M.Tech (Structural Engineering) Rishi B. Mathur (2K23/STE/04) 2025

COMPARATIVE STUDY OF SEISMIC PERFORMANCE ANALYSIS OF STEEL STRUCTURES WITH DIFFERENT STRUCTURAL SYSTEM AS PER IS 18168:2023

A Dissertation Submitted

In Partial Fulfilment of the Requirements for the

Degree of

MASTER OF TECHNOLOGY

in

Structural Engineering

by

RISHI B. MATHUR

(Roll No. 2K23/STE/04)

Under the Supervision of

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Shahbad Daulatpur , Main Bawana Road,Delhi 110042

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CANDIDTATE’ S DECELARATION

I, **Rishi B. Mathur** , (M.Tech Structural Engineering) student, having **Roll no:2K23/STE/04** hereby certify that the work which is being presented in the thesis entitled “**Comparative Study of Seismic Performance analysis of Steel Structures with Different Structural System as per IS 18168:2023**” in partial fulfilment of the requirements for the award of the Degree of **Master of Technology in Structural Engineering** , submitted in the **Department of Civil Engineering, Delhi Technological University** is an authentic record of my own work carried out during the period from August 2024 to May 2025 under supervision of **Dr. Nirendra Dev, Professor, Department of Civil Engineering , Delhi Technological University , Delhi.**

The matter presented in the thesis has not been submitted by me for the award of any other degree of this or any other institute.

Rishi

(Rishi B. Mathur)

This is to certify that the student has incorporated all the corrections suggested by the examiners in the thesis and the statement made by the candidate is correct to the best of our knowledge.

Nirendra Dev

Signature of Supervisor **Signature of External Examiners**

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CERTIFICATE BY THE SUPERVISOR

Certified that Rishi B. Mathur (2K23/STE/04) has carried out his research work presented in this thesis entitled "**Comparative Study of Seismic Performance analysis of Steel Structures with Different Structural System as per IS 18168:2023**" for the award of **Master of Technology in Structural Engineering** from the Department of Civil Engineering, Delhi Technological University, Delhi, under our supervision. The thesis embodies the results of original work and studies are carried out by the student himself. The contents of the thesis do not form the basis for the award of any degree to the candidate or to anybody else from this or any other University/Institution.

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ABSTRACT

With the progress in codal provision for steel structure design, it had significantly reinforced the durability and flexibility in modern design. With the introduction of new Indian Standard code for design and detailing for steel structure IS 18168:2023 has replaced the earlier provision mentioned in section 12 of IS 800:2007. This study emphasizes on taking into consideration new clauses for design of steel structure and compares these provision through modelling different structural steel systems under seismic event. The structure analyzed on Loading combination defined in new code as well as Load combination outlined in IS1893:2016. The concept of capacity protected element is also introduced in this code which is explored in this study. The code mentions the limiting value of the slenderness ratio at the location of plastic hinge formation. For the current study, G+4 structure is being prepared which is situated in Earthquake Zone IV. ETABS 2019 version is used and results are compared on the parameter such as storey drift, maximum story displacement and base shear value of SMRF, SCBF &; EBF structural system.

The beam-column strength ratio is being defined in this code. The finding of this study revealed that SMRF system have maximum storey drift in X direction but least in Y-direction. In all three structural system used in this study EBF structural system exhibit the highest base shear while SMRF system shows minimum value of base shear.

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Rishi B. Mathur

(2K23/STE/04)

TABLE OF CONTENTS

CHAPTER 4: CODE-BASED COMPARISON OF SEISMIC DESIGN GUIDELINES FOR STEEL STRUCTURES

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOL

Ymo- is partial factor of safety as mention in IS 800: 2007

M_p- plastic moment capacity

DL –Dead Load as per IS 875 (Part 1)

LL- Live Load as per IS 875 (Part 2)

Ry- Ratio of expected yield stress to characteristic yield stress

Fy- Characterstic yield stress of structural steel

Ag- Gross cross sectional Area

An- Net Cross Sectional Area

Pd – Design compressive strength determined using IS 800

Zpb – Plastic section Modulus of beam

Hi – Height of Storey i

Fi- Lateral Load acting on storey i

∆i- Lateral displacement of storey I, from linea static analysis

Wi – Weight acting on storey i

Mpb- Plastic Moment capacity of beam section

D_f – Distance between of centroid of flanges of a section under bending

D_p –depth of deeper beam

B_p – width of flange plate

A_w- Area of web link

Pu – Maximum axial Load demand

Py- Yield axial Load capacity

€- Yield stress ratio

Ω - over strength factor of safety

Υ_{LL}- Partial Safety Factor for live Load

α_i – Ratio of secondary overturning moment to primary overturning moment of storey i

Z_p – Plastic section modulus

CHAPTER 1
INTRODUCTION

1.1 Steel Structure Design – India

To construct a tall structure demand, the use of flexible, lightweight and strong material. Steel due to its virtue of its high strength enables the construction of large span and lightweight structures. For the construction of high rise structure, steel structure provides variety of constructional design such as braced frame and moment resistant frame. High rise structure usually experiences greater seismic and wind forces, but the use of steel structure allows them to absorb and deflect these Load more efficiently in comparison with concrete.

Structures built by humans are meant to guard us from extreme natural phenomena, whether it's severe weather or climate-based events. But if these structures are poorly designed, then they can experience great damage during natural disasters like quakes; which can result in loss of lives and finances. With earthquakes being relatively rare but significant threats to structures, it is critical to design structures that can resist these forces and respond appropriately. Steel Structure are strong and are able to resist large forces and had a plus point in unfavourable weather condition.

The use of steel for constructing high rise structure is steadily gaining popularity in India. Structure built predominantly or entirely of steel is somewhat uncommon and is seen as modern innovation in construction sector, but the iconic structure like the Empire State Structure which is situated in New York and some other renowned structure were made decades ago. Steel structures are comparatively lighter and more efficient, with beams that can be shallower than those used in concrete structures while still effectively supporting the floors.

When compared to Reinforced-Concrete frames, steel frames provide additional space which make easier integration for service conduits with minimal impact on ceiling height. Steel structures provide column free interiors which make them suitable for open-office plan layout, spacious auditorium and concerts halls.

1.2 Capacity Based Design Approach

Capacity Design principle which was taken into consideration in the new IS 18168:2023 which was recently released form Bureau of Indian Standards (BIS) inculcate additional design measure above the standard design of structural element that are not looked to go yielding. When the designated structural element yield in elastically, the generated force Demand on the basis of their over-strength on the surrounding element that are intent to remain elastic. The code specifies the protected element for different structural system so that they can remain elastic while the adjacent member or connection get in elastically strained due to earthquake shaking. The objective of capacity design is to make sure that the structure undergoes in a controlled ductile behaviour to avoid total collapse in design level earthquake.

In case of SMRF Structures, beam-column joint and columns shall be designed for capacity protected member. A key design requirement need to be fulfilled that the strength of column exceed that of beam by a margin (ratio greater than 1.4) to ensure inelastic action occur in beam and not in column. For the braced structures, system are supposed to give inelastic deformation capacity through brace buckling and yielding in tension. Beams and columns are designed as non-yielding members. Their required strength is based on greater forces obtained from analyses which take consideration that braces will resist forces on their expected strength or their post buckling strength of steel in compression.

The code tries to match with standards and practicing prevailing in other countries specifically form American code AISC 360-16 & AISC 341-16. As per provision mentioned in IS 18168:2023 in seismic zone V, all steel structure should be made eccentrically braced frame only. Additionally, it restricts SMRF structure in zone IV & V, unless the height of structure is less than 15.

1.3 Cross-Sectional Classification of Memebers

As per IS 800:2007, the section can be classified in four behaviour classes on the basis that depends upon material yield strength, the width to thickness ratio of individual component and Loading arrangement.

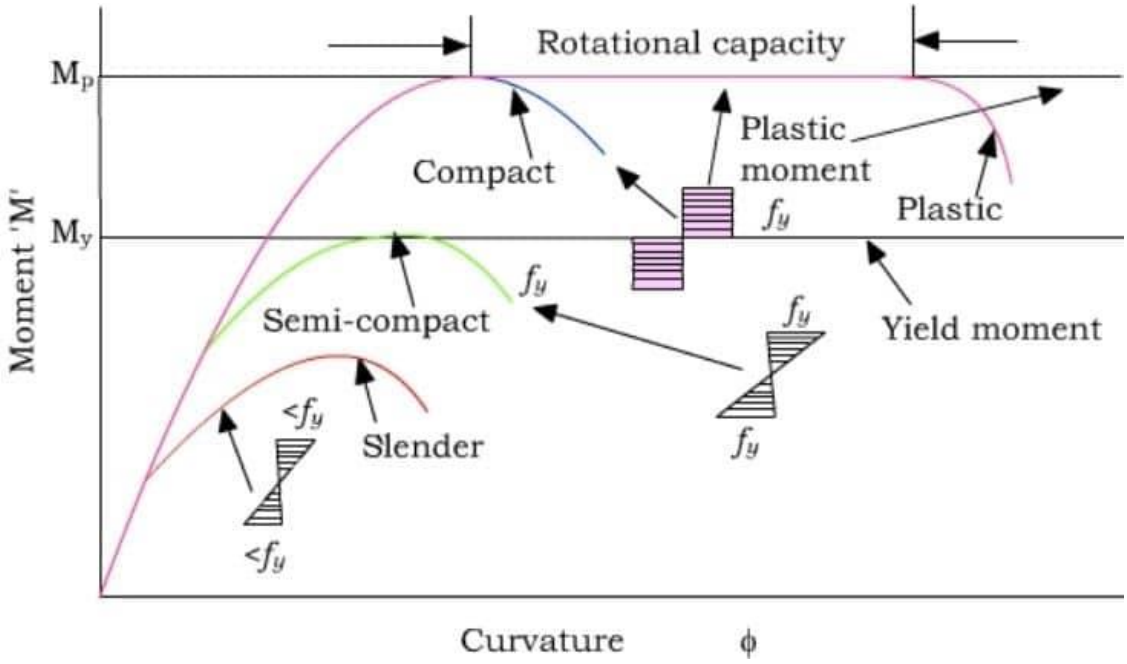


Fig. 1.1 Moment Behaviour of classes of sections as per IS 800:2007

a) Plastic or Class 1: From the figure 1.1, class 1 plastic section are fully effective under pure compression, and are capable of reaching and maintaining their full plastic moment in bending therefore can develop the plastic hinges and show sufficient rotation capacity for failure of structure due to creation of plastic hinges.

b) Compact or Class 2: These section is fully effective for pure compression and also develop their plastic moment against bending, but have inadequate plastic hinge rotation capacity as local buckling as the cause. The section has lower deformation capacity.

c) Semi Compact or Class 3: These sections are also fully effective in case of pure compression and the elastically stress in extreme compression fibre of steel member , elastic distribution of stress which can reach to yield strength. But full plastic moment of resistance against bending would not be possible due to local buckling.

d) Slender or Class 4: Section in which local buckling can occurs before the section reach to its limit of yield stress.

For calculating the Design Moment Capacity M_d, can be found by :

Table 1.1 Different classes of steel elements

1.	Plastic	$M_d = Z_p f_y$ / Ymo
2.	Compact	$M_d = Z_p f_y$ / Ymo
3.	Semi – Compact	$M_d = Z_e f_y$ / Ymo
4.	Slender	$M_d = Z_e f_y$ /Ymo

Where Z_p and Z_e are plastic and elastic section modulus respectively, and Ymo is partial factor of safety as mention in IS 800: 2007.

CHAPTER 2

Literature Review

Atul B. Pujari et.al (2023) study emphases that Bracing system not only resisting earthquake effect effectively, it has reduced the progressive collapse of ground floor removal of column up to a particular height of the structure.

Akula Prakash et al. (2024) study investigate the behaviour of multi storey with and without floating columns and taking them consider in seismic zone III. The results show that the floating columns tends to move away from similar column structure. It also shows that structure with floating column has higher value of displacement. For all the structure modelled, the storey drift kept on increasing from lower floor to upper floor.

Hajira Nausheen, & Dr.H.Eramma study the comparison between composite and conventional structure was conducted out just by varying the design of column i.e., by using composite and conventional column and keeping all other structural members same for both the structures.

Rishabh Joshi et. al. investigates the seismic performance of three types of structures—Reinforced Concrete (RCC), Composite, and Light Steel—using ETABS software for simulation and analysis. The study aims to understand how each structure type behaves under seismic Loading by comparing parameters such as displacement, storey drift, base shear, and time period. The analysis finds that RCC structures, while strong and durable, exhibit higher displacement and drift compared to Composite structures, which offer a better combination of strength and flexibility, making them more resilient to seismic forces. Light Steel structures, being lighter and more flexible, show better displacement characteristics but require careful design to withstand seismic forces effectively. The study concludes that Composite structures perform the best in seismic conditions, followed by RCC structures, while Light Steel structures, though advantageous in certain aspects, may need additional design considerations to enhance their seismic resilience. The findings emphasize the importance of selecting structure materials based on seismic risk and structural requirements.

Keshav K. S. et.al examines the vulnerability of **Special Moment Resisting Frames (SMRFs),** commonly used in earthquake-resistant structures, to progressive collapse. The study focuses on how localized failures, such as the loss of a single structural element, can lead to a chain reaction that causes the collapse of the entire frame. By conducting **non-linear dynamic analysis** in a three-dimensional context, the authors investigate the effectiveness of SMRFs in preventing progressive collapse, especially when designed for seismic resistance. The paper highlights the importance of **redundant Load paths**, which provide alternative structural routes to carry Loads in the event of failure. The findings emphasize the need for careful design considerations to enhance the resilience of SMRFs against progressive collapse, ensuring that such frames can maintain stability even under extreme conditions or unexpected damage.

