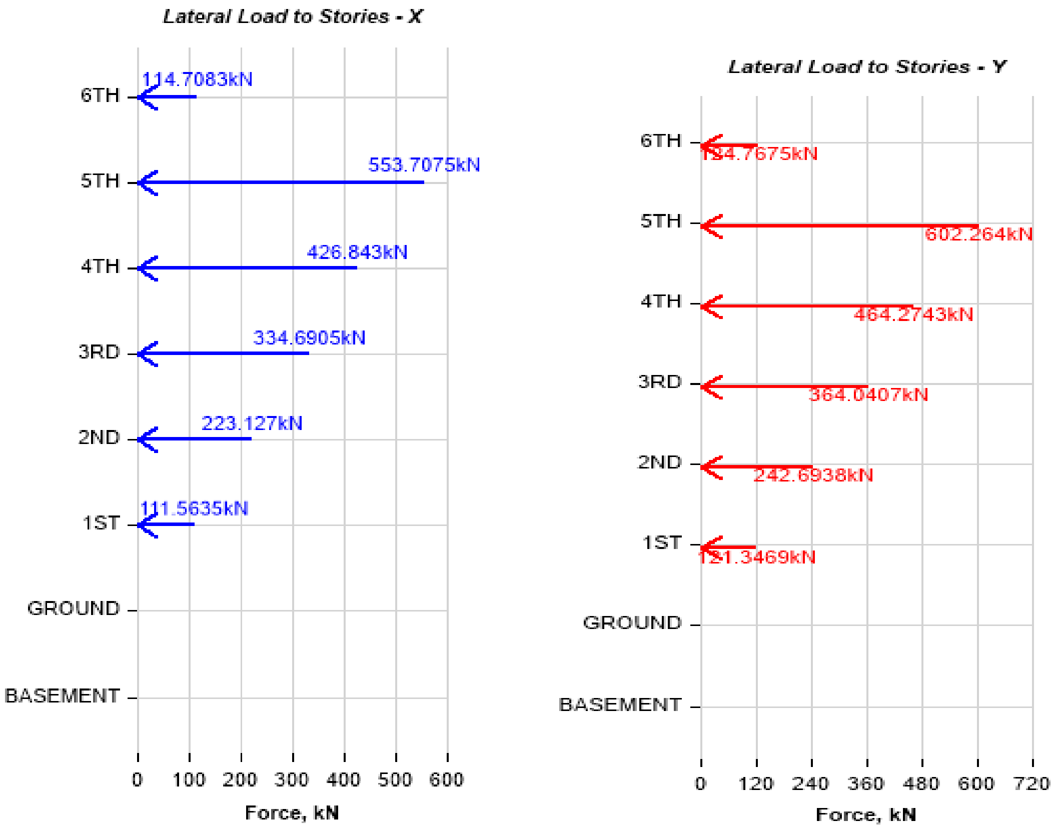


Figure 29: Lateral Load Distribution without infill (Eurocode)

5.2.2.6 Model Response Spectrum Method (Dynamic Analysis)

5.2.2.6.1 General rules

The general rules recommended for this type of analysis as described in clause 4.3.3.3 of EN1998-1:, 2004 were followed in case of the study building and are as follows.

- ✓ Modal response spectrum analysis has been performed independently for the ground excitation in two horizontal directions. The excitation in vertical direction was not considered since the structure does not have large span beams, pre-stress components or cantilever projections.
- ✓ Design spectrum for ductility class high is used
- ✓ For the combination of different modes, the “Complete Quadratic Combination (CQC) rule was used (Clause 4.3.3.3.2 of EN1998-1:, 2004).
- ✓ The results of the modal analysis in both directions were combined by the SRSS rule as described in clause 4.3.3.5.1 of EN 1998-1:2004.
- ✓ The load combinations were considered according to clause 3.2.4 of EN1998-1:, 2004.
- ✓ The accidental torsional effect was considered by overwriting the eccentricity value in the response spectrum load case which then will be considered in the analysis to calculate additional torsional moments to the buildings.

5.2.2.6.2 Response Spectrum Functions

The response spectrum function defined in ETABS model was considered as per (EN1998-1:, 2004).

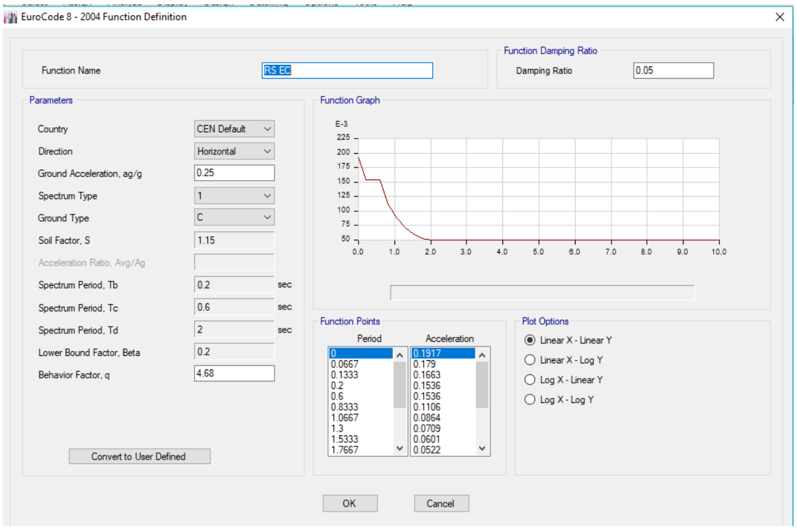


Figure 30: Response Spectrum Function Parameter applied in ETABS (EN1998-1:2004)

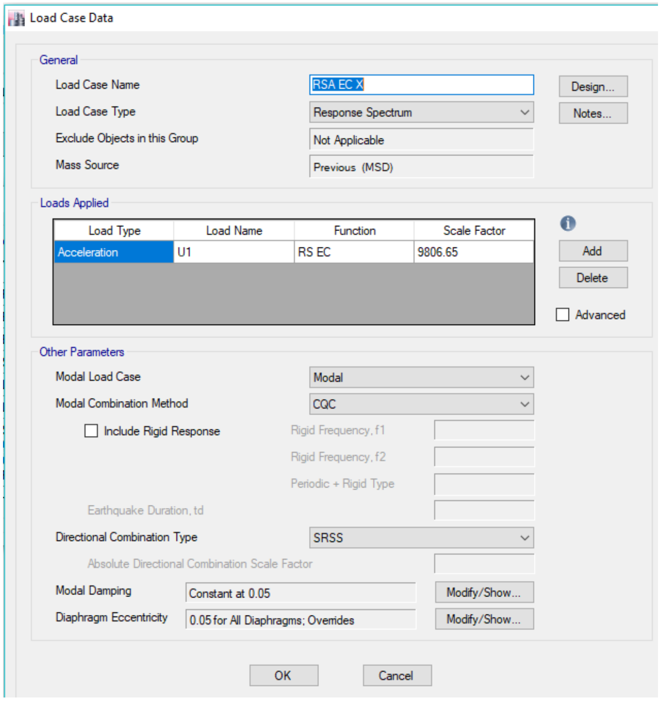


Figure 31: Response Spectrum Load Case Definition in ETABS (EN 1998-1:2004)

5.2.2.6.3 Periods and effective masses

In the modal response spectrum analysis, 24 modes of vibration were taken into account as the sum of the modal masses in each horizontal direction to exceed 90% of the total mass of the structure. All modes with effective modal masses greater than 5% of the total mass are taken into account.
The modal analysis results are same as in Indian Code design chapter 4.2.2.6.3.

5.2.2.6.4 Torsional Effects

The accidental eccentricity was taken as 5% of the floor dimension in all the storeys. In addition the calculated eccentricities (difference between CM and CR) were overwritten to the ETABS model as shown in figure below to shift the center of mass by the design eccentricity so that the seismic action produces an additional torsional moment while analyzing and designing the structure.

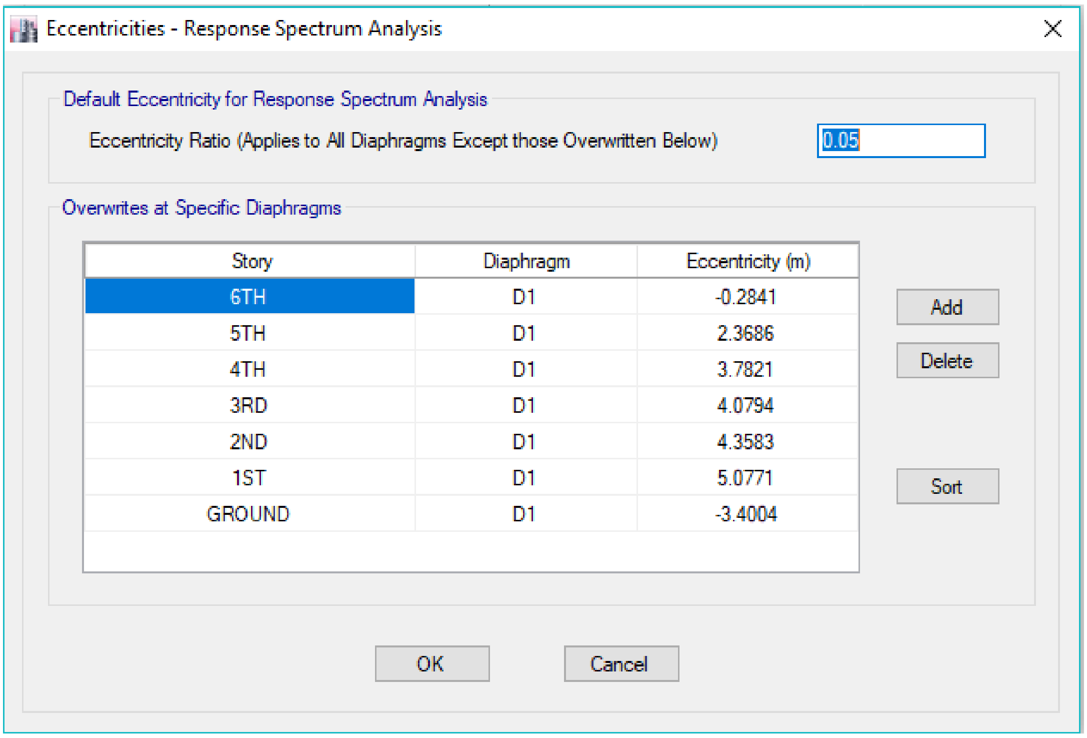


Figure 32: Eccentricities Overwrites in ETABS for torsion

5.2.2.6.5 Storey shear forces

In the case of study building, the storey shear forces at each floor level of the building were obtained by both lateral force method and response spectrum analysis for the system. Unlike in Indian code where the response spectrum base shear was modified by the factor in order to get the base shear greater than 80% of the static, Eurocode does not specify any such necessity for modification of dynamic base shear forces.

Table 58: Storey shear forces (without infill) (Eurocode)

Storey	Storey shear force (KN)			
	Lateral Force Method		Response Spectrum Method	
	X	Y	X	Y
6^{TH}	114.7083	124.7675	124.08571	181.32575
5^{TH}	668.4159	727.0315	550.08186	712.52394
4^{TH}	1095.2588	1191.3058	823.65439	1051.9206
3^{RD}	1429.9493	1555.3465	1038.27474	1320.56978
2^{ND}	1653.0763	1798.0403	1210.90792	1539.7672
1^{ST}	1764.6398	1919.3871	1330.24804	1677.07133
GROUND	1764.6398	1919.3871	1732.83276	2067.94606

5.2.2.7 Storey displacement and drift

The displacement of the center of mass (CM) of each floor level of the building was obtained by both static and dynamic analyses. The inter-storey drift (d_r) was evaluated as described in (EN1998-1:, 2004) considering the difference of the lateral displacements (d_s) in center of mass (CM) at the top and bottom of the storey, obtained by both analysis.

The inter-storey drift (d_r) was then checked for damage limitation requirement given by the following equation,

$$d_r\nu \leq (\propto).h$$

Since the structure is of importance level II, the ν value was selected to 0.5. And α is taken as 0.005 for buildings with brittle material (Brick Masonry) non- structural elements.

All parameters for the verification of the damage limitation requirement for response spectrum analysis are listed below.

Table 59: Storey Displacement (Without Infill) (Eurocode)

Storey	Storey elastic displacement, d_e (mm)				q_d	Storey design displacement, d_s (mm)			
	Static		Dynamic			Static		Dynamic	
	X	Y	X	Y		X	Y	X	Y
6^{TH}	41.8	53.6	29.79	49.71	4.68	195.624	250.848	139.4172	232.6428
5^{TH}	43	46.4	30.79	38.69	4.68	201.24	217.152	144.0972	181.0692
4^{TH}	39.3	39.8	28.75	33.08	4.68	183.924	186.264	134.55	154.8144
3^{RD}	30.6	30.6	22.64	25.63	4.68	143.208	143.208	105.9552	119.9484
2^{ND}	19.8	19.2	14.97	16.29	4.68	92.664	89.856	70.0596	76.2372
1^{ST}	8.2	7.5	6.37	6.38	4.68	38.376	35.1	29.8116	29.8584
GROUND	0.1	0.1	0.103	0.104	4.68	0.468	0.468	0.48204	0.48672

Table 60: Parameters defining the criteria for damage limitation requirement (Without Infill) (Eurocode)

Storey	d_r (m), max		h (m)	ν	α = d_r* ν/h		Remarks
	X-Dir	Y-Dir			X-Dir	Y-Dir	
6^{TH}	0.005616	0.033696	3.657	0.5	0.00077	0.00461	α (<0.005)
5^{TH}	0.017316	0.030888	3.657	0.5	0.00237	0.00422	
4^{TH}	0.040716	0.043056	3.657	0.5	**0.00557**	**0.00589**	
3^{RD}	0.050544	0.053352	3.657	0.5	**0.00691**	**0.00729**	
2^{ND}	0.054288	0.054756	3.657	0.5	**0.00742**	**0.00749**	
1^{ST}	0.037908	0.034632	3.657	0.5	**0.00518**	0.00474	
GROUND	0.000468	0.000468	3.657	0.5	0.00006	0.00006	

Here the damage limitation check does not satisfy in the 2^{nd}, 3^{rd} and 4^{th} floors as value of α is greater than 0.005.

5.2.2.8 P-Δ effects

As described in section 2.1.5.7, the P-Δ effects was checked according to the equation for inter-storey drift sensitivity coefficient given as,

$$\theta = \frac{P_{tot}.d_r}{V_{tot}.h} \leq 0.10$$

Where,

P_{tot} : is the total gravity load, including appropriate amount of imposed load at and above the storey considered in the seismic design situation.

d_r : is the inter-storey drift

V_{tot} : is the total seismic storey shear from response spectrum analysis.

h : Floor to floor height

Table 61: Calculation of inter-storey drift sensitivity coefficient at each level of building (Eurocode)

Storey	P_{tot}(KN)	d_r (m)		V_{tot}(KN)		h (m)	θ	
		X	Y	X	Y		X	Y
6^{TH}	893.6197794	0.0047	0.0516	124.08571	181.32575	3.657	0.009	0.070
5^{TH}	5176.300847	0.0095	0.0263	550.08186	712.52394	3.657	0.024	0.052
4^{TH}	4987.89401	0.0286	0.0349	823.65439	1051.9206	3.657	0.047	0.045
3^{RD}	5214.722693	0.0359	0.0437	1038.27474	1320.56978	3.657	0.049	0.047
2^{ND}	5214.722693	0.0402	0.0464	1210.90792	1539.7672	3.657	0.047	0.043
1^{ST}	5214.722693	0.0293	0.0294	1330.24804	1677.07133	3.657	0.031	0.025

Since the value of θ is less than 0.1 for all storey no P-Δ effects were need to be considered.

5.2.3 Model With Infill

The second model included the effects of infill wall in resisting the lateral load same as in Indian code design procedure in chapter 4.2.3 using Mainstone and Weeks (1970) model of equivalent diagonal strut (FEMA 306, 1998). All other modeling assumptions and characteristics were followed similar to earlier without infill analysis.

The same analysis procedures as earlier model were performed for this model with infill too i.e. Linear Static and Dynamic analysis methods. The design base shear, lateral force distribution, storey shear forces, model analysis results and displacement – drifts are shown in the tables below for this model.

Table 62: Design seismic base shear by lateral force method (with infill) (Eurocode)

Soil Type	Fundamental period, T_1 (s)		λ	q	$S_d(T_1)$		Seismic Weight, W (KN)	F_B (KN)	
	X	Y			X	Y		X	Y
Medium	1.247	0.927	1.0	4.68	0.0739	0.0994	26692.8649	1972.47	2653.72

Table 63: Distribution of design seismic base shear at each storey level (with infill) (Eurocode)

Storey Level	m_i (t)	Height z_i (m)	m_iz_i	Q_i (KN)	
				Q_{ix}	Q_{iy}
6TH	91.09274	21.9456	1999.084835	128.2181	172.5017
5TH	527.65554	18.288	9649.764516	618.9206	832.6813
4TH	508.44995	14.6304	7438.826148	477.1145	641.8987
3RD	531.57214	10.9728	5832.834778	374.1088	503.3172
2ND	531.57214	7.3152	3888.556519	249.4058	335.5448
1ST	531.57214	3.6576	1944.278259	124.7029	167.7724
TOTAL			**30753.34505**	1972.47	2653.72

Table 64: Storey Shear forces (with infill) (Eurocode)

Storey	Storey shear force (KN)			
	Lateral Force Method		Response Spectrum Method	
	X	Y	X	Y
6TH	128.2181	172.5017	132.38158	194.54915
5TH	747.1387	1005.183	672.17653	893.36188
4TH	1224.2532	1647.0818	1036.74792	1416.37966
3RD	1598.362	2150.399	1316.61875	1842.23913
2ND	1847.7678	2485.9438	1544.55995	2154.25766
1ST	1972.4707	2653.7162	1691.71274	2319.66272
GROUND	1972.4707	2653.7162	2025.64161	2613.18811

Table 65: Storey Displacement (with infill) (Eurocode)

Storey	Storey elastic displacement, d_e (mm)				q_d	Storey design displacement, d_s (mm)			
	Static		Dynamic			Static		Dynamic	
	X	Y	X	Y		X	Y	X	Y
6^{TH}	46.3	37.26	36.88	34.59	4.68	216.68	174.38	172.60	161.88
5^{TH}	41.13	32.27	36.29	27.67	4.68	192.49	151.02	169.84	129.50
4^{TH}	34.96	28.39	32.67	24.31	4.68	163.61	132.87	152.90	113.77
3^{RD}	26.81	22.25	25.59	19.17	4.68	125.47	104.13	119.76	89.72
2^{ND}	17.09	14.64	16.75	12.66	4.68	79.98	68.52	78.39	59.25
1^{ST}	6.94	6.07	7.11	5.35	4.68	32.48	28.41	33.27	25.04
GROUND	0.20	0.10	0.11	0.11	4.68	0.94	0.47	0.51	0.51

Table 66: Parameters defining the criteria for damage limitation requirement (With Infill) (Eurocode)

Storey	d_r (m), max		h (m)	ν	$\alpha = d_r * \nu/h$		Remarks
	X-Dir	Y-Dir			X-Dir	Y-Dir	
6^{TH}	0.02420	0.02335	3.657	0.5	0.00331	0.00319	
5^{TH}	0.02888	0.01816	3.657	0.5	0.00395	0.00248	
4^{TH}	0.03814	0.02874	3.657	0.5	**0.00521**	0.00393	
3^{RD}	0.04549	0.03561	3.657	0.5	**0.00622**	0.00487	α (<0.005)
2^{ND}	0.04750	0.04011	3.657	0.5	**0.00649**	**0.00548**	
1^{ST}	0.03154	0.02794	3.657	0.5	0.00431	0.00382	
GROUND	0.00094	0.00047	3.657	0.5	0.00013	0.00006	

Here the damage limitation check does not satisfy in the 2^nd^, 3^rd^ and 4^th^ floors as value of α is greater than 0.005.

6.0 SEISMIC ANALYSIS ACCORDING TO NEPAL BUILDING CODE (NBC 105: 1994)

6.1 Design seismic action

6.1.1 Zone factor, Z

The zone factor, Z for different zones in Nepal is obtained from Figure 8.2 of (NBC 105, 1994). For Kathmandu it is taken as **1.0.**

6.1.2 Importance factor, I

This is an office purpose commercial building having 7 storeys. Therefore the importance factor according to table 8.1 of (NBC 105, 1994) has been selected as **1.0.**

6.1.3 Structural Performance Factor, K

For Ductile moment resisting frame complying with the detailing for ductility requirement of IS 13920, the structural performance factor, K is taken as **1.0.**

6.1.4 Design Spectrum and Basic Seismic Coefficient, C

For seismic response calculation the natural period of vibration of the structure is used as the building is existing one. The time period in both directions is greater than 1.

For medium type of soil and 5% Damping from Figure 8.1 of (NBC 105, 1994) the design spectrum and basic seismic coefficient, C or $C(T_1)$ can thus be taken as:

$C = 0.04/T_1$

6.2 Method of analysis

6.2.1 Structural Model

Since the selected building is irregular in configurations a modal response spectrum method shall be used as per section 5.2 of (NBC 105, 1994). However, a static lateral force method of analysis has also been performed since the base shear force obtained by dynamic analysis has to be compared against that of static lateral force method.

A three dimensional mathematical model was used in this analysis since it can depict the special distribution of the mass and the stiffness of the structure adequately.

For NBC approach we consider only model without infill as NBC does not provide any description on how to consider the masonry infill. This NBC analysis result is compared with similar model with Indian code and Eurocode method analysis results.

The reinforced concrete frame system was considered as the only lateral load resisting system in the building and therefore, the presence of masonry infill walls were not considered in making the model. However, their weight was considered in the calculation of seismic weight of the building. The assumptions and basic characteristics considered to model the building in ETABS software were same as that of Indian code approach in section 4.2.2.1.

6.2.2 Seismic coefficient method (Linear Static analysis)

Analysis according to seismic coefficient method also known as lateral force method is carried out in three main steps as follows.

I. Estimating the self-weight and seismic weight of the building

II. Calculating the seismic base shear in relevant directions

III. Distribution of lateral forces at each floor level.

6.2.3 Seismic weight of the building

The seismic weight of the building was calculated considering same ways as in IS code method chapter 4.2.1.1.1.1.

Table 67: Center of Mass, Rigidity and eccentricity (NBC 105)

Story	Diaphragm	XCM	YCM	XCR	YCR	Static Ecc - X	Static Ecc - Y	Design Ecc - X	Design Ecc - Y
		m	m	m	m	m	m	m	m
6TH	D1	3.0781	18.6059	7.7272	18.3218	4.6491	-0.2841	5.5711	1.3309
5TH	D1	9.7956	15.2972	9.4304	17.6658	-0.3652	2.3686	1.6818	4.7436
4TH	D1	9.5964	14.162	9.4423	17.9441	-0.1541	3.7821	1.8929	6.1571
3RD	D1	9.7341	14.1593	9.3774	18.2387	-0.3567	4.0794	1.6903	6.4544
2ND	D1	9.7341	14.1593	9.2265	18.5176	-0.5076	4.3583	1.5394	6.7333
1ST	D1	9.7341	14.1593	8.8458	19.2364	-0.8883	5.0771	1.1587	7.4521
GROUND	D1	9.6685	12.878	9.7961	9.4776	0.1276	-3.4004	2.1606	-0.9744

6.2.4 Design seismic base shear

The horizontal seismic shear force acting at the base of the structure, in the direction being considered is given in section 10.1 of (NBC 105, 1994) as:

$V = C_d W$, $C_d = CZIK$

Where, C_d = Design horizontal seismic force coefficient

W = Seismic weight of the building

The fundamental natural period of vibration (T_1) has been obtained by model analysis performed on the three dimensional computer model of the building.

The design base shear force acting in each horizontal direction is shown in table 31 below.

Table 68: Design Seismic base shear by static seismic coefficient method (NBC)

Soil Type	Fundamental period, T_a (s)		Z	I	K	C = 0.04/T		Seismic Weight, W (KN)	V_B (KN)	
	X	Y				X	Y			
Medium	1.395	1.282	1.0	1.0	1.0	0.029	0.031	27165.368	787.80	842.13

6.2.5 Distribution of lateral forces

The design base shear (V_B) is distributed along the height of the building as per expression in section 10.2.1 of (NBC 105, 1994).

Table 69: Distribution of design seismic base shear at each storey level (NBC)

Storey Level	W_i (KN)	h_i (m)	W_ih_i	Q_i (KN)	
				Q_{ix}	Q_{iy}
6TH	945.770	21.9456	20755.49011	53.7672	57.4753
5TH	4936.842	18.288	90284.9665	233.8839	250.0138
4TH	5110.402	14.6304	74767.22542	193.6851	207.0426
3RD	5390.646	10.9728	59150.48043	153.2297	163.7973
2ND	5390.646	7.3152	39433.65362	102.1532	109.1982
1ST	5390.646	3.6576	19716.82681	51.0766	54.5991
TOTAL			**304109**	**787.80**	**842.13**

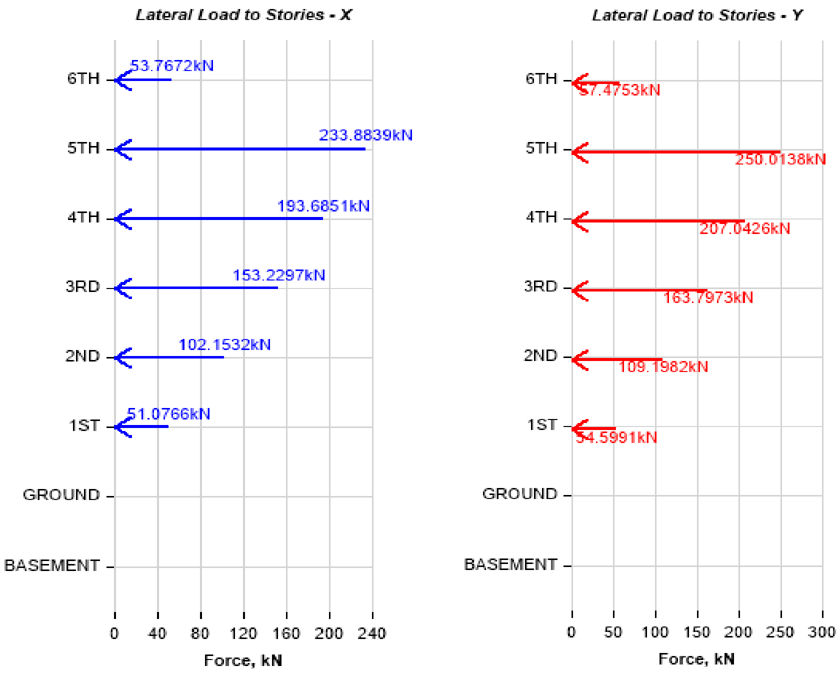


Figure 33: Lateral Force Distribution (NBC Code)

6.2.6 Model Response Spectrum Method (Dynamic Analysis)

6.2.6.1 General rules

The general rules recommended for this type of analysis were followed in case of the study building and are as follows.

- ✓ Modal response spectrum analysis has been performed independently for the ground excitation in two horizontal directions. The excitation in vertical direction was not considered.
- ✓ The design spectrum defined in section 8.1.2 of NBC 105, 1994 was used for the study building.
- ✓ For the combination of different modes, the “Complete Quadratic Combination (CQC) rule was used.
- ✓ The results of the modal analysis in both directions were combined by SRSS rule.

- ✓ The accidental torsional effect was considered by overwriting the eccentricity value in the response spectrum load case which then will be considered in the analysis to calculate additional torsional moments to the buildings.

6.2.6.2 Response Spectrum Functions

The response spectrum functions defined in ETABS model was considered as user defined calculated from NBC 105.

Table 70: Response Spectrum Function - NBC 105:1994

Name	Period sec	Acceleration	Damping %
NBC105 RS	0	0.08	5
NBC105 RS	0.2	0.08	
NBC105 RS	0.5	0.08	
NBC105 RS	0.55	0.0727	
NBC105 RS	0.6	0.0667	
NBC105 RS	0.8	0.05	
NBC105 RS	1	0.04	
NBC105 RS	1.2	0.0333	
NBC105 RS	1.4	0.0286	
NBC105 RS	1.6	0.025	
NBC105 RS	1.8	0.0222	
NBC105 RS	2	0.02	
NBC105 RS	2.2	0.0182	
NBC105 RS	2.4	0.0167	
NBC105 RS	2.6	0.0154	
NBC105 RS	2.8	0.0143	
NBC105 RS	3	0.0133	
NBC105 RS	3.2	0.0125	
NBC105 RS	3.4	0.0118	
NBC105 RS	3.6	0.0111	
NBC105 RS	3.8	0.0105	
NBC105 RS	4	0.01	

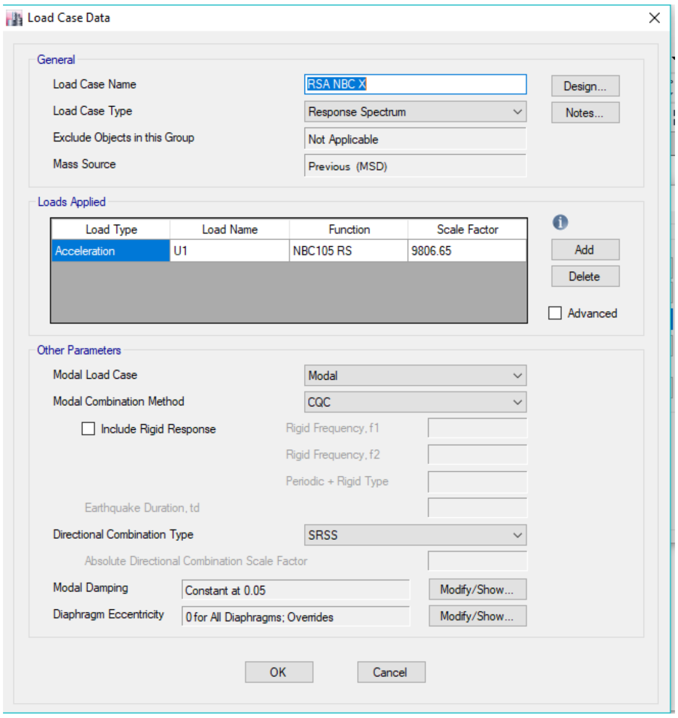


Figure 34: Response Spectrum Load Case Definition in ETABS (NBC 105:1994)

6.2.6.3 Periods and effective masses

In the modal response spectrum analysis, 24 modes of vibration were taken into account as the sum of the modal masses in each horizontal direction to exceed 90% of the total mass of the structure.

The modal analysis results are same as in Indian Code design chapter 4.2.1.1.2.3.

6.2.6.4 Torsional Effects

The accidental eccentricity was taken as 5% of the floor dimension in all the storeys. In addition the calculated design eccentricities as per section 8.2.2 of (NBC 105, 1994) were overwritten to the ETABS model as shown in figure below to shift the center of mass by the design eccentricity so that the seismic action produces an additional torsional moment while analyzing and designing the structure.

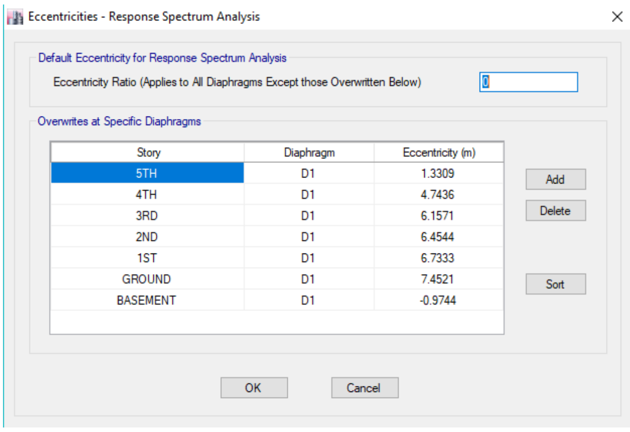


Figure 35: Eccentricities Overwrites in ETABS for torsion (NBC)

6.2.6.5 Storey shear forces by modal response spectrum analysis method

When the design base shear (V_B), obtained by response spectrum analysis is lesser than 80% the base shear ($\overline{V_B}$), obtained by static method, then as per section 7.6 of IS 1893 (part 1): 2002, the response quantities are scaled up by the factor $0.80x\overline{V_B}/V_B$. In our analysis the response spectrum base shear is greater than 80% of the static base shear, hence no up scaling was needed.

Table 71: Storey shear forces by modal response spectrum analysis (NBC)

Storey	Storey shear force (KN)	
	X	Y
6^{TH}	59.2461	82.7253
5^{TH}	241.9747	315.8217
4^{TH}	355.8192	457.4859
3^{RD}	441.6424	566.0529
2^{ND}	519.2539	666.4374
1^{ST}	580.0058	737.0202
GROUND	766.5545	915.1637

Table 72: Summary of Base Shear Forces (NBC)

Direction	Base Shear Force (KN)		
	Static (V_B)	Dynamic ($\overline{V_B}$)	$\overline{V_B}/V_B$
X	787.7957	766.5545	97.30%
Y	842.1263	915.1637	108.67%

6.2.6.6 Storey displacement and drift

The displacement of the center of mass (CM) of each floor level of the building was obtained by both static and dynamic analyses. The drift ratio at each floor levels of the structure was evaluated considering the difference of the deflections (d) in the center of mass (CM) at the top and bottom of the storey divided by the corresponding storey height. The inter-storey drift ratios (dr) at each floor levels were checked against the maximum allowable value for damage limitation requirement, given as 0.010 according to section 9.3 of (NBC 105, 1994). The design lateral deformations shall be taken as the deformations resulting from the linear analysis specified above multiplied by the factor 5/K.

Table 73: Storey Displacement (NBC 105)

Storey	Storey displacement, d in (mm)			
	Static Analysis		Dynamic Analysis	
	X	Y	X	Y
6^{TH}	18.9	25.2	12.7	21
5^{TH}	19.9	20.3	13	16.5
4^{TH}	18.6	17.4	12.2	14.2
3^{RD}	14.6	13.4	9.7	11
2^{ND}	9.5	8.4	6.4	7
1^{ST}	4	3.3	2.8	2.8
GROUND	0.1	0.1	0.1	0.1

Table 74: Storey Drift criteria for damage limitation (NBC 105)

Storey	Storey drift, (d_r),				Storey height (h), m	Drift limit. 0.010
	Static Analysis		Dynamic Analysis			
	X	Y	X	Y		
6^{TH}	0.000695	0.000634	0.000535	0.000572	3.6576	0.010
5^{TH}	0.001142	0.00095	0.000902	0.000905	3.6576	0.010
4^{TH}	0.001859	0.00135	0.001399	0.001234	3.6576	0.010
3^{RD}	0.002458	0.001643	0.001814	0.001479	3.6576	0.010
2^{ND}	0.002705	0.001674	0.002055	0.001553	3.6576	0.010
1^{ST}	0.001842	0.001066	0.001427	0.001034	3.6576	0.010
GROUND	2.50E-05	1.70E-05	2.10E-05	1.60E-05	3.6576	0.010

Here all the storey drifts values are less than the code limit of 0.010 and the inter-storey deflections are also less than 60 mm.

Table 75: Design Storey Displacement (NBC105)

Storey	Multiplier 5/K	Design Storey Displacement			
		Static Analysis		Dynamic Analysis	
		X	Y	X	Y
6TH	5	94.5	126	63.5	105
5TH	5	99.5	101.5	65	82.5
4TH	5	93	87	61	71
3RD	5	73	67	48.5	55
2ND	5	47.5	42	32	35
1ST	5	20	16.5	14	14
GROUND	5	0.5	0.5	0.5	0.5

7.0 COMPARISON OF ANALYSIS AND DESIGN RESULTS OF STUDY BUILDING WITH DIFERENT CODES

7.1 General

As described in analysis chapters, the case study building was analyzed as per three different codes of practice. In order to be more general, the structure was analyzed using same parameters (like same soil condition, same PGA, same building member size, material etc) which was present in the actual site. In this way total of 5 cases (3 without masonry infill and 2 with masonry infill) were analyzed. The outputs of those analyses were tabulated in respective subsection of the analysis chapter.

This chapter presents a detail and graphical comparison and study on analysis output. The output values were compared under different criteria to find out possible varying patterns.

7.2 Comparison based on Spectrum

The shape of elastic spectrum according to Indian Code and Eurocode are almost alike. Nepal Code does not have any elastic spectrum defined.

The design horizontal acceleration spectrum however is different in magnitude for Eurocode, Indian Code and Nepal Code. Indian Code design horizontal acceleration spectrum is almost close to Nepal Code design basic seismic coefficient spectrum. However the Eurocode design spectrum is high in magnitude than both Indian and Nepal Code spectrums.

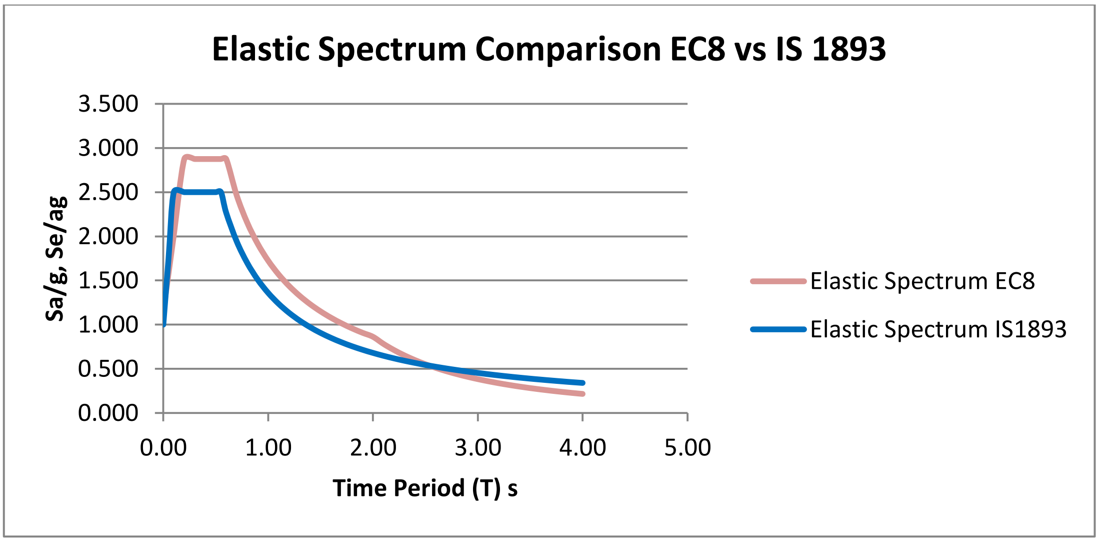


Figure 36: Elastic Spectrum comparison Eurocode vs Indian Code

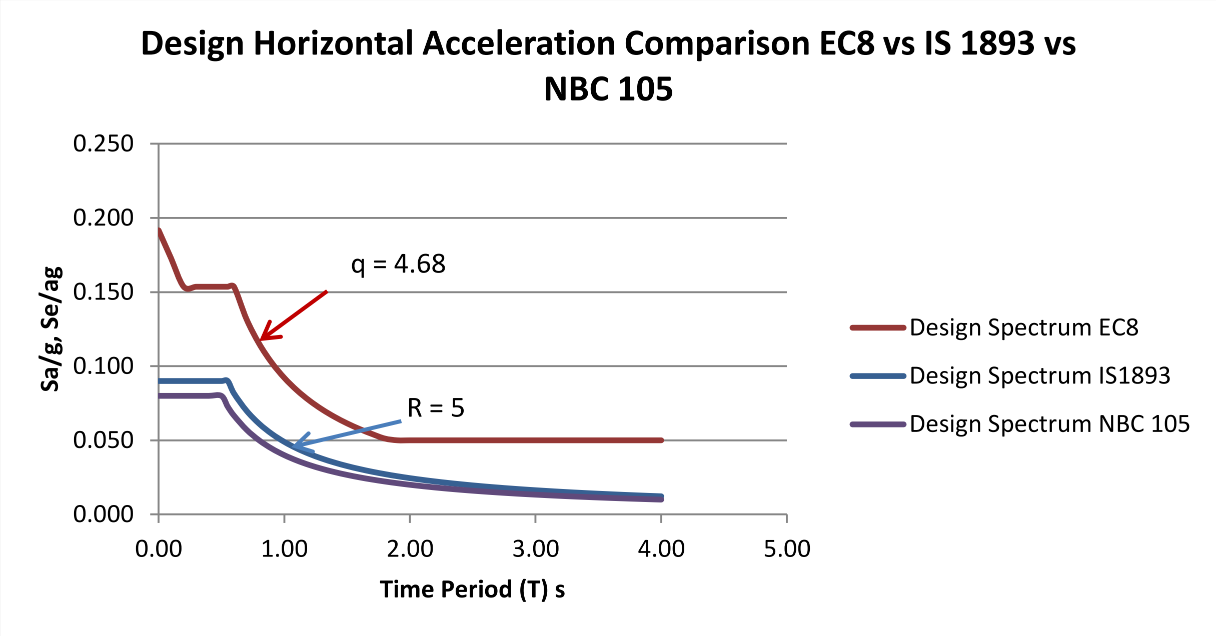


Figure 37: Design horizontal spectrum comparison Eurocode vs Indian Code vs Nepal Code

7.3 Comparison based on design base shear force

The design base shear is an important parameter that can be used as a basis for a comparison of analysis results. The design base shear forces obtained by each analysis case are presented in table 76 below.

