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# SEISMIC VULNERABILITY ANALYSIS OF STEEL FRAME BUILDING UNDER EARTHQUAKE

**Shekhar Ghildyal*,**
**Mirza Aamir Baig**

**ABSTRACT**
Moment-resistant steel frames (MRFs) are widely used globally due to numerous advantages in modern civil engineering construction practices. Their rapid design, faster fabrication and assembly capabilities, availability of practical and artistic shapes of steel sections, high strength, ductility, reliability, and sustainability against adverse conditions contribute to their worldwide acceptance.
For seismic-resistant design in India, the strong column-weak beam (SCWB) concept is employed to design moment frames. This approach promotes the formation of plastic hinges primarily in beams, away from the column face, minimizing the risk of brittle failure at beam-column connections. Typically, outer moment frames are designed to handle lateral loads from the entire structure, while interior frames are designed for gravity loads with simple shear connections. The major drawback of this methodology is that local damages tend to occur in the perimeter frames, resulting in eccentricities. If these eccentricities and the resulting additional torsional moments are not properly accounted for, they can lead to extensive damage or even complete structural collapse. Steel structures are crucial in the construction industry, especially for seismic performance. The Indian code (IS 800-2007) mandates the design of multi-story steel-framed buildings with various bracings, such as X-braced, diagonally braced, alternately diagonally braced, V-braced, inverted V-braced, and K-braced. A study analyzed the performance of diagonal, X, V, and inverted-V eccentric bracings using the SAP-2000 software package. The study found that braced steel frames significantly reduce lateral displacements, have a shorter modal period, and have higher frequencies. The ductility of a moment-resisting steel frame is affected by its height, and this height dependency of ductility is magnified when bracing systems are included.
The findings indicate that lateral displacements experience a notable decrease in braced steel frames. Furthermore, the modal period for various modes of braced steel frames is relatively shorter compared to unbraced frames, with the frequencies of braced steel frames being higher. The ductility of a moment-resisting steel frame is influenced by its height, and this dependency is accentuated when bracing systems are incorporated.

**Keywords:**
Moment resistant steel frame (MRF); push over analysis; time history analysis; comparative study; performance point; modal time-period; base shear.

## 1. INTRODUCTION

The performance of steel buildings under earthquakes depends on various factors, including design principles, structural systems, materials, and construction practices. Advances in technology and a better understanding of seismic behaviour have significantly improved the resilience of steel structures, making them a reliable choice for earthquake-prone regions. Steel buildings exhibit remarkable performance under earthquake conditions due to their inherent ductility, strength, and capacity for energy dissipation. Modern seismic design principles ensure these structures can withstand significant ground motion by incorporating systems such as moment-resisting frames, braced frames, and shear walls, which enhance stability and flexibility. The adoption of performance-based seismic design (PBSD) allows for precise tailoring of buildings to meet specific performance objectives, from operational continuity to life safety and collapse prevention.
Failures of steel buildings under earthquakes can be attributed to a range of factors including design flaws, construction deficiencies, and unanticipated seismic demands. One of the critical vulnerabilities in steel structures is connection failures, particularly in welded joints. Bolted connections, if inadequately designed or installed, can also slip or fracture under seismic loads, compromising the building’s integrity. Buckling of structural members like columns and braces is another common failure mechanism, exacerbated by high lateral loads during an earthquake, leading to a reduction in load-carrying capacity and potential collapse. Additionally, poor design and

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detailing of beam-column joints can result in failures due to high shear forces and bending moments. Foundation issues, such as differential settlement caused by soil liquefaction or inadequate foundation design, can lead to severe structural instability or collapse. Inelastic deformation and the P-Delta effect further compound the risk of failure, where large displacements induce additional moments that can destabilize the structure. Design and construction errors, including insufficient redundancy and poor construction practices, significantly undermine the seismic resilience of steel buildings.

Seismic retrofitting of steel buildings involves the strategic modification of existing structures to enhance their resistance to seismic forces, ensuring they comply with current seismic codes and standards. This process begins with a comprehensive assessment of the building's condition, identifying vulnerabilities such as inadequate connections, insufficient lateral strength, and foundation weaknesses. Common retrofitting techniques include adding concentric or eccentric bracing to increase lateral strength and stiffness, and installing steel shear walls to provide substantial lateral load resistance. Improving connections, such as reinforcing welded joints and enhancing bolted connections with high-strength bolts, is crucial for preventing brittle fractures and ensuring structural integrity. Base isolation systems, which decouple the building from ground motion through flexible bearings, significantly reduce seismic forces transmitted to the structure. Seismic retrofitting is meticulously planned and executed to enhance safety, often involving phased construction to minimize disruption. These measures collectively ensure that retrofitted steel buildings can better withstand earthquakes, protecting occupants and reducing potential damage.