Rama , K. et.al. (2013) research emphasis on importance of taking into consideration the site –specific lateral forces, like wind and seismic Loads.in parallel to vertical Loads to access the behaviour of tall structure. The increase in height of structure, make it important to use good quality of material. Tall structure design requires conceptual planning, approximate modelling, design to make sure the safety from lateral force. The research focuses on taking limit state of design for analysing and designing of reinforced concrete structure (three basement + ground +40 storey) which were subjected to wind and earthquake forces in accordance with relevant IS Codes.

CHAPTER 3

Seismic Design Philosophy of Steel Frames

3.1 Type of sesmic Resistant Steel frame

On the basis of inelastic rotation capability , sesmic resistant steel frame can be classified as :

- Special Moment Resisting Frames (SMRF)
- Special Concentrically Resisting Frame (SCBF)
- Eccentrically Braced Frame (EBF)

3.2 Seismic Behaviour and Design Philosophy Specials Moment Resisting frames (SMRF)

Special Moment Resisting frame are designed in accordance to give substantial inelastic deformation capacity through flexural yielding of the beams, limiting yield of panel zone and little or almost negligible yielding of columns except the base. It rely on ability of the frame to act as partially or fully rigid jointed frame to sustains lateral seismic Load. Further to evaluate the deign process, IS 18168:2023 provide procedure that help to estimate the demand on, and behaviour of capacity protected element. Special Steel Moment Resisting Frames (MRFs) exhibit enhanced energy dissipation capabilities by undergoing plastic deformations, primarily through bending-induced yielding in beams and columns[1]

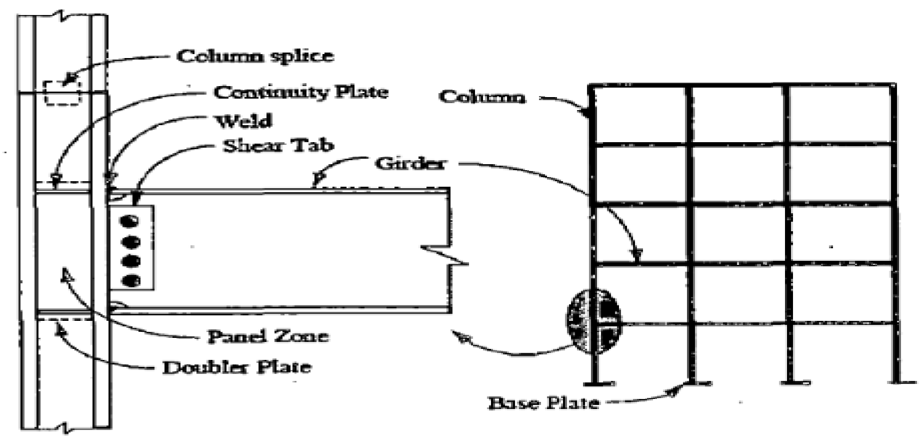


Fig. 3.1 Connection detail of MRF structure

Special Moment Frames (SMFs) shall be constructed of E250B steel of IS 2062 and must show to withstand inelastic deformation associated to a joint rotation of 0.004 radian without any reduction strength and stiffness below the full yield value (M_p).

Beam to Column Joints and Connection: All beam-to-column joints must be rigid and capable of resisting a bending moment of no less than 1.2 times the full plastic moment (Mp) of the connected beam. In cases where a Reduced Beam Section (RBS) is utilized, the section's minimum flexural capacity should be at least 80% of the full plastic moment of the original, unmodified beam. Additionally, the connection must be designed to resist shear forces resulting from the load combination 1.2 times the Dead Load (DL) plus 0.5 times the Live Load (LL), along with the shear generated by applying 1.2 times Mp at both ends of the beam in the same direction

Reduced Beam Sections (RBS): The concept of introduced by Plumier (1990), to use RBS also termed as dogbone which he found as a replacement for altering the location of plastic hinge formation at distance (away) from the face of column by reducing the plastic moment of beam at more shorter distance from the face of column.

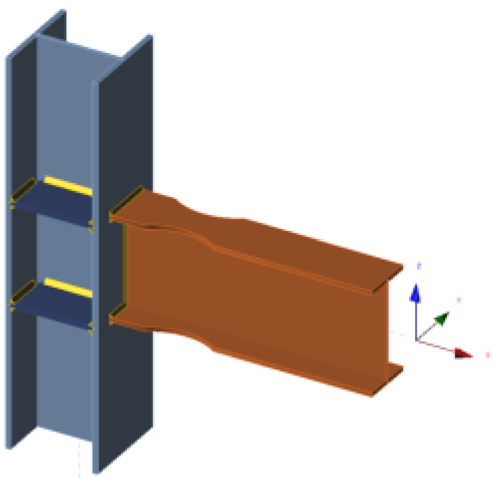


Fig 3.2 Finite element Analysis Model of RBS Connection[2]

Further studies on RBS revealed by reducing the width of beam flange leads to delay in flanges local buckling it also increases the chances of web local buckling and lateral torsional buckling.

3.3 Seismic Behaviour and Design Philosophy Special Concentrically Braced Frames (SCBF)

Steel Structure because of their higher ductility and energy dissipation properties are prone for less damage during an earthquake when compared to concrete structure. Bracings plays important role to resist lateral force. Fig 3.3 shows various concentring bracings which are widely used globally.

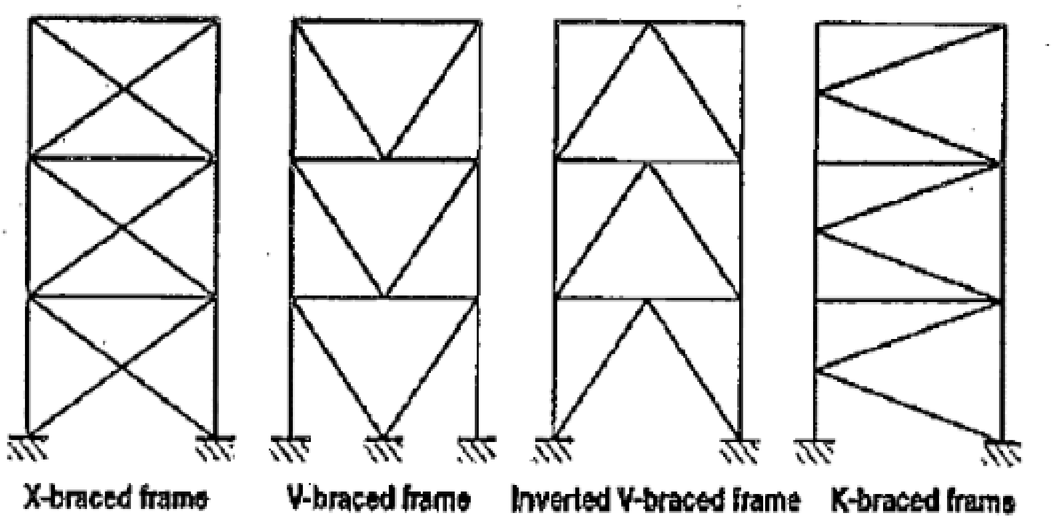
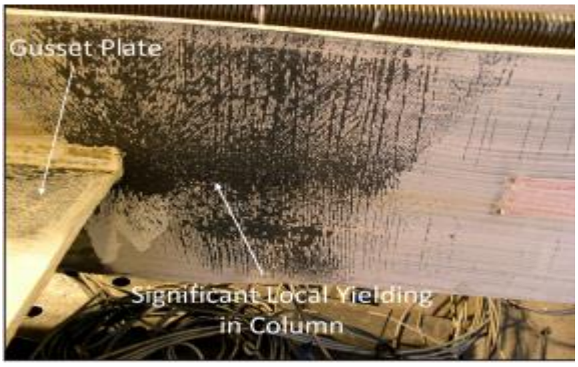


Fig 3.3 Different Configuration of bracings

SCBFs are made to accommodate to maximum expected inelastic- drift capacity[3]. Capacity based design approach based on methodology to emphasis and focuses on the ductility of structure to ensure by systematic and controlled collapse of braces first and then beams and next comes columns[4] which is inculcated in IS 18168:2023. Both V and X- braced frame, also referred as chevron braces perform poor because of deformation of braces that is due to buckling and excessive flexure of beam at the middle of span at the location of the braces extract. K-bracings are not allowed in high seismic zones. In the time of moderate to severe earthquake, the connection and members are expected to experience and to go significant elastic inelastic deformation in post buckling range. In severe earthquake, braces may undergo post buckling axial deformation from a range between 10 - 20 times their yield deformation. From the research conducted by *Lumpkin et.al.* the capacity of post buckling compressive strength the HSS tubular braces were more stable then wide flange braces[5]. Tremblay[6] in his study investigate in high seismic activity areas the utilisation of hollow section by evaluating braces compressive and tensile resistance.The braces show the tendency to buckle when under compression and yield in tension. Compressive buckling strength of braces are initially lower than tensile yield capacity. Adding to each new cycle, this compressive strength gradually starts diminishing due to inelastic deformation. For preventing it, balanced way to be designed to provide comparable lateral resistance in both directions.

(a)

(b)



(c)

Fig 3.4 Various failure behaviour of braced frame, (a) Deformation because of brace buckling, (b) Deformation at gusset plate, (c) Local Yielding at beams and columns

Slenderness ratio is critical parameter for concentrically braced frame as it directly governs the behaviour of structure. As slenderness ratio increase energy dissipation capacity of brace decreases, and as it decreases failure ductility increase. As per code , value should be less than 160 in case of effective ratio of slenderness, whereas the upper limit of slenderness ratio is mentioned 200 in AISC 341-16 for SCBFs.

The section can be used that are either rolled or can be built-up section or closed box section, the flange width-to-thickness ratio and web depth-to-thickness ratio should be under limit as prescribed in *Table 2* of IS 18168:2023. If built-up braces are employed in the structure, minimum of two connecters should be evenly provided at uniform spacing such that slenderness ratio of individual plate elements shall remain less than and does not exceed 0.4 times the governing ratio.

The tensile strength of brace connection shall be calculating as the maximum of expected tensile strength determined as $1.1R_yf_yA_g$ and $R_uf_uA_n$. The strength of braces connection shall be equal to brace strength in compression which is usually governed by buckling.

The required strength of capacity protected element i.e column, beams, structs shall be designed as per Cl 12.2.2 in IS 18168:2023. The tensile strength (T_e) of braces shall be calculated as

$$T_e = R_yf_yA_g$$

The provision of code for the compressive strength (P_e) of braces shall be taken as :

$$P_e = R_yY_{mo}P_d$$

The anticipated post buckling compressive strength of the braces under compression should be considered to be taken as 0.2 times the expected compressive strength (P_e) while the bracing connection shall be assumed to remain elastic.

The tension braces are required to resist between 30% - 70% of total horizontal force, along its line of bracing. Diagonal and X braces are permitted to use in SCBFs. K-brace frame need to be avoided in SCBFs.

Beam-to- column Connections: For brace-gusset plate assembly connection at beam-to-column connection, the assembly need to be designed so to resist beam moment taken equal to $1.1R_yf_{yb}Z_{pb.}$ And the sum of expected column flexural strength shall exceed $1.1R_yf_{yb}Z_{pb}$.

3.4. Seismic Behaviour and Design Philosophy Eccentrically Braced Frames (EBFs)

EBFs are a form of Structural System in which the lateral force induced in braces during a seismic activity get transferred either to column or other brace through bending and shear in small portion of beam called as link.

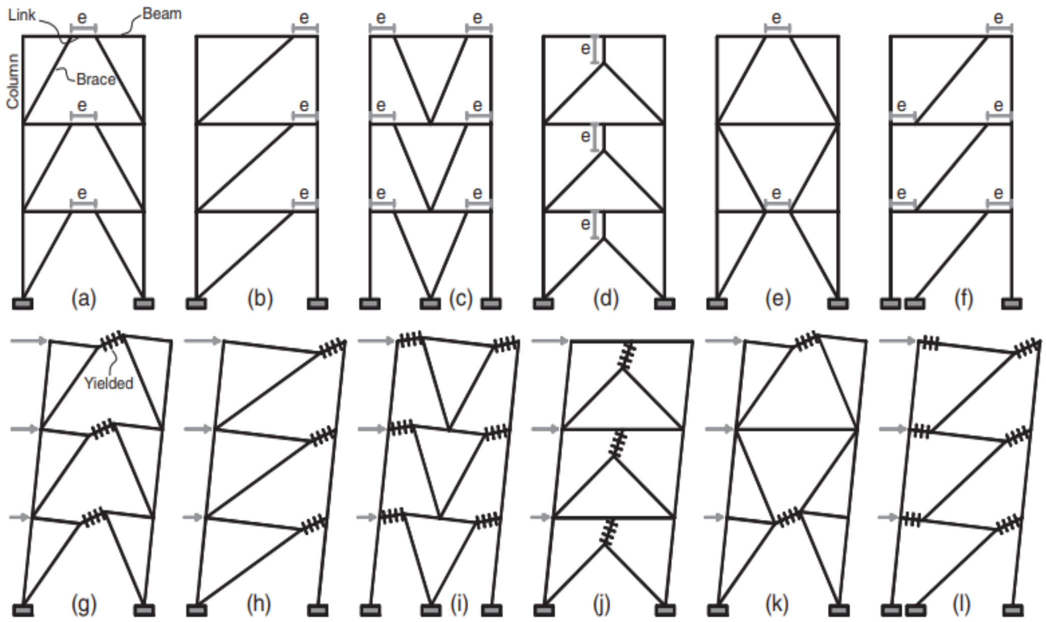


Fig 3.5 Various EBFs configuration and corresponding Plastic Mechanism [7]

The link segment length (e) is useful as it controls and govern the stiffness, ductility, strength and behaviour of EBFs structure. The link act like a component which help to dissipate earthquake induced energy in stable manner. The link length ratio, $\beta=e/(M_p/V_p)$, in which M_p is the plastic moment capacity and V_p is the plastic shear capacity of link.