Table 76: Design base shear force of the three codes

Model Type	Design base shear force (KN)											
	Indian Code				Eurocode				Nepal Code			
	Static		Dynamic		Static		Dynamic		Static		Dynamic	
	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
Without infill	953.49	1037.16	787.36	962.32	1764.64	1919.39	1669.19	2000.22	787.80	842.13	766.55	915.16
With infill	1065.49	1430.26	957.38	1261.54	1972.47	2653.72	2025.64	2613.19	NA	NA	NA	NA

Based on the values from table 76 we can see that the design base shear as per Eurocode is greater than both Indian code and Nepal Code. The Eurocode base shear force is between 40-50 % higher than from Indian code and Nepal Code. The Indian code and Nepal code base shear are almost equal for dynamic analysis whereas for static Indian code is slightly higher than Nepal code. Also for the same model when masonry infill wall contribution were considered the forces increased by 11.7% in X direction and up to 38% in Y direction *(infill wall direction).*

The design storey forces can also be compared in the graphical representation form as below:

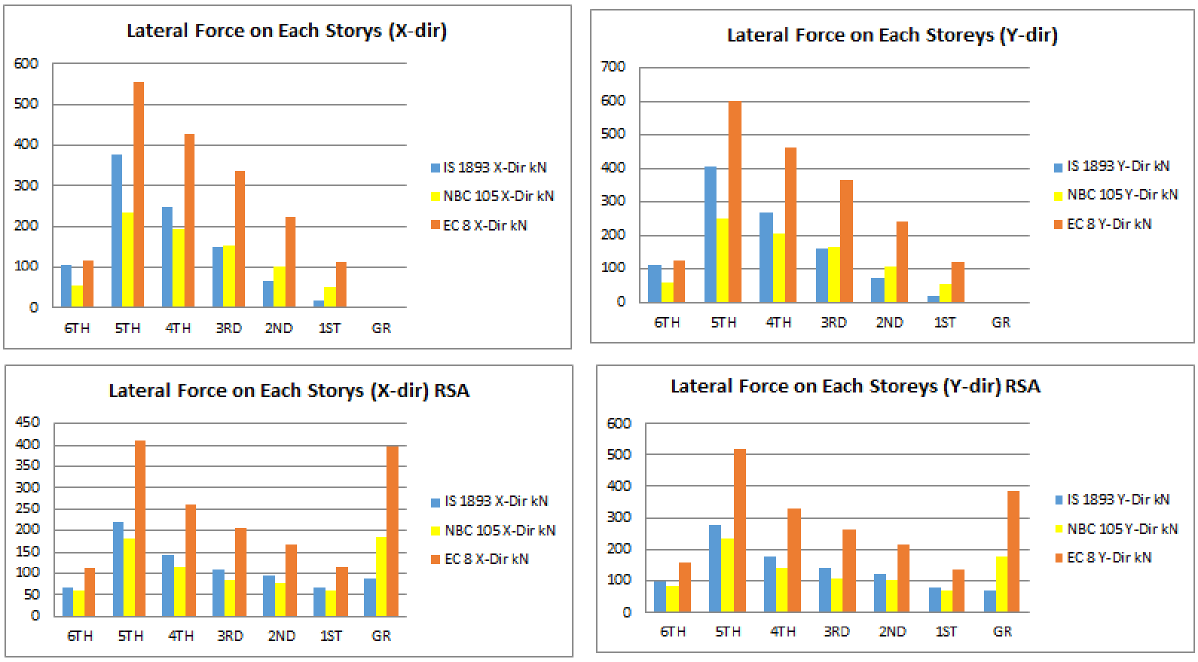


Figure 38: Graphical comparison of design lateral forces (KN) on each storey (Static and Dynamic) without infill

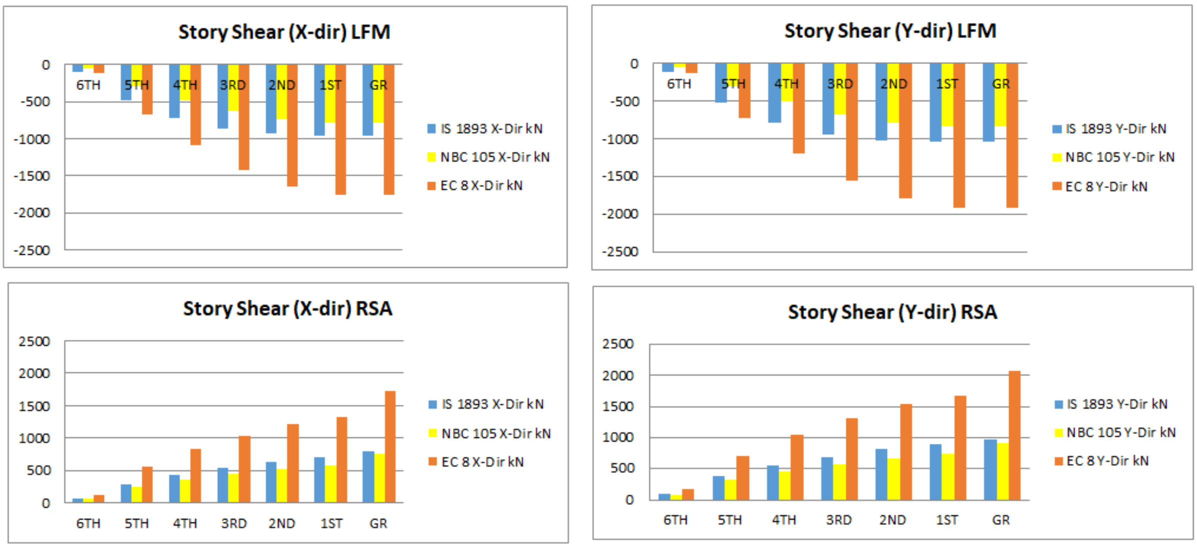


Figure 39: Graphical comparison of story shear forces (KN) (Static and Dynamic) without infill

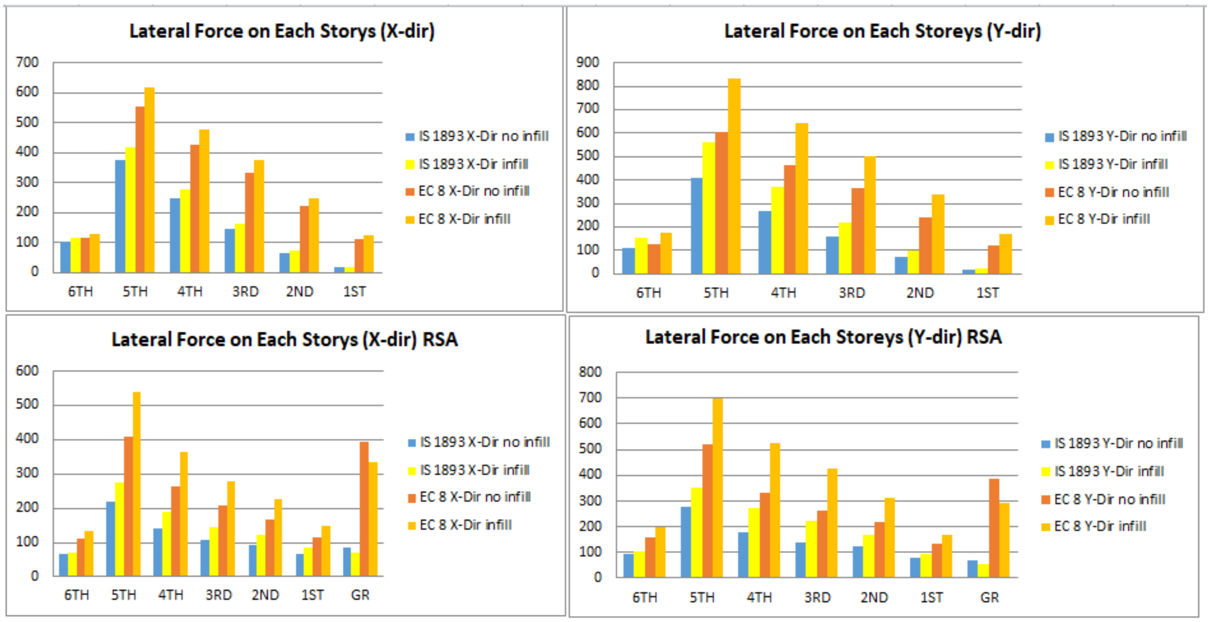


Figure 40: Graphical comparison of design lateral forces (KN) on each storey (Static and Dynamic) with infill

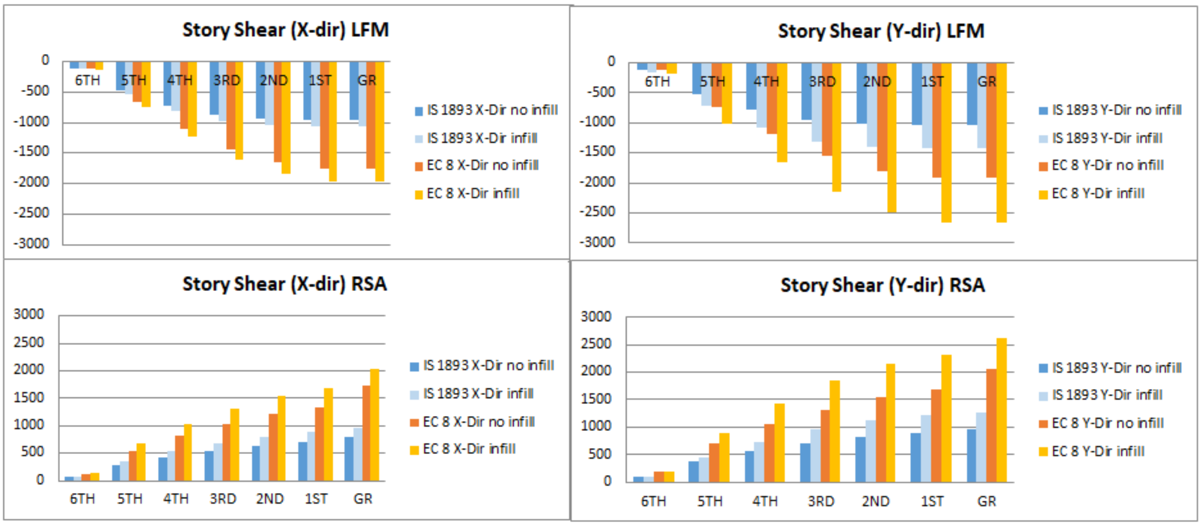


Figure 41: Graphical comparison of story shear forces (KN) (Static and Dynamic) with infill

From the comparative results presented above it can clearly be seen that the Eurocode has given the highest design base shear force values at all 5 occasions. Further, the Indian code has given low base shear values. The reason seems to be that the Indian code recommends to use a reduced zone factor (Z/2) to represent Design Basis Earthquake (DBE) regardless of the site peak ground acceleration, which tends to give lower response quantities consequently (Refer Clause 6.4.2 of (IS 1893-1, 2002). Another reason can also be due to the response reduction factor value adopted ***(5 and 4.68)***. Eurocode considers structure irregularity by 20% reduction in ‘q’ factor whereas Indian code does not consider any ‘R’ factor adjustment.

Also since Nepal Code derives most of its rules from Indian code so it was obvious that its results were close to Indian code.

7.4 Comparison based on storey deflection

The storey deflection is another important parameter to be considered as a basis in comparison of three code based analysis results. The storey deflections obtained in each analysis case are represented in graphical form below.

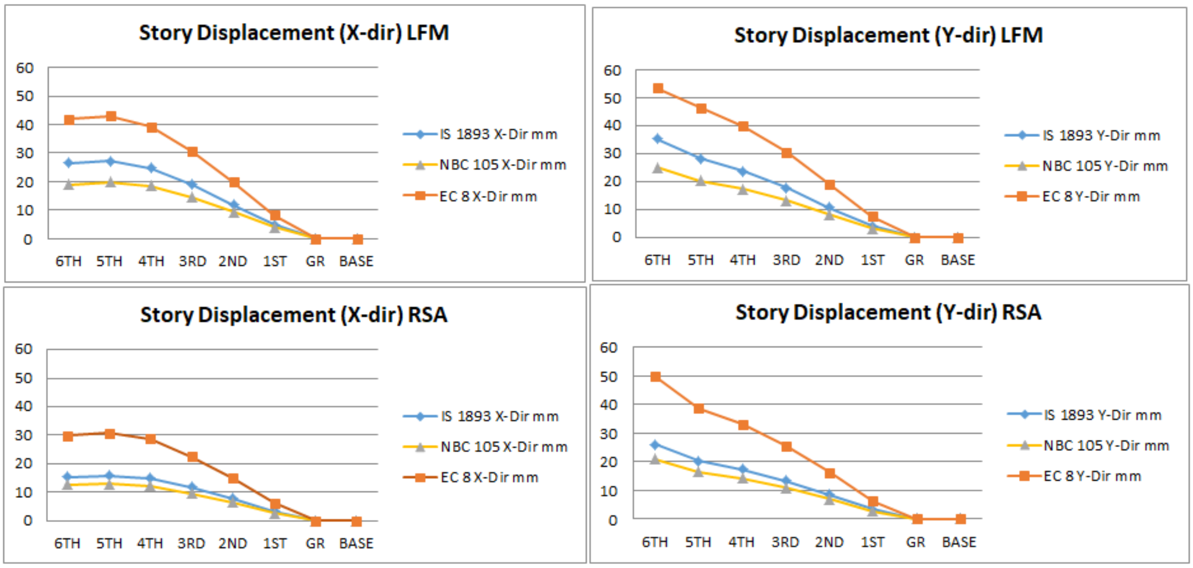


Figure 42: Graphical comparison of story displacement (Static and Dynamic) without infill

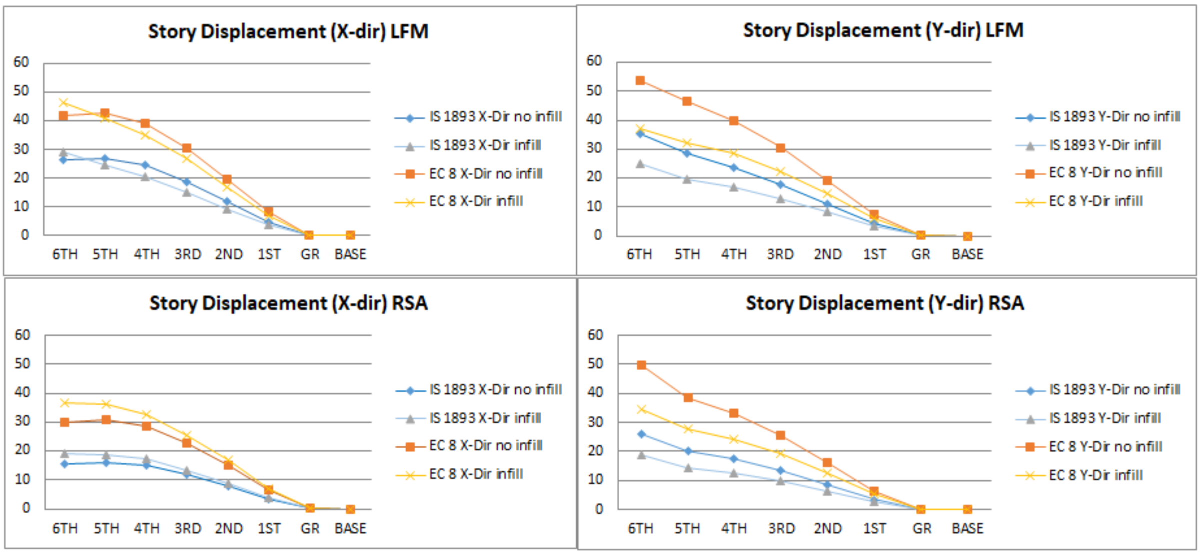


Figure 43: Graphical comparison of story displacement (mm) (Static and Dynamic) with infill

From the graphical comparison above it is evident that the Eurocode gives higher story displacements than Indian code and Nepal code. When the infill masonry effect is considered there is a substantial reduction in story deflection along the infill direction (Y-dir)

7.5 Comparison based on Inter-Story Drift ratio

The inter story drift ratio is an important parameter to be considered in finding out the performance of a structure. The inter-story drift ratios achieved when the structure were analyzed according to different codes of practice are given in graphical way below.

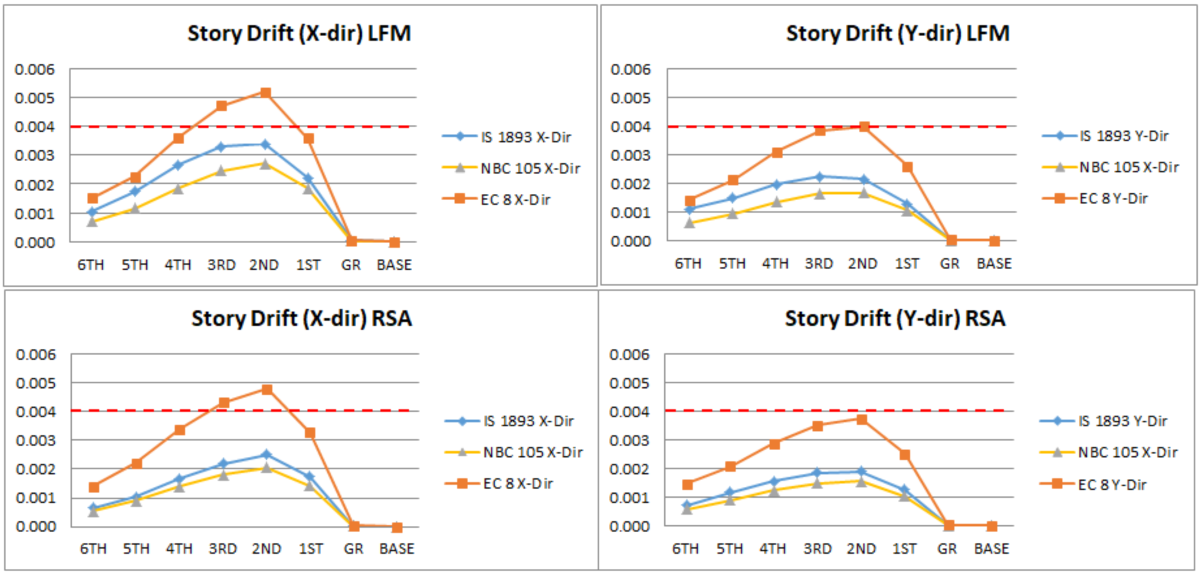


Figure 44: Graphical comparison of inter-story drift (Static and Dynamic) without infill

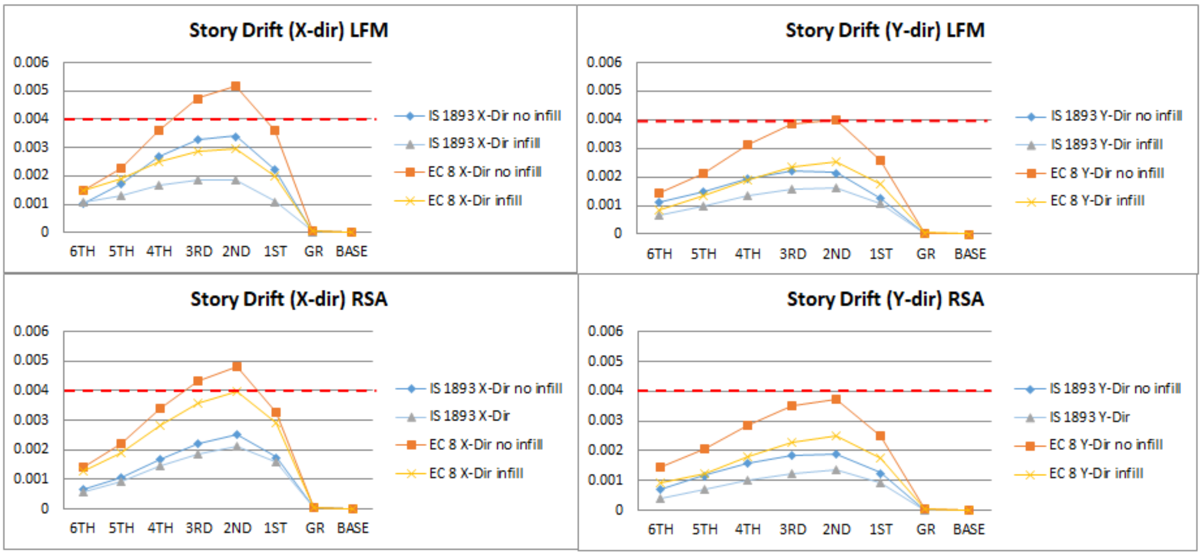


Figure 45: Graphical comparison of inter-story drift (Static and Dynamic) with infill

The distribution of inter-story drift ratio also follows almost the same pattern as story displacement and base shear force of the structures. The Eurocode gave higher drift ratio compared to Indian code and Nepal Code. Also consideration of masonry infill walls in the model as diagonal strut resulted in significant reduced drift ratio in the direction of the infill masonry considered in both Eurocode and Indian Code.

It must be noted that although the drift ratio satisfied the Indian code limit as shown in chapter 4, the drift limit for Eurocode did not satisfy for 2nd, 3rd and 4th floors both with and without infill walls diagonal struts as shown in chapter 5.

7.6 Comparison based on Design of Frame elements

7.6.1 Design Load Combination Considered

Design Load combinations are defined as the various factored combinations of three load cases for which the structure is to be designed. The design loading combinations are obtained by multiplying the characteristic loads by appropriate partial factor of safety. The design load combinations adopted in different seismic codes for member design and verification process are given in tables below.

Table 77: Indian Code Design load combination

S.N	Load combination multiplier				
	DL	LL	EQX	EQY	RSA
1	1.5	-	-	-	-
2	1.5	1.5	-	-	-
3	1.2	1.2	±1.2	-	-
4	1.2	1.2	-	±1.2	-
5	1.5	-	±1.5	-	-
6	1.5	-	-	±1.5	-
7	0.9	-	±1.5	-	-
8	0.9	-	-	±1.5	-
9	1.2	1.2	-	-	1.2
10	1.5	-	-	-	1.5
11	0.9	-	-	-	1.5

Table 78: Eurocode Design Load Combination

S.N	Load combination multiplier					
	DL	LL	Stair Live	EQX	EQY	RSA
1	1.35	-		-	-	-
2	1.35	1.5	1.5	-	-	-
3	1	0.3	0.8	±1.0	±0.3	-
4	1	0.3	0.8	±0.3	±1.0	-
5	1	-		±1.0	±0.3	-
6	1	-		±0.3	±1.0	-
7	1	0.3	0.8	-	-	1
8	1	-		-	-	1

Table 79: Nepal Code Design Load Combination

S.N	Load combination multiplier				
	DL	LL	EQX	EQY	RSA
1	1.5	-	-	-	-
2	1.5	1.5	-	-	-
3	1	1.3	±1.25	-	-
4	1	1.3	-	±1.25	-
5	0.9	-	±1.25	-	-
6	0.9	-	-	±1.25	-
7	1	-	±1.25	-	-
8	1	-	-	±1.25	-
9	1	1.3	-	-	±1.25
10	0.9	-	-	-	±1.25
11	1	-	-	-	±1.25

As it can be seen from the tables above that in seismic design situation, the Indian code has higher load combination multiplier of **1.5 and 1.2** than the Eurocode (**1.0)** and Nepal Code (**1.25**). Due to this combination multiplier the design requirements in Indian code may be higher than the Eurocode despite the base shear force is higher in later code.

The design for components like columns and beams are carried out with the help of ETABS auto design process using respective codes (ETABS, 2016). While designing and selecting reinforcement detailing in the frames the "**Strong Column – Weak Beam**" concept was followed. The table 80 shows the concrete members verification status under different codes and conditions.

Table 80: Concrete member verification checks status under different situations

Member	Total No.	Failure Type	Total failed members Eurocode	%	Total failed members Indian code	%	Remarks
Beam	394	***Shear failure***	6	1.5%	4	1.0%	w/o infill
			4	1.0%	4	1.0%	with infill
Column	155	***Axial comp. capacity in seismic condition <0.55***	3	1.9%	13	8.4%	w/o infill
		P-M2-M3	3	1.9%	1	0.6%	with infill

7.6.2 Column Reinforcement Design

The reinforcement design comparisons for a typical ground floor column using different codes are provided below:

Table 81: Column Maximum Axial Force (P_{Ed}) KN (without infill)

Col Type	IS Code	NBC Code	Eurocode
C1	433	314	416
C2	1955	1447	1417
C3	1854	1467	1342
C4	1549	1278	1212
C5	1494	1183	1110
C6	1854	1301	963

Table 82: Column Maximum Moment (M_{Ed}) (without infill)

Col Type	M_{Ed2}, KNm			Col Type	M_{Ed3}, KNm		
	IS Code	NBC Code	Eurocode		IS Code	NBC Code	Eurocode
C1	25	19	27	C1	19	20	18
C2	104	61	209	C2	374	288	266
C3	131	95	184	C3	309	243	227
C4	156	112	250	C4	302	240	184
C5	153	94	220	C5	287	285	95
C6	40	28	33	C6	149	108	116

Table 83: Column Maximum Shear Force (V_{Ed}) (without infill)

Col Type	V_{Ed2}, KN			Col Type	V_{Ed3}, KN		
	IS Code	NBC Code	Eurocode		IS Code	NBC Code	Eurocode
C1	10	138	9	C1	13	130	12
C2	106	81	103	C2	112	95	89
C3	84	64	87	C3	121	91	105
C4	89	74	77	C4	114	84	108
C5	58	45	37	C5	99	81	79
C6	107	79	75	C6	109	96	74

Table 84: Maximum Column Reinforcement Ground Floor (mm^2) (without infill)

Col Type	Provided	IS Code	NBC Code	Eurocode
C1	1256.64	720	720	900
C2	7147.12	**7140**	5496	5746
C3	6440.26	6118	4869	4714
C4	5890.49	**6014**	4771	4712
C5	4830.20	**5217**	4637	3154
C6	4830.20	3091	2246	3502

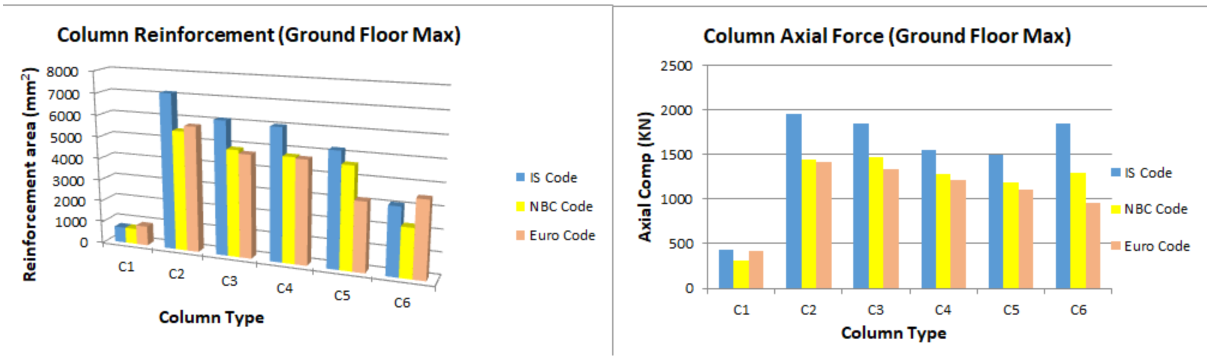


Figure 46: Column Design Graphical Comparison (without infill)

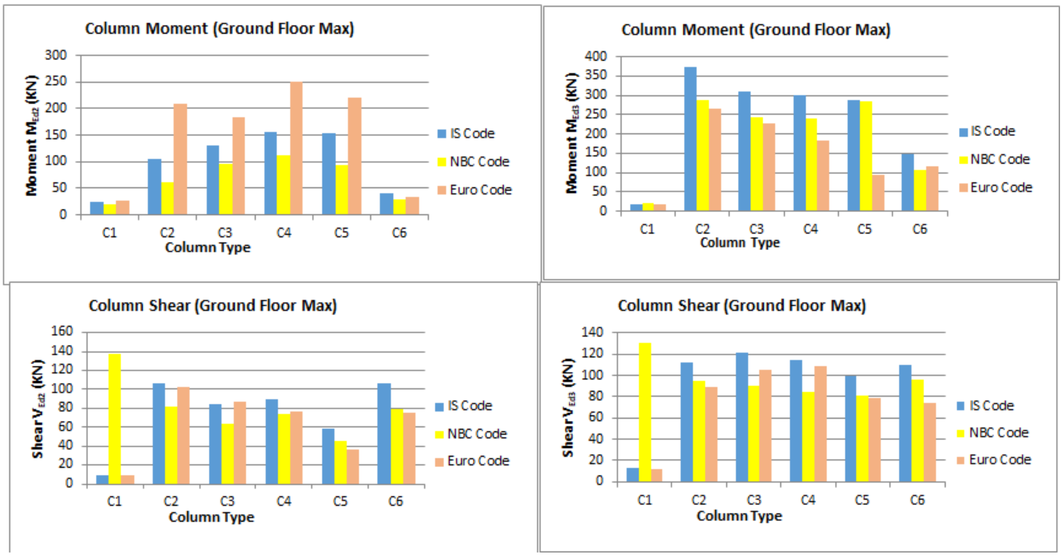


Figure 47: Column Design Forces comparison (without infill)

Table 85: Column Maximum Axial Force (P_{Ed}) KN (with infill)

Col Type	IS w/o infil	IS with infil	EC w/o infill	EC with infill
C1	433	433	416	416
C2	1955	1706	1417	1434
C3	1854	2044	1342	1432
C4	1549	2900	1212	1110
C5	1494	1802	1110	1378
C6	1854	845	963	1922

Table 86: Column Maximum Moment (M_{Ed}) KNm (with infill)

Max Column M_{Ed2}, KNm					Max Column M_{Ed3}, KNm				
Col Type	IS w/o infil	IS with infil	EC w/o infill	EC with infill	Col Type	IS w/o infil	IS with infil	EC w/o infill	EC with infill
C1	25	25	27	27	C1	19	20	18	18
C2	104	55	209	139	C2	374	232	266	244
C3	131	79	184	147	C3	309	200	227	199
C4	156	63	250	146	C4	302	76	184	204
C5	153	39	220	77	C5	287	152	95	178
C6	40	18	33	33	C6	149	108	116	131

Table 87: Column Maximum Shear Force (V_{Ed}) (with infill)

Max Column V_{Ed2}, KN					Max Column V_{Ed3}, KN				
Col Type	IS w/o infil	IS with infil	EC w/o infill	EC with infill	Col Type	IS w/o infil	IS with infil	EC w/o infill	EC with infill
C1	10	10	9	9	C1	13	13	12	12
C2	106	96	103	96	C2	112	105	89	80
C3	84	115	87	78	C3	121	97	105	93
C4	89	31	77	80	C4	114	69	108	87
C5	58	91	37	73	C5	99	96	79	62
C6	107	111	75	77	C6	109	101	74	67

Table 88: Maximum Column Reinforcement Ground Floor (mm^2) (with infill)

Col Type	Provided	IS w/o infil	IS with infil	EC w/o infill	EC with infill
C1	1256.64	720	720	900	900
C2	7147.12	7140	5060	5746	4817
C3	6440.26	6118	4888	4714	4122
C4	5890.49	6014	4465	4712	3858
C5	4830.20	5217	3347	3154	2792
C6	4830.20	3091	3173	3502	3830

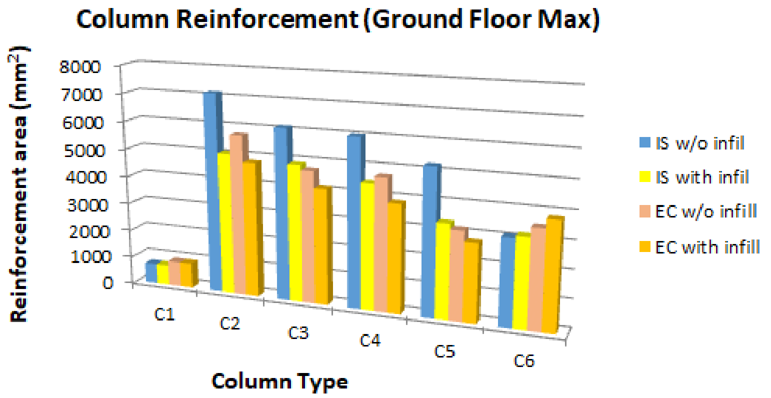


Figure 48: Column Design comparison (with infill)

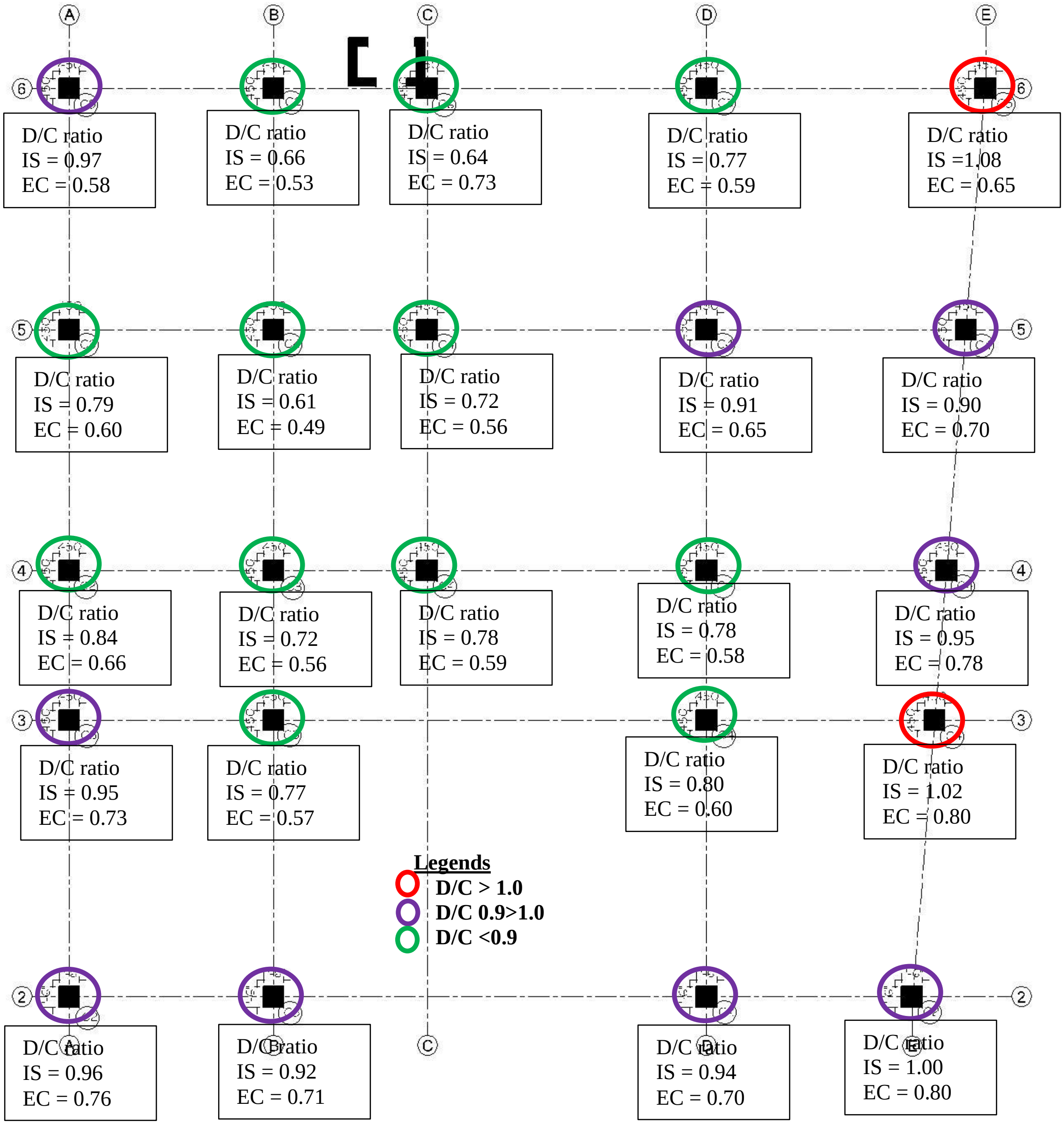


Figure 49: Column reinforcement demand/capacity comparison without infill (At Ground Floor)

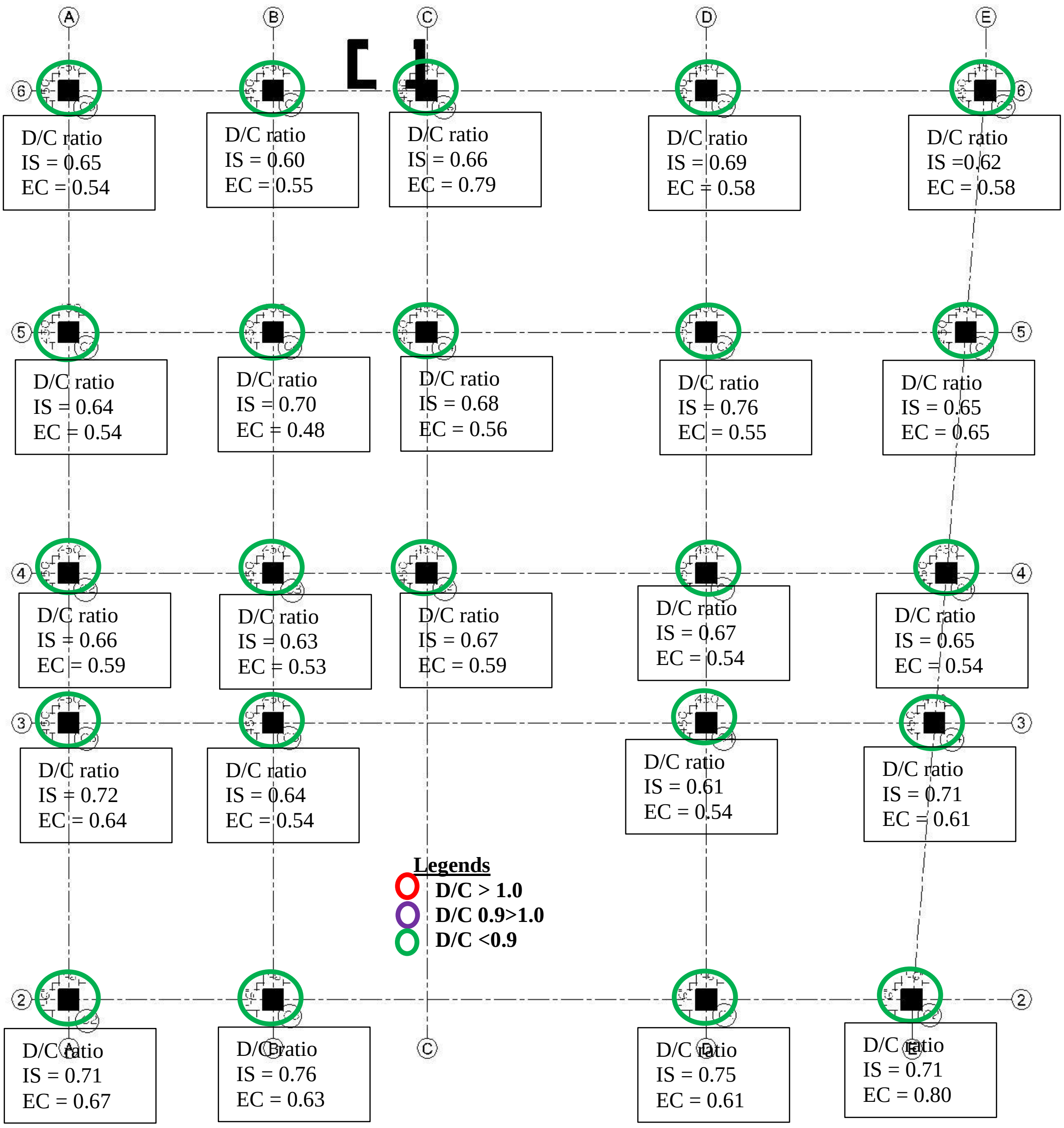


Figure 50: Column reinforcement demand/capacity comparison with infill (At Ground Floor)

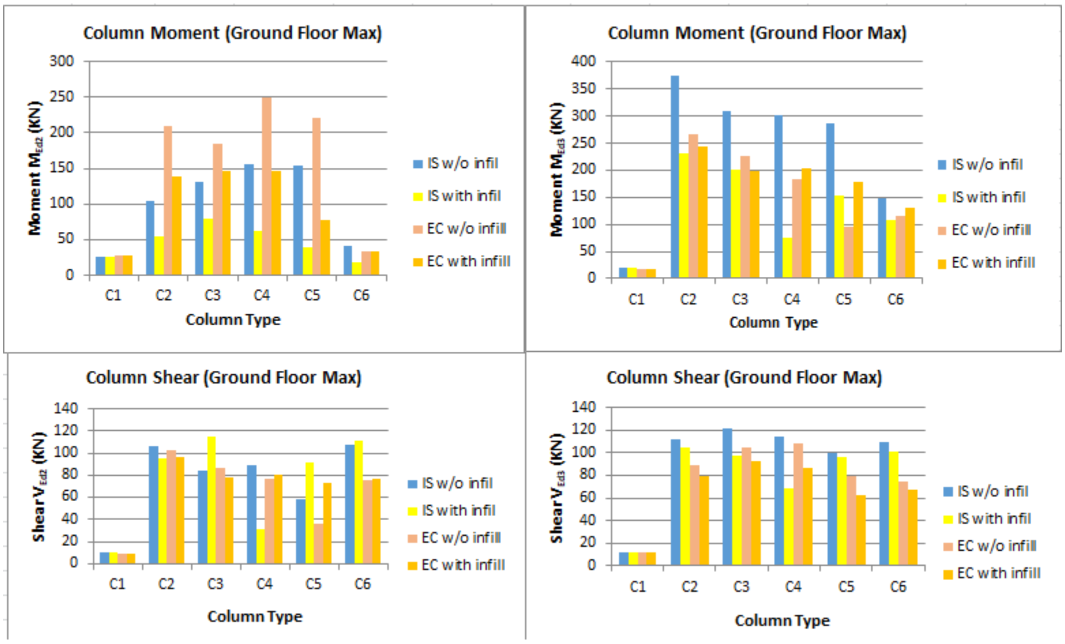


Figure 51: Column Design Forces Comparison (with infill)

7.6.3 Beam Reinforcement Design

The reinforcement design comparisons for the typical second floor beams in two orthogonal axes using different codes are provided below:

Table 89: Maximum Beam Design Forces (without infill)

Beam grid	Max Beam M_{Ed3}, KNm			Max Beam V_{Ed2}, KN		
	IS Code	NBC Code	Eurocode	IS Code	NBC Code	Eurocode
A(2-3)	249	182	234	205	154	142
A(3-4)	217	150	231	266	196	210
A(4-5)	240	167	224	227	163	162
A(5-6)	219	151	213	188	136	137
6(A-B)	155	103	146	116	80	82
6(B-C)	334	216	338	323	229	420
6(C-D)	186	135	165	118	99	82
6(D-E)	151	115	135	106	95	74

Table 90: Maximum Beam Reinforcement Second Floor (mm^2) (without infill)

Beam grid	Provided	IS Code	NBC Code	Eurocode
A(2-3)	1884.96	1545	1167	1510
A(3-4)	1884.96	1635	981	1492
A(4-5)	1884.96	1495	1084	1454
A(5-6)	1884.96	1377	996	1363
6(A-B)	1570.00	1019	621	869
6(B-C)	2415.00	2028	1364	2097
6(C-D)	1344.00	1192	859	999
6(D-E)	1344.00	967	698	774

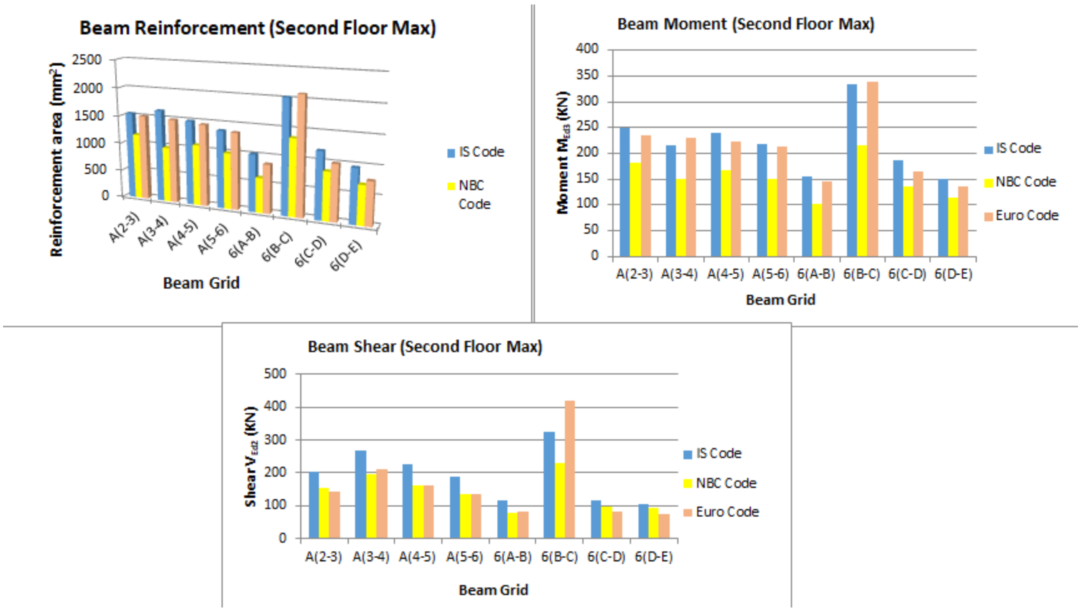


Figure 52: Beam Design Comparison (without infill)

Table 91: Maximum Beam Design Forces (with infill)

Beam grid	Max Beam M_{Ed3} KNm				Max Beam V_{Ed2} KN			
	IS Code w/o infill	IS Code with infill	EC w/o infill	EC with infill	IS Code w/o infill	IS Code with infill	EC w/o infill	EC with infill
A(2-3)	249	210	234	184	205	179	142	130
A(3-4)	217	173	231	173	266	223	210	167
A(4-5)	240	204	224	178	227	215	162	148
A(5-6)	219	179	213	161	188	173	137	118
6(A-B)	155	160	146	160	116	121	82	88
6(B-C)	334	343	338	362	323	332	420	361
6(C-D)	186	190	165	176	118	120	82	86
6(D-E)	151	154	135	146	106	114	74	76

Table 92: Maximum Beam Reinforcement Second Floor (mm^2) (with infill)

Beam grid	Provided	IS Code w/o infill	IS Code with infill	EC w/o infill	EC with infill
A(2-3)	1884.96	1545	1329	1510	1142
A(3-4)	1884.96	1635	1121	1492	1060
A(4-5)	1884.96	1495	1294	1454	1094
A(5-6)	1884.96	1377	1153	1363	973
6(A-B)	1570.00	1019	1045	869	966
6(B-C)	2415.00	2028	2079	2097	2226
6(C-D)	1344.00	1192	1213	999	1082
6(D-E)	1344.00	967	996	774	840

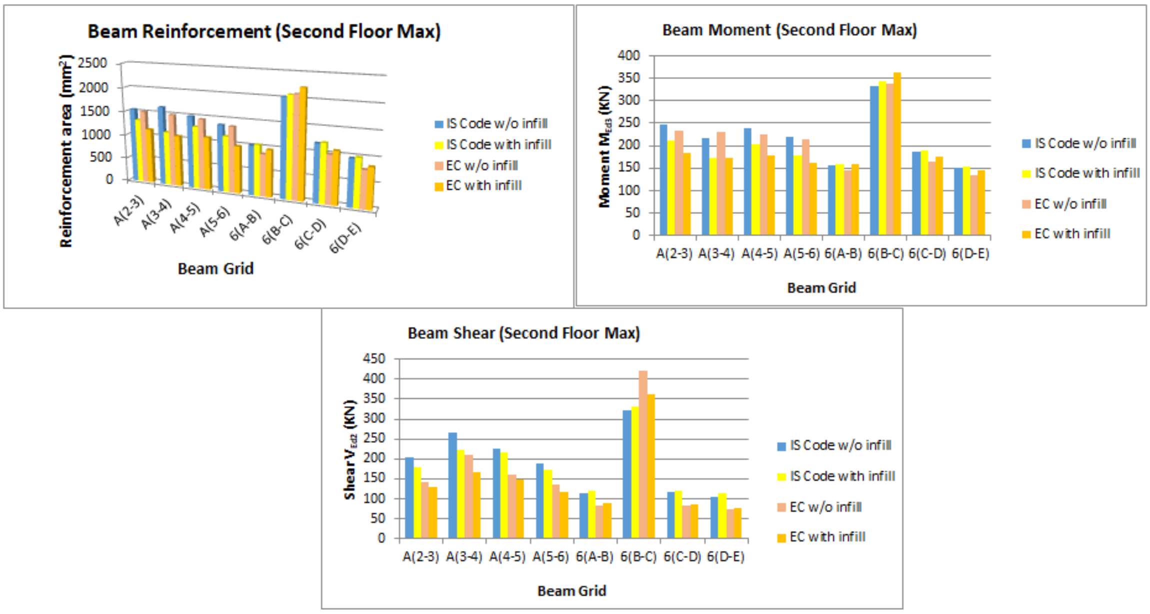


Figure 53: Beam Design Comparison (with infill)

From the above design comparisons of column and beam reinforcement it can be seen that the reinforcement demand for Indian code method is higher in columns even though lateral force is higher in Eurocode. This is probably due to the higher scale factor considered in the design load combination in Indian code than in Eurocode. The demand/ capacity for 11 columns (column type C2, C4 and C5) do not satisfy at ground floor in Indian code method (without infill) i.e. d/c > 0.9.