This research is concerned with the pushover analysis of the steel frames. The uses of pushover analysis of the steel frames have been studied extensively in previous studies for experimentally and analytical studies, but limited work is done on the study of pushover analysis of steel frames. Push over analysis is this study is carried out using SAP software. Push over analysis attained importance in the past few decades due to its simple approach and the results are effective. Single Diagonal, X, V and Inverted V frames are the different types of bracings which will be considered for this study. Performance of each frame is studied through nonlinear static analysis (pushover analysis) using a software package SAP-2000. Investigation have been made for the nonlinear damage assessment of the steel frame subjected to a series of Indian Standard (Standard 1893) response spectrum compatible earthquakes.

## 2. LITERATURE REVIEW

This literature review focuses on recent contributions related to pushover analysis of steel frames and past efforts most closely related to the needs of the present work. Various simplified nonlinear analysis procedures and approximate methods to estimate maximum inelastic displacement demand of structures are proposed in literature. The widely used simplified nonlinear analysis procedure and pushover analysis is discussed in detail.

[15] L. Di Sarno, A.S. Elnashai (2008), in this paper, the seismic performance of steel moment resisting frames (MRFs) retrofitted with different bracing systems like; special concentrically braces (SCBFs), buckling-restrained braces (BRBFs) and mega-braces (MBFs) are studied. A 9-storey steel perimeter MRF designed with lateral stiffness insufficient to satisfy code drift limitations in zones with high seismic hazard. The frame then retrofitted with SCBFs, BRBFs and MBFs. Inelastic time-history analysis carried out to assess the structural performance for earthquake ground motions. Maximum storey drifts of MBFs are 70% lower than MRFs and about 50% lower than SCBFs. The amount of steel for structural elements and their connections in configurations with mega-braces is 20% lower than in SCBFs. This reduces the cost of construction and renders MBFs attractive for seismic retrofitting applications.

[16] SandaKoboevic, Jonathan Rozon and Robert Tremblay (2012), this paper is giving the information related to eccentrically bracing system. Analytical models built in three computer programs. Similar maximum forces but the inelastic deformation predictions at the element and global structural levels showed sensitivity to the modelling employed. These features considered in seismic performance assessment or design. Current design methods failed to predict interstory drifts and plastic link rotations, but the study confirmed the strong correlation between the two parameters associated with rigid-plastic behaviour. The influence of yielding and flexural buckling of frame members, other than the links on the global frame performance evaluated using the OpenSees model. Inelastic response of braces and columns in such frames did not negatively affect the overall frame behaviour.

[17] Meng-Haotsai (2012) in this study, performance-based design approach studied for retrofitting regular building frames with steel braces. The pseudo-static response analysis of an idealised elastic-plastic, single degree-of-freedom system for retrofit design approach is used. Three model frames used to study with incremental dynamic analysis. Analytical relationship between the increment of collapse resistance and structural characteristics is derive to determine the design strength & stiffness of the braces. Non-linear dynamic analysis

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results indicate that the column loss response of the braced frames is approximate to the performance target and thus the proposed is feasible for practical applications.

[18] ZasiahTafheem, ShovonaKhusru (2013) modelled six story steel buildings and then analysed due to lateral earthquake and wind loading, dead and live load. The performance of the same steel building has been investigated for different types of bracing system such as concentric (crossed X) bracing and eccentric (V type) bracing using HSS sections. For different types of bracing system in comparison to building with no bracing has reduction in lateral displacement. From the present study, the concentric (X) bracing reduced more lateral displacement and thus significantly contributes to greater structural stiffness to the structure. The inter-storey drift is greatly reduces using of bracing system. From result, the bracing system has more influence on the restriction to relative floor-to-floor lateral displacement.

[19] Panagiota Katsimpini, George Hatzigeorgiou (2020) The seismic performance of steel structure systems subjected to earthquakes was evaluated using nonlinear time-history seismic analyses. The structural outcomes included maximum values for residual interstory drift ratios, base shears, and overturning moments, alongside maximum residual settlement and tilting of the foundations. To examine the impact of soil-structure interaction on these response results, steel building-foundation systems were designed in accordance with Eurocode 8, initially assuming fixed base conditions and subsequently compliant base conditions. The study concluded that for near-fault seismic motions, steel building-foundation hybrid systems designed to European Codes did not reliably ensure good seismic performance. Specifically, it was noted that while the seismic performance of the foundations was generally acceptable, the performance of the steel structures was likely to be unacceptable under near-fault seismic conditions.

[20] Moileen Semdok and Babita Saini (2022) Traditional building codes have been found to be inadequate under seismic conditions, leading to a growing demand for structures that ensure life safety, prevent collapse, and meet post-earthquake occupancy time and repair costs. Performance-based seismic design has been developed to address these requirements. This study examines the performance of a 5-story steel special moment resisting frame building using non-linear dynamic analysis, assessing its ability to meet target performance goals. The results show that performance-based seismic design is a reliable approach for improving seismic performance.