Some studies found there are substantial difference in behaviour of links . Experimental investigation tells performance of shorter link under cyclic Loading are better than long links in respect of flexibility and compression [8].

Based on link length (e), the failure mode of EBFs are designated as following

- e < 1.6 – Shear Yielding – Short link
- 1.6 < e < 2.6 – Shear and Flexural Yielding – Intermediate Link
- e > 2.6 – Shear Yielding – long links

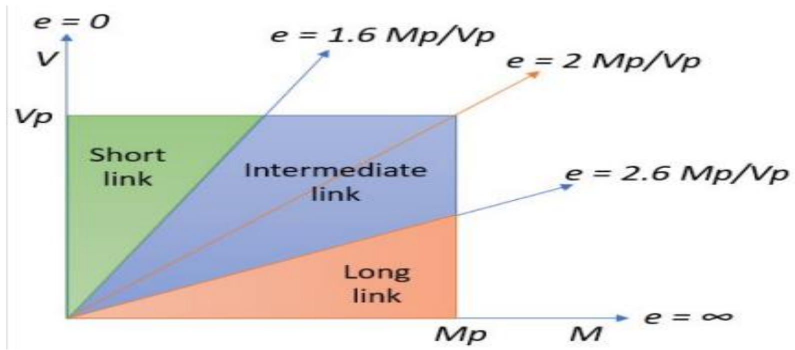


Fig 3.6 Classification of links,[9]

3.4.1 Deformation Pattern in EBFs

To calculate the plastic rotation demand of links, energy dissipation configuration can be utilised. γ_p is calculated by considering the EBF bay rotates as a rigid body and

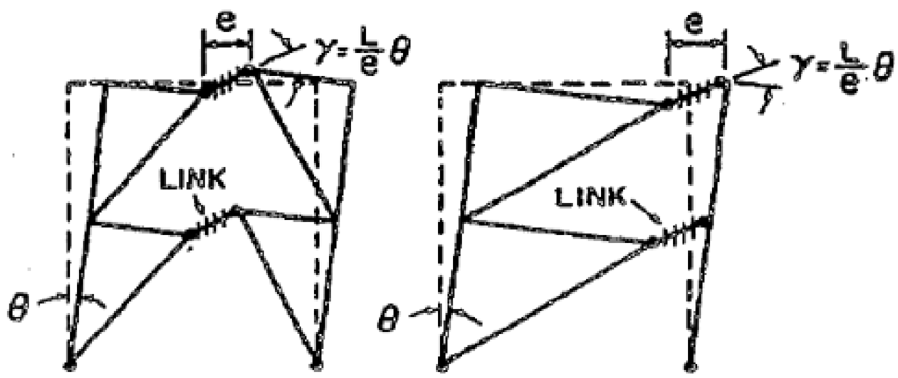


Fig 3.7 Eccentric braced frame deformation mechanism

EBFs structure are required to be designed such that inelastic behaviour is restricted to links only. EBFs are expected to be designed to provide significant inelastic deformation capacity primarily through shear yielding in link. It need to make sure, links are not directly connected to columns. For brace attached eccentrically to beam and column, torsional moment get induced in the members. Hence at the connection of diagonal brace and beam, the intersection of centrelines should lie at the end of link. Also it need to be taken care that no stress concentration is generated in connection.

Brace members, the beams that are outside the link and column shall satisfy the limitation on width-to thickness as specified at Table 2 of IS 18168:2023. Other than columns the beams and columns may subject to significant bending and axial force; hence they need to be designed of their capacities as for beam – column member as per IS 800.

As per Indian Standard, in EBFs system shear links shall be used as structural fuse. These are expected to get bending and shear force and also yielding during seismic activity. The code also two section to be utilised as link one is I-shaped cross section and another box section with flange as per limit mentioned in codes provision.

The links are protected zone in this structural system and the no shear studs as per the standard.

Beam-to-column Connection – Similar to SCBFs, the code put certain condition in connection design. The area of plate that conjunction of beam and column connection, the assembly to be made able to limit beam moment valued equal to $1.1R_y f_{yb} Z_{pb}$ and sum of expected column flexural strength to exceed this value.

Braces of frame shall be designed as they not undergo yielding in tension and buckling in strength with link shear force to be 1.2Ry times strength of link.

CHAPTER 4

Code-Based Comparison of Seismic Design Guidelines for Steel Structures

4.1 Introduction

The chapter presents a comparative highlights of design codes from various countries to provide a clear understanding of their similarities and difference. The code includes: IS 1893, American code: ASCE 7, Euro code: EC 8 and New Zealand Standard: NZS 1170.5 and also with complimentary codes for steel design - Indian Standard: IS 800, IS 18168:2023, Euro code: EC 3, New Zealand Standard: NZS 3404 and American Standard: AISC 341. This code is compared on their ductility classification, Response Reduction, base shear coefficient for all thress frames used in this study.

4.2 Cross Sectional Classification

Behavious and overall performance of elements are important parameters to determine the strength of structural steel component. Various codes define cross section utilising different parameters such as yield strength, width-to-thickness ratio of structural component web and flanges, plastic mechanism etc.

IS 800 : 2007 classify section into four classes : Plastic, Compact , Semi Compact & Slender. AISC 341 classify the cross sections into two classes i.e Moderately Ductile Member & Highly Ductile Member which are based on limiting web-to –thickness ratio as well as lateral bracing requirements Both member is capable to seismic force resisting to resist inelastic deformation. Euro code also defines section as same as Indian Standard in and specify class 1section for plastic design.

NZS 3404 classifies section as Category 1-4 on basis of section geometry and displacement Ductility.

Table 4.1 Classes of cross section of different codes

IS 800	**ASCE 7**	**EC 8**	**NZS 3404**
Class 1 (Plastic)	Highly Ductile	Class 1	Category 1
Class 2 (Compact)	Low Ductile	Class 2	Category 2
Class 3 (Semi-Compact)	-	Class 3	Category 3
Class 4 (Slender)	-	Class 4	Category 4

From the study it has been observed that NZS 3505 give most detailed requirement for lower ductility class, whereas the limit for higher ductility classes are more or less the same. Different width-to-thickness ratio is being adopted from different codes. The limit value for the ease had been defined in term of constant (€). The value of (€) is different in different codes.

For EC 3 , € = $\sqrt{\frac{235}{f_y}}$

For AISC , € = $\sqrt{\frac{E}{f_y}}$

For IS 800 & IS 18168, € = $\sqrt{\frac{250}{f_y}}$

It is having been found that AISC provide less stringent limit when compared with other countries code. Additionally, EC 3 are more stringent for their class 3 sections. AISC limit is 2.1 times higher than EC3 for flange under flexure [10]. From all codes it's been found AISC provide more stringent limit on width to thickness ratio for seismically compact section i.e Class 1. For class 2 & 3 , EC 3 had been more stringent limit. For class 3, AISC provide significantly higher limit from rest three codes. The moment capacity of section is lesser in EC 3 as only yielding is expected from class 3 section on other hand plastic moment capacity is expected from Class 1 & Class 2 element. It's been found that slenderness ratio in IS 800[11] is less conservative as compared to AISC 341.

4.3 Classification of structural system based on Ductility

During strong ground motion, it's not practical for structure to remain elastic during strong earthquake. For that reason, the structure is designed or reduced force level. The structure is expected to sustain post yield displacement by providing special ductile detailing requirements for members and connections. Earlier for ductile design different countries have specialised code for steel structure as American code ASCE 7 is as per AISC 341, New Zealand Standard NZS 1170.5 is as per NZS 3404 and EC 8 as per EC 3. For Indian standard we used to refer IS 1893 as per IS 800: 2007. But as per latest release of IS 18168 : 2023 **" Earthquake Resistant Design and Detailing of Steel Structures- Code of Practices**" by Bureau of Indian Standard (BIS) the earlier code used is replaced. The Indian standard code classify the steel structure in three types namely (i) Special Moment Resisting Frame (SMRFs), (ii) Special Concentrically Braced Frames (SCBFs) and (iii) Eccentrically Braced Frames (EBFs).

If compared with other codes, American Code classify moment resisting frame in three ductility classes , OMF, IMF and Special Moment Frame SMF. Concentrically frame in two ductility classes, OCBFs and SCBFs. Single Ductlity class of Eccentrically Braced Frame (EBF).

Eurocode 8 specifies ductility classes of MRFs, CBFs and EBFs in three class: Low-Ductility Class (DCL), Medium-Ductility Class (DCM) and High-Ductility Class (DCH) .

New Zealand Standard NZS 1170.5 divides ductility classes in four categories :

- Category 1- Fully Ductile Structure (FDS)

- Category 2- Structure of limited ductility (LDS)
- Category 3- Nominal Ductile Structure (NDS)
- Category 4 –Elastic Structure (ES)

4.4 Ductility Classification and Response Behaviour/Reduction Factor

While modelling the structure, during strong ground movement, it it make shuru structure get damage and not collapse which can be permitted. It is though advisable for a structure to get deteriorate in extensive shaking, the structure need to be modelled so for earthquake generated forces much lesser than what is expected at the time of strong shaking. Here comes the role of reduction factor, also mentioned as behaviour factor in foreign codes. In Indian Standard, reduction factors are designated as Response Reduction factor, in American Standard it is mentioned as Response Modification factor, Euro code designate them as Behaviour Factor. This factor is constant for particular type of structure, but Euro code has defined the reduction factor different for different type of concentrically braced frame. NZS 1170.5 consider soil type of structure and time period for providing response reduction factor.

Following graphs provide the response reduction factor of different ypes of structural system and the values mention in different codes around the globe.

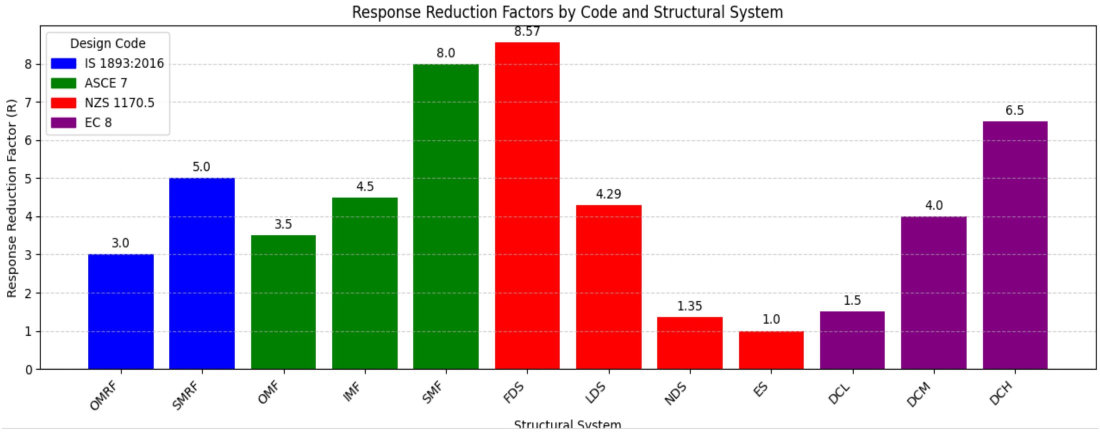


Fig 4.1 Comparing the value of Response Reduction factors for moment frame as mention in different codes

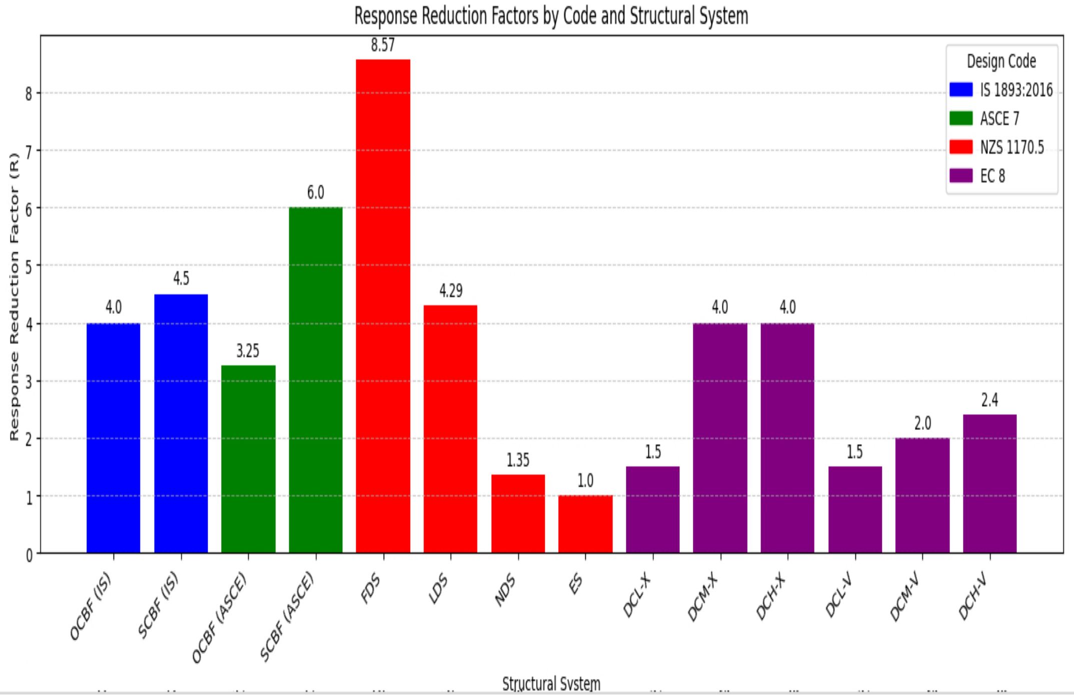


Fig 4.2. Comparing the value of Response Reduction factors for concentrically braced frame as mention in different codes