Similarly when the masonry infill wall was considered for lateral load resistance significant reduction in reinforcement demand for both Indian code and Eurocode can be observed. The demand in column is reduced by up to **35%** for Indian Code and by **18%** for Eurocode when the contribution of infill wall is considered. In their research in Seismic response assessment of a real masonry-infilled RC building by (Mazzolani, Fiorino, & Della Corte, 2009), they also found similar contribution of masonry walls to the strength of RC building. However they quote in their research that such results cannot be generalized to every building type and situation as the contribution depends upon the number of lateral load resisting elements like columns and shear walls in the building.

On the other hand in case of beam the reinforcement demand is almost similar in both codes. Although there is reduction in demand for long span beams when infill walls are considered, the short span beams tend to be critical due to increase in shear force.

The above results comparison suggests that Eurocode has higher lateral forces applied to the building than Indian and Nepal code under same circumstances. The damage limitation demands are also higher for Eurocode than the two codes. The existing building which was designed according to linear static method of Indian code when checked against these 5 cases it can be seen that the building performance do not satisfy for Eurocode drift limitations and Indian code column reinforcement demands for without infill walls case.

The building was earlier designed using approximate fundamental time period ***(T_1 = 0.075$H^{3/4}$ = 0.76 sec) (lower than actual 1.395 sec)*** resulting higher seismic forces applied in the initial structure design.

The above comparison shows that it is not reasonable to directly compare the two codes with just the linear analysis results. It can be seen that the principles and assumptions considered for seismic design are not same for all codes. For instance the use of reduction factors **(R / q)** and use of scale factors in load combination are different in all three codes. Also the confidence factor considered by the formulators of code also play vital role in effectiveness of code. So it is not correct to generalize and interpret any code as faulty.

Thus, to know the actual performance level of the given building and efficiency of any code either a much advance analysis technique like a Non-Linear Static (Pushover) analysis is required or the linear comparison should be carried out with more sample buildings with different scenarios like site conditions, different storeys and geometrical configuration etc.

8.0 NON-LINEAR STATIC ANALYSIS (PUSHOVER ANALYSIS)

8.1 General

Although it is clear from the previous comparative linear static and dynamic analyses for different codes that the existing building structure have some members not satisfying the code requirements in terms of drift ratio and reinforcement demand, a pushover analysis determines the actual performance level of the structure and demonstrates the trend of the failure at the event of earthquake.

In this chapter we discuss in detail the steps to perform a nonlinear static pushover analysis with N2 method on the existing reinforced concrete building to check its performance. Again for this analysis we will consider the given study building with and without infill wall consideration. The results will then be checked and verified in order to decide whether the building should go under retrofitting intervention or not. The CSI software (ETABS, 2016) was used to perform the pushover analysis.

8.2 Modeling of the structure

The same three dimensional spatial model used before in linear analysis was again used for the pushover analysis. All the basic characteristics and assumptions considered were also same. One very important point that should be considered for nonlinear analysis is that the structural components must be defined as close to reality as possible with the exact material strength and reinforcement details in the columns and beams. This allows the analysis to correctly predict the nonlinear behavior of the structural components and get proper numerical convergence giving us better results.

In ETABS software, the section designer tool is used to model the column sections with the same reinforcement details as provided in the site.

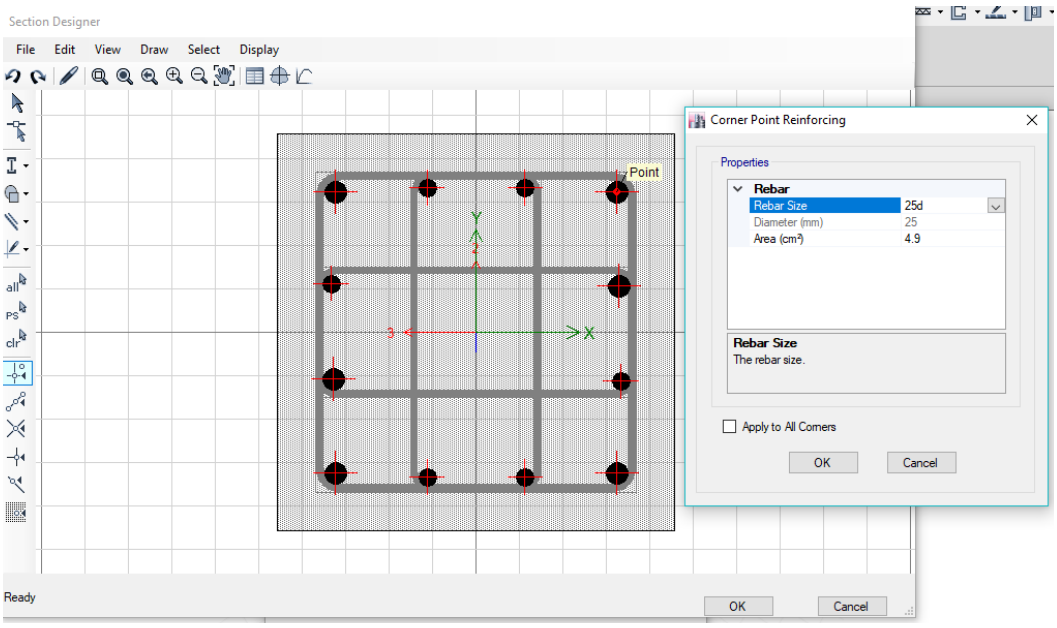


Figure 54: ETABS Section Designer to define actual column section

8.3 Plastic Hinges

The following types of plastic hinges were considered in the ETABS modeling.

Beams: plastic hinges type M3

Columns: plastic hinges type P-M2-M3

ETABS auto hinge property is used to assign hinges to the existing building frame. The hinges were assigned according to ATC-40 and FEMA 356 parameters.

Table 6-7 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Reinforced Concrete Beams

Conditions			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d\sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01

Figure 55: Modeling parameters for Reinforced Concrete Beams (FEMA 356, 2000)

Table 6-8 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Reinforced Concrete Columns

Conditions			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d\sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008

Figure 56: Modeling parameters for Reinforced Concrete Columns (FEMA 356, 2000)

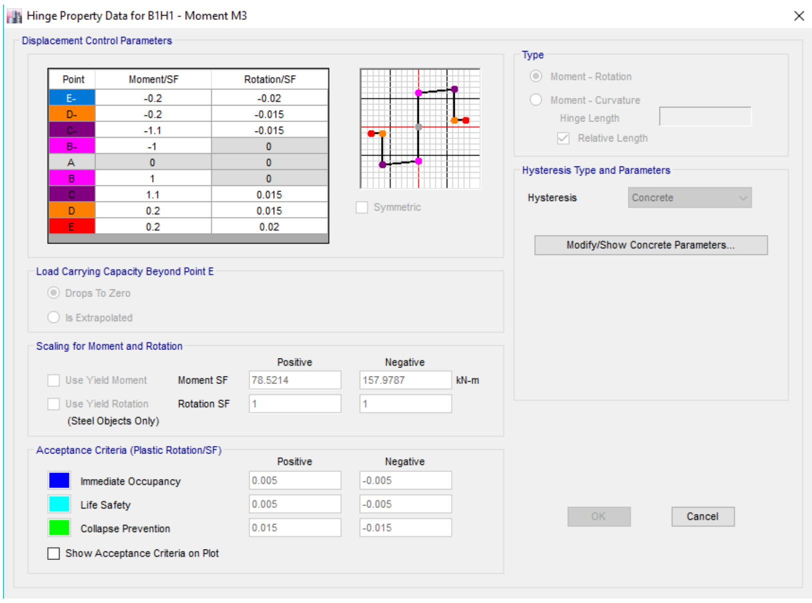


Figure 57: ETABS Hinge property data for Beam

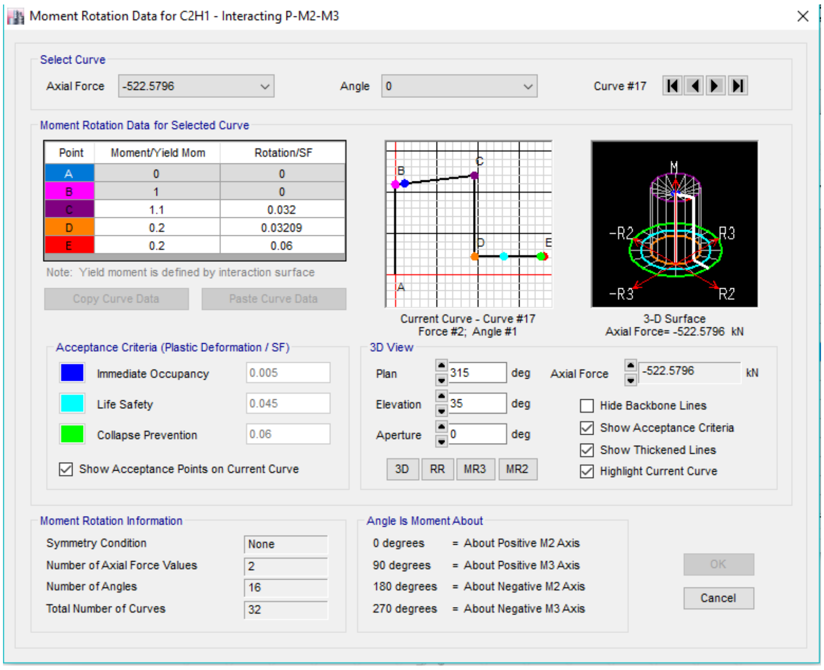


Figure 58: ETABS Hinge property data for Columns

Figure 59: Nonlinear Hinge Assignments in ETABS

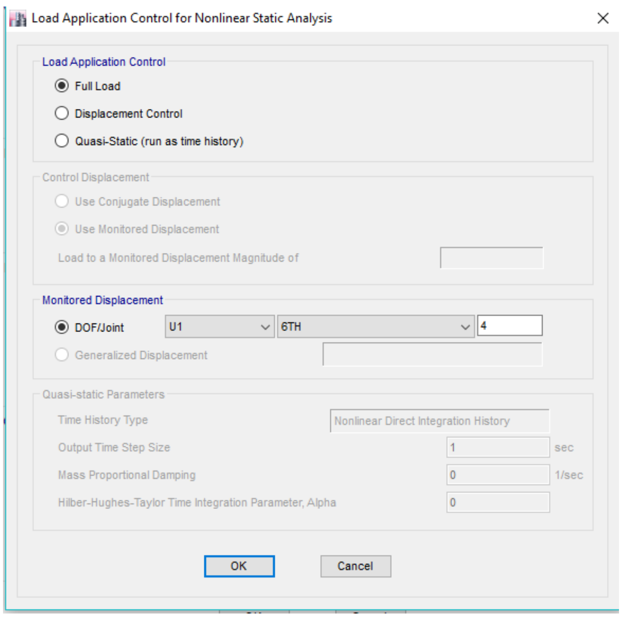
8.4 Pushover Analysis Steps in ETABS

The pushover method using N2 Method in ETABS is given in following steps.

STEP 1: Defining of Nonlinear load cases

Gravity Loading: Force Control

Pushover Loading: Displacement Control



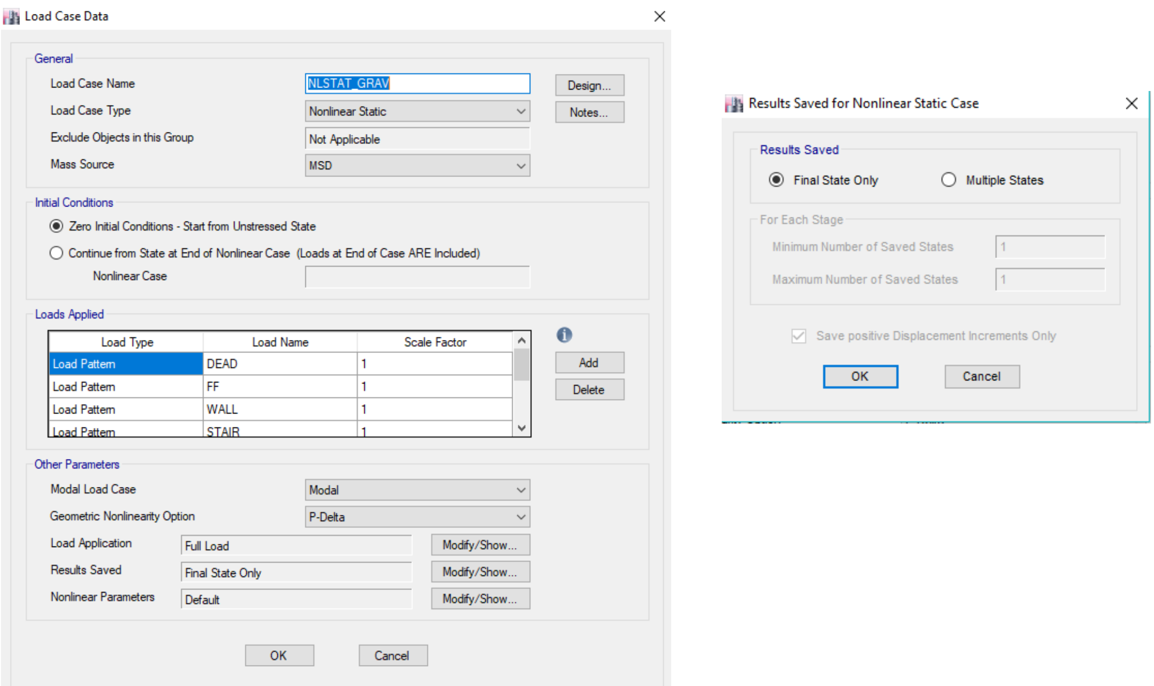


Figure 60: Gravitational Nonlinear Load Case

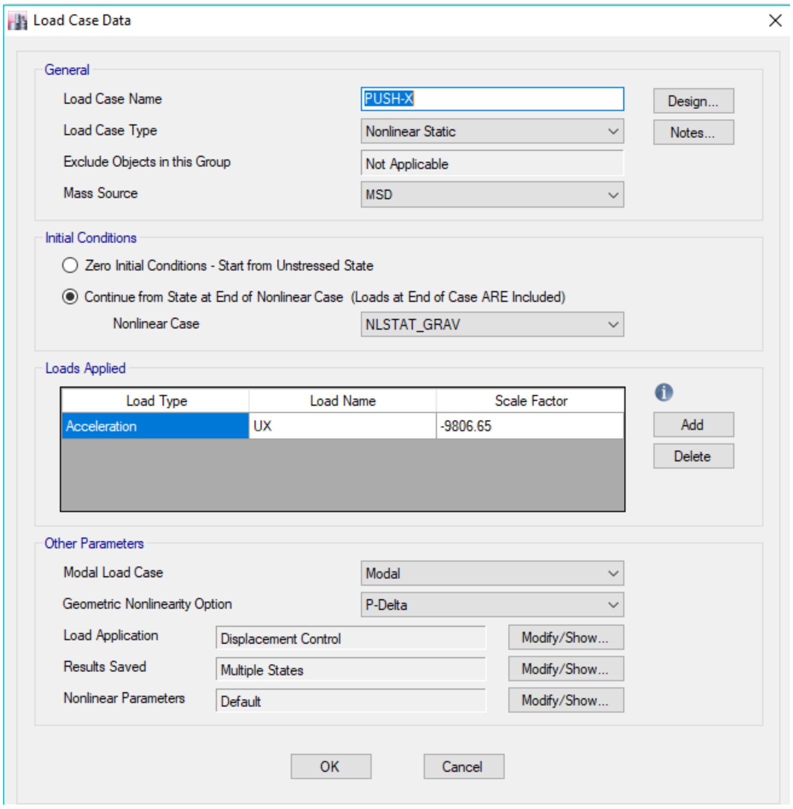


Figure 61: Pushover Load Case

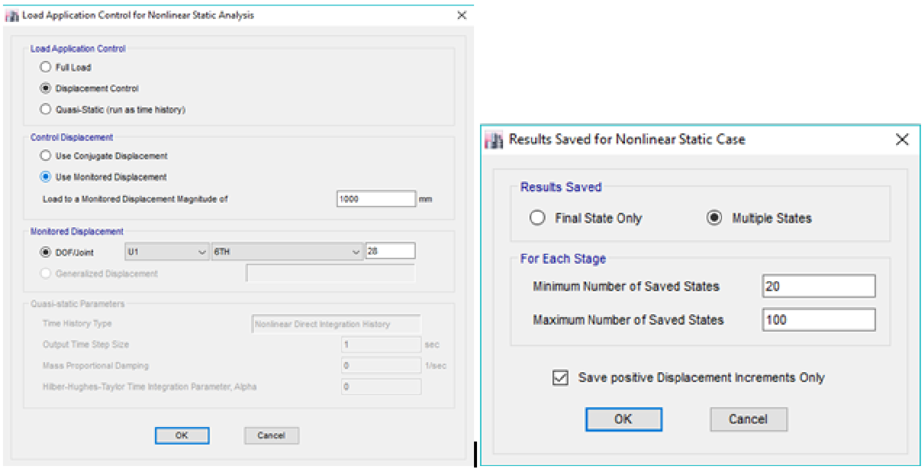


Figure 62: Deformation Control Steps

STEP 2: Run the analysis with initial arbitrary monitored displacement (eg: 1000 mm)

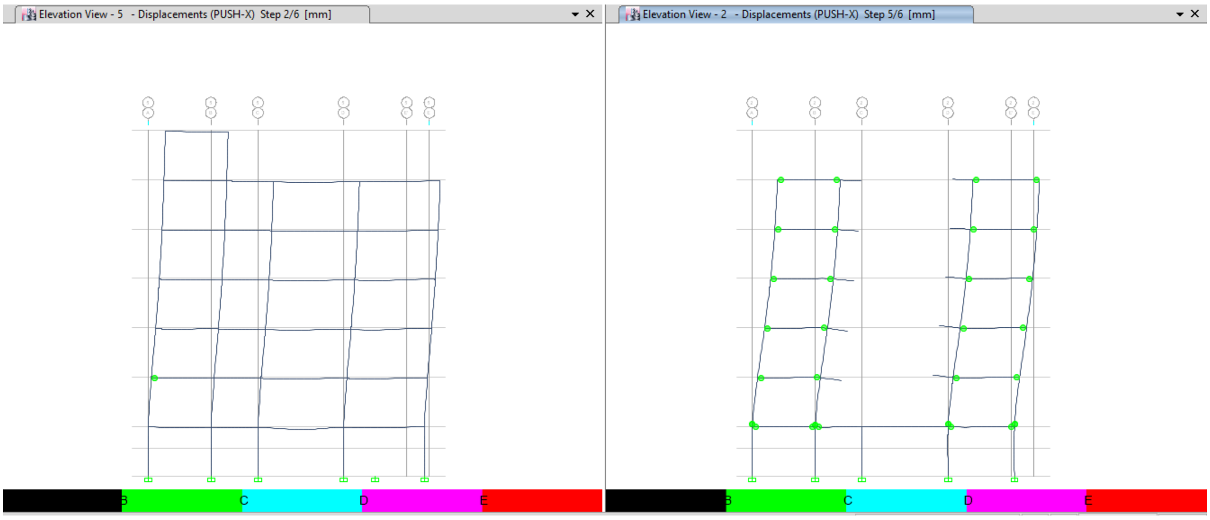


Figure 63: Plastic redundancy (First hinge and mechanism formation)

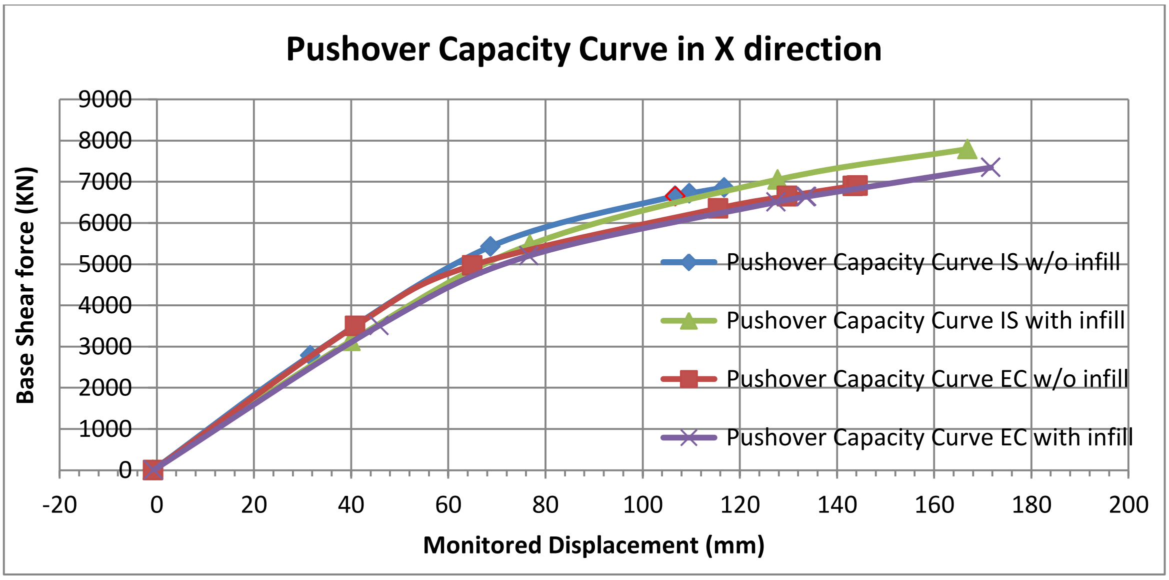


Figure 64: Force-Displacement curve in X direction

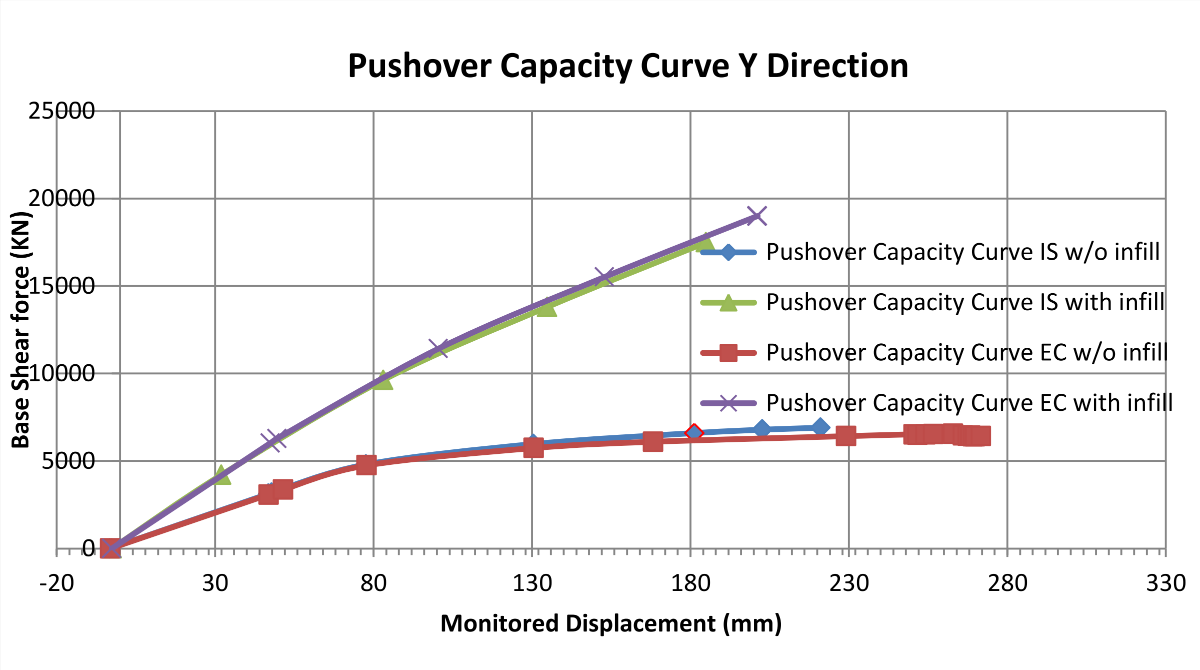


Figure 65: Force-Displacement curve in Y direction

Here the pushover force-displacement relationship is regardless of the code followed. That is the pushover curves are alike for Eurocode and Indian Code. Since N2 method is adopted to perform displacement coefficient approach of pushover analysis, we will consider only Eurocode based model for detail calculation.

STEP 3: Determination of Target Displacement (EN1998-1:, 2004)

STEP 3-1: MDOF to SDOF

The MDOF model is transformed in to an equivalent SDOF model by dividing the force – displacement (F_b-d_n curve) relation with a transformation factor Γ (EN1998-1:, 2004). It is given by,

$$\Gamma = \frac{m^*}{\sum m_i \phi_i^2}, \; m^* = \sum m_i \phi_i$$

Where, m_i is the mass in the i-th storey

Φ_i is normalized displacement

The force F* and displacement d* of the equivalent SDOF system are computed as:

$F^* = F_b/\Gamma$ and $d^* = d_n/\Gamma$.

Where F_b and d_n are, respectively, the base shear force and the control node displacement of the Multi Degree of Freedom (MDOF) system (EN1998-1:, 2004).

Table 93: Calculation of Transformation Factor Γ (without infill and with infill)

Without infill – X-Direction

Storey	mi (t)	Φi	mi* Φi		mi*Φi^2
6	91.09274	1	91.0927		91.0927
5	527.65554	1.28105	675.951		865.924
4	508.44995	1.23529	628.085		775.87
3	531.57214	0.97386	517.675		504.141
2	531.57214	0.63399	337.01		213.66
1	531.57214	0.26797	142.447		38.1722
G					
		m^*=	2392.26	$\Sigma m^* \Phi i^2$	2488.86
			Γ	0.96	

Without infill – Y-Direction

Storey	mi (t)	Φi	mi* Φi		mi*Φi^2
6	91.09274	1	91.0927		91.0927
5	527.65554	0.95088	501.736		477.089
4	508.44995	0.82105	417.464		342.76
3	531.57214	0.63158	335.73		212.04
2	531.57214	0.39649	210.764		83.566
1	531.57214	0.15439	82.0673		12.67
G	701.08934	0.00351	2.45996		0.00863
		m^*=	1641.31	$\Sigma m^* \Phi i^2$	1219.23
			Γ	1.35	

With infill – X-Direction

Storey	mi (t)	Φi	mi* Φi		mi*Φi^2
6	91.09274	1	91.0927		91.0927
5	527.65554	0.98851	521.591		515.595
4	508.44995	0.88123	448.059		394.842
3	531.57214	0.68199	362.528		247.241
2	531.57214	0.44061	234.218		103.199
1	531.57214	0.18391	97.7604		17.9789
G	701.08934	0.00383	2.68617		0.01029
		m^*=	1757.93	$\Sigma m^* \Phi i^2$	1369.96
			Γ	1.28	

With infill – Y-Direction

Storey	mi (t)	Φi	mi* Φi		mi*Φi^2
6	91.09274	1	91.0927		91.0927
5	527.65554	0.84365	445.156		375.555
4	508.44995	0.74593	379.267		282.906
3	531.57214	0.58958	313.402		184.775
2	531.57214	0.38762	206.049		79.8692
1	531.57214	0.15961	84.8438		13.5418
G	701.08934	0.00326	2.28368		0.00744
		m^*=	1522.09	$\Sigma m^* \Phi i^2$	1027.75
			Γ	1.48	

<u>STEP 3-2: Determination of the idealized force-displacement relationship</u>

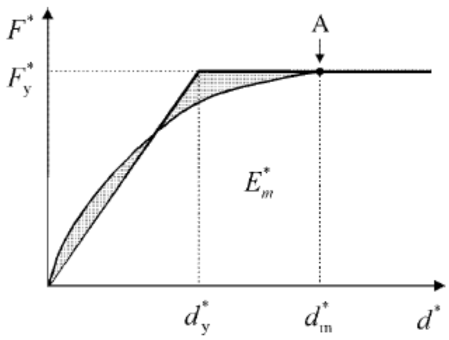


Figure 66: Bilinear approximation of the F*-d* curve

The yield displacement of the idealized SDOF system d*y is given by:

$$d_y^* = 2\left(d_m^* - \frac{E_m^*}{F_y^*}\right)$$

Where,

F^*_y = Force at the plastic mechanism

d^*_m = displacement at plastic mechanism

E^*_m = area under the F*-D*curve corresponding to the formation of plastic mechanism.

The bilinear approximation is done such that the shaded areas in Fig. 66 are equal.

<u>STEP 3-3: Determination of the period of the idealized equivalent SDOF system</u>

The period T* of the idealized equivalent SDOF system is determined by (EN1998-1:, 2004):

$$T^* = 2\pi\sqrt{\frac{m^* d_y^*}{F_y^*}}$$

<u>STEP 3-4: Determination of target displacement for the equivalent SDOF system</u>

The target displacement of the structure with period T^* and unlimited elastic behavior is given by (EN1998-1:, 2004):

$$d_{et}^* = S_e(T^*)\left[\frac{T^*}{2\pi}\right]^2$$

Where, $S_e(T^*)$ is the elastic acceleration response spectrum at the period T^*.

For the determination of target displacement d_t^* for structures in the short-period range and for structures in the medium and long period ranges different expressions should be used as indicated in annex B.5 of (EN1998-1:, 2004).

STEP 3-5: Determination of target displacement for the MDOF system

The target displacement of the MDOF system is given by (EN1998-1:, 2004):

$d_t = \Gamma d_t^*$

The target displacement corresponds to the control node (top floor node 4 in this case). The pushover calculations for 2 cases (model without infill and with infill) in X and Y directions separately can be shown in tables below.

Table 94: Pushover calculations X-Direction without infill

Pushover Capacity Curve for SDOF EC w/o infill (PUSH X)						Γ	0.96									
LoadCase	Step	Displacement	D*	Fb*	Em*	BaseForce	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
Text	Unitless	mm	mm	KN	KN-mm	KN										
Push-Modal	0	-1	-1	0		0	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	1	41	42	3640	78668	3499	1076	6	0	0	0	1082	0	0	0	1082
Push-Modal	2	65	68	5172	110540	4971	978	104	0	0	0	1082	0	0	0	1082
Push-Modal	3	116	120	**6612**	310304	6355	885	197	0	0	0	1018	0	64	0	1082
Push-Modal	4	130	135	6920		6651	863	219	0	0	0	996	0	86	0	1082
Push-Modal	5	130	135	6919		6650	863	219	0	0	0	996	0	86	0	1082
Push-Modal	6	130	135	6922		6653	863	219	0	0	0	996	0	86	0	1082
Push-Modal	7	143	149	7179		6900	845	237	0	0	0	976	0	104	2	1082
Push-Modal	8	144	150	7187		6908	845	237	0	0	0	975	0	105	2	1082
Push-Modal	9	144	150	7189		6910	845	235	2	0	0	974	0	104	4	1082
					499511.61											

Bilinearized SDOF

d*y	F*y
mm	KN
0	0
89	6612
121	6612

Target Displacement Finding — From Elastic Spectrum

m*	D_y^*	F_y^*	T*	$S_e(T^*)$	D^*_{et}	D_t
Ton	mm	KN	sec	m/s^2	mm	mm
2392.261	89	6612	1.13	3.74	121	116

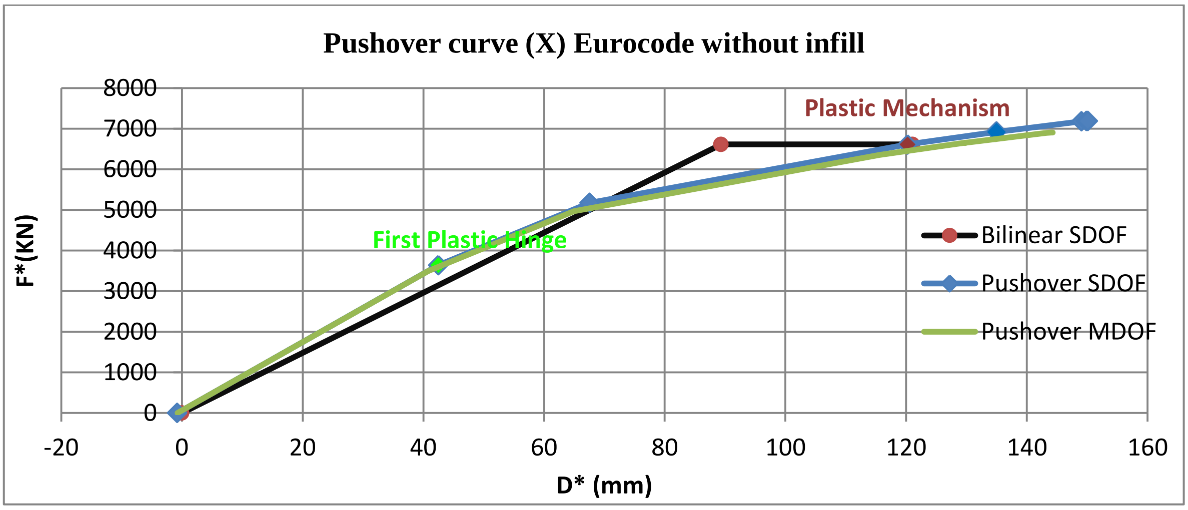


Figure 67: Bilinearized curve X-direction (without infill)

Table 95: Pushover calculations X-direction with infill

Pushover Capacity Curve for SDOF EC with infill (PUSH X)						Γ	1.28									
LoadCase	Step	Displacement	D*	Fb*	Em*	BaseForce	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
Text	Unitless	mm	mm	KN	KN-mm	KN										
Push-Modal	0	-1	-1	0		0	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	1	46	36	2736	49821	3511	1070	4	0	0	0	1074	0	0	0	1074
Push-Modal	2	77	60	4055	81499	5203	966	108	0	0	0	1074	0	0	0	1074
Push-Modal	3	128	99	5073	181039	6510	899	175	0	0	0	1032	0	42	0	1074
Push-Modal	4	134	104	5175	24359	6641	894	180	0	0	0	1015	0	59	0	1074
Push-Modal	5	134	104	5172		6636	894	180	0	0	0	1015	0	59	0	1074
Push-Modal	6	134	104	5177		6643	894	180	0	0	0	1015	0	59	0	1074
Push-Modal	7	172	134	5724		7346	866	208	0	0	0	962	0	112	0	1074
Push-Modal	8	172	134	5724		7346	866	208	0	0	0	962	0	112	0	1074
					336718.71											

Bilinearized SDOF

d*y	F*y
mm	KN
0	0
78	5175
110	5175

Target Displacement Finding

From Elastic Spectrum

m*	D_y*	F_y*	T*	$S_e(T^*)$	D^*_{et}	D_t
Ton	*mm*	*KN*	*sec*	*m/s²*	*mm*	*mm*
1757.935	78	5175	1.02	4.13	110	141

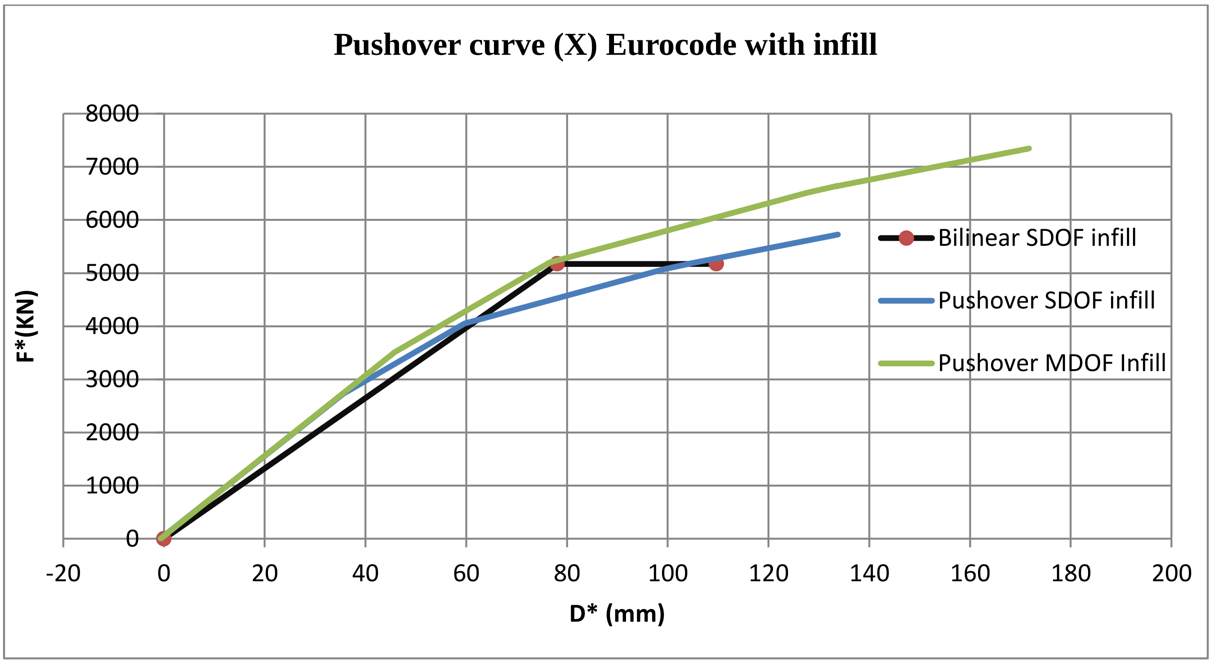


Figure 68: Bilinearized curve X-direction (with infill)

Table 96: Pushover calculations Y-direction (without infill)