[21] Ayman Z. Abdulhameed, Abdulamir A. Karim (2023) This study focuses on designing a multi-story steel building using ETABS v16 software to handle vertical loads. Five Mega Braced Frame (MBF) systems were used in two scenarios to enhance seismic performance. The building was analyzed using SAP 2000 V20 software, comparing parameters like maximum roof displacement, inter-story drift ratio, and base shear. Results showed improvements in the building's response, with the first scenario reducing maximum roof displacement by 36.08% to 48.29%, and the second scenario reducing it by 36% to 44%. The best configuration was found to be pattern 5 in the first scenario, while pattern 4 was the best in the second scenario.

## 3. METHODOLOGY

### 3.1 Pushover Analysis

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral force with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analyses, superimposed to approximate the force displacement curve of the overall structure. A two- or three-dimensional model which includes bilinear or trilinear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. The structure is subjected to predefined lateral load patterns which are distributed along the building height. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve.

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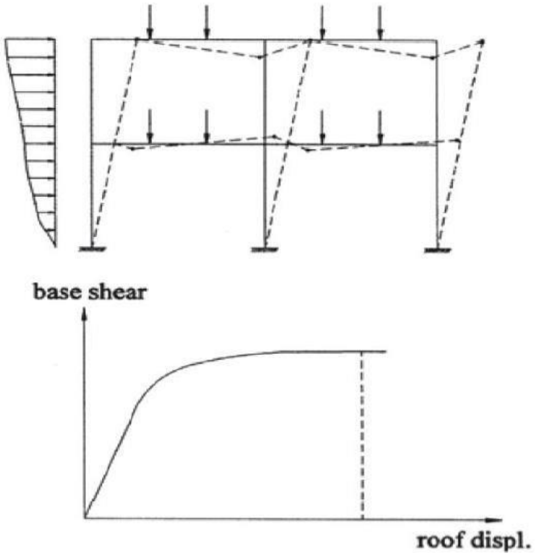


*Figure 1. Illustration of a Pushover Analysis*

Pushover analysis can be performed as either force-controlled or displacement controlled depending on the physical nature of the load and the behaviour expected from the structure. Force-controlled option is useful when the load is known (such as gravity loading) and the structure is expected to be able to support the load. Displacement controlled procedure should be used when specified drifts are sought (such as in seismic loading), where the magnitude of the applied load is not known in advance, or where the structure can be expected to lose strength or become unstable.

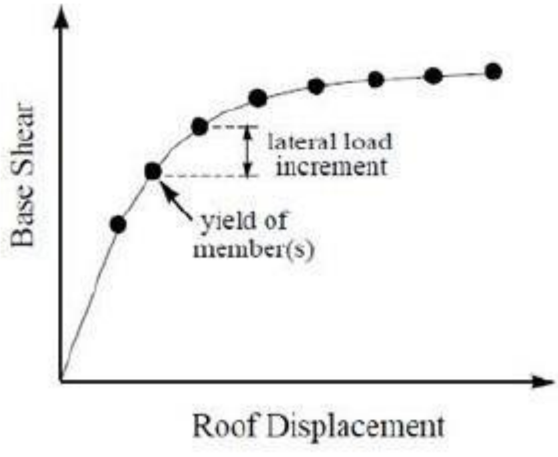


*Figure 2. Global Capacity (Pushover) Curve of Structure*

### 3.2 Time History Analysis

Non-linear time history analysis (NLTHA) is a sophisticated technique used in structural engineering to evaluate the dynamic response of structures under time-dependent loads, such as earthquakes. Unlike linear analysis, NLTHA accounts for material and geometric non-linearities, providing a more accurate representation of how structures behave under extreme conditions. This method involves applying a time-varying load or displacement to a detailed finite element model of the structure and calculating its response at discrete time intervals using iterative solution methods. By incorporating factors such as plastic deformation, cracking, large deformations, and varying boundary conditions, NLTHA enables engineers to predict complex behaviours that linear methods might miss. This detailed analysis is essential for designing structures that can withstand dynamic events, ensuring safety and resilience against natural and man-made hazards.

#### 3.2.1 Selection of Earthquake Records

The selection of earthquake ground motions for seismic analysis in India is a critical task, given the country's diverse seismic zones and the varied geological characteristics. Engineers and researchers must consider historical

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earthquake records from different regions, such as the Himalayas, which are prone to high seismic activity, and the relatively less active peninsular regions. Ground motion records should reflect the range of magnitudes and distances relevant to the site under study, ensuring that the selected motions capture both the frequency content and amplitude characteristics of potential earthquakes. Factors such as soil conditions, local site effects, and the presence of nearby fault lines also play a crucial role in this selection process. Utilizing a comprehensive database of Indian earthquakes, including significant events like the 2001 Gujarat earthquake and the 2015 Nepal earthquake, can provide valuable insights. This rigorous selection process helps in performing accurate non-linear time history analyses, leading to more robust and earthquake-resistant structural designs tailored to the specific seismic demands of different regions in India. All the ground motions listed in table are scaled from 0.1g to 0.6g.