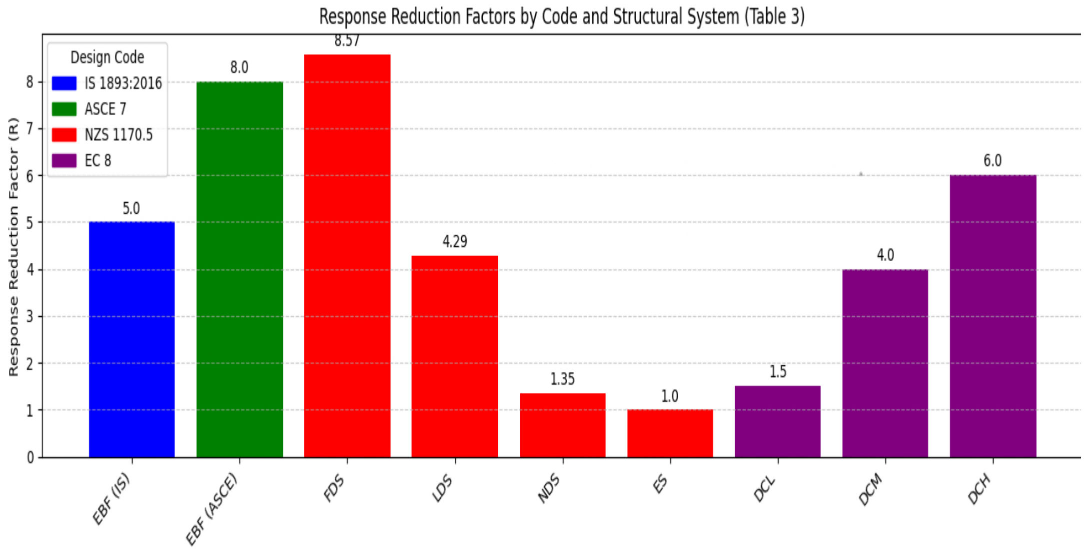


Fig 4.3. Comparing the value of Response Reduction factors for **eccentrically braced frame** as mention in different codes

From the codes, it’s been found the structural classes specified in different codes vary widely, particularly in codal provisions and reduction factors as seen from figures 3.1 to 3.3.

4.5 Comparison of code provision for seismic design of Special Moment Frame Structures

IS 18168:2023 clearly states that the modelling and creation of steel structures or structures must follow the guidelines of IS 800, except where this new standard provides specific modifications for members involved in resisting lateral loads during seismic events Hence we will also specify the difference made in current and advance Indian standard code with the IS 800 and rest of different nation code. The aim of this philosophy is to make earthquake resistant design of structure that safe the structure during extreme earthquake y enabling controlled inelastic deformation in the structure to reduce and absorb seismic energy input[1].

4.5.1 Material & Usage

As per the provision addressed in IS 800:2007, Frame (SMF) shall be made of only prescribed steel and should be able to withstand inelastic deformation w.r.t to joint rotation of 0.04 radians without degradation in strength and stiffness below full yield value. Also as per Clause 12.11.1.1 the SMF systems were allowed to be made in any seismic zones and of any importance factor value of structure.

These provisions were redefined in new code IS 18168:2023. The code allows to make structural steel sections and plates of grade E250 – E350 as per IS 2062. Additionally, the new code limits the use of SMRFs till the zone III and advised only to use SMRFs in certain zone only if the height of structure is less than 15m.

4.5.2 Beam-Column Connection

It is recommended that capacity of columns should be greater than capacity of beam so as to form the plastic hinges at the beam and not in columns, and ductility at global level is maintained. This concept is defined as strong column- weak beam design criteria.

Table 4.2 Comparison of capacity design criteria for moment resisting frame

Capacity Design Criteria	EC 8	NZS 3404	IS 800	AISC 341	IS 18168
Strong Column-Weak Beam Criteria	$\frac{\sum Mpc}{\sum Mpb} \geq 1.3$	$\frac{\sum Mpc}{\sum Mpb} \geq 1.35$	$\frac{\sum Mpc}{\sum Mpb} \geq 1.2$	$\frac{\sum Mpc}{\sum Mpb} \geq 1.0$	$\frac{\sum Mpc}{\sum Mpb} \geq 1.4$

In the codes, $\sum Mpc$ is column moment capacity of section and $\sum Mpb$ the moment of beam section for IS 18168:2023. AISC 341 specifies $\sum Mpb$ the expected of beam and $\sum Mpc$ the expected of column. Whereas EC 8 consider these parameters as nominal moment capacity of sections. As per the provision of NZS 3404, it suggests to consider the column should have designed for combined effect of bending and axial force by taking into consideration the over strength factor. The code also specifies if using composite beam-slab system, to take into consideration the effect of slab is to increase the over strength demand of the column. The IS 800 doesn't specify whether the moment capacity of the column should be calculated for pure flexure. This beam-column strength ratio need not to be checked at roof level as per provision of IS 18168:2023.

4.5.3 Minimum Rotation at Joints

For the design of strong column-weak beam philosophy, the design should help distribute inelasticity. So to achieve this the connection should be properly designed to have sufficient plastic rotation capacity without strength loss.

Table 4.3 Comparison of Join Rotation for moment resisting frame in different national codes

Capacity Design Criteria	EC 8	NZS 3404	IS 800	AISC 341
Minimum Inelastic to be endured at beam- column joint (in radians)	0.0035	0.013[1] 0.03[2] 0.04[3]	0.04	0.04

[1]- for moderate axially Loaded member

[2]- for low axially Loaded member

[3]- for negligibly axial Loaded member

4.5.4 P-Δ Effect

For a specific moment resisting frame, a comprehensive geometric non- linear analysis should be carried out. But the analysis can be neglected for any storey, when the ratio of α_i of the secondary moment compliance with:

$$\alpha_i = \frac{w_i \Delta_i}{H_i \cdot F_i}$$

$\alpha_i > 0.1$ is not permitted as per IS 18168: 2023.

4.5.5 Stability Bracing of Beams at Plastic Hinge Location

So as to avoid lateral torsional buckling of beams, stability of bracing is necessary check at plastic hinge location so as to provide structure with lateral support. The purpose of bracing is to control lateral movement and torsional twist that may occur during a seismic event. Earlier IS 800 doesn't give any specified condition for stability bracing whereas AISC 341 specifies that lateral braces should provide a strength equal to 6% of the anticipated flexural capacity of the beam flange at the specified location. But we found a special requirement for bracings in IS 18168:2023. The code suggests to use panel bracing and not point bracing. Special Bracing shall be located adjacent to expected location of formation of plastic hinge. It's necessary that both flanges of beam are laterally supported. Axial Strength of bracing is given by:

$$P_{br} = 0.06 R_y M_{pb} / d_f$$

4.5.6 Protected Zones

The area located at both ends of a beam, extending a length equal to twice the beam's depth, is subject to inelastic straining and is defined as the protected zone. According to the code, steel headed stud anchors and other fabrication or erection attachments should not be installed on the beam flanges within these protected zones.

4.5.7 Thickness Criteria of column web and doubler plate

Different National Codes shows similar criteria on thickness requirement of column web and doubler plate. This thickness is an important check so to prevent the column web yielding and crippling.

Table 4.4 Column Web and doubler plate thickness comparison

Capacity Design Criteria	AISC 341	NZS 3404	EC 8	IS 800	IS 18168
Criteria of column web and doubler plate thickness	$t > (d_p + b_p)/90$	Limitation has been specified	Slenderness limit for panel zone elements are specified	$t \geq (d_p + b_p)/90$	$t \geq (d_p + b_p)/90$

4.6 Comparison of code provision for seismic design of Special Concentrically Braced Frame Structures

The approach of capacity based design for SCBFs structure are to make columns and beam protected and let braces undergo inelastic failure. Braces includes lateral stiffness and strength to structure frames. The braces shall be laid over full height of structure frames. In SCBFs structure, Earthquake energy is dissipated by:

1. Yielding of tension braces
2. Buckling of Compression braces

4.6.1 Types of Braces

Braces of shape are allowed to used in SCBFs. V and inverted V-braced frame can also be used with some given requirements. Beams that will intersect by braces which are away from beam-to-column connection shall satisfy below conditions:

- Beams should span continuous between columns and adequately braced to prevent lateral torsional buckling
- A set of braces is required at the point of intersection of V-type or inverted V-type frames, unless the connecting beam possess adequate out of plane strength and stiffness to maintain stability between adjacent braces.

K –braces shall be avoided for SCBFs.

4.6.2 Capacity Design approach in SCBFs

The provision in the code is focused on restricting the inelastic behaviour to the braces and to design beam and columns for over strength action from braces.

4.6.3 Lateral force Distribution

Various codes specify the criteria for lateral distribution so as to ensure that under lateral Loading all the braces at same time are not under tension or compression. This balanced the strength of braces in all direction of Loading and energy dissipation of capacity increased.

Table 4.5 Different codes lateral force distribution provision

CODE	Lateral force distribution provision
IS 18168:2023	Bracing should be arranged so that, under lateral Loads in both way, the tension braces carry between 0.3% and 0.7% of the total horizontal force
NZS 3404	Braces to be provided such that Loading in either direction , difference of force component between tension and compression limited to 20%
AISC 341-16	Bracing should be arranged so that, under lateral Loads in both way, the tension braces carry between 0.3% and 0.7% of the total horizontal force

4.6.4 Rotation Capacity

The new Indian Standard code does not give any value for limiting the rotation but put the condition that brace connection can have rotation due to buckling. Also, inelastic rotation of the connection is permitted. Earlier code IS 800 had put the conviction for rotation for concentrically braced frames should be limited to 0.04 radians. Other national code didn't put any limit for rotation capacity.

4.6.5 Multi-tiered braced frames

A special concentrically braced frame can be compose as a multi-tiered braced frame (MT-SCBFs) provided that the braces are placed in opposite pairs at every tier level and horizontal struts provided at every tier level. For such configuration , vertical member are torsional supported packed at the desired location and column should have adequate strength to resist force arising from braces buckling. For all Load combination, columns that are subjected to axial compression need to be designated to resist bending moment due to second order and geometric imperfection effects.

Lateral drift in each tier of multi-tiered concentrically braced frame must not exceed 0.4 percent of tier heights. The structural elements such as column, braces struts and beams in multi-tiered concentrically braced shall comply with width-to –thickness requirements specified in table 2 of IS 18168:2023.

4.7 Comparison of code provision for seismic design of Eccentrically Braced Frame Structures

EBF are designed in accordance with the provision of new Indian standard code [12] is required to provide significant inelastic deformation capacity primarily to shear yielding in the link. It need to be taken into consideration that links shall not be connected directly to columns.

The link must be designed to resist expected shear force found based on analysis required by IS 1893 (Part 1)[13]. The braces member. beam that are outside the links and column shall satisfy the width to thickness ration of table 2 of the code.[12]. It is observed apart for columns, the beam and brace can be subjected to adequate axial and bending forces, therefore their design capacities shall be determined as for beam-column member as per IS 800.

4.7.1 Beam-Column Connection

When a brace or gusset plate is connected to both the beam and column at their intersection, the connection assembly must be designed to resist a beam moment equal to 1.1 Ry fyb Zpb. Additionally, the combined expected flexural capacities of the columns should exceed this moment value. This design moment must be considered alongside the required strengths of the brace and beam connections, including diaphragm collector forces based on the over strength seismic Load.

4.7.2 Link-Rotation Angle

The rotation angle is important parameter basically a variable used to describe inelastic link angle. It is inelastic rotation angle between the link and beam that is outside the link when the condition of total storey height equal to design storey height fulfils.

Table 4.6 Different codes Link-Rotation Angle limitation

Code	**Link Rotation Angle (radian)**
AISC	0.08 (for short link) 0.02 (for long link)
NZS 3404	0.09 (for short link) 0.045(for long links)
EC 8	0.08 (for short link) 0.02 (for long link)
IS 18168:2023	0.08 (for short link)

4.7.3 Link Shear Strength Criteria

IS 18168 take into consideration the shear strength of link on two parameters on is shear yielding and another flexural yielding. AISC 314 and EC8 consider shear force-axial force interaction moment –axial force interaction for the determination of nominal shear and flexural strength but NZS 3404 only take into consideration moment axial interaction which depend on section slenderness parameter and is measure on the relative importance of local buckling and yielding.