Pushover Capacity Curve for SDOF EC w/o infill (PUSH Y)					Γ	1.35										
LoadCase	Step	Displacement	D*	Fb*	Em*	BaseForce	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
Text	Unitless	mm	mm	KN	KN-mm	KN										
Push-Modal	0	-3	-2	0		0	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	1	47	35	2293	42578	3086	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	2	51	38	2505	8249	3372	1081	1	0	0	0	1082	0	0	0	1082
Push-Modal	3	78	58	3530	58996	4752	1010	72	0	0	0	1082	0	0	0	1082
Push-Modal	4	131	97	4266	152792	5742	916	166	0	0	0	1045	0	37	0	1082
Push-Modal	5	168	125	4531	123081	6100	870	212	0	0	0	969	0	113	0	1082
Push-Modal	6	229	170	4770	210507	6422	854	228	0	0	0	904	0	178	0	1082
Push-Modal	7	250	186	4848	75841	6526	849	232	1	0	0	891	0	190	1	1082
Push-Modal	8	252	187	4835		6508	849	232	0	1	0	891	0	190	1	1082
Push-Modal	9	254	189	4845		6522	849	232	0	1	0	890	0	191	1	1082
Push-Modal	10	257	191	4850		6529	849	232	0	0	1	886	2	191	3	1082
Push-Modal	11	263	195	4874		6561	849	232	0	0	1	884	2	193	3	1082
Push-Modal	12	266	198	4795		6455	849	227	0	3	3	884	2	188	8	1082
Push-Modal	13	268	199	4781		6437	849	226	0	3	4	884	2	187	9	1082
Push-Modal	14	268	199	4785		6442	849	226	0	3	4	884	2	187	9	1082
Push-Modal	15	269	200	4764		6414	849	225	0	4	4	884	2	186	10	1082
Push-Modal	16	270	200	4759		6407	849	225	0	2	6	884	2	186	10	1082
Push-Modal	17	272	202	4768		6418	849	223	2	2	6	884	2	184	12	1082
					672044.08											

Bilinearized SDOF

d*y	F*y
mm	KN
0	0
95	4848
186	4848

Target Displacement Finding

From Elastic Spectrum

m*	D_y*	F_y*	T*	$S_e(T^*)$	D^*_{et}	D_t
Ton	*mm*	*KN*	*sec*	*m/s^2*	*mm*	*mm*
1641.31	95	4848	1.126	3.76	121	162

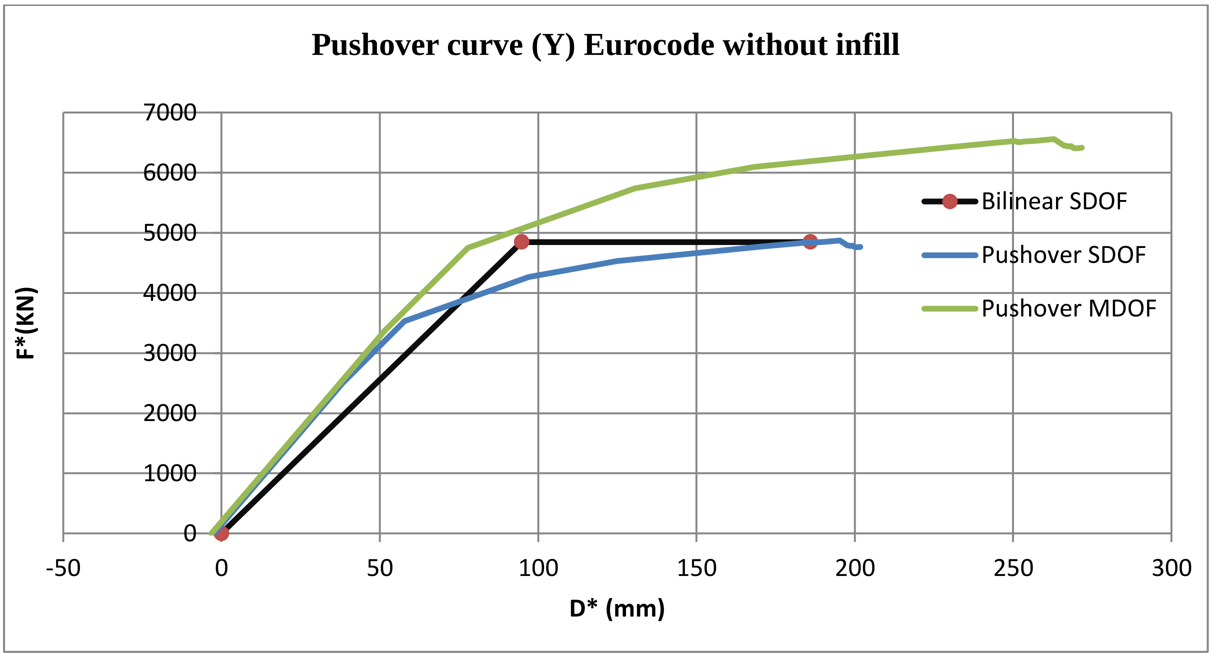


Figure 69: Bilinearized curve Y-direction (without infill)

Table 97: Pushover calculations Y-direction (with infill)

Pushover Capacity Curve for SDOF EC with infill (PUSH Y)						Γ	1.48									
LoadCase	Step	Displacement	D*	Fb*	Em*	BaseForce	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
Text	Unitless	mm	mm	KN	KN-mm	KN										
Push-Modal	0	-3	-2	0		0	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	1	47	32	4066	68998	6022	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	2	50	33	4241	5890	6281	1081	1	0	0	0	1082	0	0	0	1082
Push-Modal	3	101	68	7713	205830	11423	1010	72	0	0	0	1082	0	0	0	1082
Push-Modal	4	153	103	10475	321763	15514	916	166	0	0	0	1045	0	37	0	1082
Push-Modal	5	201	136	12826		18995	870	212	0	0	0	969	0	113	0	1082
Push-Modal	6	201	136	12826		18995	854	228	0	0	0	904	0	178	0	1082
Push-Modal	7	201	136	12825		18994	849	232	1	0	0	891	0	190	1	1082
Push-Modal	8	201	136	12822		18990	849	232	0	1	0	891	0	190	1	1082
Push-Modal	9	201	136	12822		18990	849	232	0	1	0	890	0	191	1	1082
Push-Modal	10	201	136	12823		18991	849	232	0	0	1	886	2	191	3	1082
					602481.08											

Bilinearized SDOF

d*y	F*y
mm	KN
0	0
91	10475
103	10475

Target Displacement Finding — Spectrum

m*	D_y^*	F_y^*	T*	$S_e(T^*)$	D^*_{et}	D_t
Ton	*mm*	*KN*	*sec*	*m/s^2*	*mm*	*mm*
1522.09	91	10475	0.724	5.84	78	115

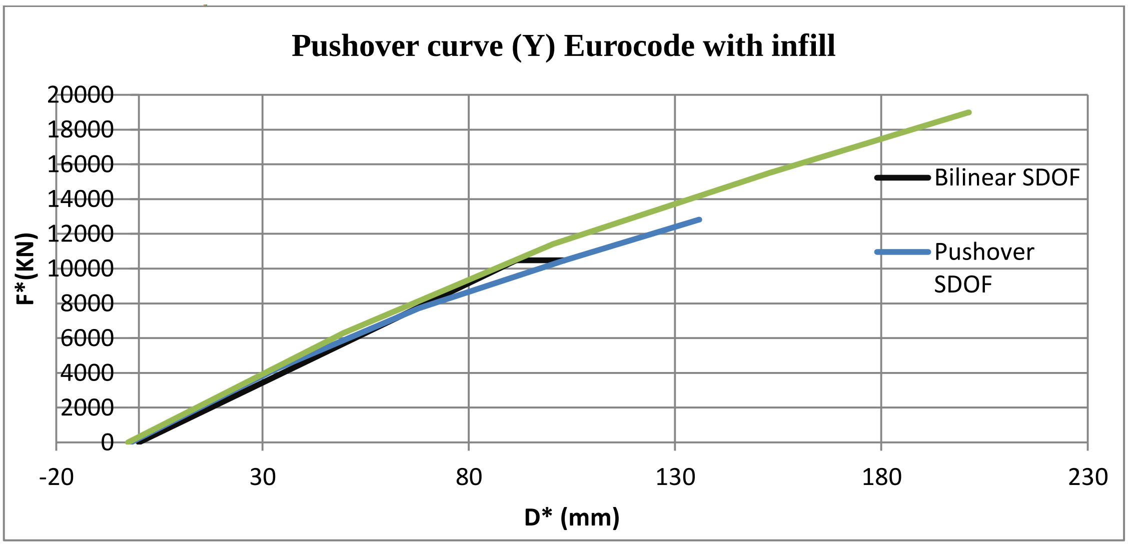


Figure 70: Bilinearized curve Y-direction (with infill)

The model was now again pushed to this new target displacement d_t as calculated above. Pushover is carried out in an iterative process until a desired convergence is met. We can also evaluate the behavior factor of the building in each direction using the ductility factor and over-strength factor calculated from the idealized curve. The expression for behavior factor is given by (Chaulagain & al, 2014):

$q = \mu \times \Omega$, $\mu = \frac{\Delta_{max}}{\Delta_y}$, $\Omega = \frac{V_y}{V_s}$

Where, μ = ductility factor (ratio between ultimate displacement and yield displacement)

Ω = overstrength factor (ratio between force at yielding and force at first hinge formation)

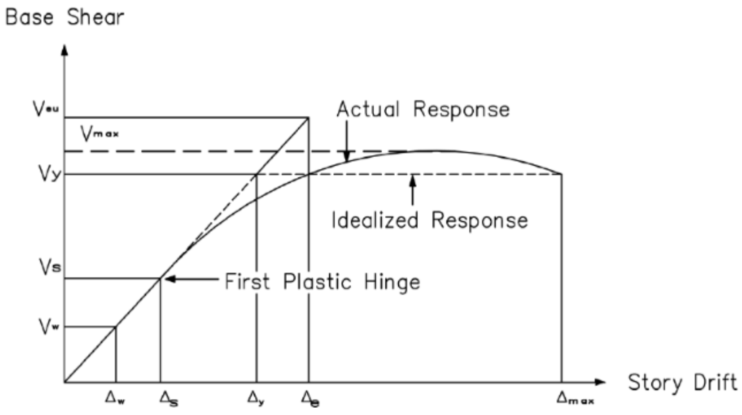


Figure 71: Seismic Demand Parameters

Table 98: Pushover calculations X-direction without infill at D_t

Pushover Capacity Curve for SDOF EC w/o infill (PUSH X)						Γ	0.96									
LoadCase	Step	Displacement	D*	Fb*	Em*	BaseForce	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
Text	Unitless	mm	mm	KN	KN-mm	KN										
Push-Modal	0	0	0	0		0	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	1	6	6	349	1063	336	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	2	11	12	699	3189	672	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	3	17	18	**1048**	5316	1007	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	4	23	24	1397	7442	1343	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	5	29	30	1747	9568	1679	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	6	35	36	2096	11694	2015	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	7	41	42	2445	13821	2351	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	8	47	48	2795	15947	2686	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	9	52	55	3144	18073	3022	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	10	53	55	3194	2766	3070	1081	1	0	0	0	1082	0	0	0	1082
Push-Modal	11	61	63	3640	27060	3498	1076	6	0	0	0	1082	0	0	0	1082
Push-Modal	12	67	70	3976	24087	3822	1065	17	0	0	0	1082	0	0	0	1082
Push-Modal	13	73	76	4286	26066	4119	1049	33	0	0	0	1082	0	0	0	1082
Push-Modal	14	79	82	4559	28992	4383	1031	51	0	0	0	1082	0	0	0	1082
Push-Modal	15	85	89	4779	29870	4593	1009	73	0	0	0	1082	0	0	0	1082
Push-Modal	16	92	96	4978	33035	4785	991	91	0	0	0	1082	0	0	0	1082
Push-Modal	17	98	102	5153	33242	4953	981	101	0	0	0	1082	0	0	0	1082
Push-Modal	18	105	109	5327	38018	5120	962	120	0	0	0	1082	0	0	0	1082
Push-Modal	19	112	116	5471	35538	5258	951	131	0	0	0	1081	0	1	0	1082
Push-Modal	20	117	121	5583	29820	5367	944	138	0	0	0	1078	0	4	0	1082
					394606.90											

CALCULATION							
$0,6*F_U=F_y$	$d\ (0,6*F_u)$	area bil	$A_{push}-A_{bil}$	d_Y	k*	m*	T*
[KN]	[m]	[m^2]		[m]	[KN/m]	[ton]	[sec]
3350.04	0.05815	394.6069	0.0	0.089019	57607.7	2392	1.280

A_{push}	du [m]	Fu [KN]
394.607	0.121458	5583.4049

INTERPOLATION CALCULATION d*		
S_{sup}	V_{sup}	0.05815
0.06	3194.26	
S_{inf}	V_{inf}	
0.06	3639.62	

$$d^* = \frac{\left(S_{inf} - S_{sup}\right) * \left(F^* - V_{sup}\right)}{\left(V_{inf} - V_{sup}\right)} + S_{sup}$$

Bilinearized SDOF

d*y	F*y
mm	KN
0	0
58	3350
89	5128
121	5128

μ	Ω
1.36	**1.61**
q	2.19

Target Displacement Finding

From Elastic Spectrum

m*	D_y*	F_y*	T*	$S_e(T^*)$	D^*_{et}	D_t
Ton	*mm*	*KN*	*sec*	*m/s^2*	*mm*	*mm*
2392.26	89	5128	1.28	3.30	137	132

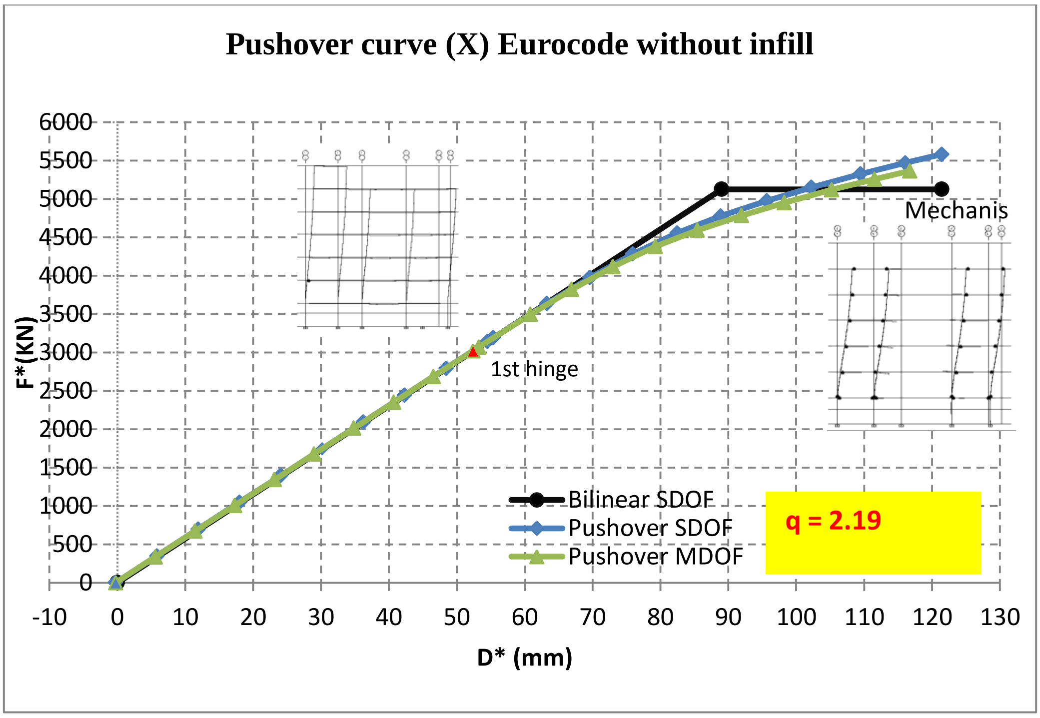


Figure 72: Bilinearized curve X direction without infill at D_t

Table 99: Pushover calculations X-direction with infill at D_t

Pushover Capacity Curve for SDOF EC with infill (PUSH X)						Γ	1.28									
LoadCase	Step	Displacement	D*	Fb*	Em*	BaseForce	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
Text	Unitless	mm	mm	KN	KN-mm	KN										
Push-Modal	0	0	0	0		0	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	1	7	5	353	996	453	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	2	14	11	707	2892	907	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	3	21	16	1060	4888	1360	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	4	28	22	1414	6747	1814	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	5	35	27	1767	8799	2267	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	6	42	33	2120	10603	2721	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	7	48	37	2399	9861	3078	1073	1	0	0	0	1074	0	0	0	1074
Push-Modal	8	55	43	2753	14453	3533	1070	4	0	0	0	1074	0	0	0	1074
Push-Modal	9	62	48	3096	16409	3973	1055	19	0	0	0	1074	0	0	0	1074
Push-Modal	10	69	54	3416	18778	4384	1036	38	0	0	0	1074	0	0	0	1074
Push-Modal	11	77	60	3702	20802	4750	1002	72	0	0	0	1074	0	0	0	1074
Push-Modal	12	84	65	3921	21088	5031	978	96	0	0	0	1074	0	0	0	1074
Push-Modal	13	91	71	4115	22856	5280	953	121	0	0	0	1074	0	0	0	1074
Push-Modal	14	99	77	4280	24858	5492	940	134	0	0	0	1074	0	0	0	1074
Push-Modal	15	106	83	4429	25111	5684	931	143	0	0	0	1074	0	0	0	1074
Push-Modal	16	115	89	4579	29133	5875	917	157	0	0	0	1071	0	3	0	1074
Push-Modal	17	126	98	4761	40759	6109	908	166	0	0	0	1063	0	11	0	1074
Push-Modal	18	134	104	4887	30826	6271	903	171	0	0	0	1054	0	20	0	1074
Push-Modal	19	141	110	4989	25783	6402	901	173	0	0	0	1043	0	31	0	1074
					335641.13											

CALCULATION							
$0,6*F_U=F_y$	d $(0,6*F_u)$	area bil	$A_{push}-A_{bil}$	d_Y	k*	m*	T*
[KN]	[m]	[m^2]		[m]	[KN/m]	[ton]	[sec]
2998.12	0.04663	336.6457	0.0	0.070121	64290.2	1755	1.038
				?			

A_{push}	du [m]	Fu [KN]
336.646	0.109737	4996.8594

INTERPOLATION CALCULATION d*		
S_{sup}	V_{sup}	0.04663
0.04	2757.10	
S_{inf}	V_{inf}	
0.05	3100.73	

$$d^* = \frac{\left(S_{inf} - S_{sup}\right) * \left(F^* - V_{sup}\right)}{\left(V_{inf} - V_{sup}\right)} + S_{sup}$$

Bilinearized SDOF			
d*y	F*y	μ	Ω
mm	KN	1.56	1.88
0	0	q	2.94
47	2998		
70	4508		
110	4508		

Target Displacement Finding — From Elastic Spectrum

m*	D_y*	F_y*	T*	$S_e(T^*)$	D^*_{et}	D_t
Ton	*mm*	*KN*	*sec*	*m/s^2*	*mm*	*mm*
1757.93	70	4508	1.04	4.07	111	143

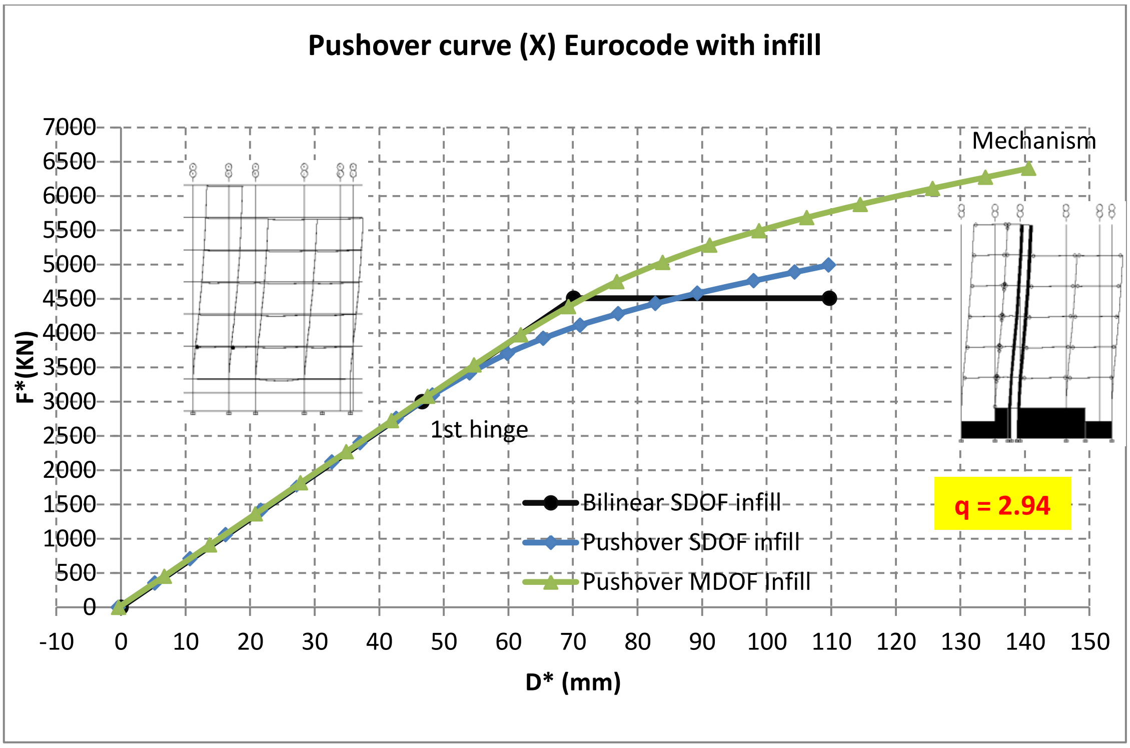


Figure 73: Bilinearized curve X direction with infill at D_t

Table 100: Pushover calculations Y-direction without infill at D_t

Pushover Capacity Curve for SDOF EC w/o infill (PUSH Y)						Γ	1.35									
LoadCase	Step	Displacement	D*	Fb*	Em*	BaseForce	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
Text	Unitless	mm	mm	KN	KN-mm	KN										
Push-Modal	0	-3	-2	0		0	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	1	5	4	371	1117	500	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	2	13	10	743	3352	1000	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	3	21	16	1114	5587	1500	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	4	29	22	1486	7822	2000	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	5	37	28	1857	10057	2500	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	6	45	34	2229	12292	3000	1082	0	0	0	0	1082	0	0	0	1082
Push-Modal	7	51	38	2505	10587	3372	1081	1	0	0	0	1082	0	0	0	1082
Push-Modal	8	60	45	2903	17843	3908	1072	10	0	0	0	1082	0	0	0	1082
Push-Modal	9	69	51	3245	19017	4368	1047	35	0	0	0	1082	0	0	0	1082
Push-Modal	10	78	58	3531	23093	4754	1010	72	0	0	0	1082	0	0	0	1082
Push-Modal	11	87	64	3722	24123	5011	975	107	0	0	0	1082	0	0	0	1082
Push-Modal	12	95	71	3857	22993	5192	965	117	0	0	0	1082	0	0	0	1082
Push-Modal	13	105	78	3995	30058	5378	942	140	0	0	0	1081	0	1	0	1082
Push-Modal	14	115	85	4108	29208	5530	925	157	0	0	0	1077	0	5	0	1082
Push-Modal	15	124	92	4198	26709	5651	922	160	0	0	0	1059	0	23	0	1082
Push-Modal	16	133	99	4286	28804	5770	910	172	0	0	0	1043	0	39	0	1082
Push-Modal	17	141	105	4362	27920	5872	897	185	0	0	0	1022	0	60	0	1082
Push-Modal	18	151	112	4430	30230	5964	883	199	0	0	0	994	0	88	0	1082
Push-Modal	19	159	118	4480	26836	6031	877	205	0	0	0	983	0	99	0	1082
					357647.66											

CALCULATION							
$0,6*F_U=F_y$	$d\ (0,6*F_u)$	area bil	$A_{push}-A_{bil}$	d_Y	k*	m*	T*
[KN]	[m]	[m^2]		[m]	[KN/m]	[ton]	[sec]
2692.00	0.04134	358.7170	0.0	0.063871	65115.0	1639	0.996

A_{push}	du [m]	Fu [KN]
358.717	0.118187	4486.6597

INTERPOLATION CALCULATION d*		
S_{sup}	V_{sup}	0.04134
0.04	2508.43	
S_{inf}	V_{inf}	
0.04	2907.65	

$$d^* = \frac{\left(S_{inf} - S_{sup}\right) * \left(F^* - V_{sup}\right)}{\left(V_{inf} - V_{sup}\right)} + S_{sup}$$

Bilinearized SDOF

d*y	F*y
mm	KN
0	0
41	2692
64	4159
118	4159

µ	Ω
1.85	1.66
q	3.07

Target Displacement Finding				From Elastic Spectrum		
m^*	D_y^*	F_y^*	T^*	$S_e(T^*)$	D^*_{et}	D_t
Ton	*mm*	*KN*	*sec*	*m/s^2*	*mm*	*mm*
1641.31	64	4159	0.998	4.03	101	137

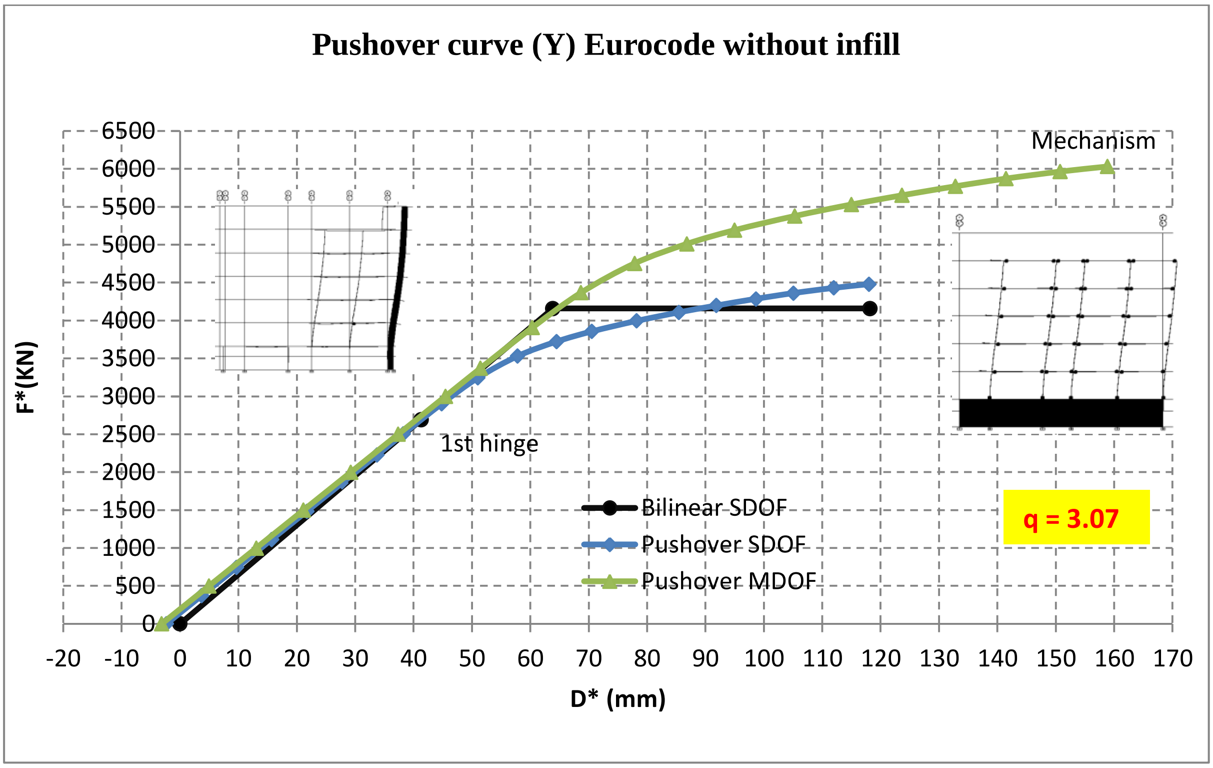


Figure 74: Bilinearized curve Y direction without infill at D_t

Table 101: Pushover calculations Y-direction with infill at D_t

Pushover Capacity Curve for SDOF EC with infill (PUSH Y)						Γ	1.48									
LoadCase	Step	Displacement	D*	Fb*	Em*	BaseForce	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
Text	Unitless	mm	mm	KN	KN-mm	KN										
Push-Modal	0	-3	-2	0		0	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	1	3	2	467	940	692	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	2	9	6	935	2746	1384	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	3	15	10	1402	4498	2077	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	4	20	14	1870	6407	2769	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	5	26	18	2338	8097	3462	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	6	32	22	2805	10070	4155	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	7	38	25	3273	11697	4847	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	8	43	29	3741	13734	5540	1074	0	0	0	0	1074	0	0	0	1074
Push-Modal	9	47	32	4062	10274	6016	1073	1	0	0	0	1074	0	0	0	1074
Push-Modal	10	54	36	4565	18349	6761	1069	5	0	0	0	1074	0	0	0	1074
Push-Modal	11	61	41	5142	23595	7615	1059	15	0	0	0	1074	0	0	0	1074
Push-Modal	12	67	45	5589	21738	8278	1041	33	0	0	0	1074	0	0	0	1074
Push-Modal	13	73	49	6018	23905	8913	1022	52	0	0	0	1074	0	0	0	1074
Push-Modal	14	79	54	6444	26927	9544	1006	68	0	0	0	1074	0	0	0	1074
Push-Modal	15	85	58	6812	26405	10089	990	84	0	0	0	1074	0	0	0	1074
Push-Modal	16	91	61	7165	27369	10611	981	93	0	0	0	1074	0	0	0	1074
Push-Modal	17	97	65	7504	29218	11113	966	108	0	0	0	1073	0	1	0	1074
Push-Modal	18	103	70	7861	32681	11643	950	124	0	0	0	1072	0	2	0	1074
Push-Modal	19	110	74	8247	37524	12214	941	133	0	0	0	1067	0	7	0	1074
Push-Modal	20	112	76	8371	12904	12398	939	135	0	0	0	1063	0	11	0	1074
					349077.21											

CALCULATION							
$0,6{*}F_U=F_y$	$d\ (0,6{*}F_u)$	area bil	A_{push}-A_{bil}	d_Y	k*	m*	T*
[KN]	[m]	[m^2]		[m]	[KN/m]	[ton]	[sec]
5030.22	0.04011	350.0808	0.0	0.062066	125413.2	1520	0.691

A_{push}	du [m]	Fu [KN]
350.081	0.076008	8383.7039

INTERPOLATION CALCULATION d*		
S_{sup}	V_{sup}	0.04011
0.04	4571.69	
S_{inf}	V_{inf}	
0.04	5149.56	

$$d^{*}=\frac{\left(S_{inf}-S_{sup}\right)*\left(F^{*}-V_{sup}\right)}{\left(V_{inf}-V_{sup}\right)}+S_{sup}$$

Bilinearized SDOF

d*y	F*y
mm	KN
0	0
40	5030
62	7784
76	7784

μ	Ω
1.22	1.92
q	2.35

Target Displacement Finding

From Elastic Spectrum

m*	D_y*	F_y*	T*	$S_e(T^*)$	D^*_{et}	D_t
Ton	*mm*	*KN*	*sec*	*m/s²*	*mm*	*mm*
1522.09	62	7784	0.692	6.04	73	109

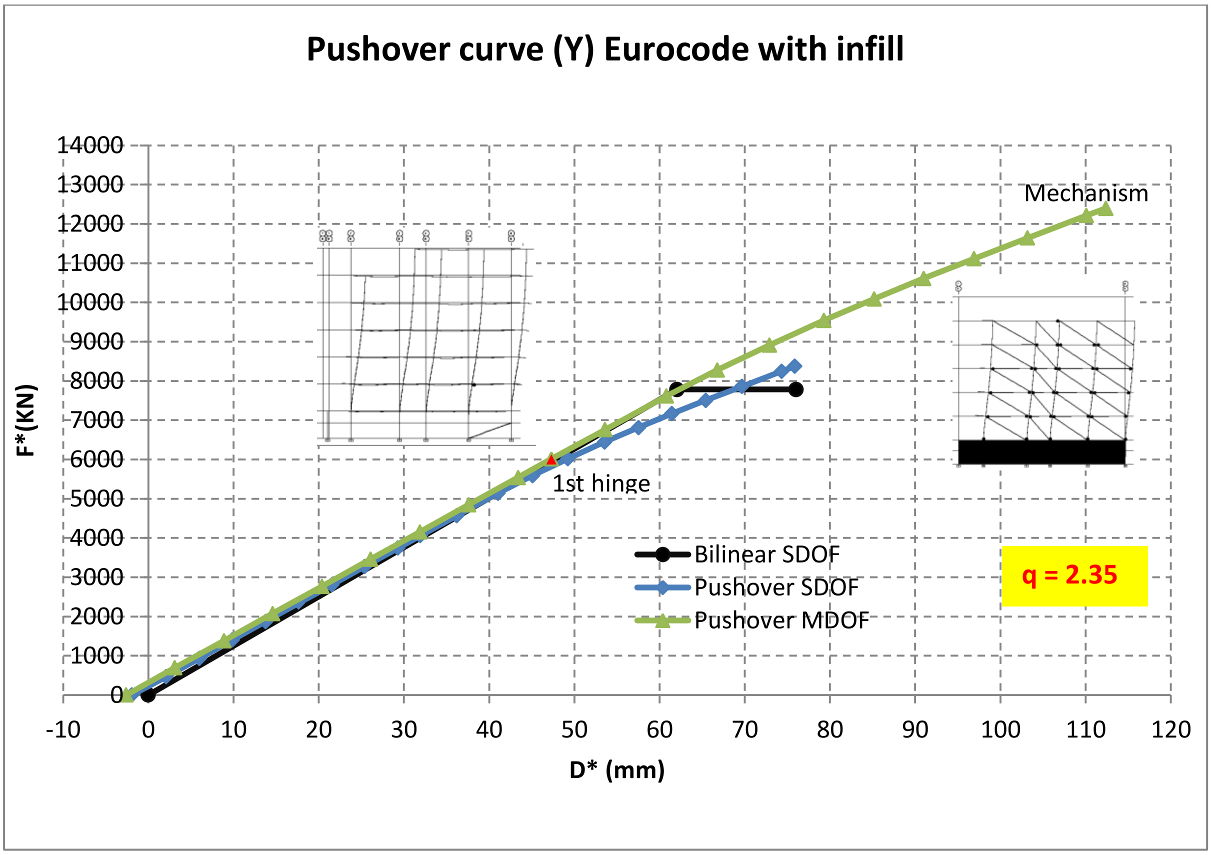


Figure 75: Bilinearized curve Y direction with infill at D_t

The model was now again pushed to the target displacement d_t as calculated above. The extent of damage experienced by the building at this final target displacement is considered to be representative of the damage experienced by the building when subjected to design level ground shaking.

Table 102: Seismic Demand Parameter for different cases

Case	Δ_s (mm)	Δ_y (mm)	Δ_{max} (mm)	V_s (KN)	V_y (KN)	V_{max} (KN)	μ	Ω	q
X – direction without infill	55	89	121	3194	5128	5583	1.36	1.61	2.19
X – direction with infill	37	70	110	2399	4508	4989	1.56	1.88	2.94
Y – direction without infill	38	64	118	2505	4159	4480	1.85	1.66	3.07
Y – direction with infill	32	62	76	4062	7784	8371	1.22	1.92	2.35

Table 103: Behavior Factor (q) for different cases from Pushover analysis

Behavior Factor/Cases	X – direction without infill	X – direction with infill	Y – direction without infill	Y – direction with infill
q	**2.19**	2.94	3.07	2.35

8.5 ADRS (Demand – Capacity Curve)

The behavior factor calculations above give us tentative idea about the response of the study building. The smallest 'q' value **2.19** can be considered as the real behavior/reduction factor of the case structure. The adopted factor as per code was **4.68 and 5.0** (for EC and IS resp.). This suggests that the study building lacks proper ductility than assumed in linear analysis. Further verification of performance level can be determined from the Acceleration Displacement Response Spectrum (ADRS) curve given in figure 76. The ADRS is a plot containing both demand curve for a given earthquake hazard and the capacity curve of the structure. Here the capacity curve is the bilinearized curve for SDOF determined from the pushover analysis steps as described above. The demand curve is plotted for a peak ground acceleration a_g=0.25g, which is the PGA for 475 years return period in Kathmandu (Wijeyewickrema & et al, 2011). The intersection between these two curves is known as the performance point. To meet the required performance level the demand/capacity ratio should be less than 1.

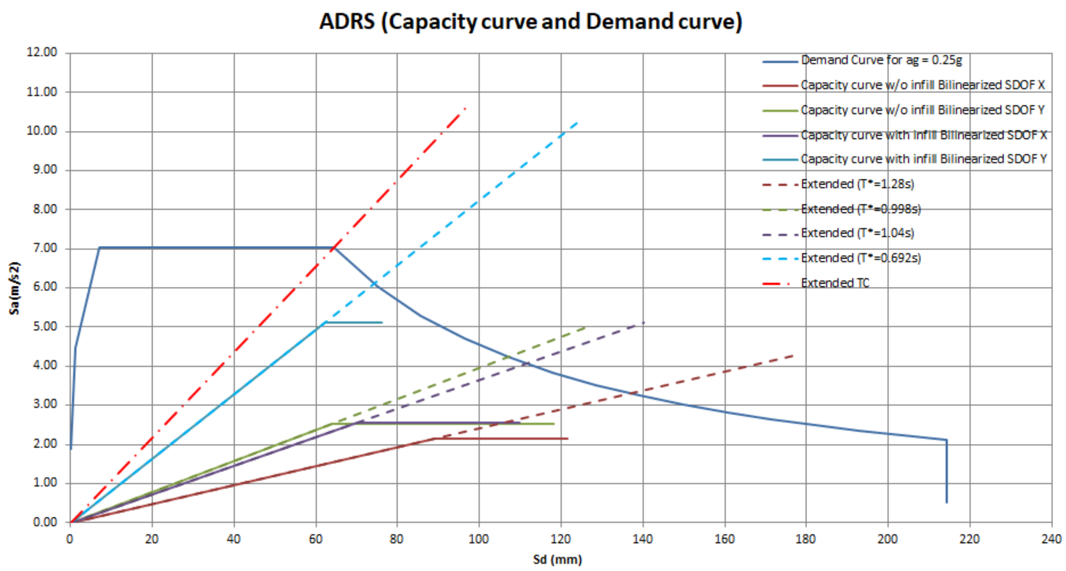


Figure 76: ADRS curve

8.6 Determining Performance Level and Vulnerability

m* 2392.26 ton
Capacity curve w/o infill
Bilinearized SDOF X

	d*y=Sd	F*y	Sa
	mm	KN	m/s2
Δ_0	0	0	0
Δ_y	89	5128	2.14366
Δ_u	121	5128	2.14366

Extended (T*=1.28s)	
Sd	Sa
0.000	0.000
89.019	2.144
178.039	4.287

Sae(T*)	Sde (T*)=de_{max} [mm]
3.3	137.5
Intersection of Extended T* with Demand curve	
Iv	
1.13	NOT VERIFIED

m* 1641.31 ton
Capacity curve w/o infill
Bilinearized SDOF Y

d*y=Sd	F*y	Sa
mm	KN	m/s2
0	0	0
64	4159	2.53393
118	4159	2.53393

Extended (T*=0.998s)	
Sd	Sa
0.000	0.000
63.871	2.534
127.742	5.068

Sae(T*)	Sde (T*)=de_{max} [mm]
4.2	107.5
Intersection of Extended T* with Demand curve	
Iv	
0.91	VERIFIED

m* 1757.93 ton
Capacity curve with infill
Bilinearized SDOF X

d*y=Sd	F*y	Sa
mm	KN	m/s2
0	0	0
70	4508	2.56442
110	4508	2.56442

Extended (T*=1.04s)	
Sd	Sa
0.000	0.000
70.121	2.564
140.242	5.129

Sae(T*)	Sde (T*)=de_{max} [mm]
4.1	112.5
Intersection of Extended T* with Demand curve	
Iv	
1.03	NOT VERIFIED

m* 1522.09 ton
Capacity curve with infill
Bilinearized SDOF Y

d*y=Sd	F*y	Sa
mm	KN	m/s2
0	0	0
62	7784	5.11391
76	7784	5.11391

Extended (T*=0.692s)	
Sd	Sa
0.000	0.000
62.066	5.114
124.131	10.228

Sae(T*)	Sde (T*)=de_{max} [mm]
6.1	75
Intersection of Extended T* with Demand curve	
Iv	
0.99	VERIFIED

Figure 77: Verification checks for the building performance level

The building is verified if the vulnerability index (**Iv**) is less than **1.0**. In the above results it can be seen that the X direction models are not verified. Also the Y direction models that are indicated verified also have Iv ratio very close to the limiting value of 1.0.

Hence from this pushover analysis it can be concluded that the study building does not meet the global performance level during the peak ground acceleration demand of 0.25g in Kathmandu. This suggests that the building needs proper retrofitting solution in order to meet the performance requirement.

The type of retrofitting intervention is being discussed in chapter 9.0.

9.0 RETROFITTING INTERVENTION SUGGESTION

9.1 General

As observed from above linear (static and dynamic) as well as static non-linear (pushover) analysis it is clear that the building needs to undergo some extent of retrofitting solution in order to meet the desired performance level. There are many types of seismic rehabilitation options of reinforced concrete buildings. There are interventions based on concrete, fiber reinforced polymers (FRPs) and steel solutions etc. Some widely used retrofitting techniques for existing RC buildings can be listed as follow:

- Concrete based solution
    - Concrete jacketing
    - Concrete Shear wall addition
    - Adding concrete lateral load resisting elements
- Fiber Reinforced Polymer based solution
    - FRP jacketing of beam, column (esp. beam column joints) and slabs
    - Use of FRP reinforcing bars
- Steel based solution
    - Jacketing of beam and columns using steel angles and plates
    - Providing steel bracing system embedded with RC frame
    - Use of Buckling Restrained Braces (BRB)
    - Use of steel shear plates
- Base Isolators
- Use of damping devices, etc.

In this thesis study an example of steel based solution using Buckling Restrained Brace technology is considered.

A buckling-restrained brace (BRB) is a structural brace in a building, designed to allow the building to withstand cyclic lateral loadings, typically earthquake-induced loading. It consists of a slender steel core, a concrete or steel casing designed to continuously support the core and prevent buckling under axial compression, and an interface region that prevents undesired interactions between the two. Braced frames that use BRBs – known as buckling-restrained braced frames, or BRBFs – have significant advantages over typical braced frames.

Advantages of using steel BRB solution:

- ✓ The BRB element can be embedded in gap near existing infill walls causing less destruction of existing structure.
- ✓ The BRB solution is often reversible solution so the existing architecture can be saved.
- ✓ Steel is environmental friendly and more sustainable due to its reusability and cleanliness in execution.
- ✓ By using BRB extremely dissipative structure can be designed resulting very ductile structure behavior.

✓ With BRBF one can achieve high response modification or behavior factor (q = 6 to 8), so the building can be designed for lower seismic forces.