**Table 1.** Earthquake Records

| Event | Year | Magnitude | PGA | Intensity |
|---|---|---|---|---|
| Bihar-Nepal | 1934 | 8.2 | 0.3g | IX |
| India-Burma | 1988 | 7.2 | 0.34 g | VIII |
| India-Bangladesh | 1988 | 5.8 | 0.1 g | VI–VII |
| Garhwal | 1991 | 7.1 | 0.3 g | VIII |
| Uttarkashi | 1991 | 7 | 0.29 g | IX |
| Chamoli | 1999 | 6.6 | 0.34 g | VIII |
| Bhuj | 2001 | 7.7 | 0.38 g | VIII |

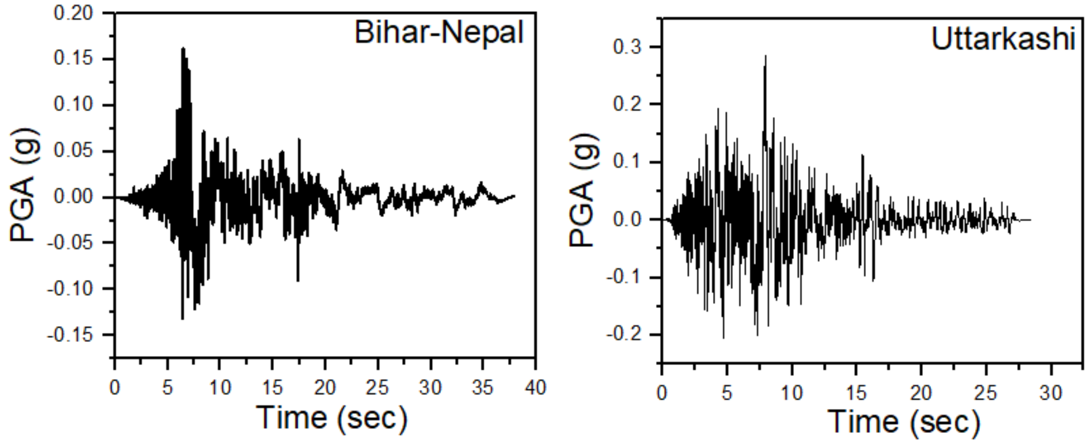


*Figure 3. Acceleration Time history for Bihar-Nepal and Uttarkashi earthquake*

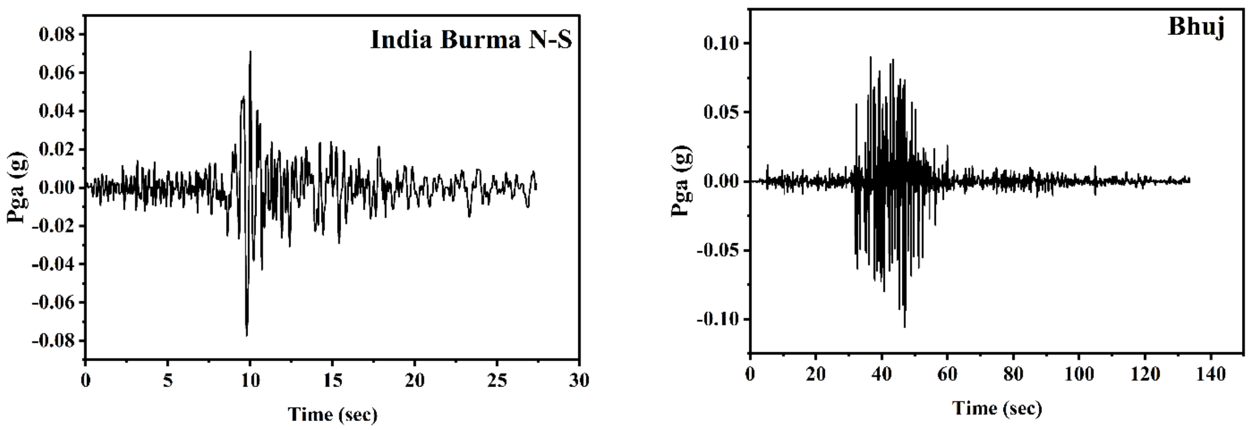


*Figure 4. Acceleration Time history for India-Burma and Bhuj earthquake*

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### 3.2.2 Performance-based Design

Performance-based design is an approach for seismic design and analysis of tall buildings to have predictable and safe performance when subjected to strong earthquake ground motions. The procedure requires a thorough understanding of ground shaking hazards, the behaviour of materials, and nonlinear dynamic response. The approach to performance-based design followed in several nations relies on component-based evaluation, where each component of the building (beam, column, wall, etc.) is assigned as normalized force/moment-deformation/rotation relation, as shown with the help of Figure 5 given below.