Table 4.7 Comparison of link section strength

Parameter	AISC 341		NZS 3404	IS 18168:2023	
V_p	$\frac{P_u}{P_y} \leq 0.15$	$\frac{P_u}{P_y} \geq 0.15$	$0.6F_yA_w$	$\frac{P_u}{P_y} \leq 0.15$	$\frac{P_u}{P_y} \geq 0.15$
	$0.6F_yA_w$	$V_p\sqrt{1-\frac{P_u}{P_y}^2}$		$0.6F_yA_w$	$V_p\sqrt{1-\frac{P_u}{P_y}^2}$
M_p	F_yZ	$M_p\frac{\left(1-\frac{P_u}{P_y}\right)}{0.85}$	$0.75F_yZ_e$	F_yZ	$M_p\frac{\left(1-\frac{P_u}{P_y}\right)}{1}$

4.7.4 Link Section Criteria

Table 4.8 Comparison of link section Criteria

Parameters	AISC 341	EC8	IS 18168	NZS 3404
Section Classification	I-Section (rolled or built up section)	I- Section	I- Section (standard rolled wide-flange section or built up section)	Doubly Symmetric Section
Webs of Links	Single thickness without double plate penetration	Single thickness without double plate penetration and hole	Single thickness without double plate penetration	Single thickness without double plate penetration and web penetration
Moment of Inertia	0.67	-	0.67	-

4.7.3 Link Stiffeners

4.7.3.1 Stiffeners for I-shaped Section

So as to prevent premature buckling, web of links shall be stiffened.

End Web Stiffeners Condition:

- ***i.*** The full depth web stiffeners should be installed on one side of each web link and at point where the diagonal braces connect to end of the link.
- ***ii.*** Stiffeners shall had combined width not less than (b_f -2t_w), where b_f is link flange width and t_w is link web thickness
- ***iii.*** The thickness hall not be lesser than the larger value of 0.75t_w or 10mm.

Intermediate Web Stiffeners Condition:

Links to be provide with intermediate web stiffeners which spaced at intervals not greater than (30t_w – 0.2d).

4.7.3.2 Stiffeners for Box-shaped Section

End Web Stiffeners Condition:

- ***i.*** The full depth web stiffeners should be installed on one side of each web link and at point where the diagonal braces connect to end of the link.
- ***ii.*** These stiffeners are allowed to be welded to outside or inside the face of links web.
- ***iii.*** The stiffeners width shall not be less than b/2, where b is width form inside of box section.
- ***iv.*** Thickness of stiffeners shall not be less than the larger of value of 0.75 t_w or 10mm.

Intermediate Web Stiffeners Condition:

i. Box links to be provided with full depth intermediate web stiffeners welded either to outside or inside face of link web.

ii. If web depth to thickness ratio exceed $19€/\sqrt{f_y}$, the full depth stiffners should be provided on one side each of link web and spacing shall not ne increasing $20t_w – (d- 2t_f)/8$

iii. If web depth to thickness ratio exceed is less than or equal to $19€/\sqrt{f_y}$, then no intermedidate web stiffeners is required.

CHAPTER 5

Performance of Steel Structure of Different Structural System

5.1 Introduction

Structures built by humans are meant to guard us from extreme natural phenomena, whether it's severe weather or climate-based events. But if these structures are poorly designed, then they can experience great damage during natural disasters such as earthquakes; which can result in unexpected number of loss of lives and finances. With earthquakes being relatively rare but significant threats to structures, it is critical to design structures that can resist these forces and respond appropriately[14]. The new IS 18168:2023 gives provision for design of detailing of steel structure to make them earthquake resistant. This study aims to implement the provision and analytically obtain and compare seismic performance of SMRFs, SCBFs and EBFs and obtain result according to provisions. All systems were prepared in ETABS 19 Version.

IS 18168:2023 code also mentions that in seismic zone V, steel structure should use eccentrically braced frame only and specially concentrically braced frame should not be used. Additionally, it limits the implementation of special moment resistant frames in zone IV and V to structures with height less than 15m [12]. With inclusion of new code IS 18168:2023, the design and detailing of steel structure had been improved in respect of Indian scenario. The newly released IS 18168:2023 outlines seismic width-to-thickness limits to achieve a ductile design, ensuring sufficient inelastic deformation capacities. These limits are derived from the AISC 341-16 code. Different countries have their own code of steel frame structure design. In general, seismic design of moment-resisting frame is governed by the serviceability requirements. From research, it was found that the story drift requirement due to serviceability limit design seismic forces in Euro Code 8 (EC8) is much more strict than the equivalent requirement in Japanese Seismic Design Code (BCJ)[15] . Parallel flange sections are more effective than the conventional tapered flange sections used in terms of strength, workability and economy. The NPB section is more efficient than corresponding MB section in bending, as it has a lower mass for the same section modulus about major axis; and NPB section is more efficient than corresponding MB section in compression, as it has a higher radius of gyration about minor axis [16].

5.1.1 Cruciform Shape Section

The structural stability of cruciform-shaped steel section was investigated in this study by comparing it with other open steel section including I, T and U under compression and shear forces [17]. Cruciform column is made by combining two universal beam section where one beam section is cut into pieces from center of web and connected to other beam at mid-depth by welding (fillet). The column section offer higher axial capacity and lesser steel weights[18]. For this study the beam section to make cruciform shape column is selected as narrow parallel flange beam from Indian standards[19].

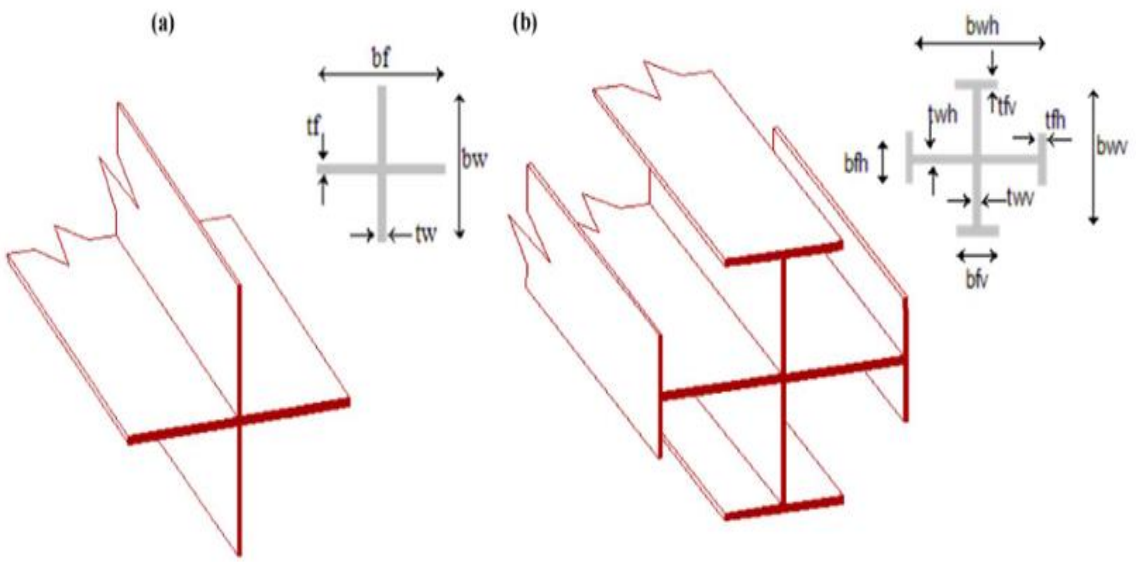


Fig 5.1 Steel Cruciform Section: (a) unstiffened, and (b) stiffened, [17]

5.1.2 Structural Systems

In last few decades, major upgradation in earthquake engineering had done to assess seismic risk and mitigate structural and non-structural damage following major earthquakes. Selecting a structural system that ensures efficient use of its components while meeting all design criteria is crucial [20] . As a result, more robust LFRS system have been developed and integrated into steel structure frames to improve their earthquake resistance. Among these, three key types of steel structural systems are commonly used as LFRS in structure frames: SMRF, SCBF, and EBF. Each of these systems is designed to manage structural damage by utilizing various mechanisms to dissipate seismic energy. Research has found that steel structure with self –centering is gaining popularity because of their prefabricated nature, simple installation and absence of deformation capability under gravity system[21]**.**

5.1.3 Capacity Protected Element

Under seismic design as per IS 1893, ASCE 7, or NZS 1170.5 and now IS 18168:2023, ductile elements are intentionally allowed to get yield, while other structural components (capacity-protected elements) must remain elastic. This ensures:

- Predictable inelastic behavior
- Concentrated energy dissipation
- Prevention of brittle failure modes

Capacity based design is a concept used in the making of earthquake force absorbing structures . It ensures that specific elements of a structures yield in a controlled manner, while other elements are protected to maintain the structure's integrity and to avert total collapse Fardis et al. (2018).The design and detailing code for steel structure specify the element in each structural system need to be treated as Capacity protected element. Table 4.1 shows the types of elements in various system.

Table 5.1 Type of element in Different Structural System

Element	**SMRF**	**SCBF**	**EBF**
Ductile Element	Beam	Braces	Links
Capacity Protected Element	Column	Beam, Column	Beam, Column, Braces

5.2 Objective of this study

The study aims to examine the behaviour of steel structures by employing various structural systems in a multi-story framework, in accordance with the provisions outlined in IS 18168:2023. The code specifies new limiting ratio for selection of strength elements which was used while modelling the structures in ETABS. The analysis of SMRF, SCBF & EBF system is done according to provision specified in the code. The performance of each system is assessed by comparing key seismic response parameters such as **storey drift, maximum story displacement, time period** and **base shear.**

5.3 Methodology and Structural Modelling

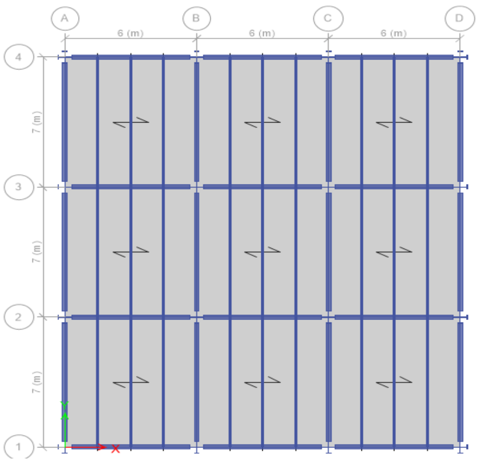
A G+4 storey commercial structure is prepared to serves as test case for this study, situated in **Earthquake Zone IV** to represent regions with moderate seismic activity. Three type of structural systems are compared: (a) SMRF structure, (b) SCBF structure & (c) EBF structure. To have an accurate comparison, all three structural system have the same height, same floor area & floor height.

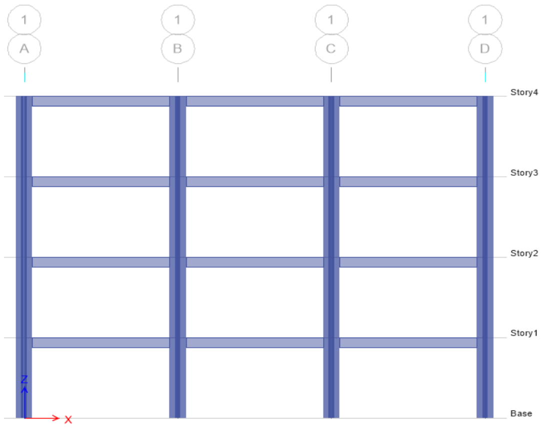
Table 5.2 Structural Configuration

Structural Parameters	SMRFs	SCBFs	EBFs
Plan Dimension	18m X 21m	18m x21m	18m X 21m
Total Height of Structure (from base plate)	14m	14 m	14m
Heights of each storey	3.5m	3.5m	3.5m
Thickness of slab	110 mm	110mm	110 mm
Seismic Zone	IVth	IVth	IVth
Soil Condition	medium Soil	medium Soil	medium Soil
Response Reduction factor	5	4.5	5
Importance factor as per IS 1893-2016	1.2	1.2	1.2
Zone Factor	0.24	0.24	0.24
Dead Load	3 kN/m^2	3 kN/m^2	3 kN/m^2
Live Load at all floors	5 kN/m^2	5 kN/m^2	5 kN/m^2
Grade of concrete	M 25	M 30	M 30
Grade of Structural Steel	Fe345	Fe345	Fe345
Grade of reinforcing steel	Fe500	Fe500	Fe500

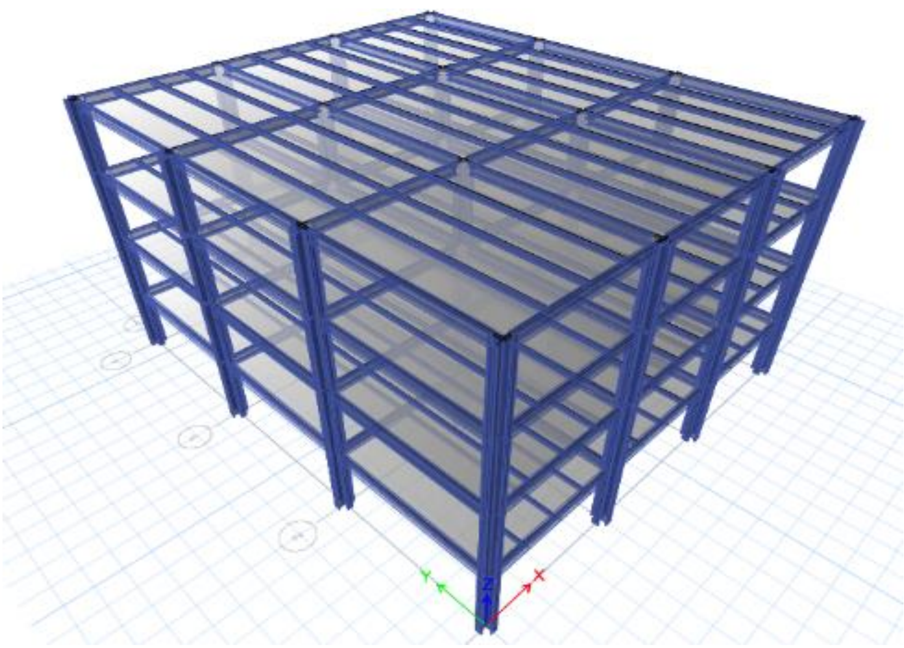
5.3.1 Modelling of Structural System Steel Structure

As per provision of IS 18168:2023, in SMRFs columns need to be designed for capacity based approach and beams will undergo inelastic failure in severe earthquake thus preventing total collapse.