Here, the complete design of the BRB and its connection design with the existing concrete members have been skipped due to limitation in time and also the objective is only to show the improvement in the global behavior of the structure after use of BRB solution. Only a preliminary sizing of BRB element has been done with a simplified calculation.

9.2 Preliminary design and configuration of BRB

The buckling restrained braces are proposed 4 in each direction at the location and alignment shown in the figure below.

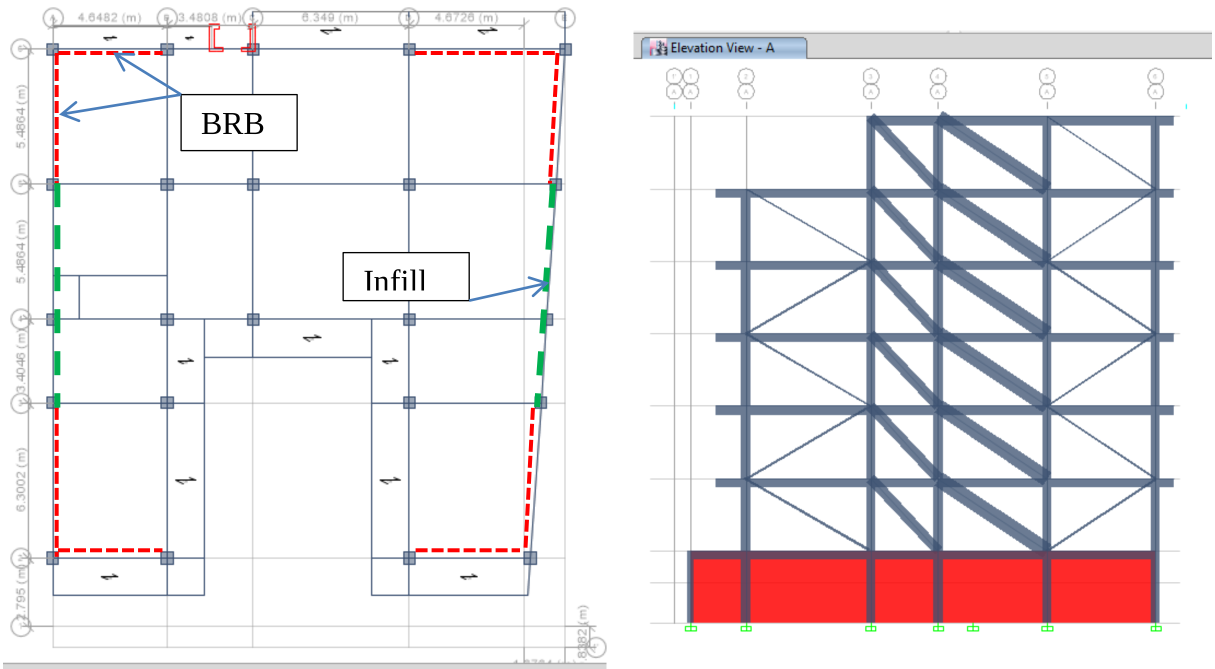


Figure 78: BRB and infill position in plan and elevation along Y-dir

The preliminary sizing of the BRB cores are done in such way that the horizontal shear force due to earthquake loading is taken by the BRB elements alone. Each BRB will carry a horizontal shear force $F_i/4$ on each floor. The horizontal shear force F_i is taken from the linear analysis in chapter 5.0 according to Eurocode (max). Also we can group the BRB area for every two floors. The detail calculations are done in table 104.

Modeling Assumptions:

- The BRB is modeled together with partial infill masonry, i.e. in the grid 3-4 and 4-5 infill masonry walls are considered where there is no BRB.
- The infill masonry walls are assumed to be in effect only until the inter storey drift reaches the limit for non-structural walls (i.e. 0.4% of height) after which the walls cracks and the frame behaves without infill afterwards.
- The eccentricity due to BRB connection with the RC frame is neglected although in reality there is some eccentricity.
- Only the yielding core dimension is considered for modeling.
- Plastic hinge in BRB is assumed to form at the center due to axial force (P).
- The connection between BRB and RC frame is considered to be pinned.

Table 104: Preliminary sizing of BRB core

Preliminary design of BRB core
Required steel core area
Material: S235 *fy* **= 235 Mp: E = 210 GPa**

In X-Direction (Grid 2) **Gap available for BRB = 170mm**

Floor	Force (KN), Fi/4	Force of group (KN)	Area required (mm2)	Height required	Adopted size (bxh) mm		Adopted area (mm2)	I min of casing	Lc req mm	Lc/L	Grid length	Height	BRB Length L	Ly	Le	Ke	Ne of casing	Ny	Ne/Ny
6	33	168	715	20	20	60	1200	1250000	2142.17	0.38	4648	3200	5643	2144	1749	144051	562830	282000	2.00
5	168							**120x50**	<-- Casing Size at side										
4	259	329	1400	40	20	75	1500	1562500	2142.17	0.38	4648	3200	5643	2144	1749	180064	703537	352500	2.00
3	329							**150x50**	<-- Casing Size at side										
2	386	423	1800	51	20	100	2000	2083333	2142.17	0.38	4648	3200	5643	2144	1749	240085	938050	470000	2.00
1	423							**200x50**	<-- Casing Size at side										

In Y-Direction Grid (5-6) **Gap available for BRB = 170mm**

Floor	Force (KN), Fi/4	Force of group (KN)	Area required (mm2)	Height required	Adopted size (bxh) mm		Adopted area (mm2)	I min of casing	Lc req mm	Lc/L	Grid length	Height	BRB Length L	Ly	Le	Ke	Ne of casing	Ny	Ne/Ny
6	49	223	950	27	20	60	1200	1250000	2142.17	0.34	5486	3200	6351	2142	2105	119739	564000	282000	2.00
5	223							**120x50**	<-- Casing Size at side										
4	354	461	1962	49	20	100	2000	2083333	2142.17	0.34	5486	3200	6351	2142	2105	199566	940000	470000	2.00
3	461							**200x50**	<-- Casing Size at side										
2	539	580	2468	71	20	125	2500	2604167	2142.17	0.34	5486	3200	6351	2142	2105	249457	1175000	587500	2.00
1	580							**250x50**	<-- Casing Size at side										

Here it must be noticed that the buckling load of the casing sleeve (N_E) is kept adequately larger than the yield load of the core (N_y) in order to satisfy the stability criterion suggested by (Watanabe & et al, 1998). $N_E/N_y \geq 2$. Also another key aspect is the ratio between the restrained yielding segment length (L_c) and the total BRB length (L) which is kept between 0.3 ~ 0.5 as done by (D'Aniello, Mazzolani, & Della Corte, 2009) in their research project.

Maximum size of BRB used is 250x160 mm.

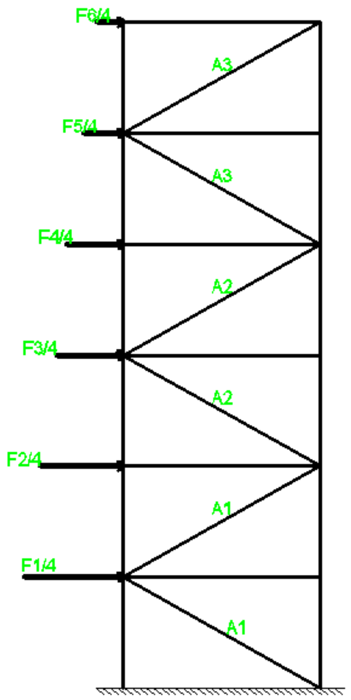


Figure 79: Preliminary sizing of BRB core

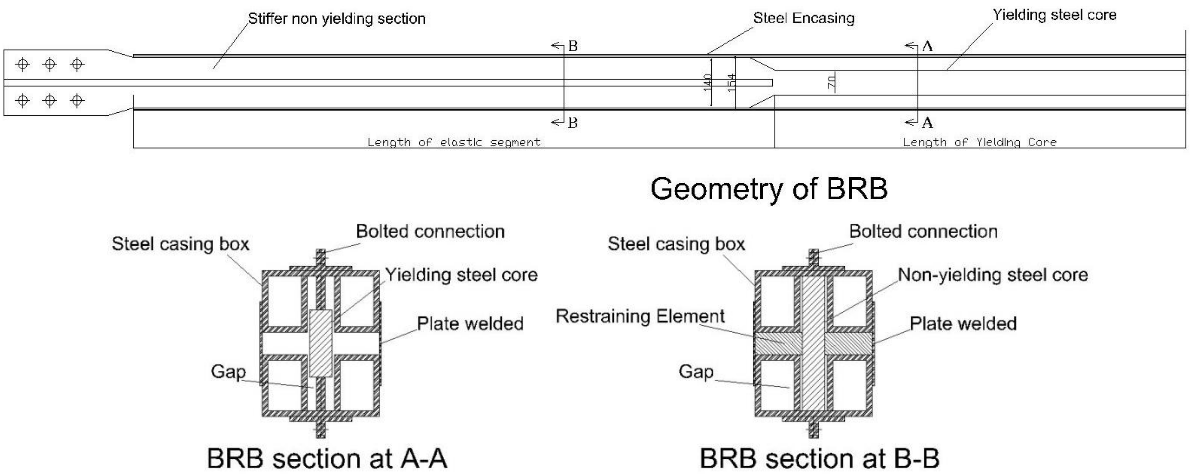


Figure 80: Typical BRB section and geometry

9.3 Non Linear Static analysis (Pushover) using BRB section

BRB Material used:

S235, F_y = 235 MPa, E = 210 GPa

BRB core sizes considered:

X- Direction, → A1 = 20x100mm, A2 = 20x75mm, A3 = 20x60mm

Y- Direction, → A1 = 20x125mm, A2 = 20x100mm, A3 = 20x60mm

The same 3D model as earlier was considered with addition of BRB elements. The BRB elements are modeled as pinned connection to the RC frames. The buckling restrained braces were assigned with axial-P type plastic hinge.

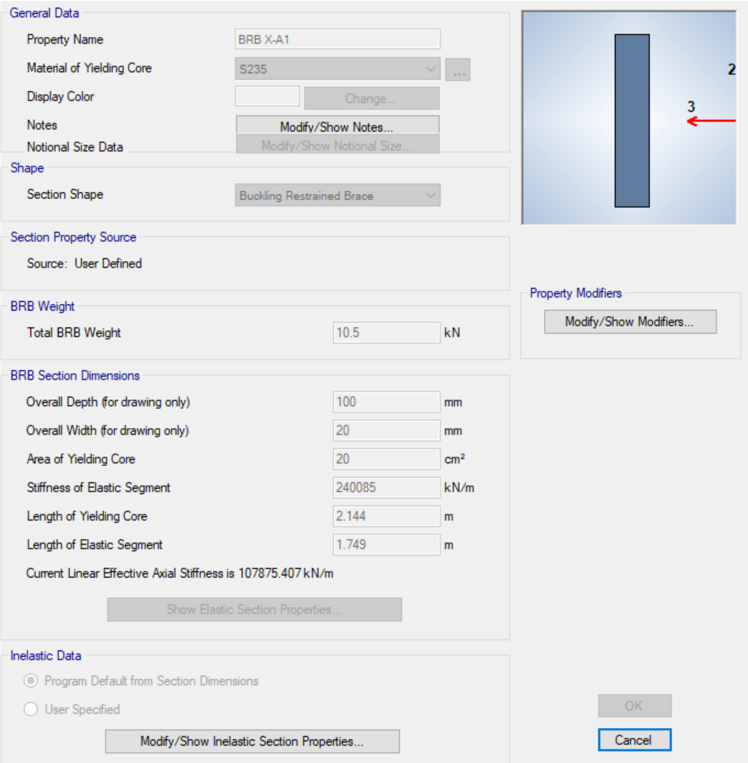


Figure 81: BRB section defined in ETABS

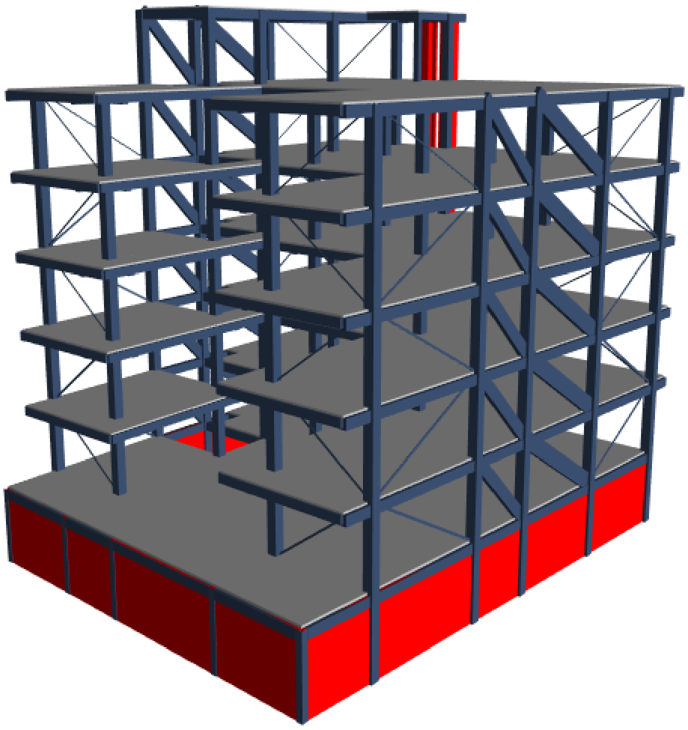


Figure 82: 3D model with BRB and partial infill

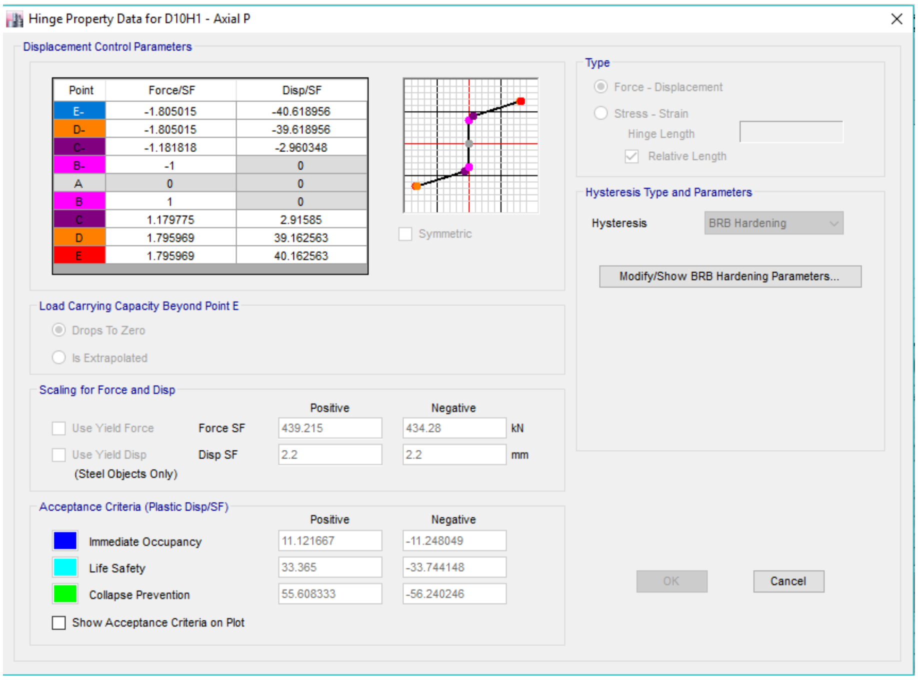


Figure 83: ETABS Hinge property data for BRB

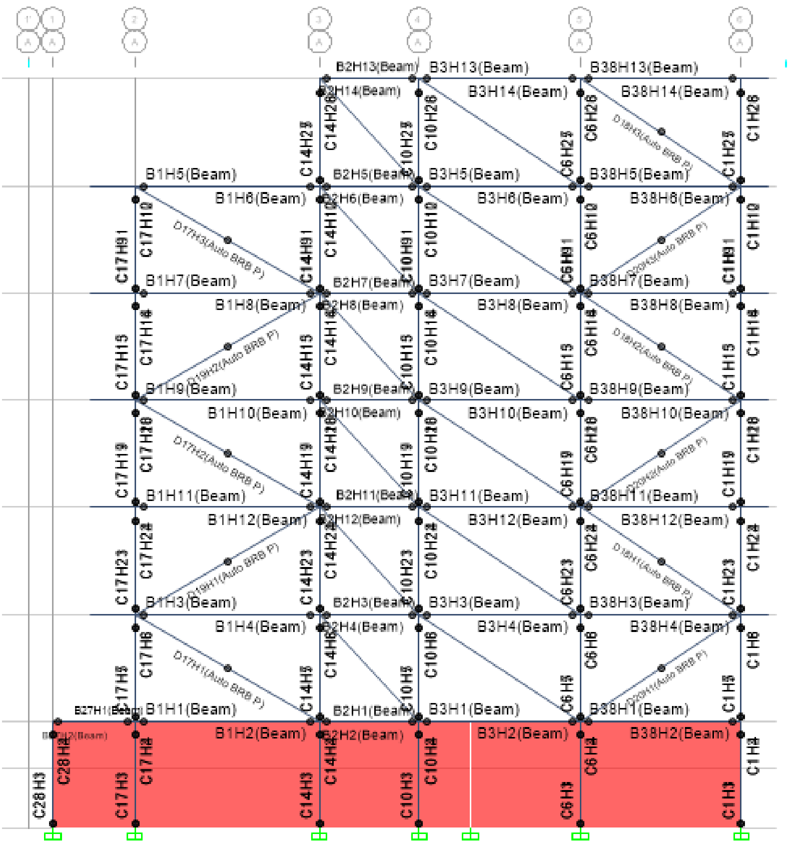


Figure 84: Nonlinear Hinge Assignments in ETABS

The pushover analysis was carried out using N2 method in ETABS as described in earlier chapter 8.0 to verify the performance of the building after retrofitting.

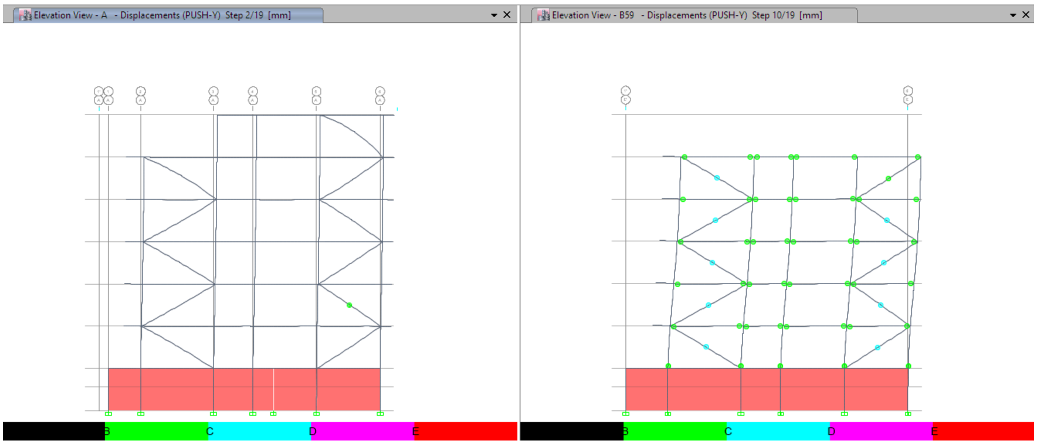


Figure 85: Plastic redundancy (First hinge and mechanism)

Here it can be noticed that the first plastic hinge is now formed at the BRB element.

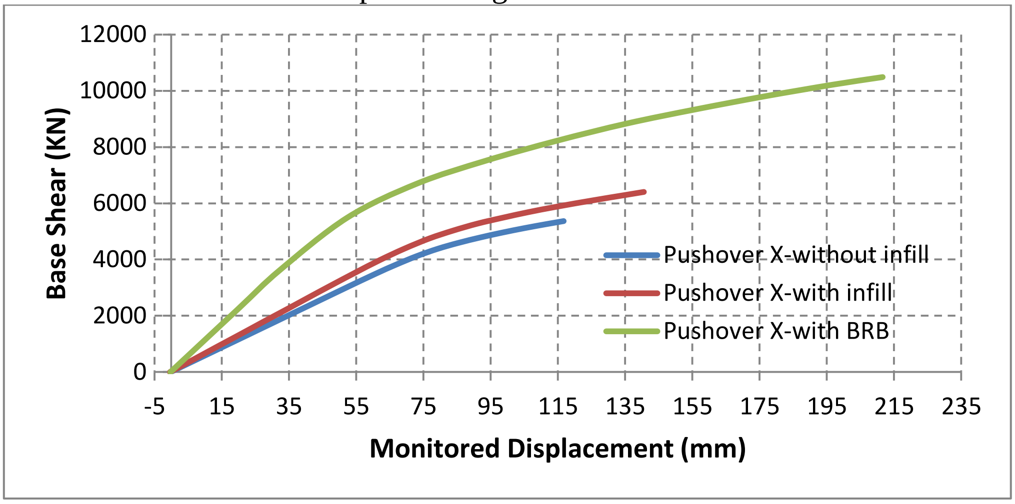


Figure 86: Pushover curve comparison before and after retrofit X-direction

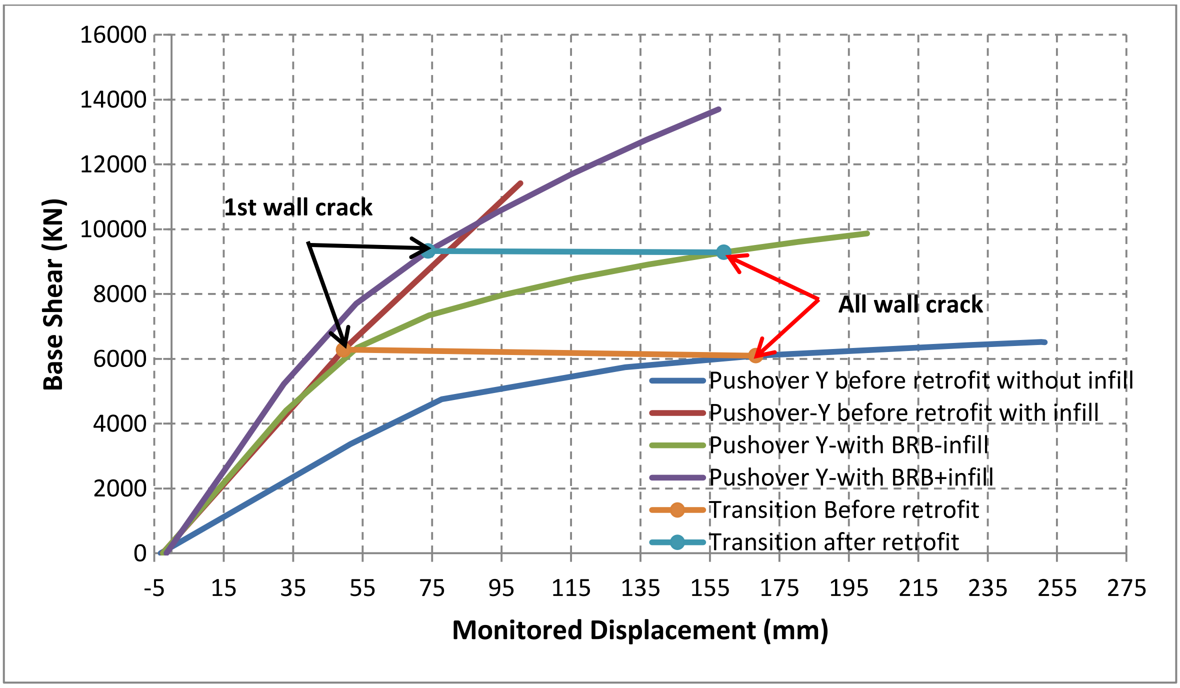


Figure 87: Pushover curve comparison before and after retrofit Y-direction

Table 105: Calculation of Transformation Factor Γ (BRB)

X-Direction					
Storey	mi (t)	Φi	mi* Φi		mi*Φi^2
6	91.2862	1	91.2862		91.2862
5	528.67005	0.97415	515.002		501.687
4	510.17466	0.8521	434.722		370.429
3	533.37945	0.65004	346.718		225.38
2	533.62626	0.41231	220.02		90.7168
1	533.87308	0.17752	94.7741		16.8245
G	702.23981	0.00496	3.4861		0.01731
		m^*=	1706.01	$\Sigma m^* \Phi_i^2$	1296.34
				Γ	1.32

Y-Direction					
Storey	mi (t)	Φi	mi* Φi		mi*Φi^2
6	91.2862	1	91.2862		91.2862
5	528.67005	0.92867	490.96		455.94
4	510.17466	0.78809	402.063		316.862
3	533.37945	0.60579	323.116		195.741
2	533.62626	0.38721	206.624		80.0063
1	533.87308	0.16191	86.4414		13.9961
G	702.23981	0.00427	2.99621		0.01278
		m^*=	1603.49	$\Sigma m^* \Phi_i^2$	1153.84
				Γ	1.39

Table 106: Pushover Calculation X-direction with BRB

Pushover Capacity Curve for SDOF EC with BRB (PUSH X)						Γ	1.32									
LoadCase	Step	Displacement	D*	Fb*	Em*	BaseForce	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
Text	Unitless	mm	mm	KN	KN-mm	KN										
Push-Modal	0	0	0	0		0	1124	0	0	0	0	1124	0	0	0	1124
Push-Modal	1	20	15	1693	12861	2227	1124	0	0	0	0	1124	0	0	0	1124
Push-Modal	2	32	25	2752	21165	3621	1123	1	0	0	0	1124	0	0	0	1124
Push-Modal	3	52	40	4168	52717	5485	1093	31	0	0	0	1124	0	0	0	1124
Push-Modal	4	73	55	5082	72282	6688	1023	100	1	0	0	1124	0	0	0	1124
Push-Modal	5	94	71	5710	85066	7514	987	129	8	0	0	1121	0	3	0	1124
Push-Modal	6	115	87	6248	95931	8223	964	150	10	0	0	1097	0	27	0	1124
Push-Modal	7	136	103	6714	102505	8835	945	165	14	0	0	1062	0	62	0	1124
Push-Modal	8	156	119	7098	108050	9341	929	179	16	0	0	1040	1	83	0	1124
Push-Modal	9	177	134	7452	114527	9808	909	198	17	0	0	1028	3	93	0	1124
Push-Modal	10	197	150	7762	115247	10215	895	211	18	0	0	1012	5	107	0	1124
Push-Modal	11	212	161	7967	88783	10485	883	219	22	0	0	995	6	118	5	1124
					869133.28											

CALCULATION							
$0{,}6^*F_U=F_y$	d $(0{,}6^*F_u)$	area bil	A_{push}-A_{bil}	d_y	k*	m*	T*
[KN]	[m]	[m^2]		[m]	[KN/m]	[ton]	[sec]
4790.17	0.05033	872.6725	0.0	0.073775	95183.8	1703	0.840

A_{push}	du [m]	Fu [KN]
872.673	0.161162	7983.6095

INTERPOLATION CALCULATION d*		
S_{sup}	V_{sup}	0.05033
0.04	4176.35	
S_{inf}	V_{inf}	
0.06	5092.64	

$$d^* = \frac{\left(S_{inf} - S_{sup}\right) * \left(F^* - V_{sup}\right)}{\left(V_{inf} - V_{sup}\right)} + S_{sup}$$

Bilinearized SDOF

d*y	F*y
mm	KN
0	0
50	4790
74	7022
161	7022

μ	Ω
2.18	2.55
q	5.57

Target Displacement Finding

From Elastic Spectrum

m*	D_y*	F_y*	T*	$S_e(T^*)$	D^*_{et}	D_t
Ton	*mm*	*KN*	*sec*	*m/s^2*	*mm*	*mm*
1706.01	74	7022	0.84	5.03	90	119

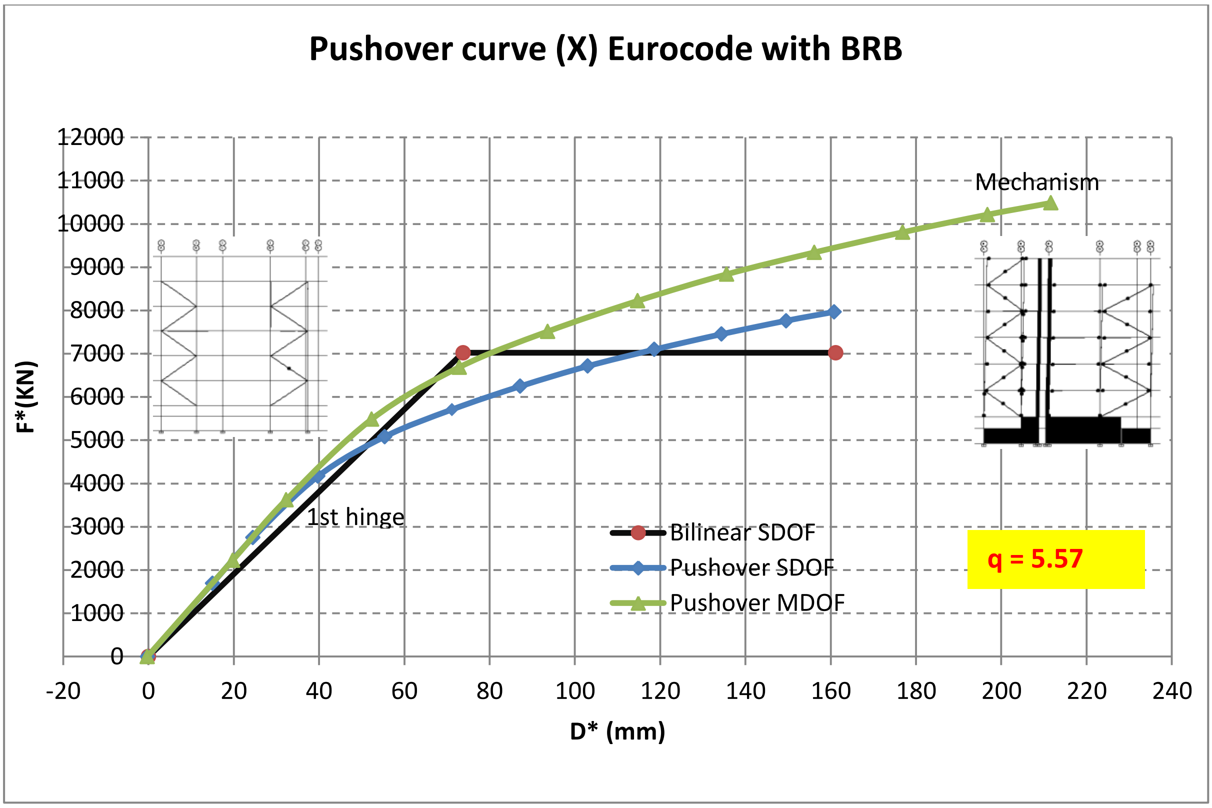


Figure 88: Bilinearized curve X direction with BRB

Table 107: Pushover Calculation Y-direction with BRB

Pushover Capacity Curve for SDOF EC with BRB actual(PUSH Y)						Γ	1.39										
LoadCase	Step	Displacement	D*	Fb*	Em*	BaseForce	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total	
Text	Unitless	mm	mm	KN	KN-mm	KN											
Push-Modal	0	-1	-1	0		0	1124	0	0	0	0	1124	0	0	0	1124	before crack
Push-Modal	1	17	12	2063	13837	2867	1124	0	0	0	0	1124	0	0	0	1124	
Push-Modal	2	32	23	3765	31871	5232	1123	1	0	0	0	1124	0	0	0	1124	
Push-Modal	3	53	38	5546	69666	7708	1066	58	0	0	0	1124	0	0	0	1124	
Push-Modal	4	74	53	6713	91340	9329	1009	111	4	0	0	1124	0	0	0	1124	
Push-Modal	8	159	114	6684	410482	9289	910	198	16	0	0	1014	4	106	0	1124	after crack
Push-Modal	9	181	130	6916	105876	9610	887	221	16	0	0	1002	8	114	0	1124	
Push-Modal	10	200	144	7103	98872	9871	878	226	20	0	0	986	8	129	1	1124	
					821943.95												

Calculation							
$0,6*F_U=F_y$	d $(0,6*F_u)$	area bil	$A_{push}-A_{bil}$	d_y	k*	m*	T*
[KN]	[m]	[m^2]		[m]	[KN/m]	[ton]	[sec]
4269.64	0.02752	825.0060	0.0	0.043328	155160.5	1600	0.638

A_{push}	du [m]	Fu [KN]
825.006	0.144382	7116.0681

INTERPOLALATION CALCULATION d*		
S_{sup}	V_{sup}	0.02752
0.02	3772.11	
S_{inf}	V_{inf}	
0.04	5556.58	

$$d^* = \frac{(S_{inf} - S_{sup}) * (F^* - V_{sup})}{(V_{inf} - V_{sup})} + S_{sup}$$

Bilinearized SDOF

d*y	F*y
mm	KN
0	0
28	4270
43	6723
144	6723

μ	Ω
3.33	1.79
q	5.95

Target Displacement Finding

From Elastic Spectrum

m*	D_y*	F_y*	T*	$S_e(T^*)$	D^*_{et}	D_t
Ton	*mm*	*KN*	*sec*	*m/s^2*	*mm*	*mm*
1603.488	43	6723	0.639	6.62	68	95

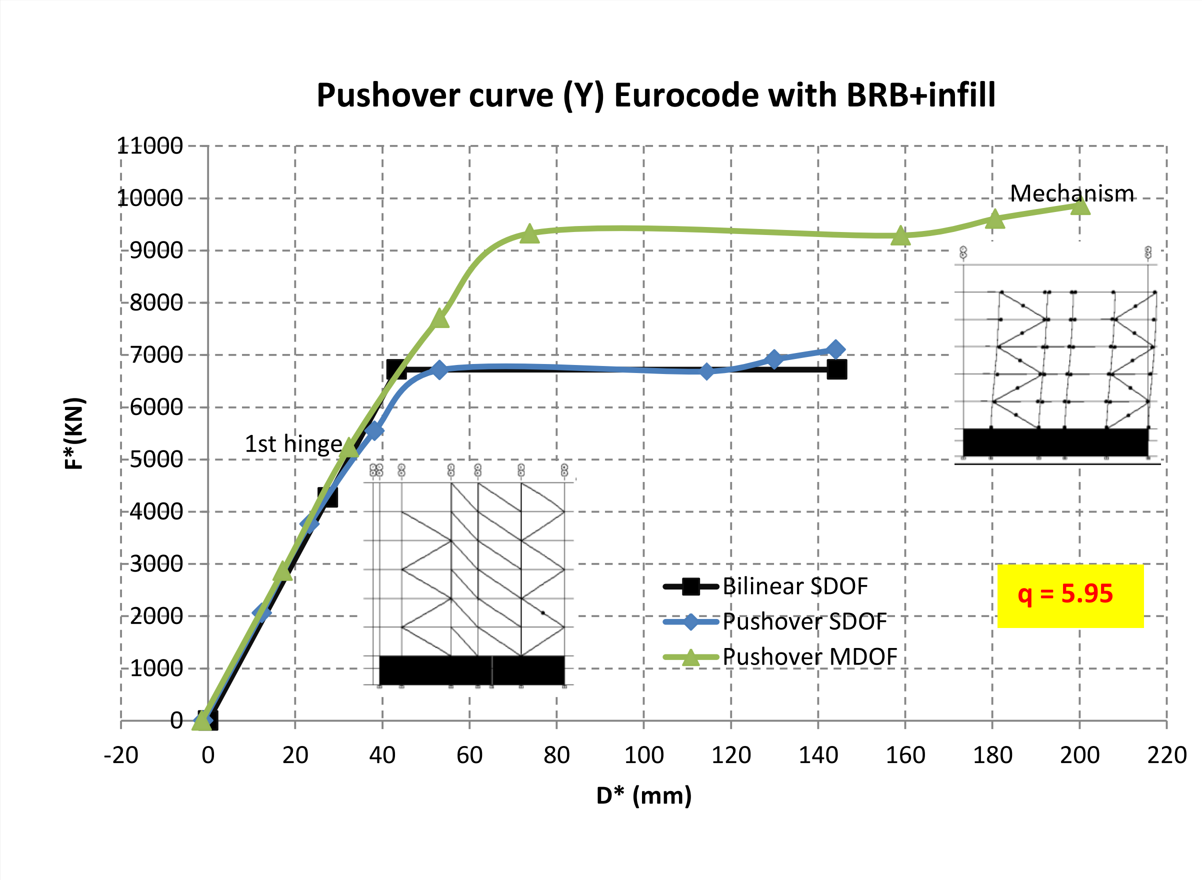


Figure 89: Bilinearized curve Y direction with BRB

9.4 Performance Evaluation of the Retrofitted Building with BRB

The pushover analysis above gives a behavior factor of **5.57** in X-direction and **5.95** in Y-direction which is more than code value (**4.68 and 5.0**). The verification of the performance level can further be done by ADRS curve.

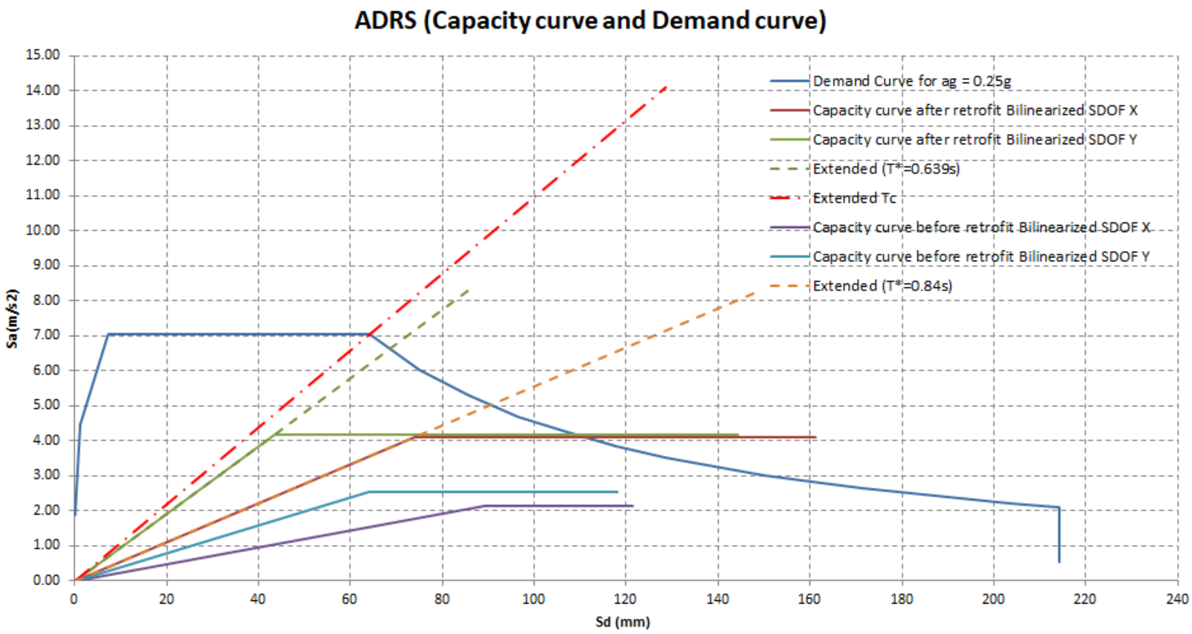


Figure 90: ADRS Curve for building after BRB retrofitting

m* 1706.01 ton

Capacity curve after retrofit

Bilinearized SDOF X

	d*y=Sd	F*y	Sa
	mm	KN	m/s2
Δ_0	0	0	0
Δ_y	74	7022	4.11613
Δ_u	161	7022	4.11613

Extended (T*=0.84s)	
Sd	Sa
0.000	0.000
73.775	4.116
147.549	8.232

Sae(T*)	Sde (T*)=de_{max} [mm]
5.0	90
Intersection of Extended T* with Demand curve	
Iv	
0.56	VERIFIED

m* 1603.49 ton

Capacity curve after retrofit

Bilinearized SDOF Y

d*y=Sd	F*y	Sa
mm	KN	m/s2
0	0	0
43	6723	4.19259
144	6723	4.19259

Extended (T*=0.639s)	
Sd	Sa
0.000	0.000
43.328	4.193
86.656	8.385

Sae(T*)	Sde (T*)=de_{max} [mm]
6.6	69
Intersection of Extended T* with Demand curve	
Iv	
0.48	VERIFIED

Figure 91: Verification checks for the building performance with BRB

From the ADRS curve the performance point of the structure is determined where the capacity curve meets the demand curve. As it is seen in above fig: 91 the vulnerability index Iv now is less than 1.0. The ratio improved by 45 ~ 55% with the use of buckling restrained braces as retrofitting solution. The ductility and strength of the structure were also found to be improved extensively. Similar results were found in one of the research for PROHITECH project (D'Aniello, Mazzolani, & Della Corte, 2009) .

Table 108: Seismic parameters comparison

Case	Δ_s (mm)	Δ_y (mm)	Δ_{max} (mm)	V_s (KN)	V_y (KN)	V_{max} (KN)	μ	Ω	q
X – direction without infill	55	89	121	3194	5128	5583	1.36	1.61	2.19
X – direction with infill	37	70	110	2399	4508	4989	1.56	1.88	2.94
X – direction with BRB	**25**	**74**	**161**	**2752**	**7022**	**7967**	**2.18**	**2.55**	**5.57**
Y – direction without infill	38	64	118	2505	4159	4480	1.85	1.66	3.07
Y – direction with infill	32	62	76	4062	7784	8371	1.22	1.92	2.35
Y – direction with BRB	**23**	**43**	**144**	**3765**	**6723**	**7103**	**3.33**	**1.79**	**5.95**

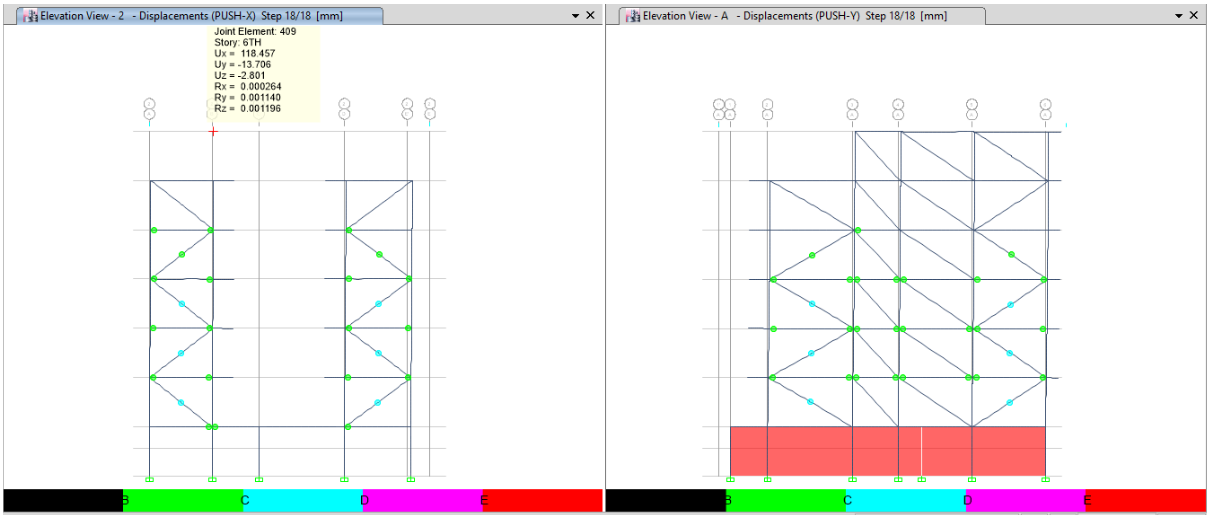


Figure 92: Deformation and distribution of plasticity at target displacement

The final state after pushover analysis at target displacement shows that the building after BRB retrofitting is now safe and within performance requirement as no plastic mechanism have formed and the hinges formed are in beams and braces which are within life safety limit for the given earthquake hazard.