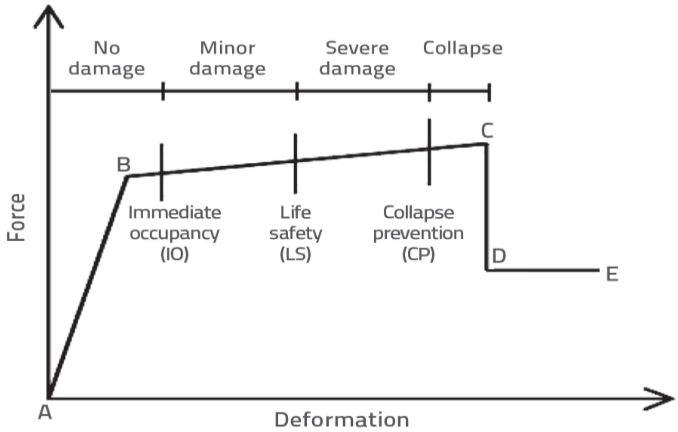


***Figure 5. Force-deformation behaviour***

In which,

AB = represents elastic behaviour, C = represents the beginning of loss of capacity, DE = represent the residual capacity of component, and E = identifies the ultimate inelastic deformation/rotation.

Components are classified as primary (P)/secondary(S) and has been assigned to different deformation limits corresponding to several performance objective. Immediate occupancy (IO), Life safety (LS), and Collapse Prevention (CP) represent the target building performance objectives and briefly explained below.

**Immediate occupancy**: When there is no significant damage in the building during an earthquake and the structure is ready to be occupied immediately. However, some damage to the non-structural system is expected. Repair and cleanup may be needed.

**Life safety**: When the structure has suffered no significant structural damage besides limited architectural failure and can be occupied with sufficient repair work without any life hazards. Such as cracking in vertical load-resisting systems, spalling of concrete, failure of wall segments, etc.

**Collapse prevention or near collapse prevention**: When the structure suffers severe damage during ground shaking and loses most of its pre-earthquake strength or stiffness and the building is near collapse. The structure becomes unsafe for occupancy, and repair and restoration are probably impossible i.e. extensive cracking and spalling of concrete, buckling, large drift, and block of the exit due to rubble.

### 3.3 Structural Modelling

The present study is based on nonlinear analysis of steel frames with and without different type of bracings models. Different configurations of frames are selected such as Single Diagonal, X, V and Inverted V frames. This section presents a summary of different parameters defining the computational models, the basic assumptions considered, and the different steel frame geometry considered in this study.

### 3.3.1 Frame Geometry

The details of framed building are as given in Figure 6. The building is assumed to be symmetric in both direction (i.e. X & Y direction). Typical bay widths in X and Y direction and column height in this study are selected as 4m, 3.5m and 3.2m respectively. A configuration of G+9 building is considered in this study.

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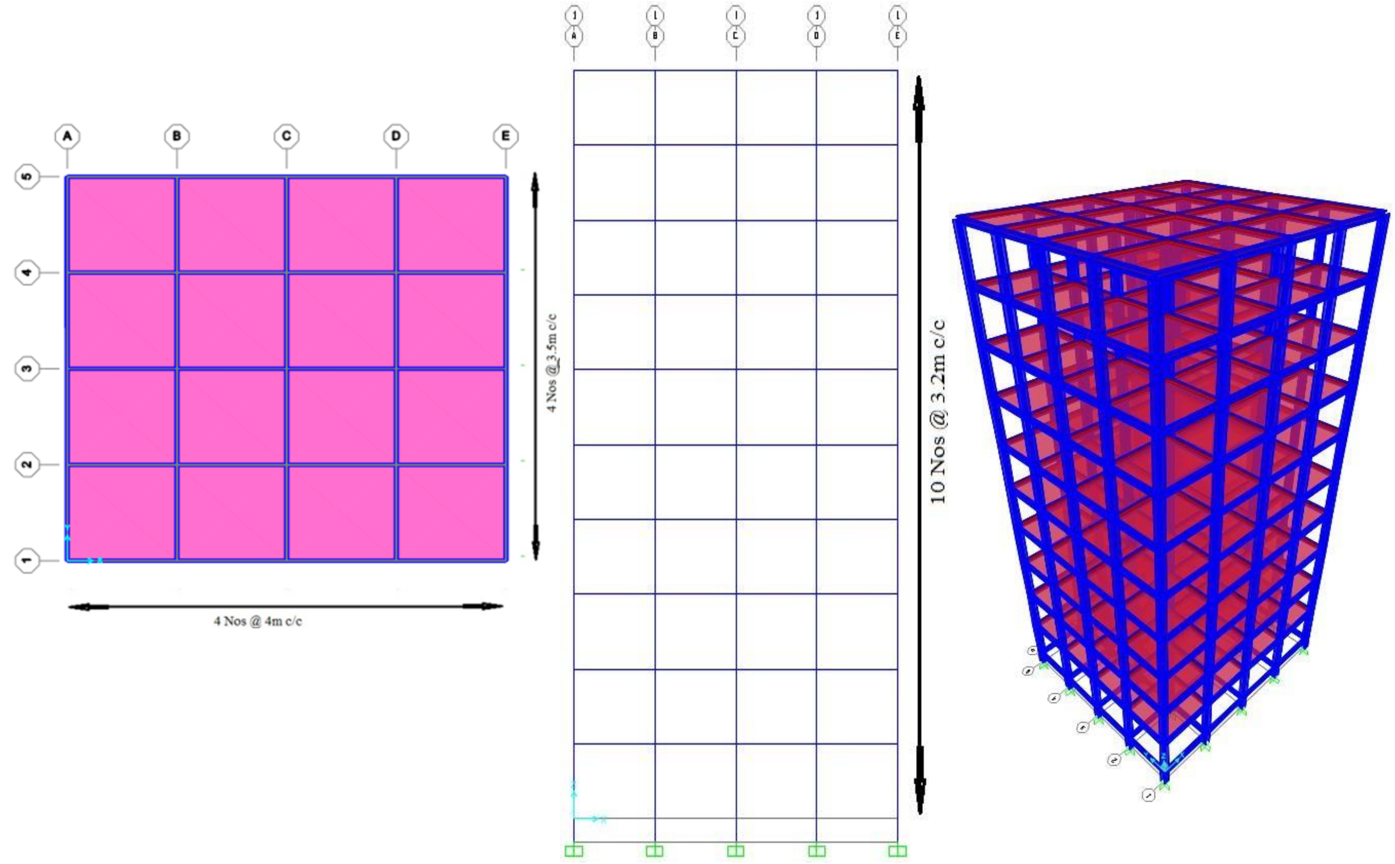