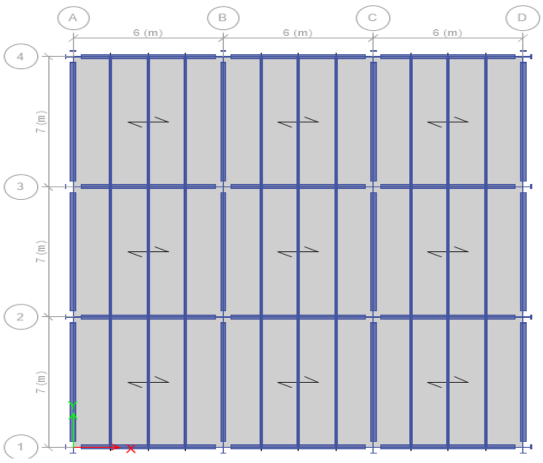
(a) **(b)**



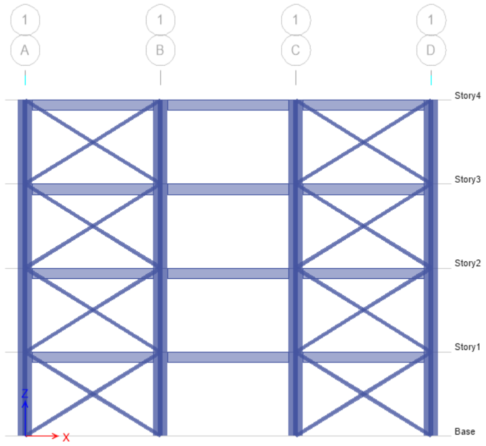
(c)

Fig 5.2 (a) Structure Plan (SMRFs), **(b)** Structure Elevation (SMRFs) and **(c)** 3D Rendered Model (SMRFs)

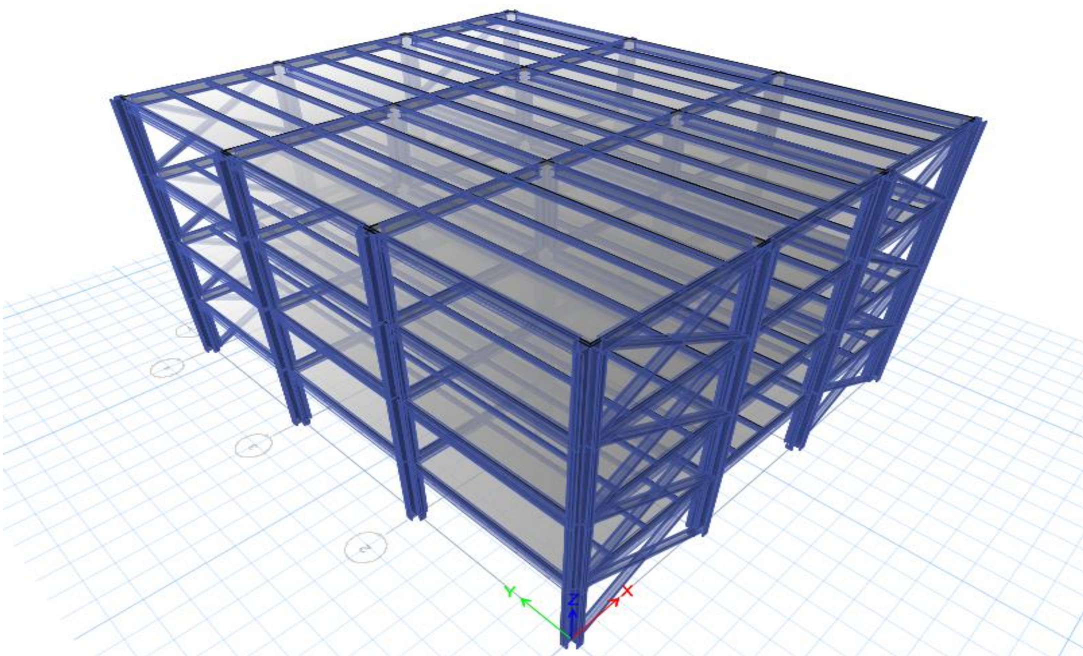
As per provision of IS 18168:2023, in SCBFs columns and beams need to be designed for capacity based approach and braces will undergo inelastic failure in severe earthquake thus preventing total collapse.



(a)



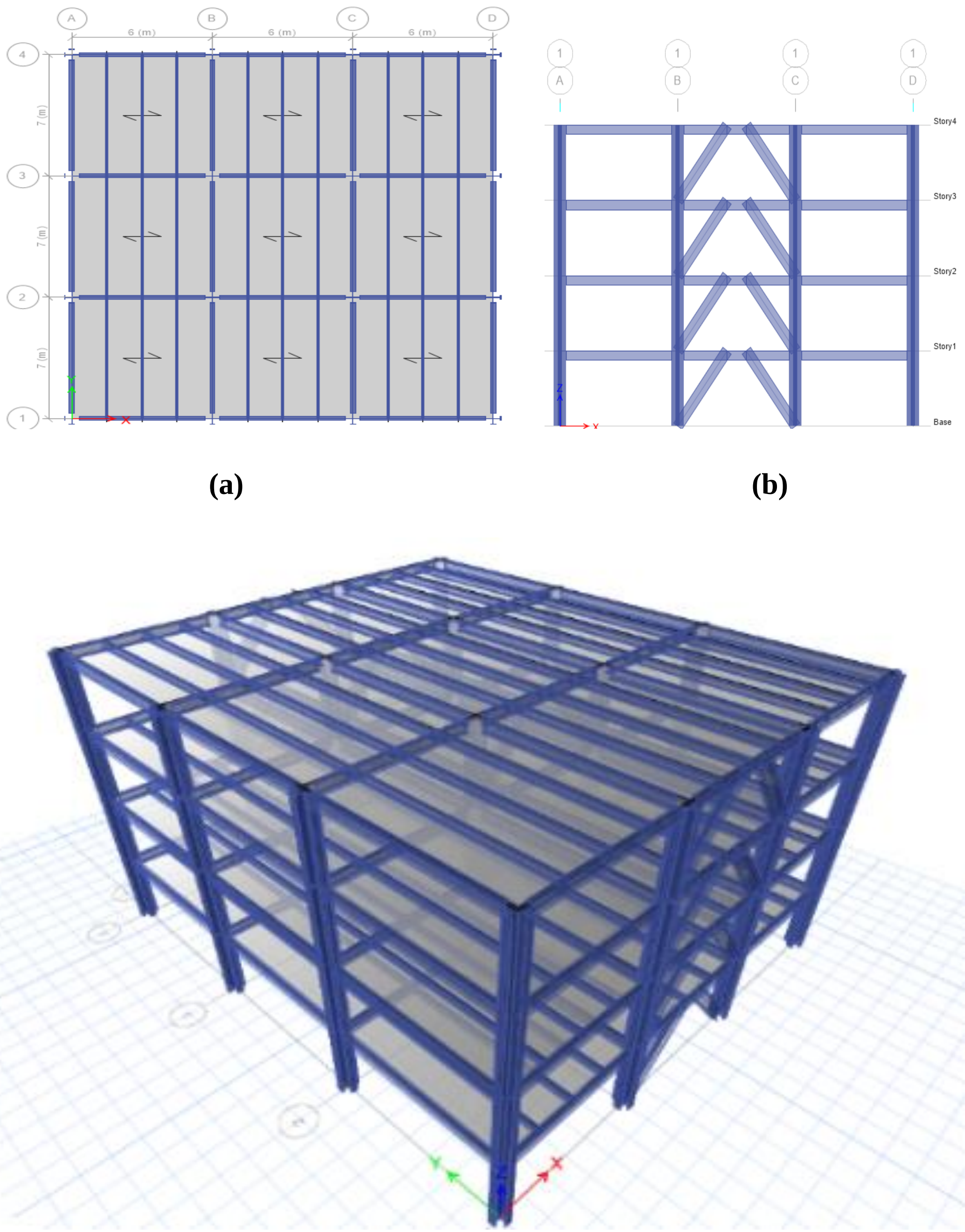
(b)



(c)

Fig 5.3 (a) Structure Plan (SCBFs), **(b)** Structure Elevation (SCBFs) and **(c)** 3D Rendered Model (SCBFs)

As per provision of IS 18168:2023, in EBFs columns, beams and braces need to be designed for capacity based approach and link will undergo inelastic failure in severe earthquake thus preventing total collapse.



(a) **(b)**

(c)

Fig 5.4 (a) Structure Plan (EBFs), **(b)** Structure Elevation (EBFs) and **(c)** 3D Rendered Model (EBFs)

5.3.2 Section Classification

As per the provision of code [12], parallel flange sections are need to be used for designing of structural element. Thus NPB section is used for this study. NPB section are best for construction projects which requires heavy duty supports such as factories, warehouse etc. NPBs are most demanding in crucial project due to its structural integrity and better Load bearing strength. Table 5.3 shows the dimension of parallel flanges used for modelling of structures. NPB sections are given with nominal depth , flange width and mass of element [16].

Table 5.3 Section Dimensions

Section	SMRF	SCBF	EBF
Beam (I-section)	NPB 400 x 200 x 67.28	NPB400 x200 x 67.28	NPB 400 x 200 x 67.28
Column (Cruciform shape section)	NPB 600 x 220 x 122.45	NPB 600x220x122.45	NPB 600 x 220 x 122.45
Secondary Beam	ISLB 175 with 110 mm deck slab	ISLB 175 with 110mm deck slab	ISLB 300 with 110mm deck slab
Bracing (I-section)	-	NPB 250 x 175x 43.94 (X brace)	NPB500 x 200 x 90.69 (Link)

5.3.3 Loads and Loading Combination

The dead Load value is 3kN/m^2 and live Load taken 5kN/m^2 for all structural systems. Equivalent static analysis performed on all 3 models for comparison of result. Load Combination were taken as prescribed by IS 1893(Part 1) and also along with those mentioned in Table 4 of IS 800. Addition Load combination are considered as per IS 18168, (a) for columns in SMRFs, SCBFs, EBFs, (b) beams used in SCBFs and EBFs, (c) braces used in EBFs. γ_{LL} (Partial factor of safety for live Load) is taken 0.5. The over strength factor (**Ω**) as mentioned in IS 18168:2023 is taken 3.

Table 5.4 Dead Load and Live Load Date

Parameters	Value
Dead Load	$3KN/m^2$
Live Load (on floors)	$5KN/m^2$
Live Load (on storey 4)	$1.5\ KN/m^2$

The structure was analysed in two principal horizontal direction for seismic Loads Seismic analysis was done as per IS 1893:2016.

Table 5.5 Load Cases and Combination

Primary Load Cases		
DL LL (EQx) (EQy) (EQ-x) (EQ-y)		
Load Combinations		
IS 1893:2016 (Part-1)	IS 800:2007	IS 18168:2023
1.5DL+1.5LL 1.2 (DL + LL ± EQx) 1.2(DL+LL ± EQy) 1.5 (DL ± EQx) 1.5 (DL ± EQy) 0.9DL ± 1.5 EQx 0.9DL ± 1.5 EQy	DL+0.5LL (Seismic Weight) DL ± EQx DL ± EQy DL +0.8LL ± 0.8EQx DL +0.8LL ± 0.8EQy	0.9DL ± 3EQx 0.9DL ± 3EQy 0.9DL ± 3EQx ± 3EQy 1.2DL +0.5 LL ± 3EQx 1.2DL +0.5 LL ± 3EQy

5.4 Designing of Models in ETABS

All three structural systems were modelled using ETABS Version 19. The primary structural elements—beams, columns, and braces—were designed using narrow parallel flange sections, selected in accordance with IS 808:2021 [18]. The columns across all three models were kept identical and employed a cruciform cross-section, created using the Section Designer tool in ETABS to evaluate the efficiency of this configuration.

The fundamental time period of each structural system was defined manually, following the guidelines and empirical formulas provided in IS 1893 (Part 1):2016 [22]. To ensure compliance with design requirements, the beam-to-column strength ratio was calculated manually to guide the selection of cross-sectional dimensions, verifying their adequacy per the relevant code provisions. Section proportions were checked using the limiting flange width-to-thickness and web depth-to-thickness ratios specified in Table 2 of IS 18168:2023.