Drift Check after retrofitting at Target Displacement (D_t)

Table 109: Damage Limitation check after retrofitting (Eurocode)

Storey	Storey Displacement, d_s(mm) at D_t		d_r (m), max		h (m)	ν	α = d_r* ν/h		Remarks
	X-Dir	Y-Dir	X-Dir	Y-Dir			X-Dir	Y-Dir	
6TH	118.872	93.531	0.0035	0.0036	3.657	0.5	0.000	0.000	α (≤0.005)
5TH	122.421	89.906	0.0105	0.0078	3.657	0.5	0.001	0.001	
4TH	111.898	82.103	0.0192	0.0135	3.657	0.5	0.003	0.002	
3RD	92.711	68.598	0.0300	0.0216	3.657	0.5	0.004	0.003	
2ND	62.705	46.996	0.0354	0.0272	3.657	0.5	0.005	0.004	
1ST	27.26	19.769	0.0266	0.0192	3.657	0.5	0.004	0.003	
GROUND	0.632	0.536	0.0006	0.0005	3.657	0.5	0.000	0.000	

The damage limitation check is now satisfied for all floors at target displacement.

Table 110: Damage limitation comparison before and after retrofit (Eurocode)

Storey	α (without infill)		α (With infill)		α (With BRB retrofit)		Remarks
	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	
6TH	0.00077	0.00461	0.00331	0.00319	0.000	0.000	α (≤0.005)
5TH	0.00237	0.00422	0.00395	0.00248	0.001	0.001	
4TH	**0.00557**	**0.00589**	**0.00521**	0.00393	0.003	0.002	
3RD	**0.00691**	**0.00729**	**0.00622**	0.00487	0.004	0.003	
2ND	**0.00742**	**0.00749**	**0.00649**	**0.00548**	0.005	0.004	
1ST	**0.00518**	0.00474	0.00431	0.00382	0.004	0.003	
GROUND	0.00006	0.00006	0.00013	0.00006	0.000	0.000	

Table 111: Column Reinforcement Demand (mm^2): Comparison (Before and after retrofit)

Col Type	Provided	IS before retrofit	IS after retrofit	EC before retrofit	EC after retrofit
C1	1256.64	720	720	900	900
C2	7147.12	**7140**	5217	5746	4045
C3	6440.26	6118	4315	4714	3285
C4	5890.49	**6014**	4477	4712	3829
C5	4830.20	**5217**	3526	3154	2560
C6	4830.20	3091	2463	3502	2608

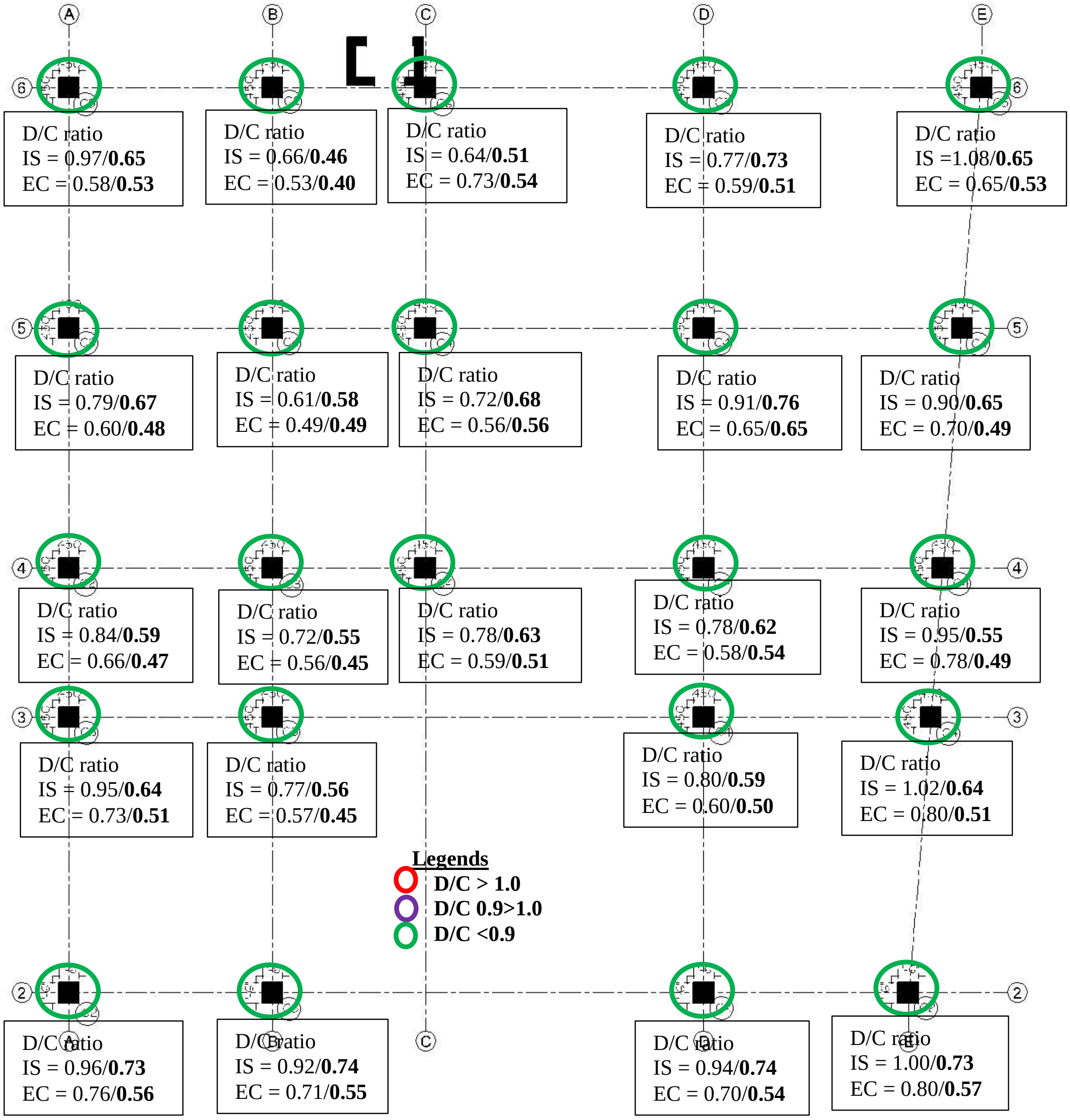


Figure 93: Column reinforcement demand/capacity ratio before and after retrofit comparison

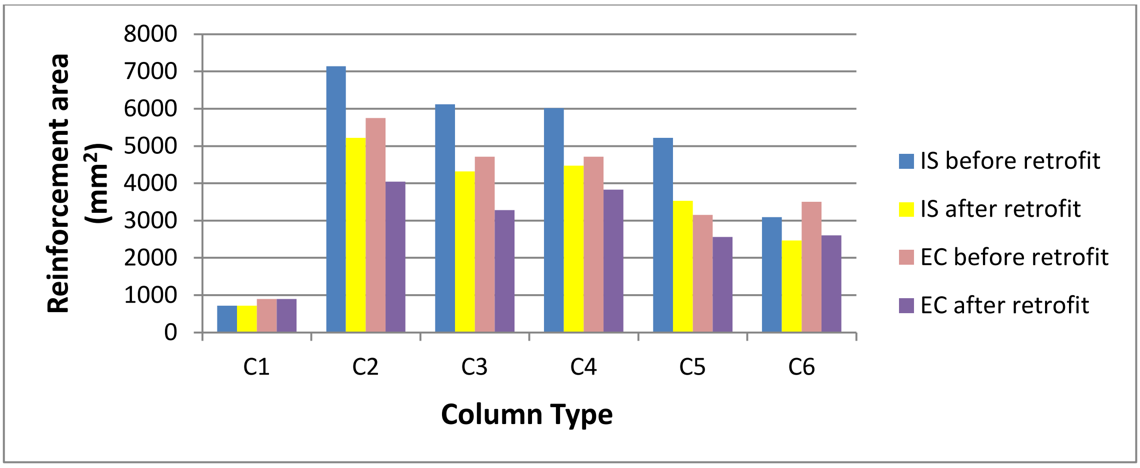


Figure 94: Column Reinforcement Comparison (Before and after retrofit)

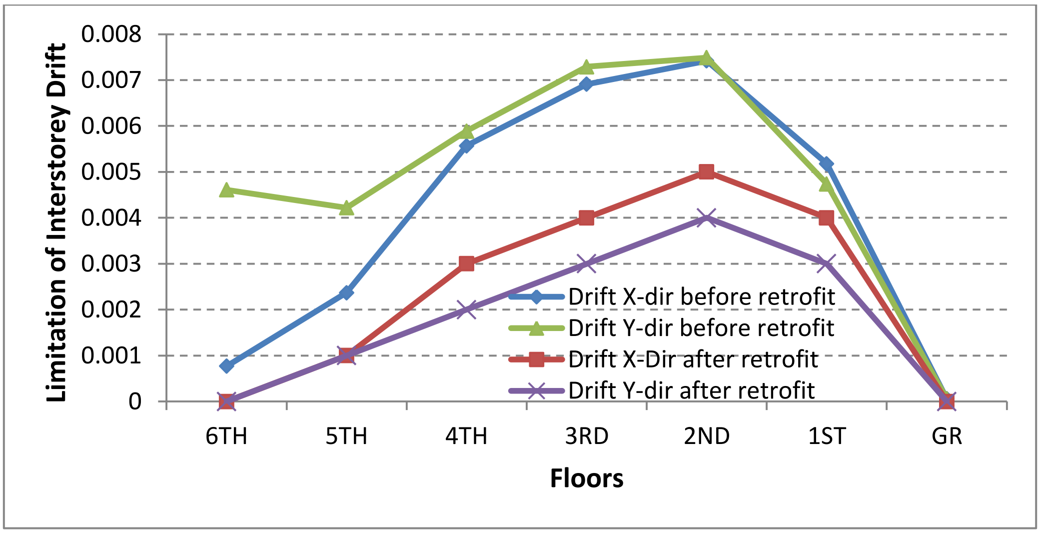


Figure 95: Interstorey Drift Comparison (Before and after retrofit)

Table 112: Beam Reinforcement Demand (mm^2): Comparison (Before and after retrofit)

Beam grid	Provided	IS before retrofit	IS after retrofit	EC before retrofit	EC after retrofit
A(2-3)	1884.96	1545	1145	1510	898
A(3-4)	1884.96	1635	1019	1492	899
A(4-5)	1884.96	1495	1209	1454	959
A(5-6)	1884.96	1377	940	1363	708
6(A-B)	1570.00	1019	639	869	555
6(B-C)	2415.00	2028	1843	2097	1832
6(C-D)	1344.00	1192	1067	999	836
6(D-E)	1344.00	967	732	774	597

The column and beam reinforcement demand is now reduced after BRB retrofitting.

Thus it is possible to strengthen and provide energy dissipation capacity to the case study building using all steel buckling restrained brace (BRB) solution. In the numerical analysis done by (Dubina, Bordea, & Stratan, 2009) for their research, they have suggested to associate BRB retrofitting with local FRP confinement of columns at least (confinement of beams would be beneficial, too) for better performance level.

10.0 CONCLUSIONS

The seismic codes commonly used in Nepal, which are the Indian Code (Ch.4) and the Nepal Code (Ch.6) are compared with the Eurocode 8 (Ch.5) in this thesis and they are applied to a selected building made of reinforced concrete (Ch.3). From this comparative study the following conclusive points can be drawn out.

- The design base shear force evaluated by Eurocode method of analysis is higher than the one coming from the Indian and Nepal code (see comparison in Ch.7). It is also higher in terms of lateral displacement and inter-storey drift. The reason is due to the use of a lesser zone seismic intensity factor (Z/2=0.18g) which represents the design base earthquake (DBE) in Indian code, whereas the Eurocode uses a value of peak ground acceleration PGA = 0.25g. Also the response reduction factor used in Indian code is higher than the 'q' factor of Eurocode, being them equal to **5** and **4.68** respectively.
- In terms of design of frame elements, the reinforcement demand in columns and beams is higher for the Indian code design. This is because the design load combination factor adopted by the Indian code is higher than the one of the Eurocode and Nepal codes. It helps to compensate the effect of lower design lateral forces.
- The analysis of the effect of infill walls on the lateral load resistance was also done in this thesis, showing a reduction in reinforcement demand for both Indian and Eurocode. The reduction was up to 35% for column reinforcement with Indian code and 18% with Eurocode. Although the use of infill wall is economical in terms of reinforcement, Eurocode does not advice designer to consider it, as the effect of infill remains until the walls cracking, after which the frames behave without infills resulting in a reduction of the building lateral capacity.
- From the comparative study of the three examined seismic codes, it can be observed that, for seismic analysis of building structures, the Eurocode describes the whole process in more details and considers more clearly the structural effects in terms of regularity, eccentricity, P-delta effects, behavior factors etc.
- Eurocode also clearly describes the process to perform non-linear pushover analysis with the N2 target displacement method and describes the effect of infill masonry walls as well. In the Indian code there is no description about infill wall effects and non-linear analysis, but it refers to other developed codes, like ACI and FEMA.
- Nepal code is less detailed than the above two codes in these matters.
- In conclusion of the preliminary part of the thesis, it can be observed that it is not practical to directly compare two codes and to assume in general that one of these two is faulty, just because principles and assumptions considered are different. Also for the simple case study building, it is not enough to apply the code provisions, but it should be more effective to apply different scenarios by using more advanced methods of analysis.
- The analysis of the case study building shows that the damage limitation checks are not fulfilled for the Eurocode drift requirements. It means that the global performance is not

satisfied, although the members are verified. This is also in accordance with the results of the pushover analysis (Ch.8), where the actual behavior factor is much lower (**q=2.19**) than the codified one (**5.0 & 4.68**). The building vulnerability index (**Iv**) is found to be more than 1.0 for the given demand earthquake of 0.25g PGA. This suggests that the building needs to be retrofitted in order to improve its global behavior.

- The building is retrofitted by using the steel buckling restrained brace (BRB) technology (Ch.9). The BRB solution was chosen because of its beneficial characteristics, being dissipative, reversible and requiring an easy erection with a minimal modification of the existing structure. Being an "*all-steel*" solution, it is sustainable and environmental friendly. The most important result with use of BRB is to obtain a highly dissipative structure with improved ductility and strength in the overall behavior.
- From the pushover analysis of the building equipped with BRBF, it can be observed that an improved performance is achieved. The behavior factor q (the response modification factor R, according to the Indian code) is also improved by the use of BRB (**q=5.57** and **q=5.95**). These values are close to the ones recommended by various researches and enough to achieve the global performance level of the existing building, according to the examined codes. Also in the ADRS curve the vulnerability index value, which originally was greater than 1, dropped to < **0.6**, thanks to the use of BRB. This proves the effectiveness of choosing BRB as retrofitting solution in this study.

Considering all above points, it can be concluded that for a seismically active country like Nepal it is urgent to either upgrade its insufficient existing code or to substitute it with more sophisticated and coherent provisions for improving the earthquake resistant design regulation in this country.

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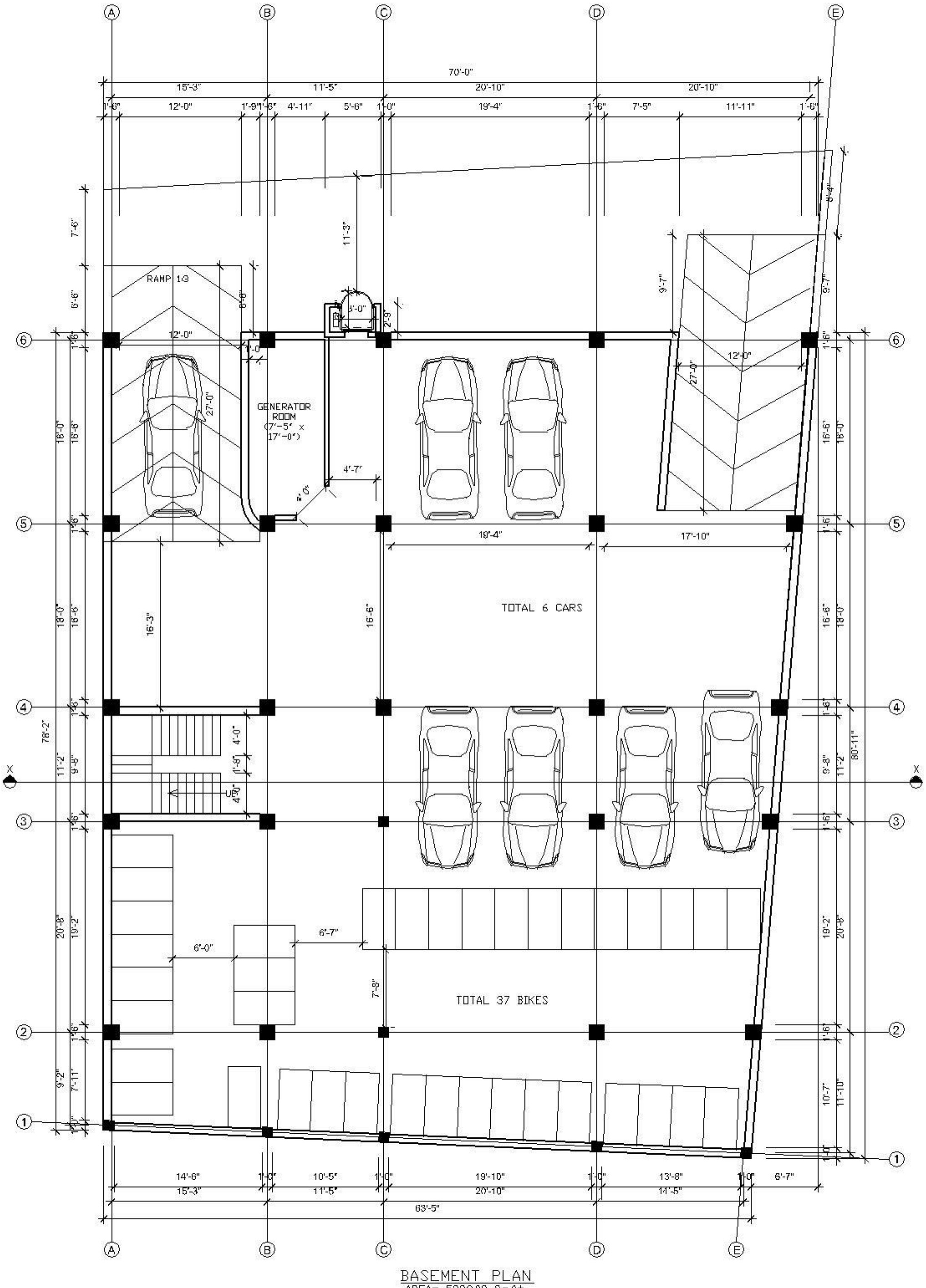
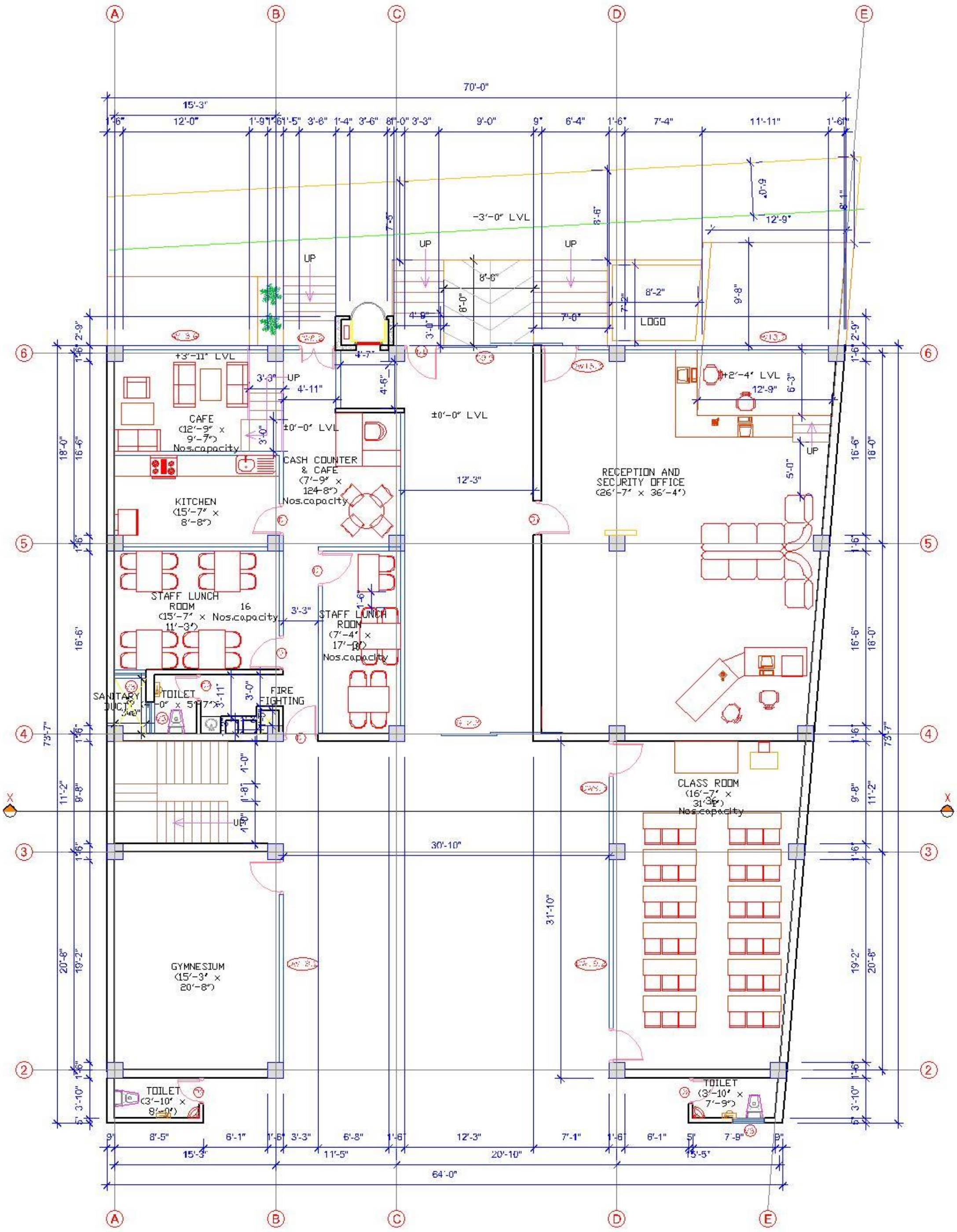
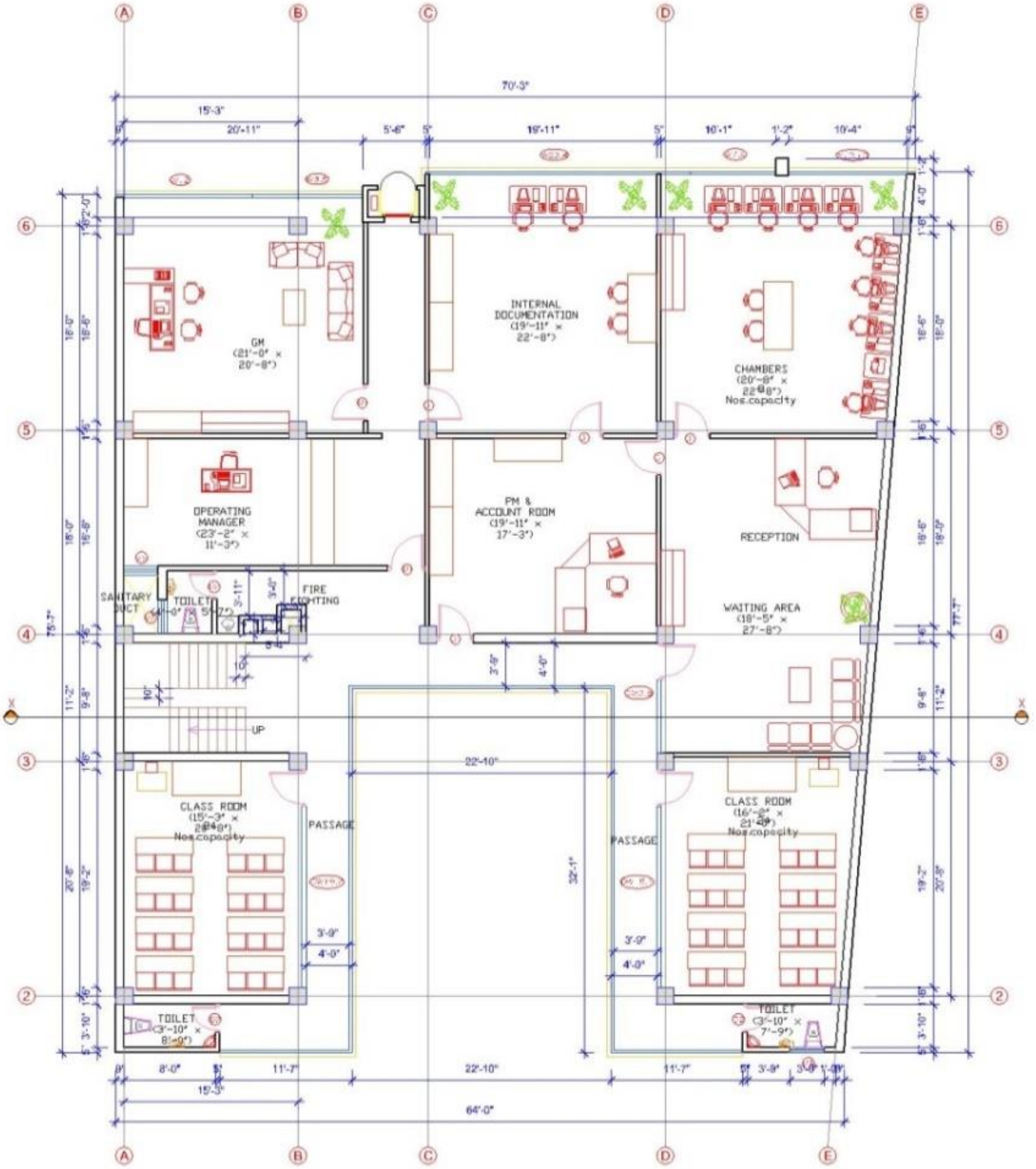
APPENDIX A: DETAIL DRAWINGS OF THE STUDY BUILDING

A1: Architectural Drawing





GROUND FLOOR PLAN
AREA= 3771.96 Sq.ft.
GCR=58.62%



FIRST FLOOR PLAN
AREA= 4431.22 Sq.ft.

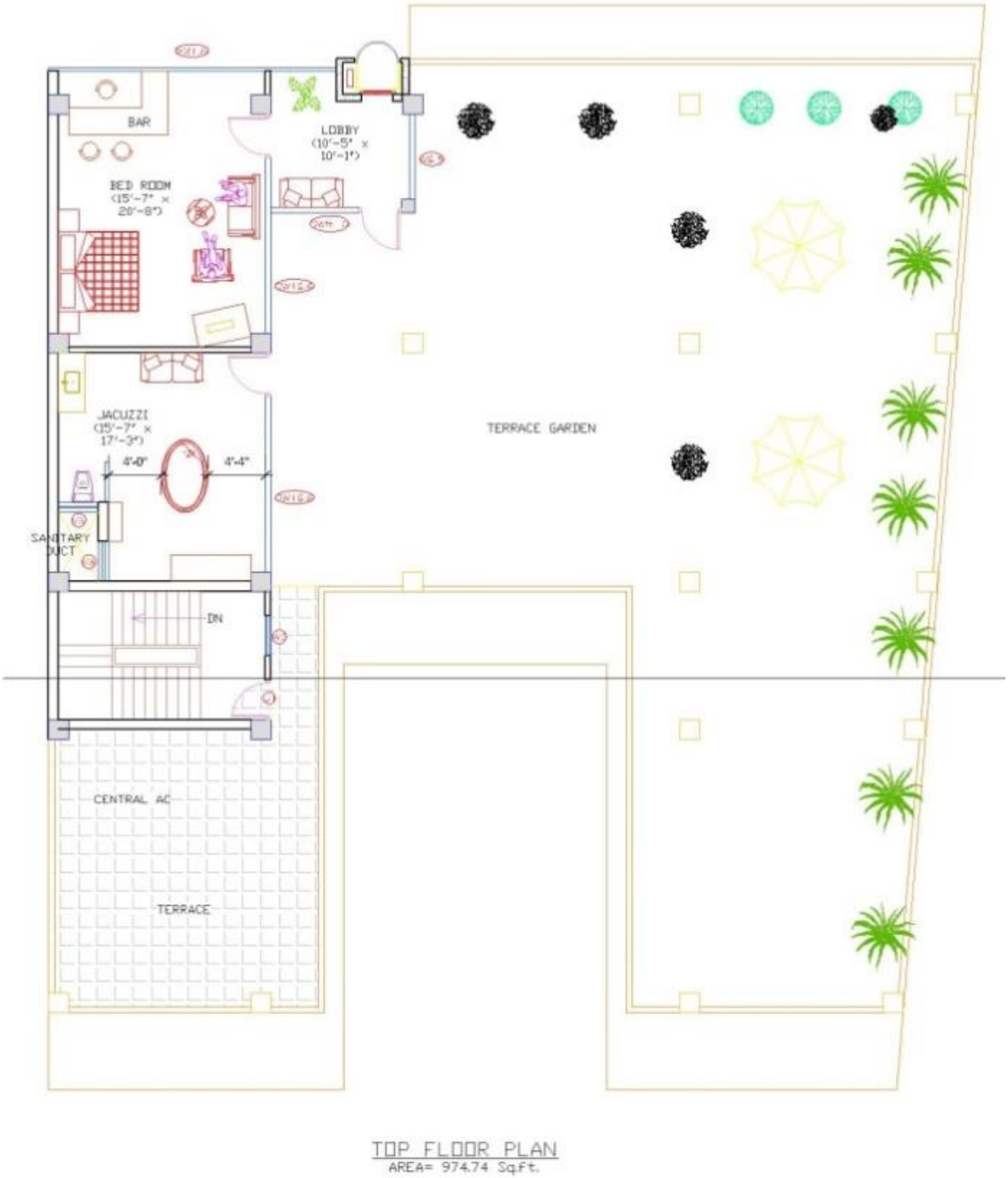
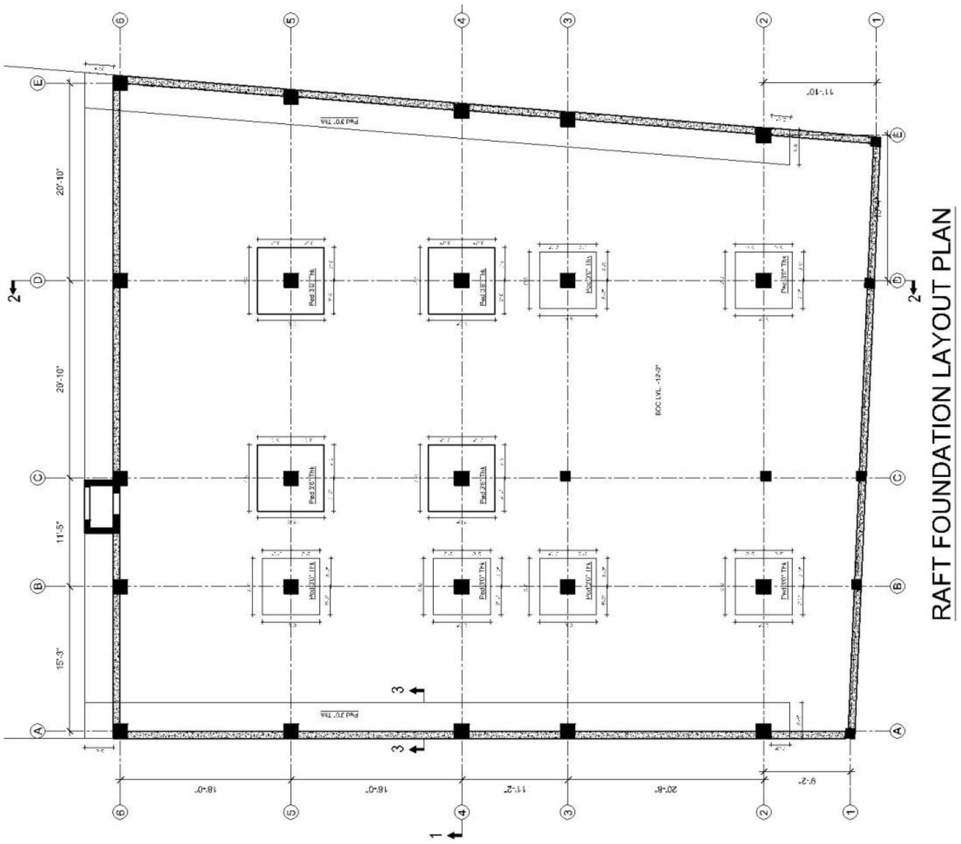
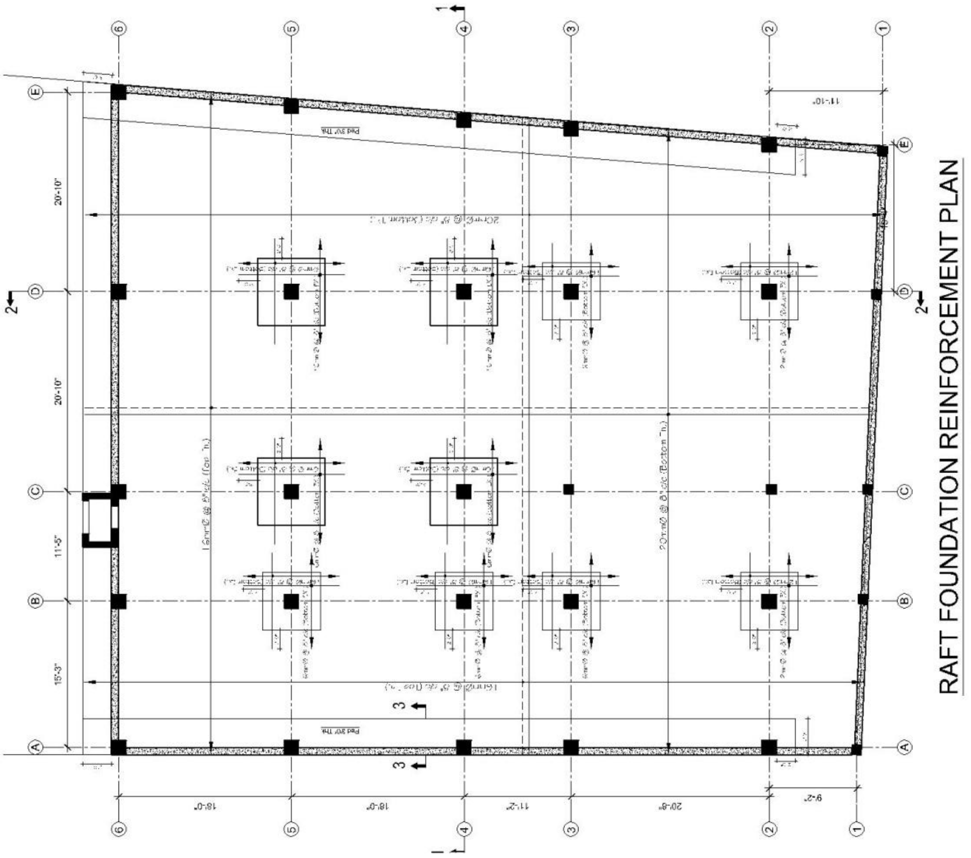