*Figure 6. Plan, elevation and 3D view of steel frame without steel bracing*

The different arrangements of steel frames such as Single Diagonal, X, V and Inverted V frames as considered for study are shown in Figure 7.

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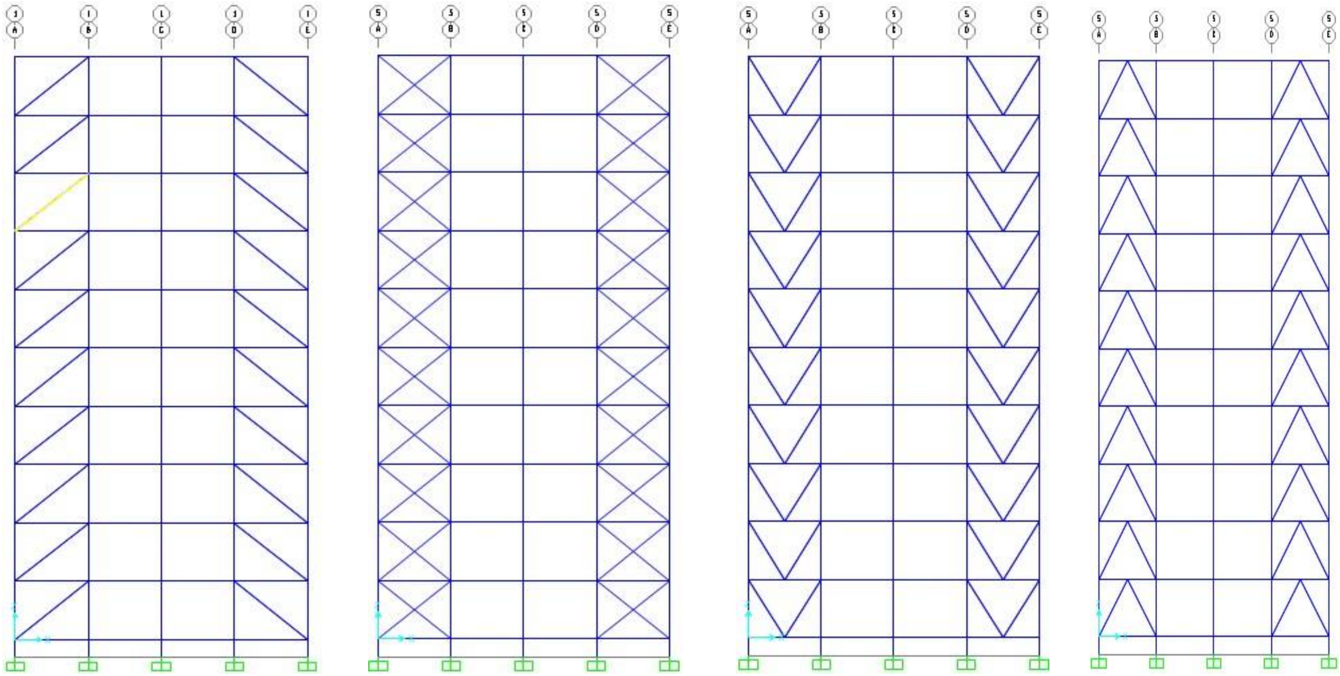