The highest permissible steel grade as per the relevant standards—E350 (B0 or C), as referenced in [13]—was utilized in the design. Slenderness ratios were considered in line with code requirements: for columns, the effective slenderness ratio was maintained below 75, and for braces, below 160, to ensure stability under axial Loads. All structural supports were modelled as fixed. Furthermore, each structural member was designed to achieve a utilization ratio (i.e., demand-to-capacity ratio) of less than 1.0, ensuring adequate strength and serviceability under design Loading conditions.

CHAPTER 6
Results and Discussion

6.1 Introduction

Steel structure had got prominence across the globes due to their flexibility, strength and faster construction timeline. This study focuses on G+4 structure located in seismic zone IV , analysed in ETABS 2019 Version.

Following Structural Systems were evaluated:

- SMRF systems
- SCBF systems
- EBF systems

The performance indicators include storey drift, maximum storey displacement, time period and base shear.

6.2 Storey Drift

Storey drift is important parameter for evaluating the performance of a structure during seismic event. As per Indian Standard code, the storey drift in any floor should not exceed 0.004 times the height of storey. The result is valued in both the direction of earthquake (EQx & EQy) as shown in table 6.1 & table 6.2 .

Table 6.1. Storey Drift in X-Direction

Storey	**SMRF**	**SCBF**	**EBF**
4^{th}	0.001258	0.000237	0.000212
3^{rd}	0.001690	0.000509	0.000261
2^{nd}	0.001735	0.000668	0.000264
1^{st}	0.000962	0.000410	0.000169

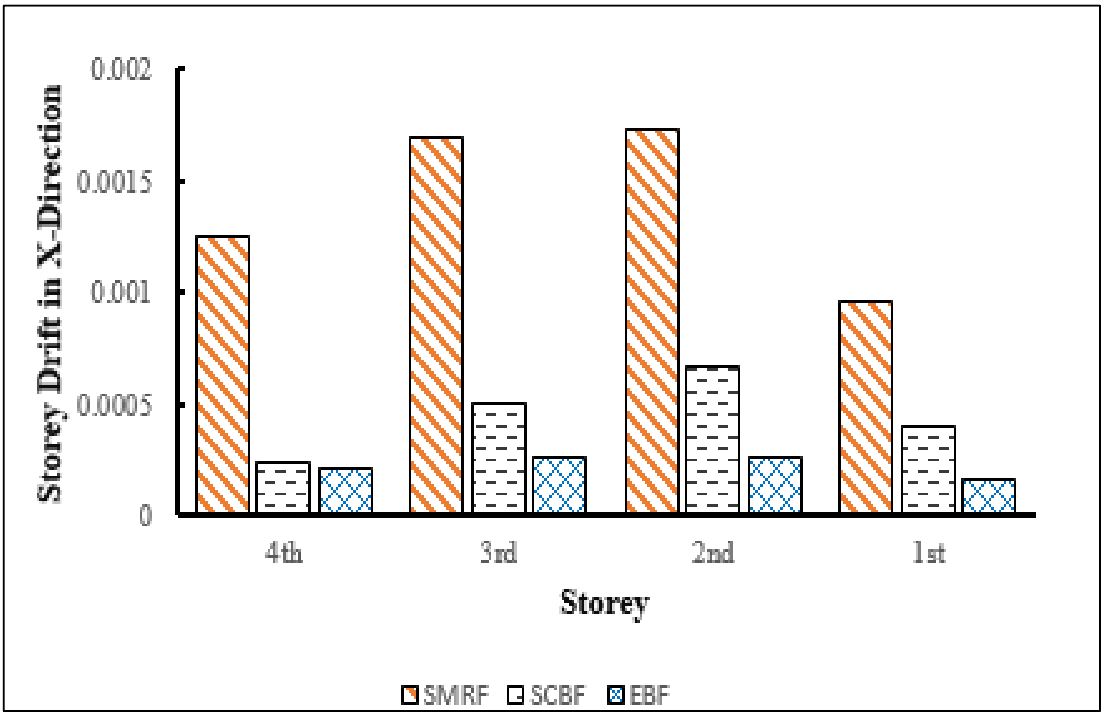


Fig 6.1 Storey Drift v/s Storey (X-Direction)

Table 6.2 . Storey Drift in Y-Direction

Storey	**SMRF**	**SCBF**	**EBF**
4th	0.001418	0.001480	0.001607
3rd	0.001868	0.001949	0.002114
2nd	0.001892	0.001976	0.002138
1st	0.001028	0.001074	0.001157

6.2.1 Result Analysis (Storey Drift in X-Direction)

- In the X-direction, SMRFs shows highest among all structural system as shown in Fig. 6.1, this is mainly because of its flexible nature.
- Ductile element were present in structures: Braces in SCBFs and Links in EBFs
- Beams and column were overdesigned to resist maximum probable force from braces/ link yielding.
- Lower Drift means good performance during seismic event.
- EBFs system would be best in higher seismic prone areas.

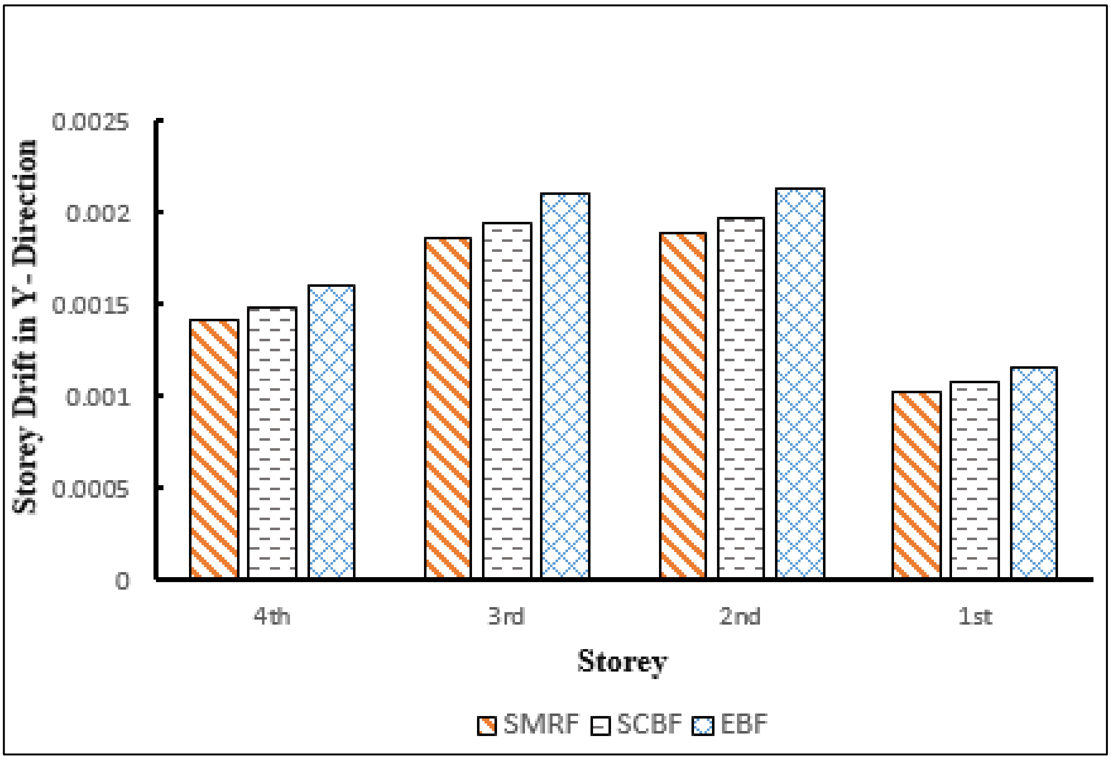


Fig 6.2 Storey Drift v/s Storey (Y-Direction)

6.2.2 Result Analysis (Storey Drift in Y-Direction)

- Drift value in the Y-Direction were higher and similar across all systems.
- SMRFs, SCBFs and EBFs effectively behave as simple the systems lateral load resisting in this direction.
- Energy Dissipation Mechanism (brace yielding or link) were missing in Y-Direction.
- Therefore, the inelastic capacity is reduced, and drift increase to accommodate the lack of energy absorption.

Therefore, lateral system must be designed in both principal directions to prevent torsional irregularities. Ductility must not be assumed unless deliberately introduced while detailing. Ductile yielding is controlled and localized in braces or links. As per new Indian code, capacity design ensures other components remain elastic.

6.3 Storey Displacement

Storey Displacement is defined as lateral horizontal displacement about a fixed reference point, generally the base of the storey. As per the result the displacement of SMRF is maximum in X-Direction but least in Y-Direction in Fig 6.3 and Fig 6.4. Table 6.3 & Table 6.4 shows the storey-wise displacement in X and Y-Direction respectively.

Table 6.3. Displacement in X-Direction (in mm)

Storey	SMRF	SCBF	EBF
1st	3.367	1.721	0.593
2nd	9.439	4.527	1.512
3rd	15.354	6.649	2.427
4th	19.757	7.553	3.170

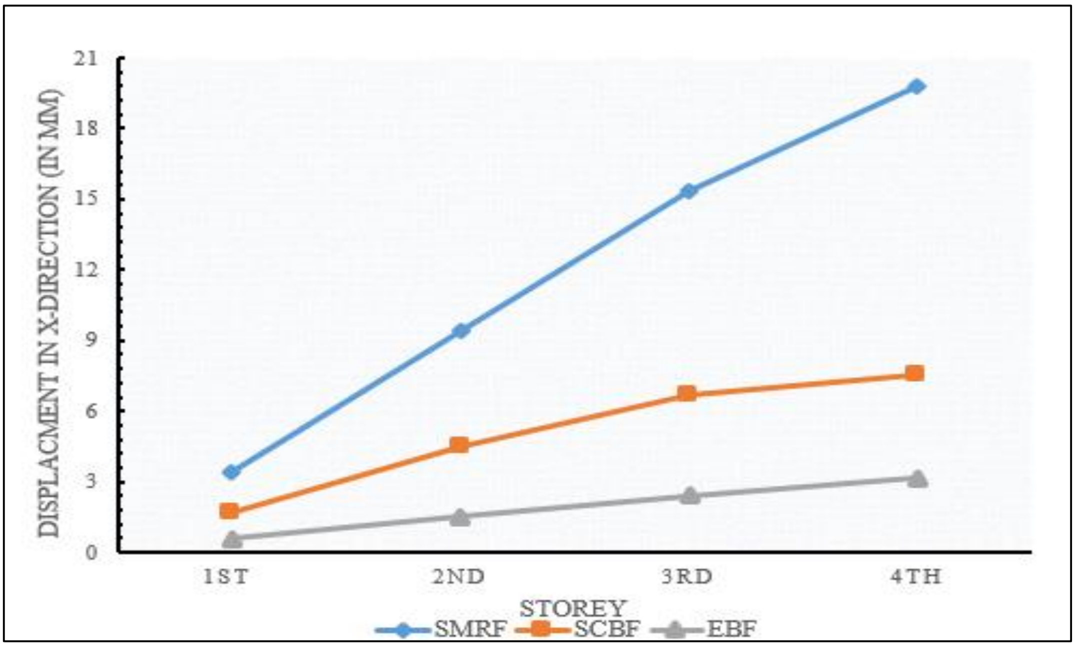


Fig 6.3 Displacement in X-Direction (in mm)

Table 6.4. Displacement in Y-Direction (in mm)

Storey	SMRF	SCBF	EBF
1^{st}	3.597	4.061	4.088
2^{nd}	10.214	11.522	11.637
3^{rd}	16.752	18.889	19.106
4^{th}	21.715	24.478	24.78

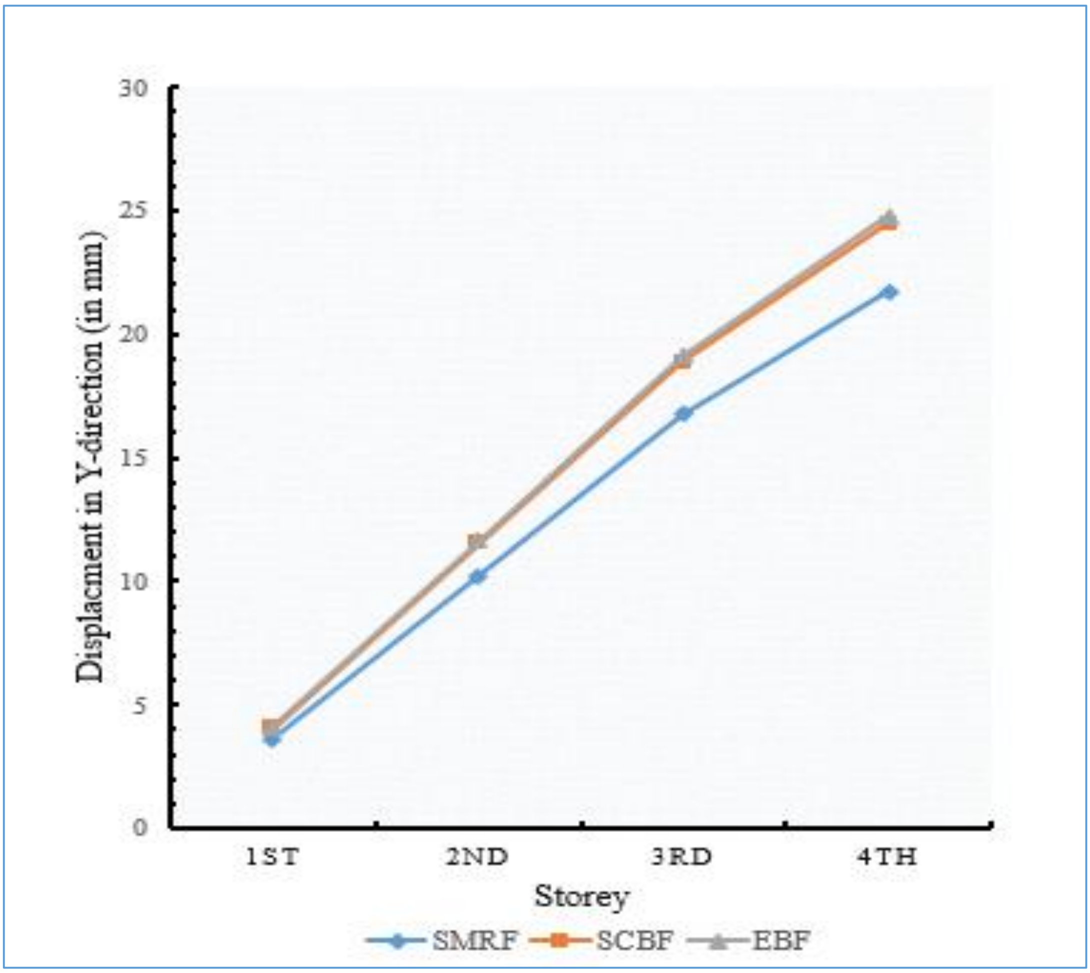


Fig 6.4 Displacement in Y-Direction (in mm)

6.3.1 Result Analysis (Storey Displacement in X –Direction)

- SMRFs in X-direction shows highest displacement as the system relies on flexural stiffness of beam and column.
- Due to braces in SCBFs (only in X-Direction), the displacement is reduced as formation of diagonal and triangulation action of braces.
- EBF shows minimum displacement among three as links provided yields in shear giving stiffness and ductility to system.
- SCBFs are quite effective but their performance depends upon the type of braces and detailing.
- EBFs show good result in controlling the lateral deformation but its complex for link detailing.