Figure 96: Plan Views

Figure 97: Elevation Views

A2: Structural Drawings





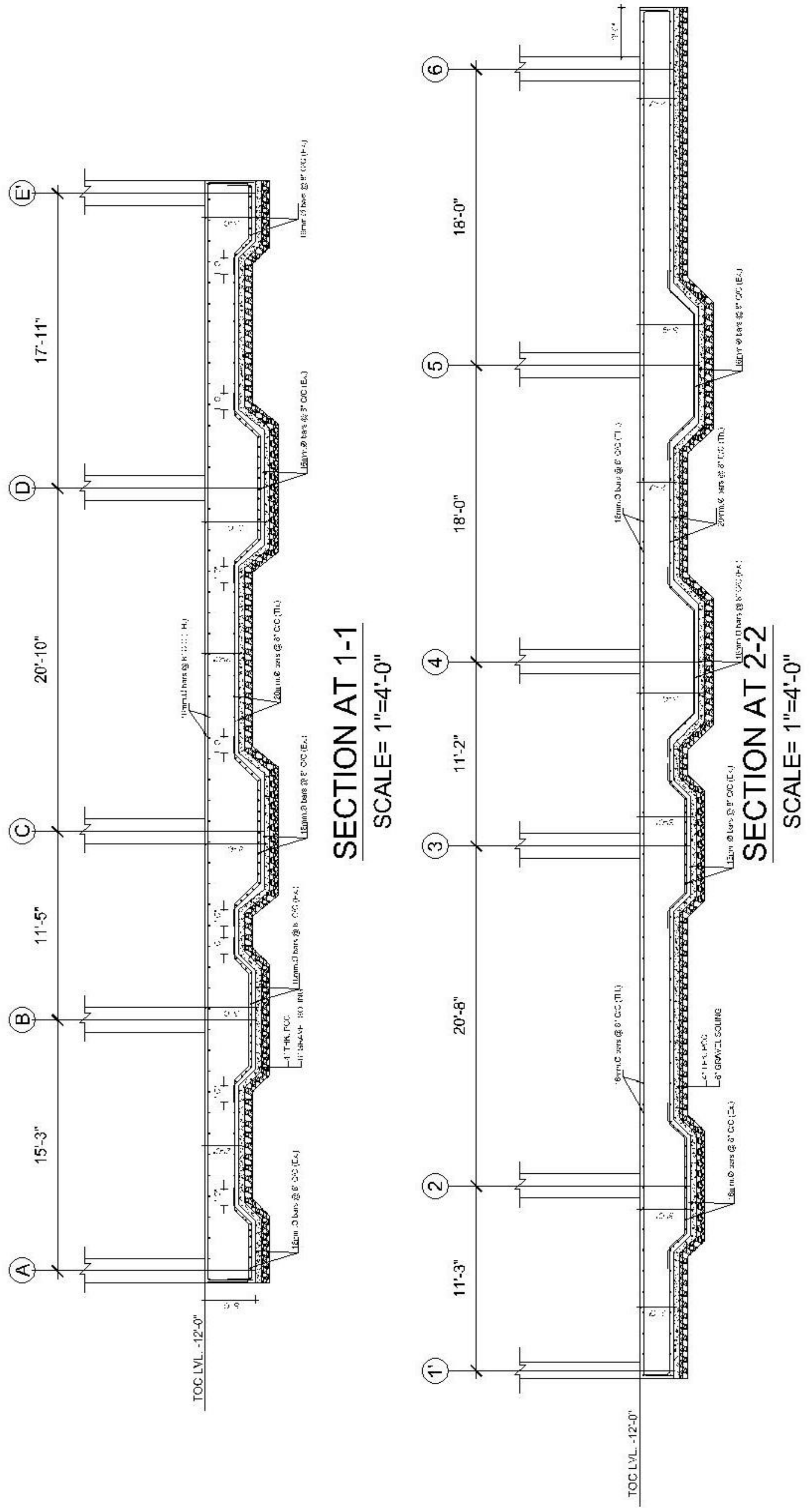


Figure 98: Foundation Details

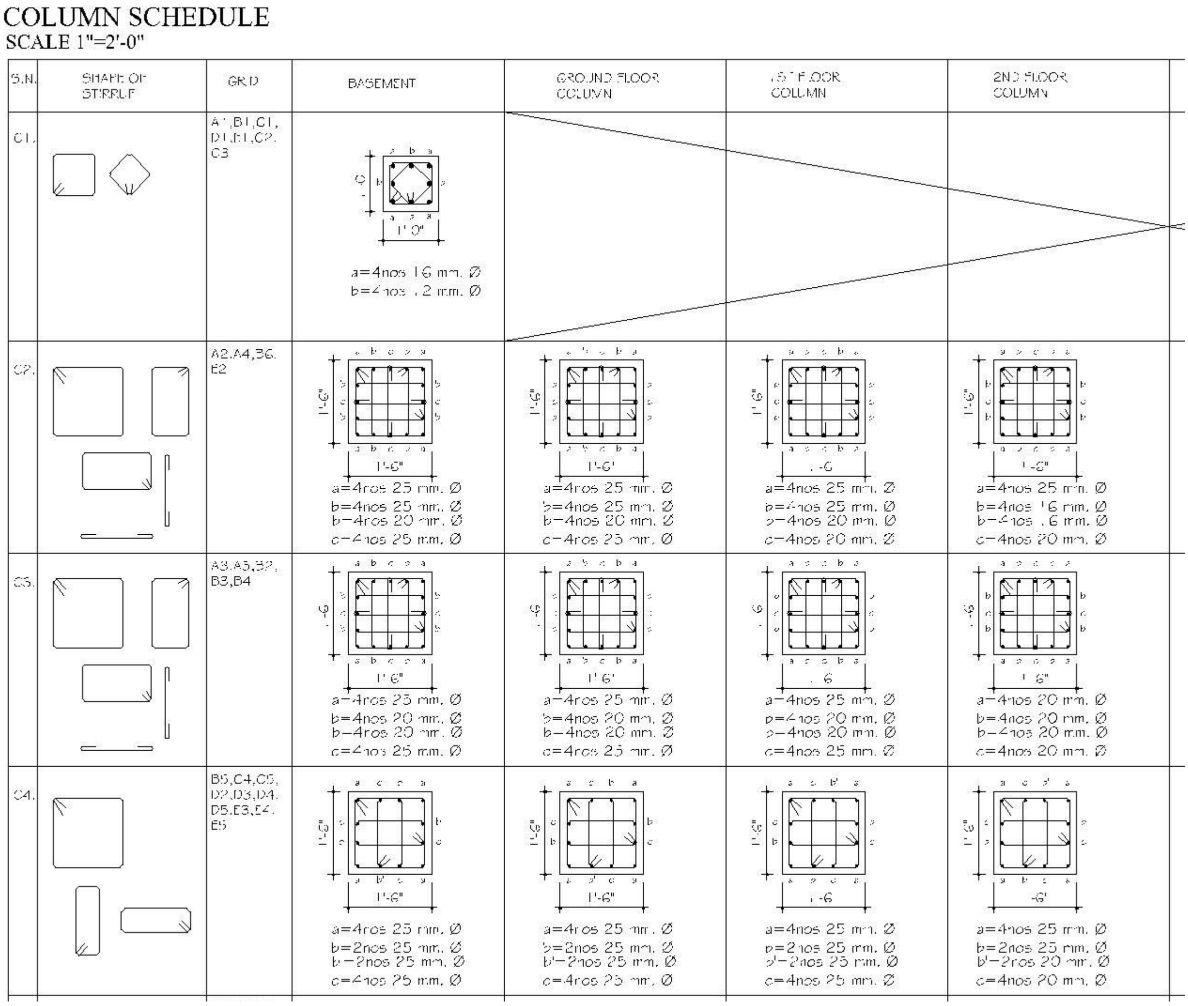


Figure 99: Typical Column Reinforcement Schedule

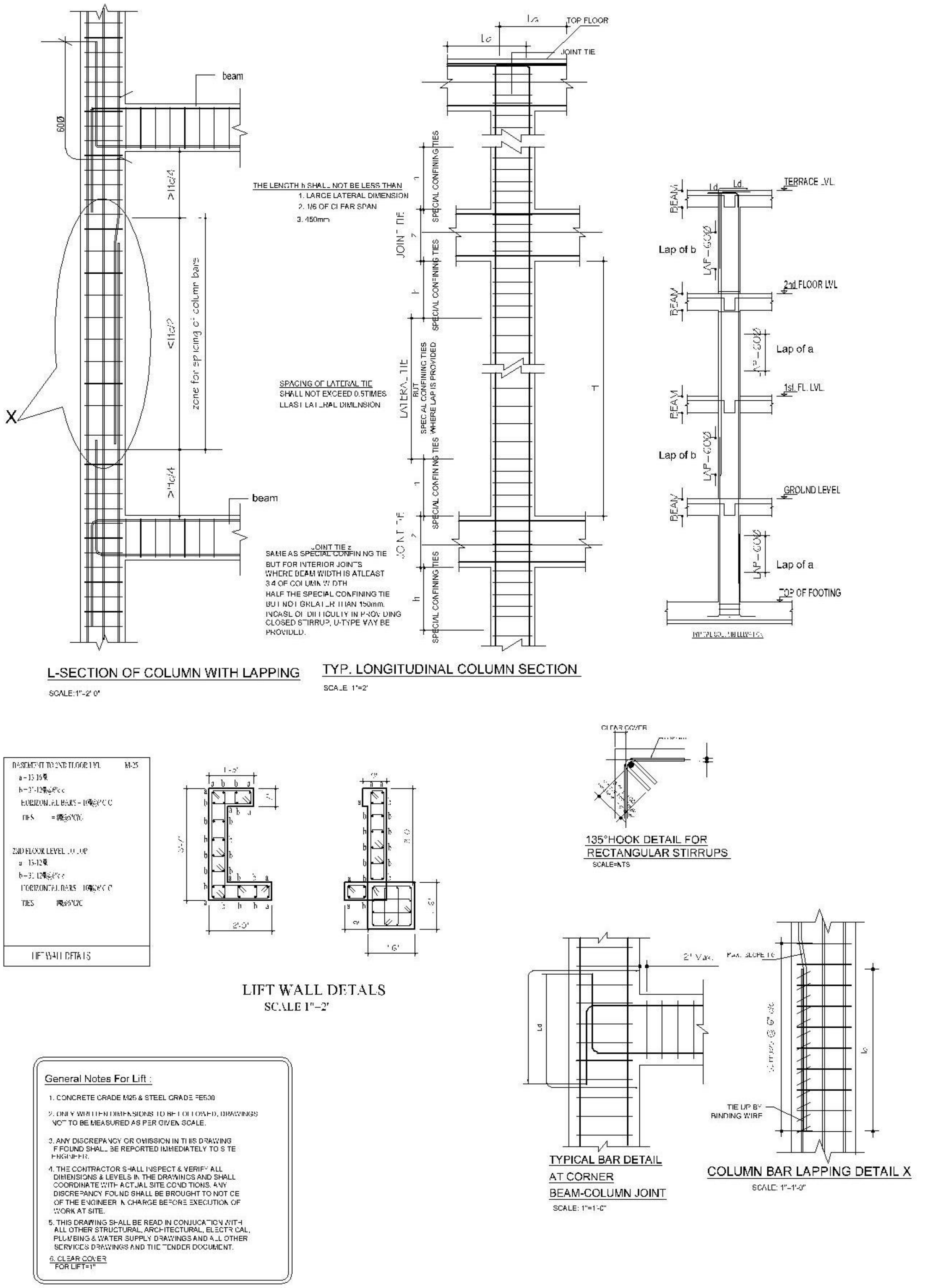


Figure 100: Column and Lift Details

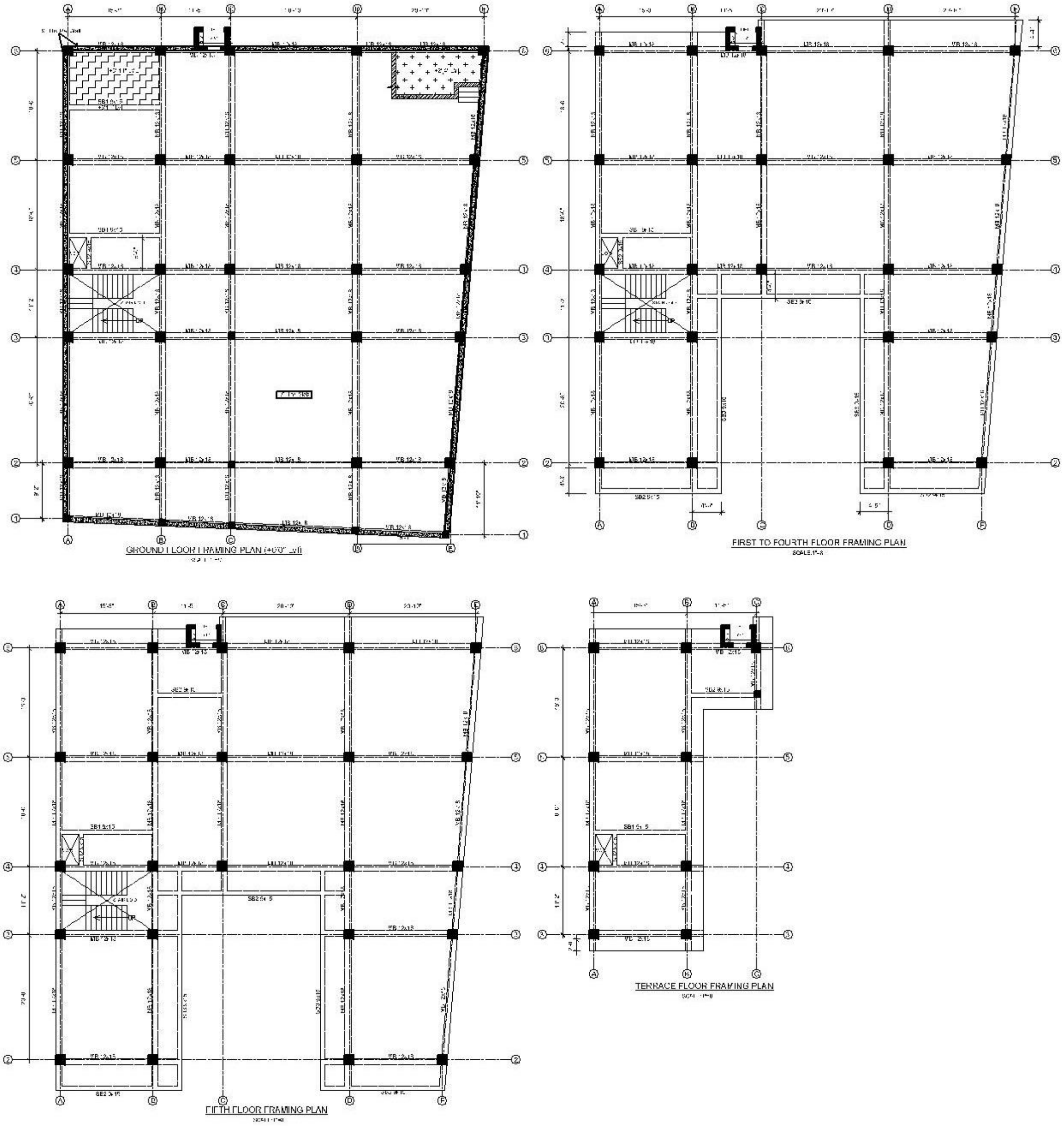


Figure 101: Beam Plans

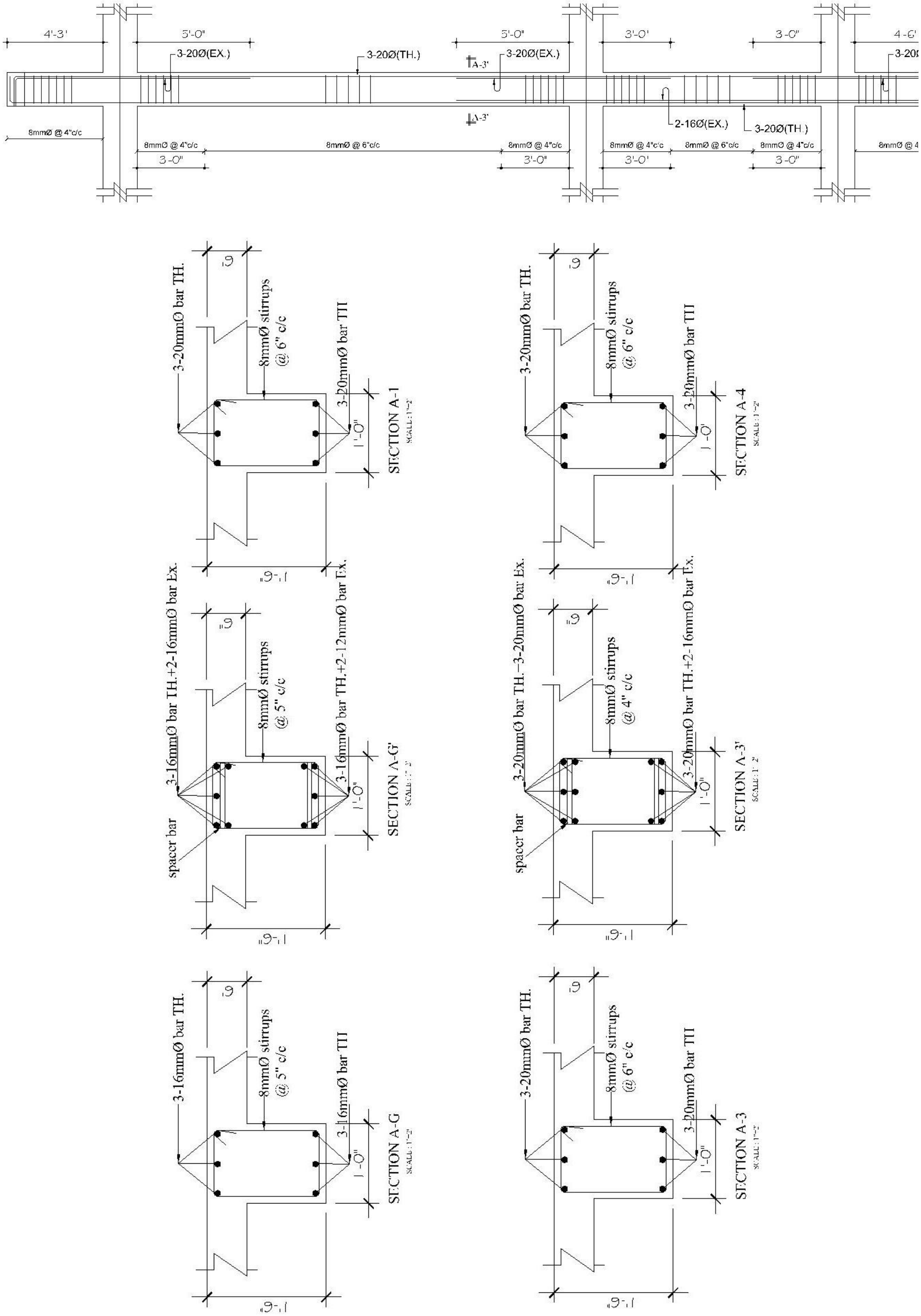
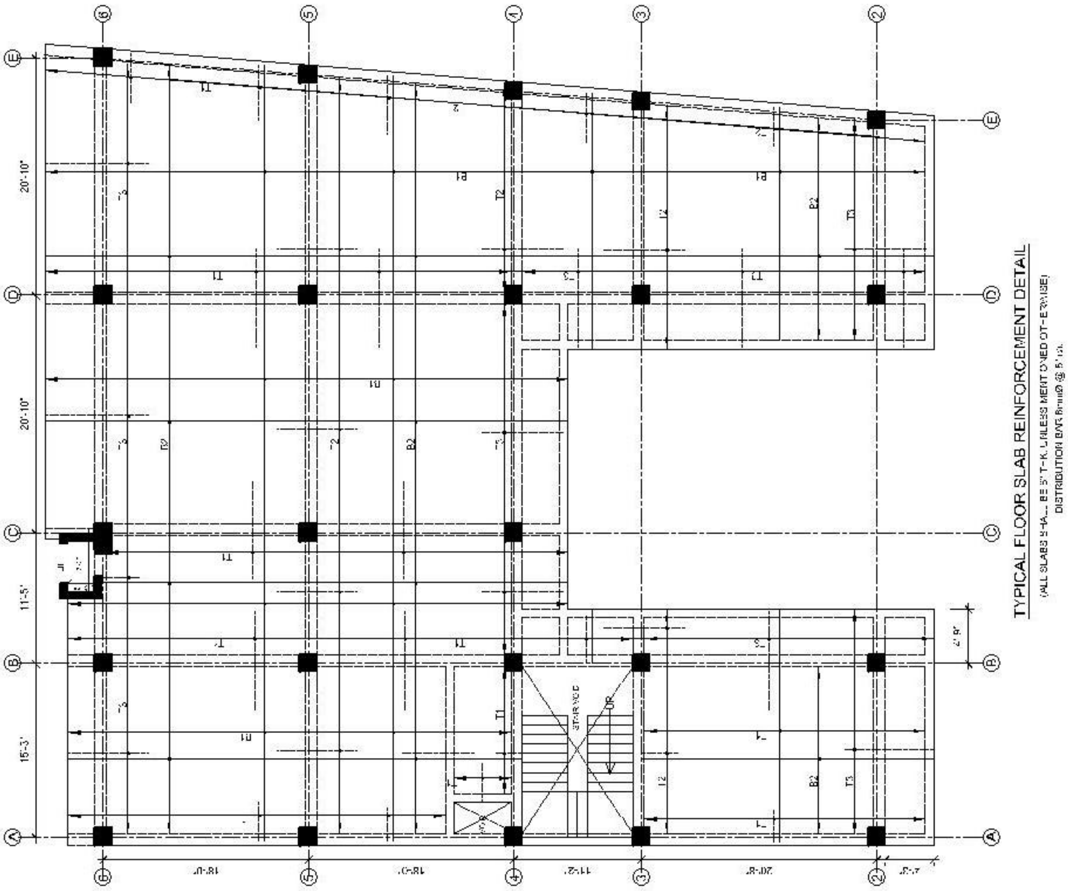
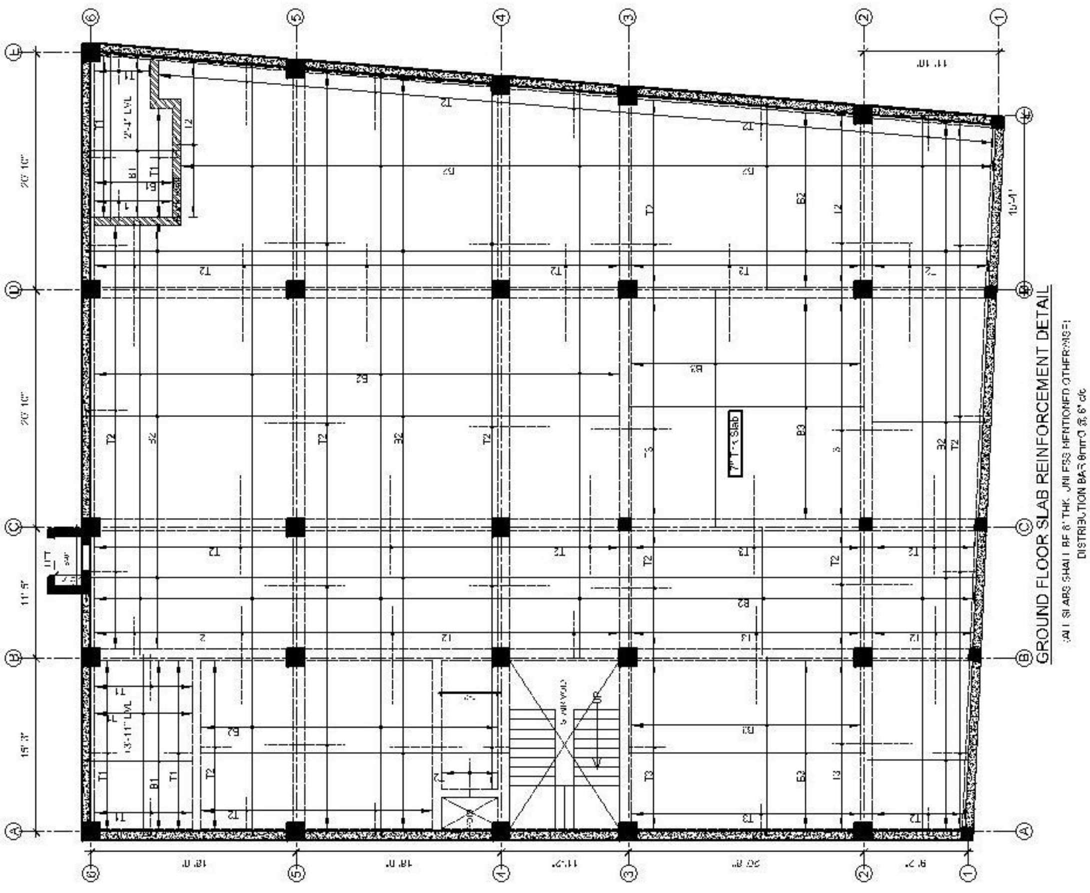


Figure 102: Typical Beam Reinforcement portion


TYPICAL FLOOR SLAB REINFORCEMENT DETAIL


GROUND FLOOR SLAB REINFORCEMENT DETAIL

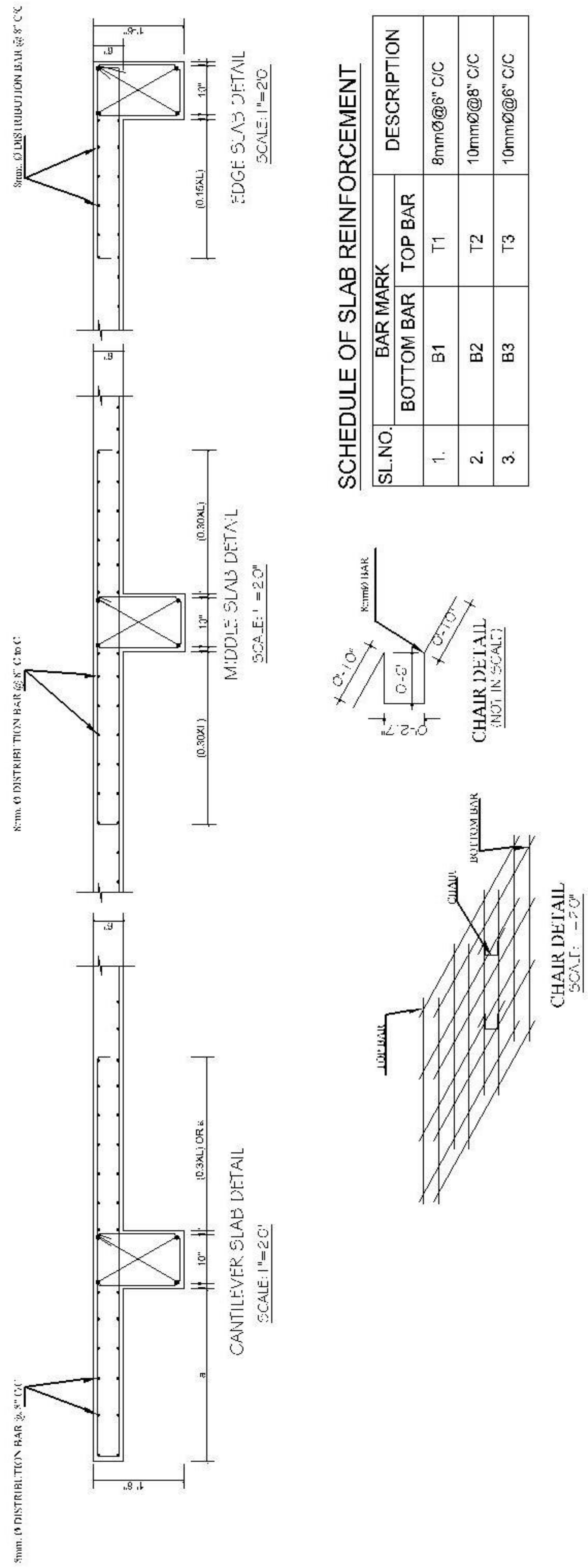


Figure 103: Slab Detail

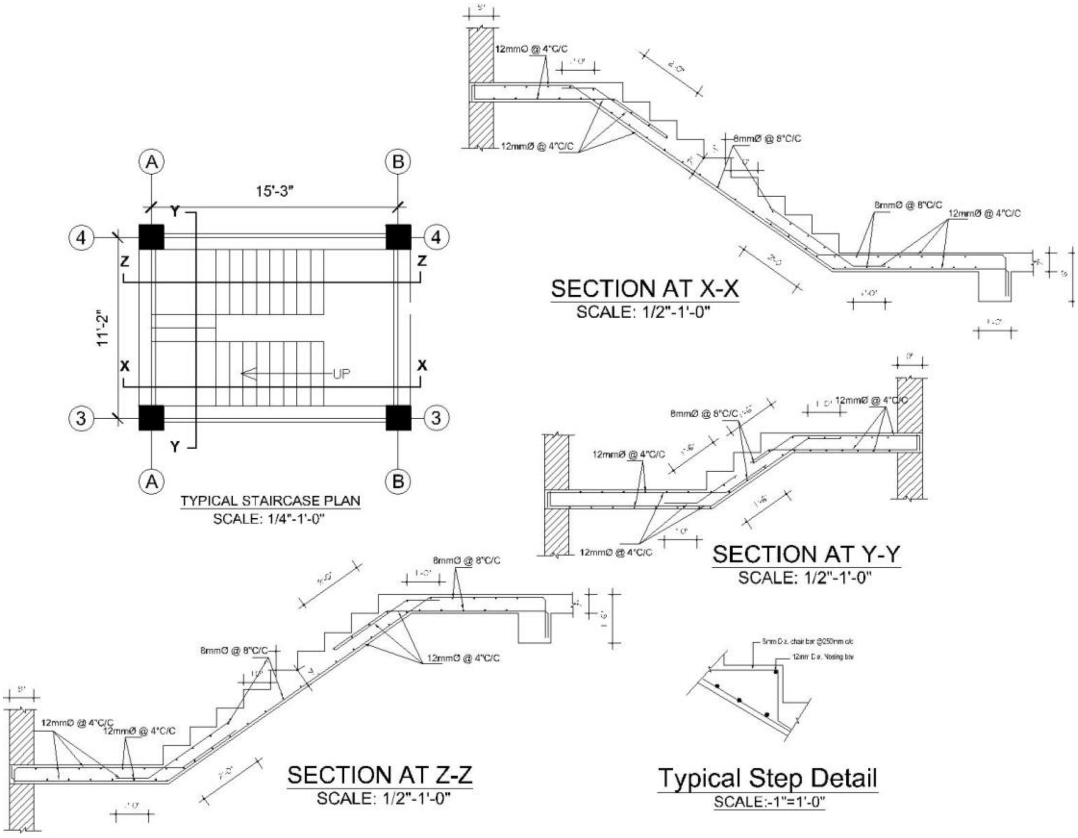


Figure 104: Staircase Detail

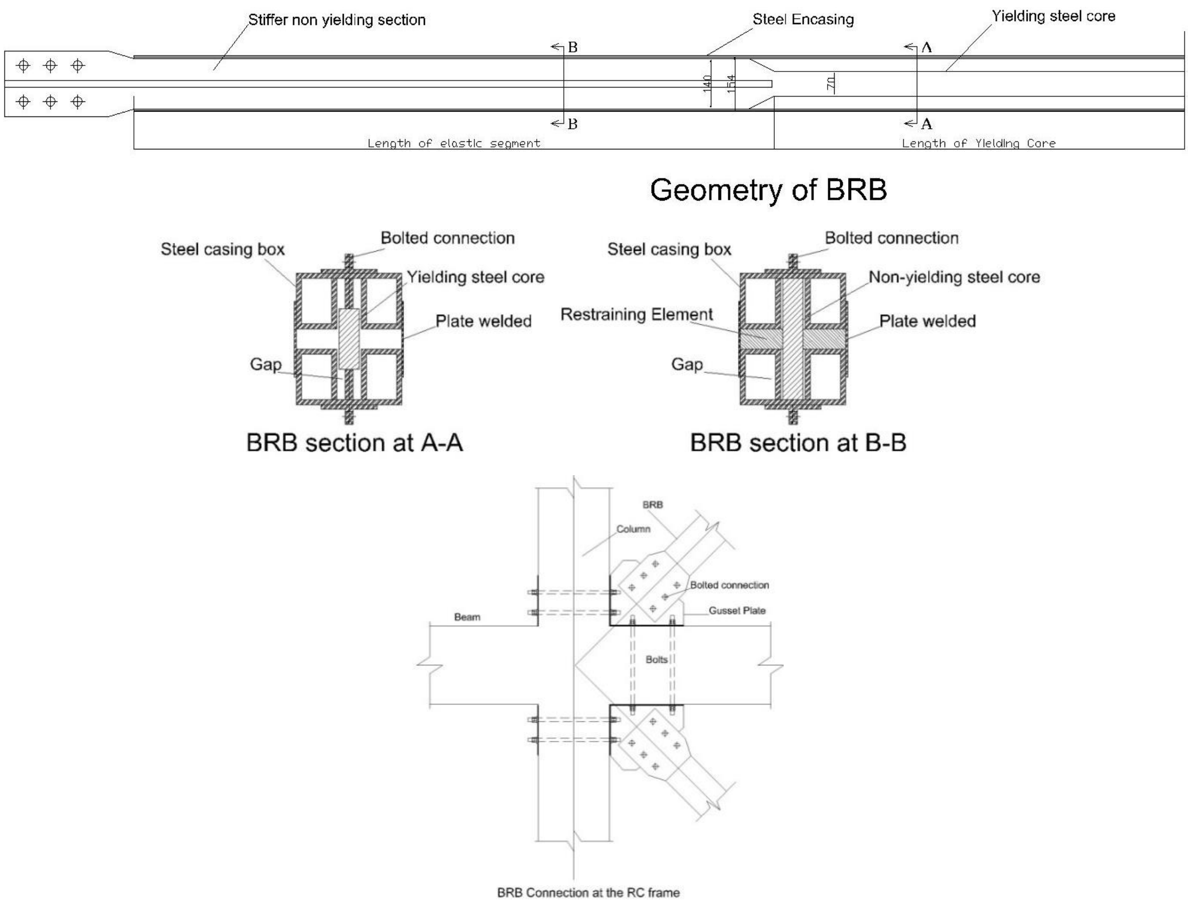


Figure 105: Typical Buckling Restrained Brace Schematic Diagrams

APPENDIX B: ETABS LINEAR ANALYSIS INTERNAL FORCE DIAGRAMS

(FRAME 5-5 Example)

B1: Bending Moment Diagrams (BMD)

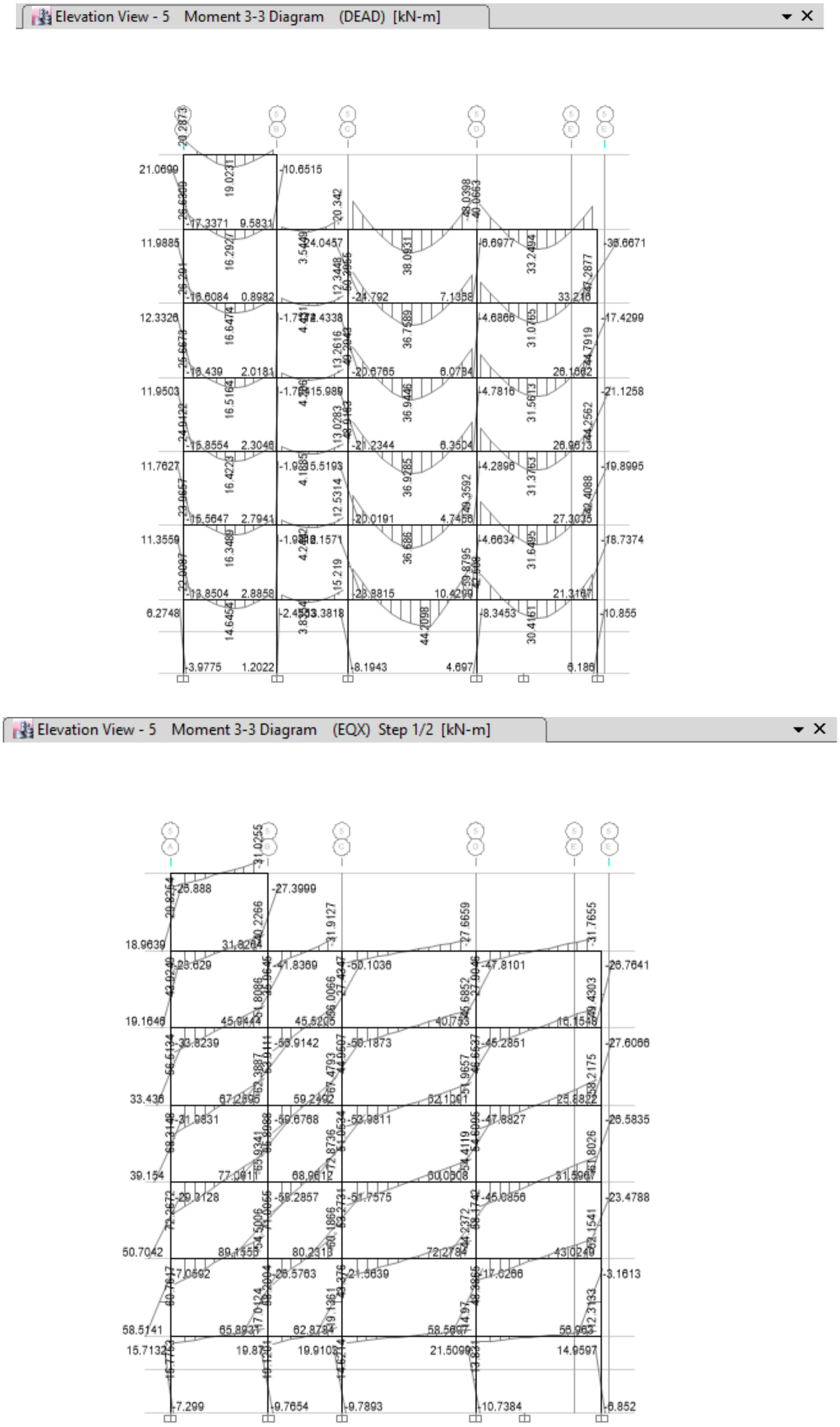


Figure 106: Typical BMD under gravity and lateral loads

B2: Shear Force Diagrams (SFD)

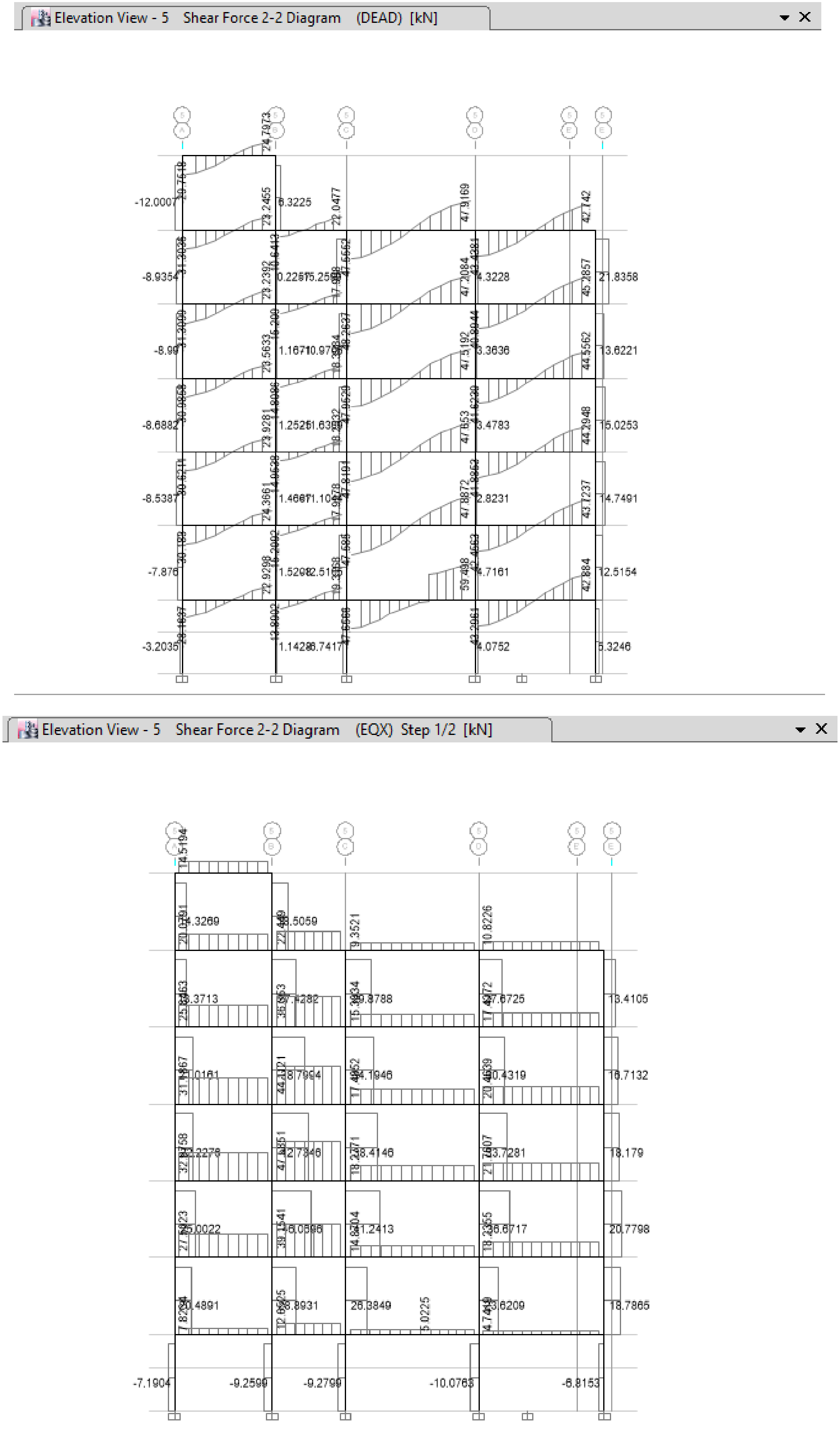
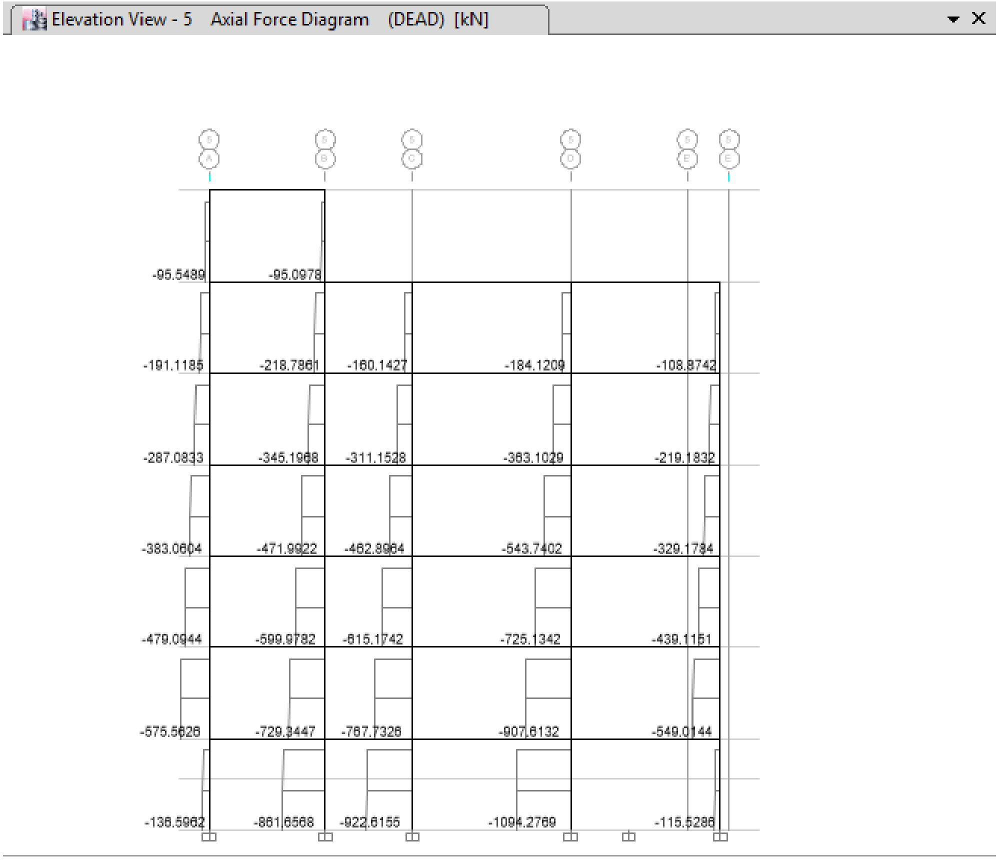


Figure 107: Typical SFD under gravity and lateral loads

B3: Axial Force Diagrams (AFD)



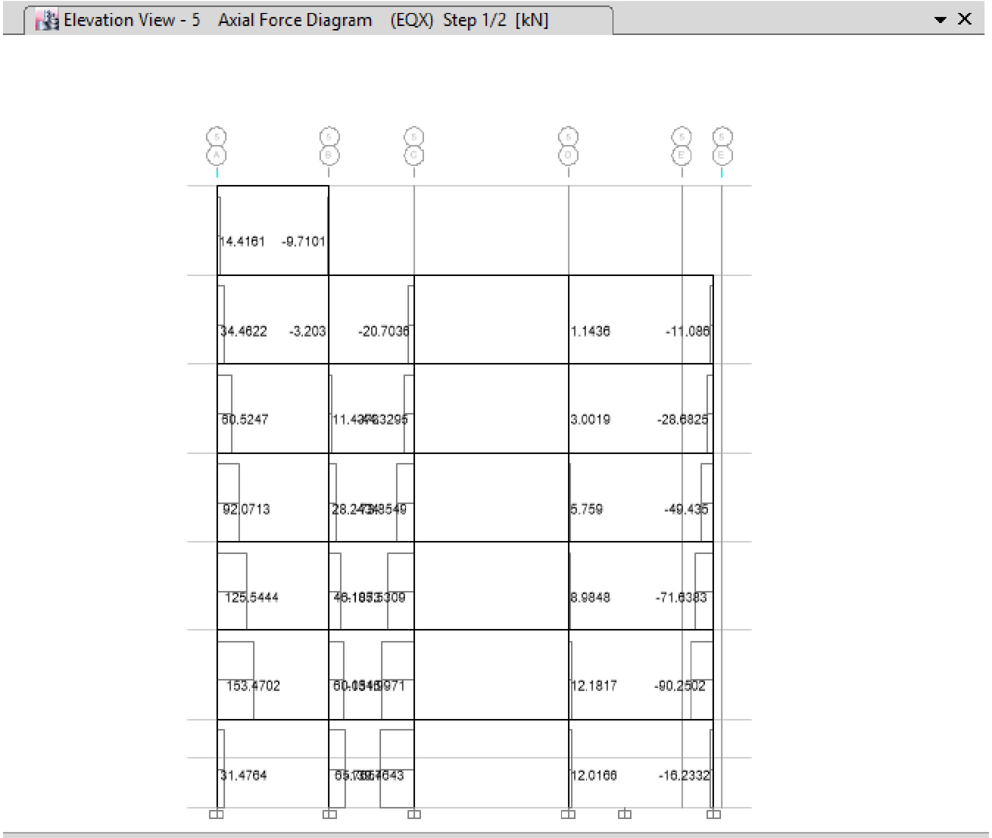
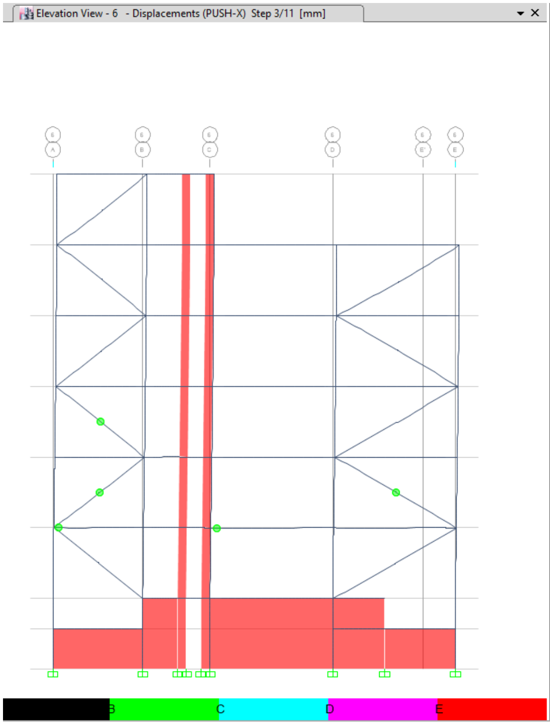
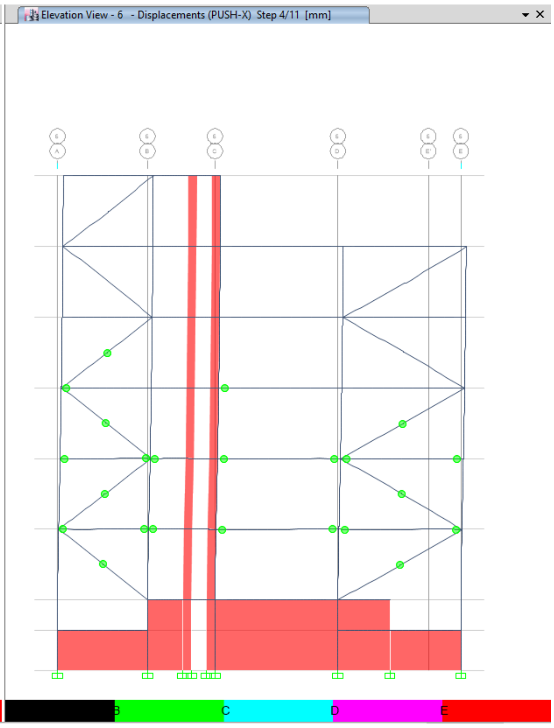