*Figure 7. Elevation of Frame in Z direction (vertical) for steel frame with Single diagonal, X, V, and inverted V- bracing*

### 3.3.2 Frame Loads

The details of various loads considered on the steel frame are as follows.

i) Self-weight: Self-weight of beams, columns and slabs is automatically calculated by analysis program.
ii) Wall load:
   a. Periphery wall = 12.57 kN/m
   b. Partition wall = 6.29 kN/m
   c. Parapet wall = 6.29 kN/m
iii) Roof treatment load: 1.5 kN/m$^2$ uniform on roof.
iv) Floor finish load: 1.0 kN/m$^2$ uniform on all floors.
v) Roof live load: 1.5 kN/m$^2$ uniform on roof.
vi) Floor live load: 3.0 kN/m$^2$ uniform on all floors.

### 3.4 Frame Design

The building frame considered in this study is assumed to be located in India seismic zone IV with medium soil conditions. The design peak ground acceleration (PGA) of this zone is specified as 0.24g. The frame is designed as per standard practice in India. Seismic loads are estimated as per IS 1893 (2016) and the design of the steel elements are carried out as per standards of IS 800 (2007). The characteristic strength of steel is considered 410MPa (Fe410 steel). The design horizontal seismic coefficient ($\alpha_h$) is calculated as per IS 1893 (2016).
Where, seismic zone factor, Z = 0.24, Importance factor I = 1.0, Response reduction factor R = 4.0.
Figure 8 shows the designed cross section details as per IS 800: 2007 of steel beams, columns and bracings used in models.

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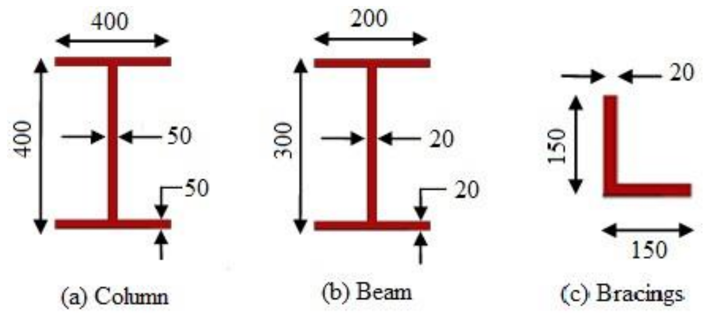


*Figure 8. Cross sectional details of the framed elements*

### 3.5 Modelling for Nonlinear Analysis

Non-linear cases for all steel frames are defined in the program as shown in Figure 9 and Figure 10 below.