6.3.2 Result Analysis (Storey Displacement in Y –Direction)

- In contrary to X-Direction displacement results, SCBFs and EBFs shows higher displacement than SMRF in the Y-Direction. This is beacause, without braces or links, these systems will act as moment resisting frame behaviour thus no additional stiffness or energy dissipation technique.
- As braces and link present in the structure doesn’t contribute to lateral resisting force, the additional mass and altered stiffness distribution, leads toward their modal response which ultimately leads to higher displacement.
- Stiffness asymmetry leads to torsional behaviour.

6.4 Base Shear Value

Base shear is the maximum anticipated lateral force that will be experienced due to seismic ground acceleration at the base of the structure. SCBFs have shown maximum value of base shear in X-direction as the braces are provided to resist lateral forces from X-direction.

The EBFs system is preferable in location with high seismic activity due to its superior ability to resist base shear in both primary directions. The base shear value of EBFs in X-direction is 14.7% higher with respect to SMRF, but lower by 8.7 % with respect to SCBF. But in Y-direction due to change in Response Reduction factor(R), the base shear of EBF is higher by 1.7% the SCBF as shown in Table 6.5 given below.

Table 6.5 X and Y Direction Base Shear (in KN)

Structural System	EQ_x	EQ_y
SMRF	1023.9	1023.9
SCBF	1282.8	1154.6
EBF	1174.3	1174.3

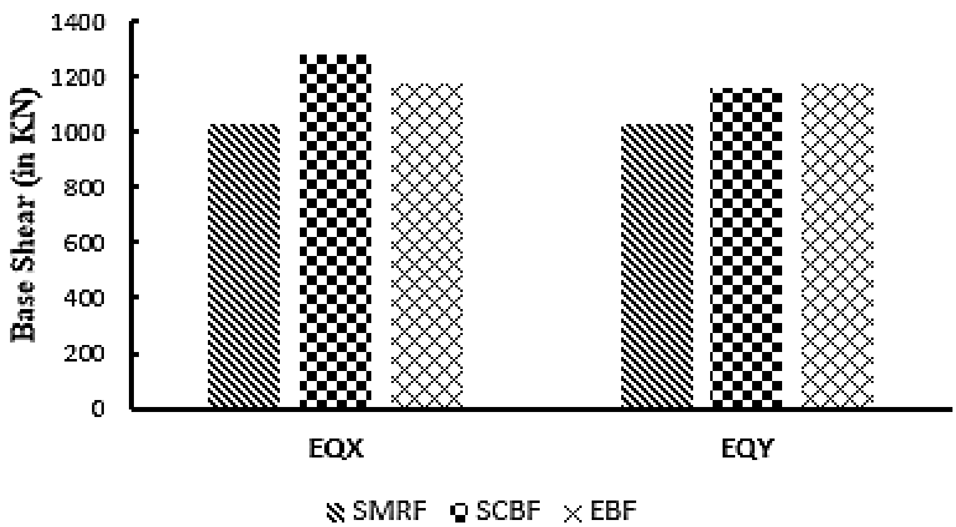


Fig 6.5 Graphical Interpretation of Base Shear in X and Y Direction

6.5 Time Period

Time period is an important parameter in seismic activity of structure as it depicts how the structure responding to ground motion. In all three model prepared, 12 different modes across the height of the structure were shown in Table 6.6. The first mode (fundamental mode) the time period of all structure are very close.

Table 6.6 Time Period

Mode	SMRF	SCBF	EBF
1.	0.868	0.870	0.874
2.	0.830	0.480	0.331
3.	0.705	0.388	0.315
4.	0.244	0.244	0.245
5.	0.236	0.136	0.118
6.	0.200	0.117	0.111
7.	0.117	0.110	0.107
8.	0.115	0.080	0.076
9.	0.097	0.076	0.062
10.	0.075	0.064	0.060
11.	0.075	0.062	0.045
12.	0.063	0.050	0.045

6.5.1 Result Analysis

- The first mode , a fundamental mode show similar value suggesting similar stiffness for first mode (global behavior).
- Mode 2 , onwards SCBFs and EBFs how significantly lower period than SMRFs.

- SMRFs are more flexible, especially in higher mode and it offer better ductility and energy dissipation.
- In SCBFs due to presence of braces it stiffens the frame and reduce time period.
- The EBF are stiffest in higher modes as eccentric bracing offer both the stiffness and energy dissipation from the links.
- SMRFs respond more in higher modes which result more drift and at the same time offer better ductility.

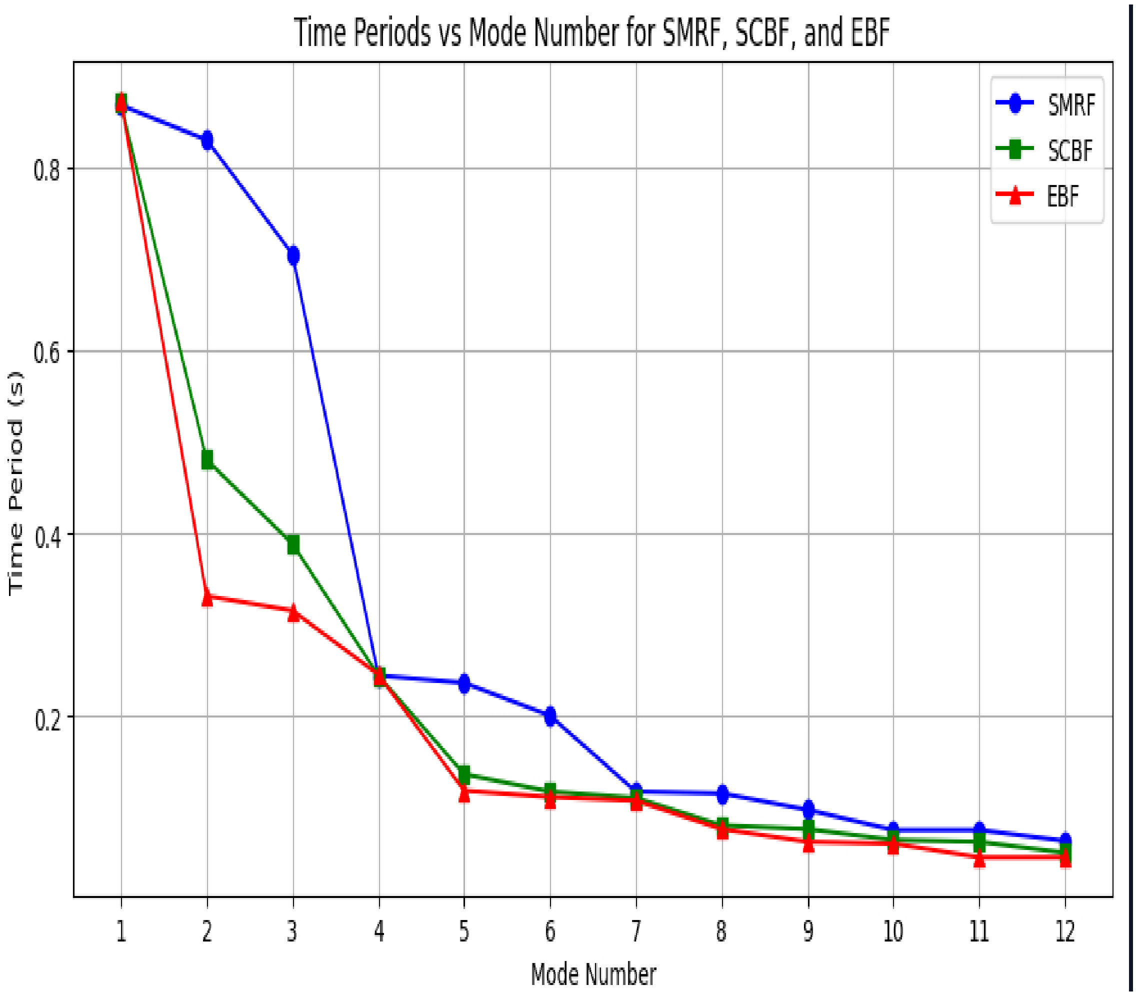


Fig.6.6 Mode v/s Time Period (in sec)

CHAPTER 7
Conclusion

7.1 Summary

The study focusses on the implementation of newly released Indian standard IS 18168:2023 by the Bureau of Indian Standard (BIS). The code was implemented for modelling of structure in ETABS 19 version. The objective of this study required an extensive reading of literature present on this topic as well as research conducted do far in steel structure design.

Chapter 2 of this thesis consist of various outcome of study conducted by research across the globe for giving detailed understanding about the topic. The literature review works as a foundational knowledge and theoretical framework that was important for this research.

Chapter 3 give the detailed knowledge on three types of structural system used for steel structure design namely SMRFs, SCBFs and EBFs that were examined in this thesis. The important structural aspects, behaviour of element during the earthquakes were reviewed. Some guideline for designing of SMRF, SCBFs and EBFs steel structure were presented on study of their behaviour and types of damages occurred in the elements.

Chapter 4 a comparative study is presented to give proper understanding of other countries code about the steel structure. The chapter compares the codes provisions, section classifications, Response reduction factor for SMRFs, SCBFs and EBFs as per American, European, New Zealand and Indian Standard in detailed manner. It also gives comparison of IS 800:2007 with the new code of design and detailing of Steel Structure IS 1816:2023.

7.2 Conclusion

Following Conclusion can be derived from this study:

- EBFs have the maximum base shear value in X-direction, indicating they are stiffer than other structural system. This is due to higher lateral stiffness that was achieved through braces that made yielding confine to certain region. This stiffness reduces time period and higher spectral acceleration, making structure to attract more seismic force.
- It is inferred that SMRF has higher value of storey drift than SCBF and EBF structure making it more flexible and absorb more energy due to seismic event. EBF structure, though having lower storey drift values may me more resistant to lateral force but it could lead to concentrated damage at localized areas of the structure. SCBF and EBF structure shows more flexibility in Y-direction as compared to X-direction. This is primarily because the eccentric link in EBF and braces in SCBFs have been installed for energy dissipation and resistance in X-direction only, they do not resist the lateral force from Y direction.
- SMRFs have the highest participation in both lower and higher mode, suggesting they are more stiff and resistant to dynamic forces. On other hand SCBFs and EBFs, have lower participation in higher mode, making them flexible making them good option in more seismic prone areas.
- It is observed in X- Direction due to presence of braces and links the lateral displacement is reduced by 61% in SCBF and by 83% in EBF when compared with SMRF. The displacement of SMRF(19.47mm) is more in X direction than other structural system and least in the Y-direction with 21.47mm.
- SMRF are highly ductile, yet simple to construct but are flexible thus resulting in higher drift value and displacement. SCBFs are stiff and economical, but while designing the brace buckling must be controlled. EBF shows best seismic performance as the structure are stiff and ductile but their detailing of links is complex.
- The study also reaffirm that braces and link shall be provided in both the principal direction so as to remove torsional irregularities. The inclusion of IS 18168:2023 had improve the predictability by using over strength factor and

advance section classification which met with global standards. The capacity design approach used in this code ensures that the failure occurs in intended and ductile region so as to prevent total collapse during strong seismic event.

7.3 Future Scope

The future scope of the research in dynamic analysis of the structure could lead more light on the code and expand the knowledge in more horizons.

The potential areas for exploration could be:

- Further studies taking into consideration different parameters such as structural configuration, material shape and section could give more insight on seismic performance on their effect on steel structure. Advance analysis techniques such as non linear time history analysis and performance based design can also give better result for understanding the behaviour during seismic event.
- Experimentally validating the result and inclusion of structural health monitoring could be employed to enhance the result and accuracy of the analysis.

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SCOPUS INDEXED CONFERENCES

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