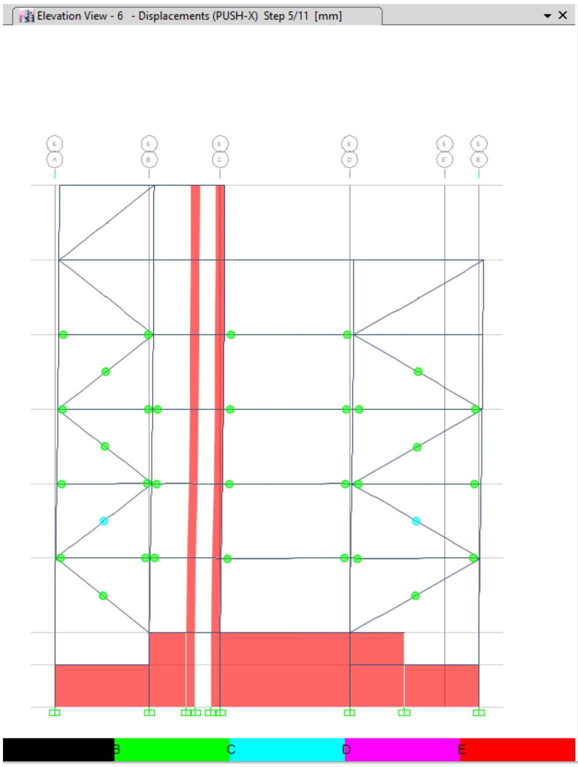
Figure 108: Typical AFD under gravity and lateral loads

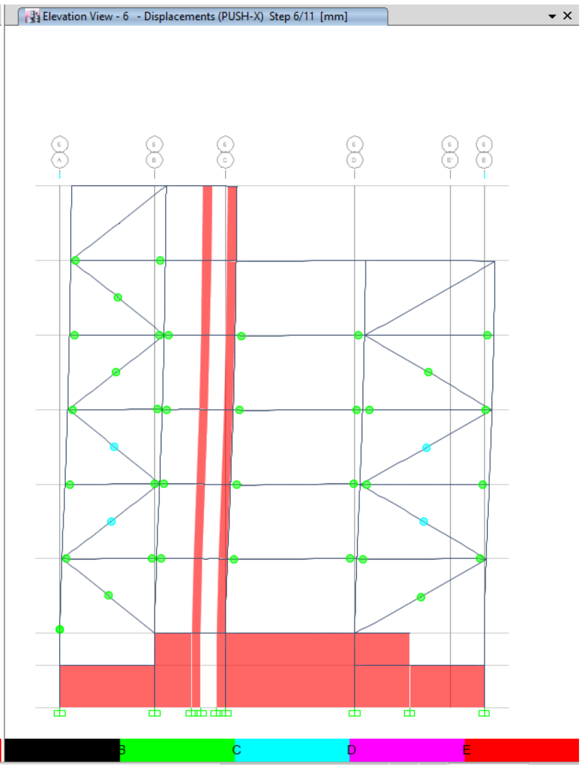
APPENDIX C: NON- LINEAR ANALYSIS PLASTIC HINGE FORMATIONS

(FRAME 6-6 Example)









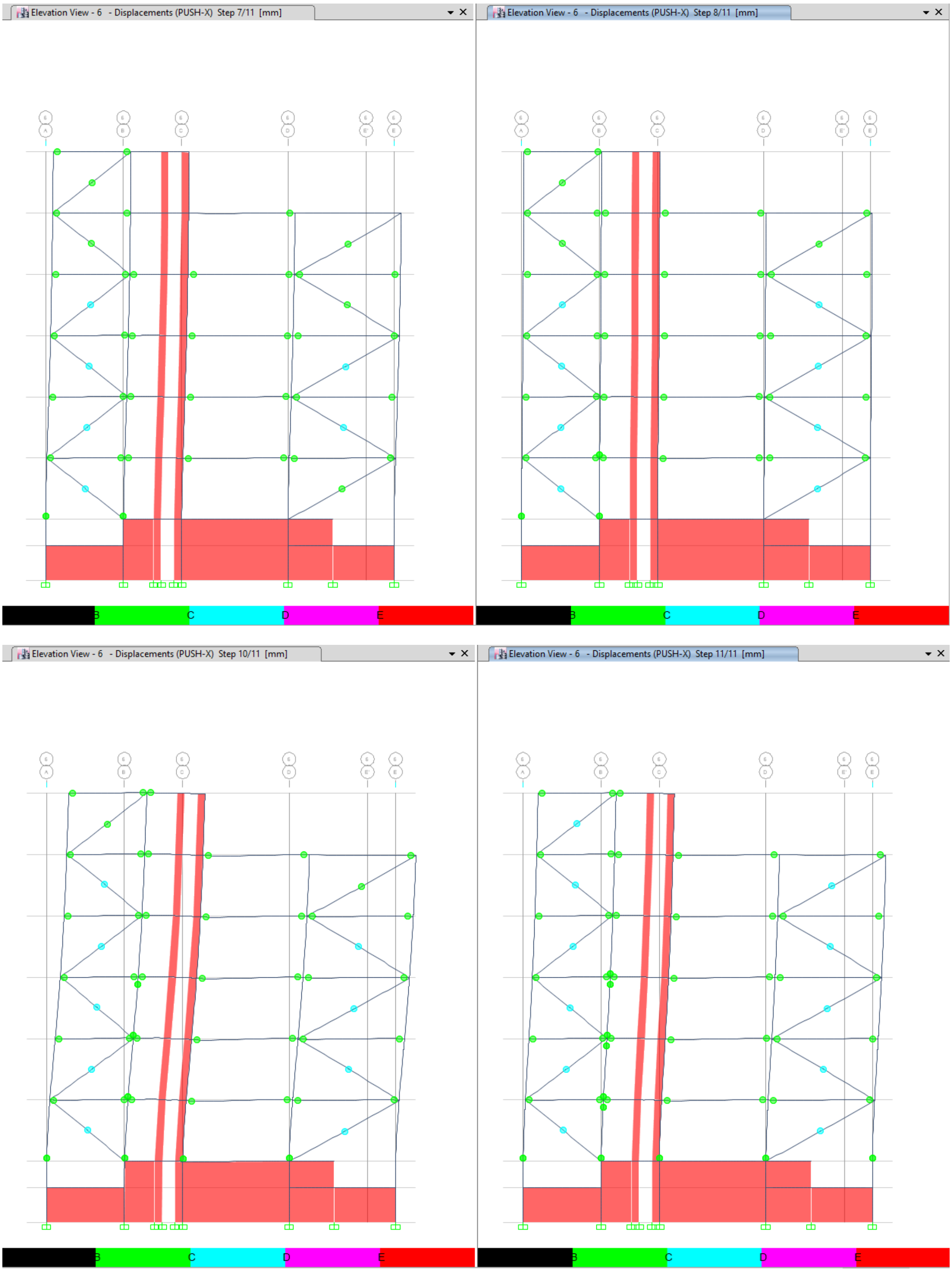
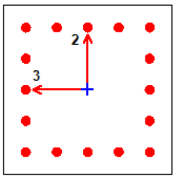


Figure 109: Plastic Hinge formation (Step 3- first hinge, step 11 - mechanism)

APPENDIX D: MEMBER DESIGNS

D1: Column Design

ETABS 2016 Concrete Frame Design (IS code)
IS 456:2000 Column Section Design (Column C2 before retrofit)



Column Element Details Type: Ductile Frame (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
1ST	C17	151	C2-B/G	UDCon8	0	3657.6	1

Section Properties

SD Section		dc (mm)	Cover (Torsion) (mm)
457.2		60.5	30

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
25000	25	1	500	500

Design Code Parameters

γ_c	γ_s
1.5	1.15

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M_2 kN-m	Minimum M_3 kN-m	Rebar % %	Capacity Ratio Unitless
1903.0368	90.406	-372.7952	41.1832	41.1832	3.42(O/S #35)	1.034(O/S #35)

Axial Force and Biaxial Moment Factors

	K Factor Unitless	Length mm	Initial Moment kN-m	Additional Moment kN-m	Minimum Moment kN-m
Major Bend(M3)	2.472854	3200.4	-130.9909	103.3805	41.1832
Minor Bend(M2)	0.831359	3200.4	51.9723	0	41.1832

Shear Design for V_{u2} , V_{u3}

	Shear V_u kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm²/m
Major, V_{u2}	108.1301	215.8808	69.6776	95.137	506.78
Minor, V_{u3}	110.5311	215.8808	69.6776	110.5311	506.78

Additional Moment Reduction Factor k (IS 39.7.1.1)

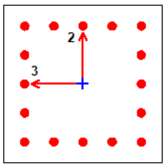
A_g cm²	A_{sc} cm²	P_{uz} kN	P_b kN	P_u kN	k Unitless
2090.3	71.5	5032.1082	1086.6889	1903.0368	0.79309

Additional Moment (IS 39.7.1)

	Consider M_a	Length Factor	Section Depth (mm)	KL/Depth Ratio	KL/Depth Limit	KL/Depth Exceeded	M_a Moment (kN-m)
Major Bending (M_3)	Yes	0.875	457.2	17.31	12	Yes	130.3516
Minor Bending (M_2)	Yes	0.875	457.2	5.82	12	No	0

O/S #35 Capacity ratio exceeds limit

ETABS 2016 Concrete Frame Design (Eurocode)
Eurocode 2-2004 Column Section Design (Column C2 before retrofit)



Column Element Details Type: DC High

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	SOM	LLRF
1ST	C17	C2-B/G	UDCon11		0	3657.6	Nominal Stiffness	1

Section Properties

SD Section		dc (mm)	Cover (Torsion) (mm)
457.2		60.5	30

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	E_s (MPa)	f_{yk} (MPa)	f_{ywk} (MPa)
31475.81	25	1	200000	500	500

Design Code Parameters

γ_C	γ_S	α_{CC}	α_{CT}	α_{LCC}	α_{LCT}
1.5	1.15	1	1	0.85	0.85

Axial Force and Biaxial Moment Design For N_{Ed} , M_{Ed2} , M_{Ed3}

Design N_{Ed} kN	Design M_{Ed2} kN-m	Design M_{Ed3} kN-m	Minimum M_2 kN-m	Minimum M_3 kN-m	Rebar % %	Capacity Ratio Unitless
1386.1377	-179.6311	-267.74	27.7228	27.7228	3.42	0.763

Axial Force and Biaxial Moment Factors

	M_{0Ed} Moment kN-m	M_{add} Moment kN-m	Minimum Ecc mm	β Factor Unitless	Length mm
Major Bend(M3)	-134.0378	0	20	1	3200.4
Minor Bend(M2)	-95.9986	0	20	1	3200.4

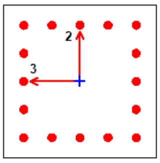
Axial Compression Ratio

Conc Capacity (α_{CC} *A*f_{cd}) kN	Compressive Ratio N_{Ed} /(α_{cc} *A*f_{cd})	Comp Ratio Limit	Seismic Load?	Ratio OKay?
3483.864	0.398	0.65	Yes	Yes

Shear Design for V_{Ed2} , V_{Ed3}

	Shear V_{Ed} kN	Shear V_{Rdc} kN	Shear V_{Rds} kN	tan(θ) Unitless	Rebar A_{sw} /s mm²/m
Major, V_{Ed2}	103.987	705.486	0	0.4	506.78
Minor, V_{Ed3}	87.7622	705.486	0	0.4	506.78

ETABS 2016 Concrete Frame Design (IS Code)
IS 456:2000 Column Section Design (Column C2 after retrofit)



Column Element Details Type: Ductile Frame (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
1ST	C17	151	C2-B/G	UDCon8	0	3657.6	1

Section Properties

SD Section		dc (mm)	Cover (Torsion) (mm)
457.2		60.5	30

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
25000	25	1	500	500

Design Code Parameters

γ_c	γ_s
1.5	1.15

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M_2 kN-m	Minimum M_3 kN-m	Rebar % %	Capacity Ratio Unitless
2527.3583	54.6941	-168.1119	54.6941	54.6941	3.42	0.73

Axial Force and Biaxial Moment Factors

	K Factor Unitless	Length mm	Initial Moment kN-m	Additional Moment kN-m	Minimum Moment kN-m
Major Bend(M3)	0.871127	3200.4	-77.9044	0	54.6941
Minor Bend(M2)	0.831359	3200.4	21.5094	0	54.6941

Shear Design for V_{u2} , V_{u3}

	Shear V_u kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm²/m
Major, V_{u2}	70.4658	215.8808	69.6776	63.0005	506.78
Minor, V_{u3}	95.8564	215.8808	69.6776	95.8564	506.78

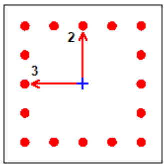
Additional Moment Reduction Factor k (IS 39.7.1.1)

A_g cm²	A_{sc} cm²	P_{uz} kN	P_b kN	P_u kN	k Unitless
2090.3	71.5	5032.1082	1086.6889	2527.3583	0.63485

Additional Moment (IS 39.7.1)

	Consider M_a	Length Factor	Section Depth (mm)	KL/Depth Ratio	KL/Depth Limit	KL/Depth Exceeded	M_a Moment (kN-m)
Major Bending (M_3)	Yes	0.875	457.2	6.098	12	No	0
Minor Bending (M_2)	Yes	0.875	457.2	5.82	12	No	0

ETABS 2016 Concrete Frame Design (Eurocode)
Eurocode 2-2004 Column Section Design (Column C2 after retrofit)



Column Element Details Type: DC High

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	SOM	LLRF
1ST	C17	C2-B/G	UDCon12-1		192	3657.6	Nominal Stiffness	1

Section Properties

SD Section		dc (mm)	Cover (Torsion) (mm)
457.2		60.5	30

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	E_s (MPa)	f_{yk} (MPa)	f_{ywk} (MPa)
31475.81	25	1	200000	500	500

Design Code Parameters

γ_C	γ_S	α_{CC}	α_{CT}	α_{LCC}	α_{LCT}
1.5	1.15	1	1	0.85	0.85

Axial Force and Biaxial Moment Design For N_{Ed} , M_{Ed2} , M_{Ed3}

Design N_{Ed} kN	Design M_{Ed2} kN-m	Design M_{Ed3} kN-m	Minimum M_2 kN-m	Minimum M_3 kN-m	Rebar % %	Capacity Ratio Unitless
1919.8924	-126.685	-91.7666	38.3978	38.3978	3.42	0.487

Axial Force and Biaxial Moment Factors

	M_{0Ed} Moment kN-m	M_{add} Moment kN-m	Minimum Ecc mm	β Factor Unitless	Length mm
Major Bend(M3)	-40.6497	0	20	1	3200.4
Minor Bend(M2)	-60.6519	0	20	1	3200.4

Axial Compression Ratio

Conc Capacity (α_{CC} *A*f_{cd}) kN	Compressive Ratio N_{Ed} /(α_{cc} *A*f_{cd})	Comp Ratio Limit	Seismic Load?	Ratio OKay?
3483.864	0.551	0.65	Yes	Yes

Shear Design for V_{Ed2} , V_{Ed3}

	Shear V_{Ed} kN	Shear V_{Rdc} kN	Shear V_{Rds} kN	tan(θ) Unitless	Rebar A_{sw} /s mm^2/m
Major, V_{Ed2}	42.9098	705.486	0	0.4	0
Minor, V_{Ed3}	73.8464	705.486	0	0.4	0

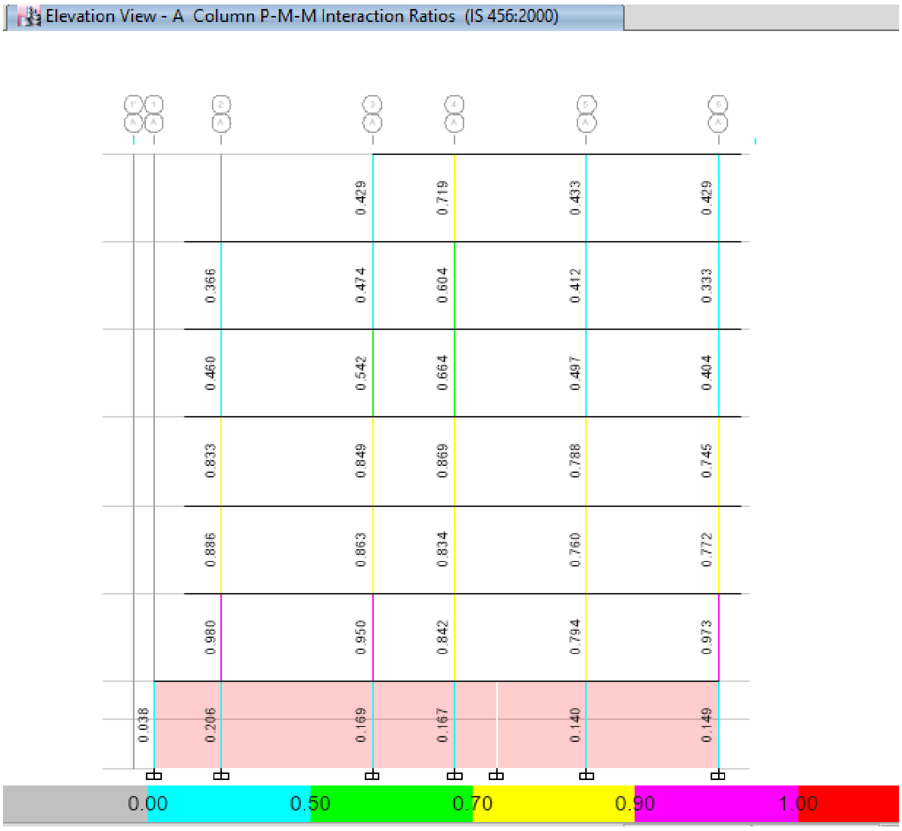


Figure 110: Column Reinforcement capacity ratio (IS code before retrofit)

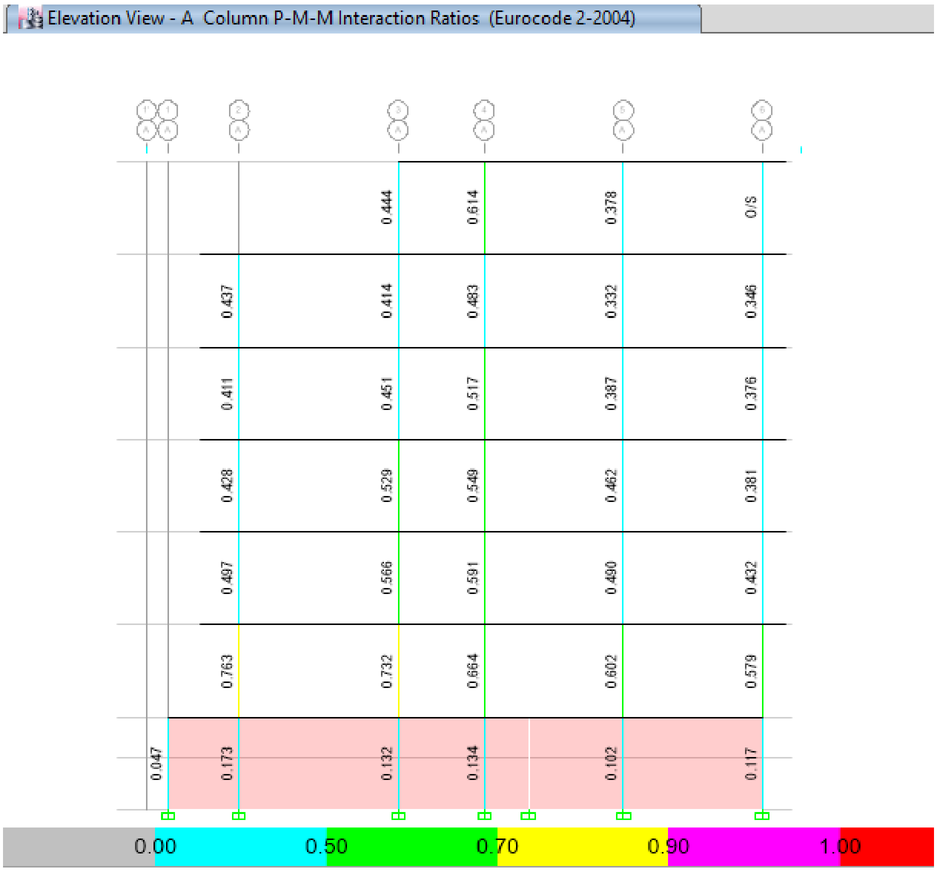


Figure 111: Column Reinforcement capacity ratio (EC before retrofit)

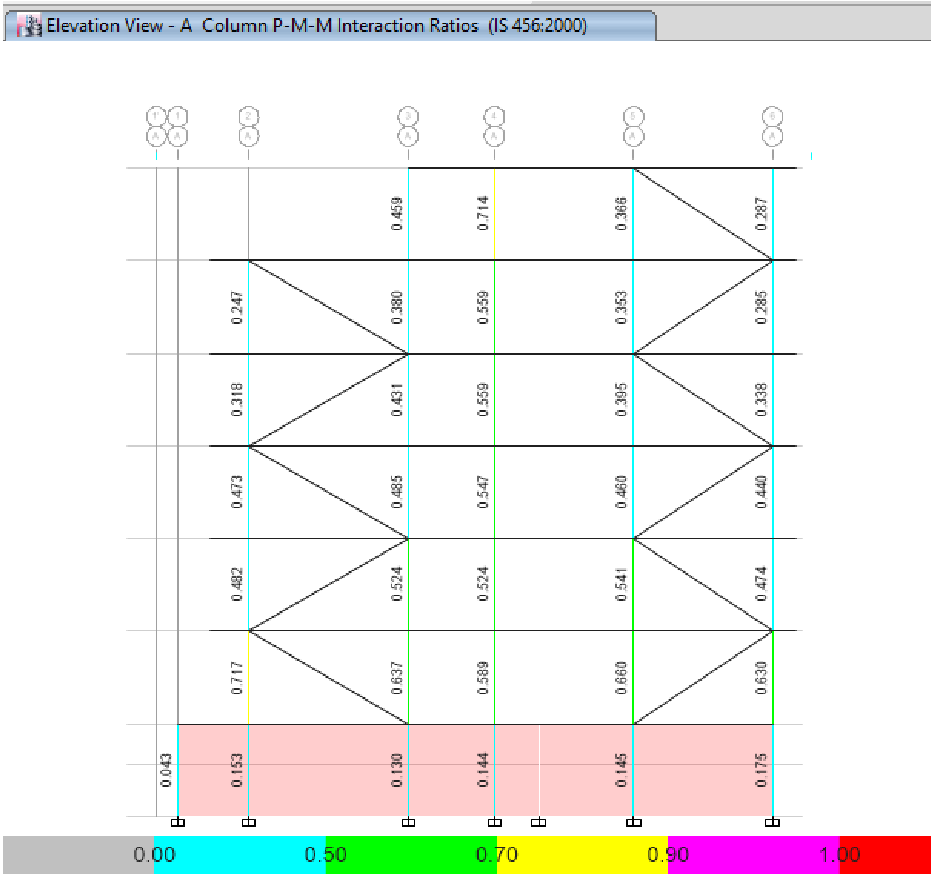


Figure 112: Column Reinforcement capacity ratio (IS code after retrofit)

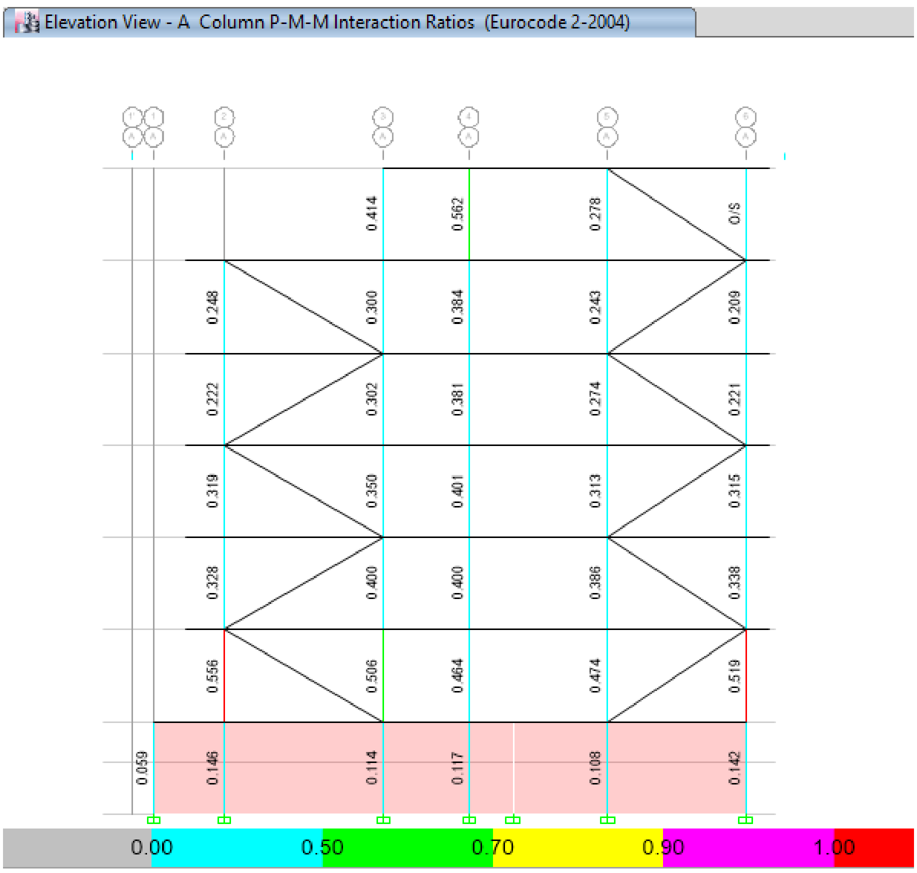


Figure 113: Column Reinforcement capacity ratio (EC after retrofit)

Column Design Summary

Design parameters:

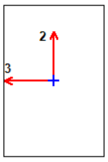
Concrete Grade:	25.00	M25	C25/30
Steel Grade:	500.00	Fe500	
Clear Cover:	40.00	40mm	

Column Floor	Column Type	b (mm)	d (mm)	Area of main steel Provided mm2	% of Steel provided	A_{st} Required as per IS code (linear analysis without infill) mm^2	A_{st} Required as per IS code (linear analysis with infill) mm^2	A_{st} Required as per Euro code (linear analysis without infill) mm^2	A_{st} Required as per Euro code (linear analysis with infill) mm^2
Column Type C1 ***NOTE:*** *The area of steel calculated is according to ETABS*									
Basement	REC	300	300	1256.64	1.40	720	720	900	900
Column Type C2 ***NOTE:*** *The area of steel calculated is according to ETABS*									
Basement	REC	450	450	7147.12	3.53	1908	2116	2025	2025
Ground	REC	450	450	7147.12	3.53	7140	5060	5746	4817
1st	REC	450	450	6440.26	3.18	5874	4566	3896	4193
2nd	REC	450	450	4828.63	2.38	4196	3230	2888	3105
3rd	REC	450	450	3669.38	1.81	2525	2598	2202	2337
4th	REC	450	450	2965.66	1.46	1791	1776	2025	2025
5th	REC	450	450	2513.27	1.24	1807	1855	2025	2025
Column Type C3 ***NOTE:*** *The area of steel calculated is according to ETABS*									
Basement	REC	450	450	6440.26	3.18	4148	4148	2937	2937
Ground	REC	450	450	6440.26	3.18	6118	4888	4714	4122
1st	REC	450	450	6440.26	3.18	5558	4476	3748	3420
2nd	REC	450	450	5026.55	2.48	4268	3071	2719	2463
3rd	REC	450	450	4121.77	2.04	2415	2263	2025	2025
4th	REC	450	450	3216.99	1.59	1620	1620	2025	2025
5th	REC	450	450	2513.27	1.24	1701	1620	2025	2025

Column Type C4 ***__NOTE:__*** *The area of steel calculated is according to ETABS*									
Basement	REC	450	450	5890.49	2.91	5360	5354	3835	3835
Ground	REC	450	450	5890.49	2.91	6014	4465	4712	3858
1st	REC	450	450	5890.49	2.91	5696	4459	3758	2986
2nd	REC	450	450	4830.20	2.39	4289	2763	2792	2178
3rd	REC	450	450	3769.91	1.86	2594	1938	2025	2025
4th	REC	450	450	3091.33	1.53	2099	1818	2025	2025
5th	REC	450	450	2412.74	1.19	1687	1620	2025	2025
Column Type C5 ***__NOTE:__*** *The area of steel calculated is according to ETABS*									
Basement	REC	450	450	4830.20	2.39	1620	1620	2025	2025
Ground	REC	450	450	4830.20	2.39	5217	3347	3154	2792
1st	REC	450	450	4830.20	2.39	3922	3362	2087	2058
2nd	REC	450	450	3769.91	1.86	3039	1987	2025	2025
3rd	REC	450	450	2865.13	1.41	1620	1620	2025	2025
4th	REC	450	450	2412.74	1.19	1620	1620	2025	2025
5th	REC	450	450	1884.96	0.93	1620	1620	2025	2025
Column Type C6 ***__NOTE:__*** *The area of steel calculated is according to ETABS*									
Basement	REC	450	450	4830.20	2.39	1620	1620	2025	2025
Ground	REC	450	450	4830.20	2.39	3091	3173	3502	3830
1st	REC	450	450	4830.20	2.39	2821	2096	2025	2025
2nd	REC	450	450	3769.91	1.86	1620	1620	2025	2025
3rd	REC	450	450	2865.13	1.41	1620	1620	2025	2025
4th	REC	450	450	2412.74	1.19	1620	1620	2025	2025
5th	REC	450	450	1884.96	0.93	1620	1620	2025	2025

D2: Beam Design

ETABS 2016 Concrete Frame Design (IS Code)
IS 456:2000 Beam Section Design



Beam Element Details Type: Ductile Frame (Summary)

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
2ND	B1	444	B12X18	UDCon10	228.6	6300.2	1

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
304.8	457.2	304.8	0	25	25

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
22360.68	20	1	500	415

Design Code Parameters

γ_C	γ_S
1.5	1.15

Factored Forces and Moments

Factored M_{u3} kN-m	Factored T_u kN-m	Factored V_{u2} kN	Factored P_u kN
-246.9936	1.0435	162.3324	0

Design Moments, M_{u3} & M_t

Factored Moment kN-m	Factored M_t kN-m	Positive Moment kN-m	Negative Moment kN-m
-246.9936	1.5346	0	-248.5282

Design Moment and Flexural Reinforcement for Moment, M_{u3} & T_u

	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm²	+Moment Rebar mm²	Minimum Rebar mm²	Required Rebar mm²
Top (+2 Axis)	-248.5282		1545	0	1545	386
Bottom (-2 Axis)		0	772	0	559	772

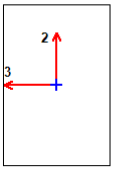
Shear Force and Reinforcement for Shear, V_{u2} & T_u

Shear V_e kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm²/m
204.7841	86.2281	124.0338	84.5927	795.25

Torsion Force and Torsion Reinforcement for Torsion, T_u & V_{U2}

T_u kN-m	V_u kN	Core b_1 mm	Core d_1 mm	Rebar A_{svt} /s mm²/m
1.0435	162.3324	274.8	427.2	523.07

ETABS 2016 Concrete Frame Design (Eurocode)
Eurocode 2-2004 Beam Section Design



Beam Element Details Type: DC High

Level	Element	Unique Name	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
2ND	B1	444	B12X18	UDCon6	228.6	6300.2	1

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
304.8	457.2	304.8	0	25	25

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	E_s (MPa)	f_{yk} (MPa)	f_{ywk} (MPa)
29961.95	20	1	200000	500	415

Design Code Parameters

γ_C	γ_S	α_{CC}	α_{CT}	α_{LCC}	α_{LCT}
1.5	1.15	1	1	0.85	0.85

Design Moment and Flexural Reinforcement for Moment, M_{Ed3}

	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm²	+Moment Rebar mm²	Minimum Rebar mm²	Required Rebar mm²
Top (+2 Axis)	-234.2823		1510	0	291	1510
Bottom (-2 Axis)		117.1411	64	681	291	681

Shear Force and Reinforcement for Shear, V_{Ed2}

Shear V_{Ed} kN	θ deg	Shear V_{Rdc} kN	Shear V_{Rds} kN	Rebar A_{sw} /S mm²/m
133.7484	45	75.4524	142.431	1014.68

Torsion Force and Torsion Reinforcement for Torsion, T_{Ed}

Torsion T_{Ed} kN-m	T_{cr} kN-m	Area A_k cm²	Perimeter, u_K mm	Rebar A_t /s mm²/m	Rebar A_{sl} mm²
1.4547	0	780.4	1158.2	48.57	25

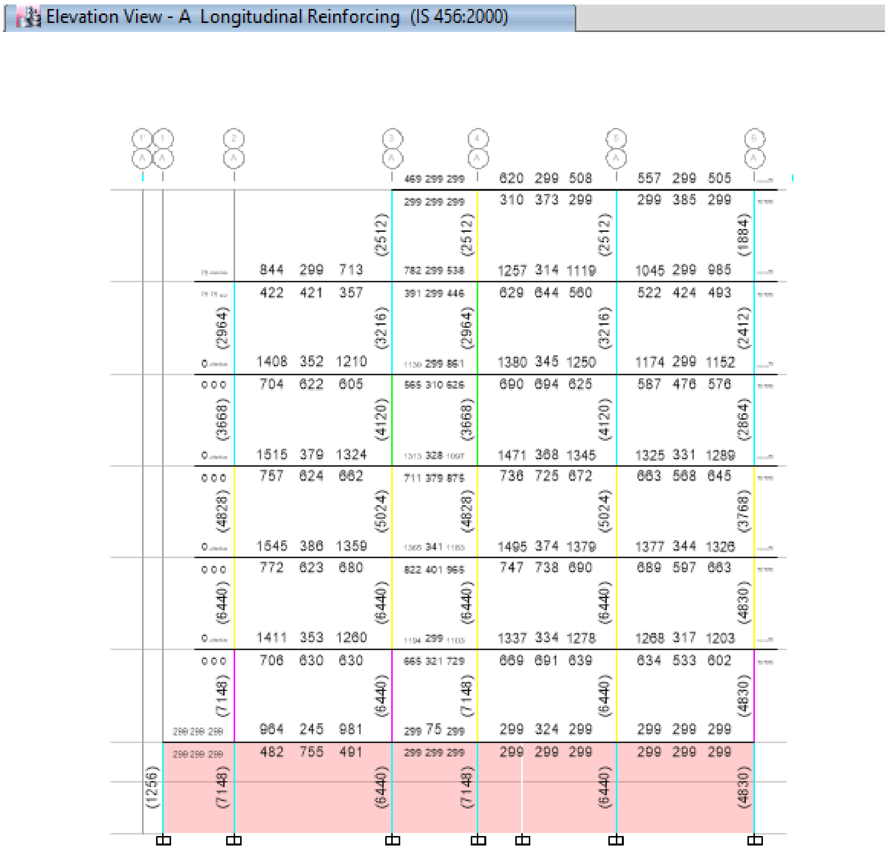


Figure 114: Beam Reinforcement (IS code before retrofit)

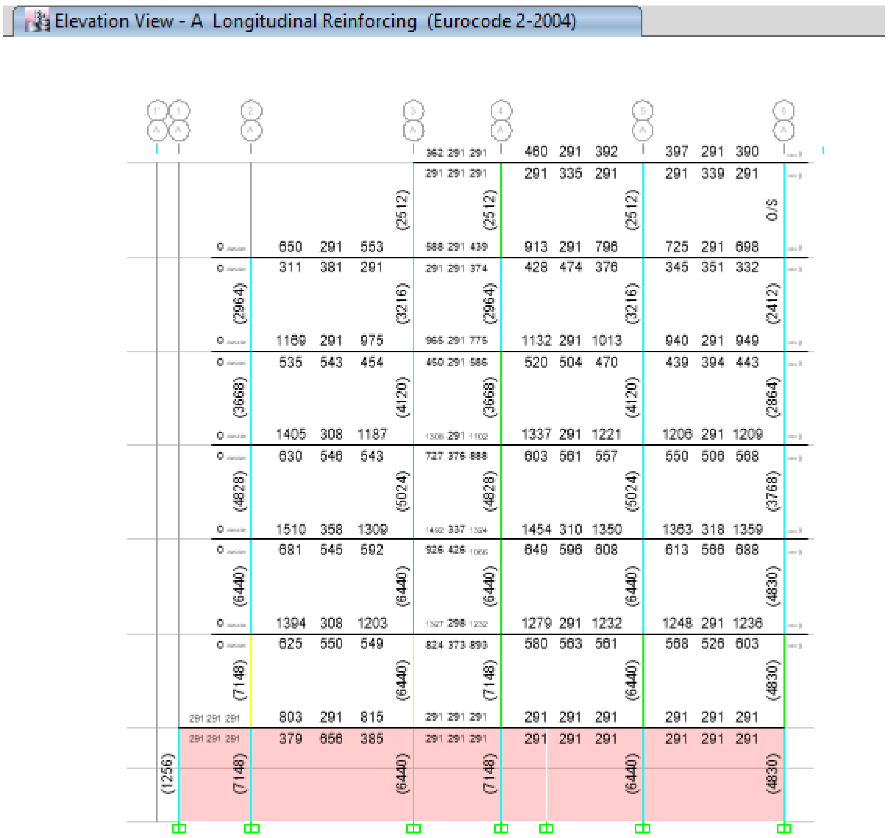


Figure 115: Beam Reinforcement (EC before retrofit)

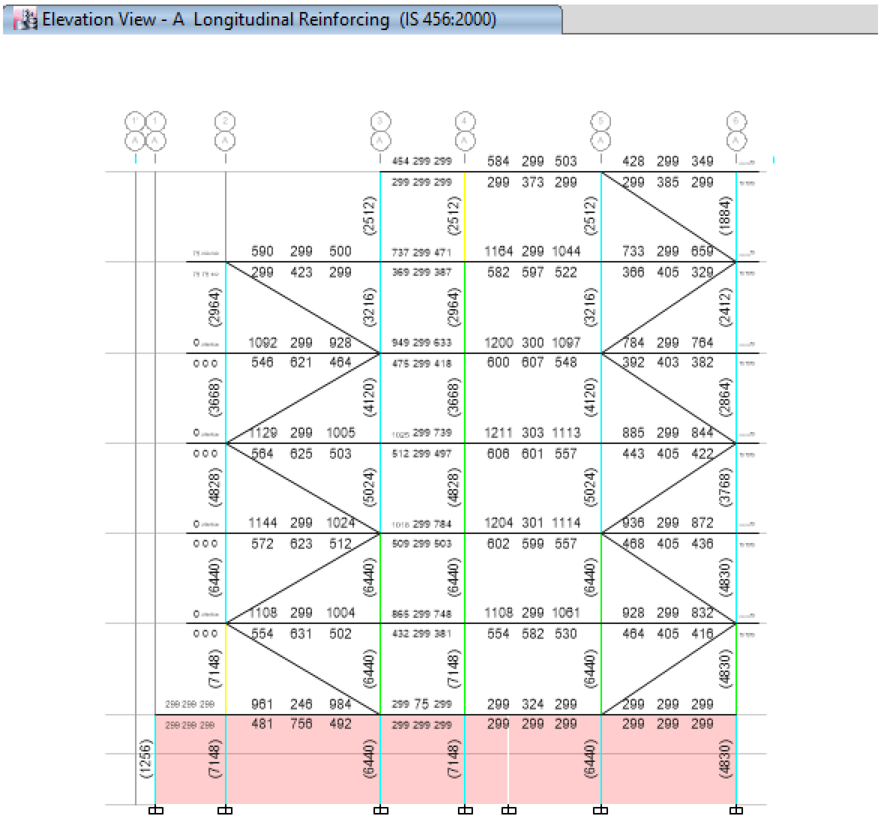


Figure 116: Beam Reinforcement (IS code after retrofit)

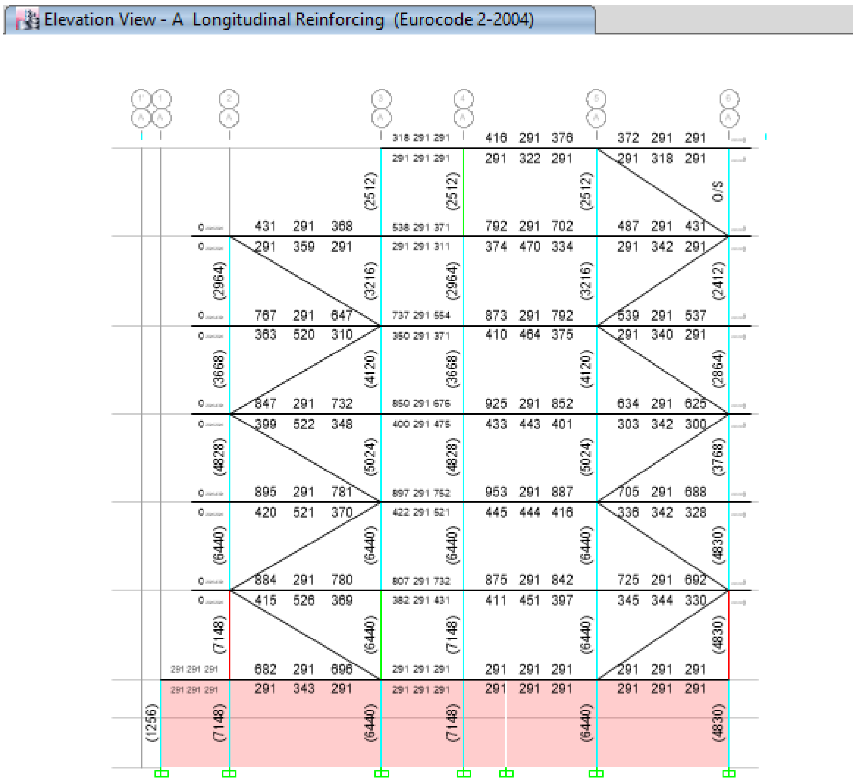


Figure 117: Beam Reinforcement (EC after retrofit)