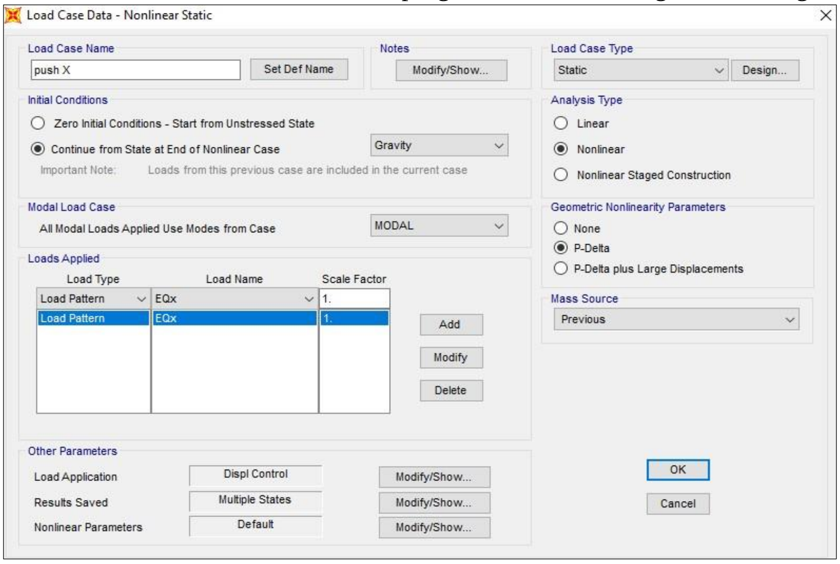


*Figure 9. Push case in global X-direction*

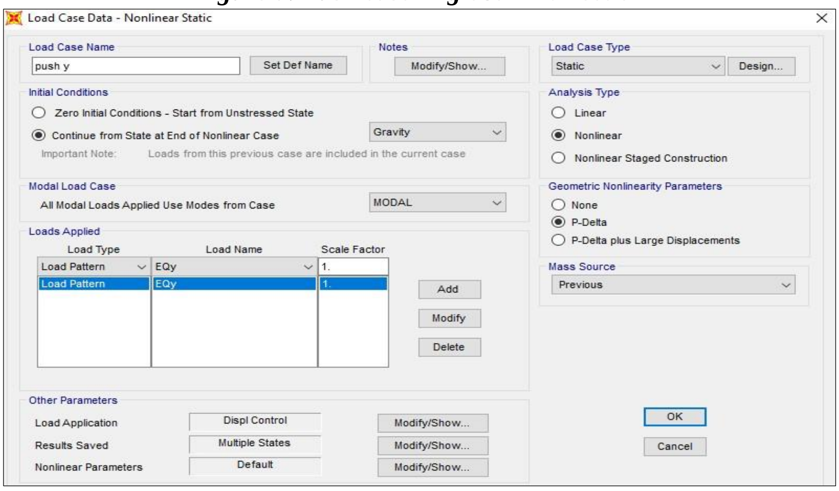


*Figure 10. Push case in global Y-direction*

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## 4. ANALYSIS RESULTS

### 4.1 Static Pushover Curve

Static pushover curve are obtain from program considered FEMA 440 equivalent linearization and displacement modification as stated below. Similar performance curve are obtained for X and Y-direction with Y-direction curve with slightly higher displacements.

**FEMA 440 Equivalent Linearization**

From this method, performance point is calculated comparing capacity curve and demand curve. In Figure 11 to Figure 15, red and green lines represent demand and capacity curves respectively. Capacity curve calculated using the spectral acceleration vs spectral displacement. Demand curve calculated from ground acceleration and period of the structure. The point where capacity curve and demand curve cross each other is called performance point of the structure in the expected seismic activity. Pushover curve gives us various information related to base shear, displacement, effective period and effective damping at the performance point.

**FEMA 440 Displacement Modification**

In this method, the displacement modification factors are applied to the maximum deformation of an equivalent elastic single-degree-of-freedom (SDOF) system, hence estimate the maximum inelastic displacement demand of the multi degree of freedom (MDOF) system. In the FEMA-356 document, the Displacement Coefficient Method (DCM) used to characterize the displacement demand as shown in Figure 11 to Figure 15. This method estimates the elastic displacement of an equivalent single-degree-of-freedom SDOF system assuming initial linear properties and damping for the ground motion. In this method, the demand represented by reducing the elastic demand spectra by the correction factor to the inelastic demand spectra (constant-ductility demand spectrum) which are more accurate than the elastic spectra, with equivalent viscous damping.

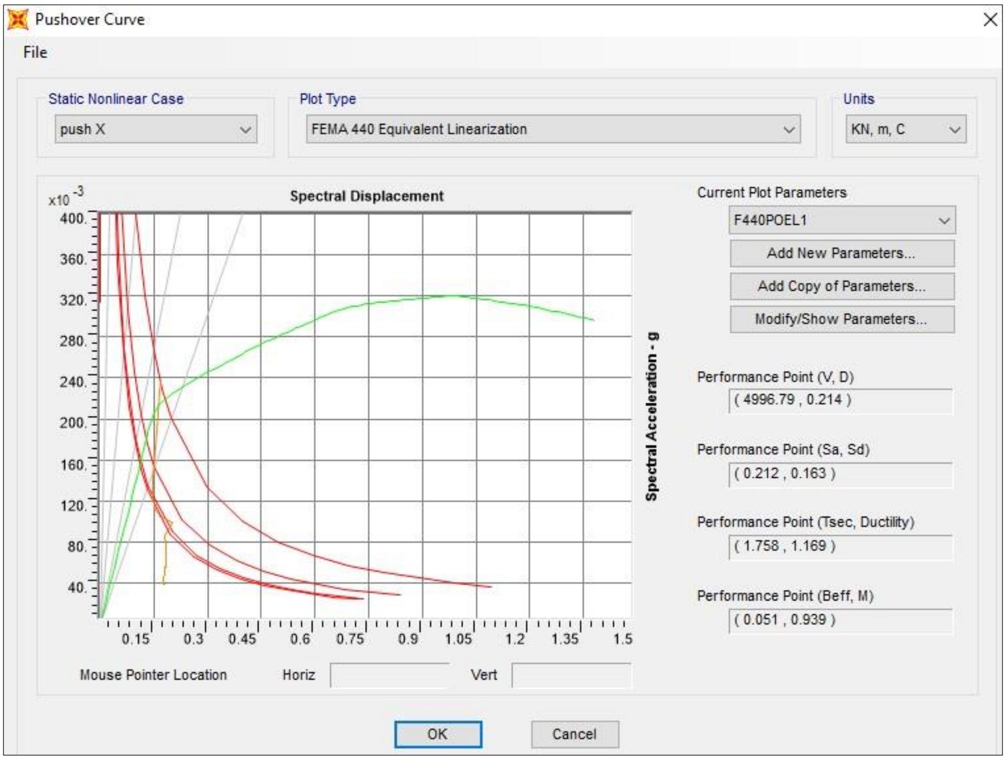


*Figure 11. Static pushover curve for push X case in frame model without bracing*

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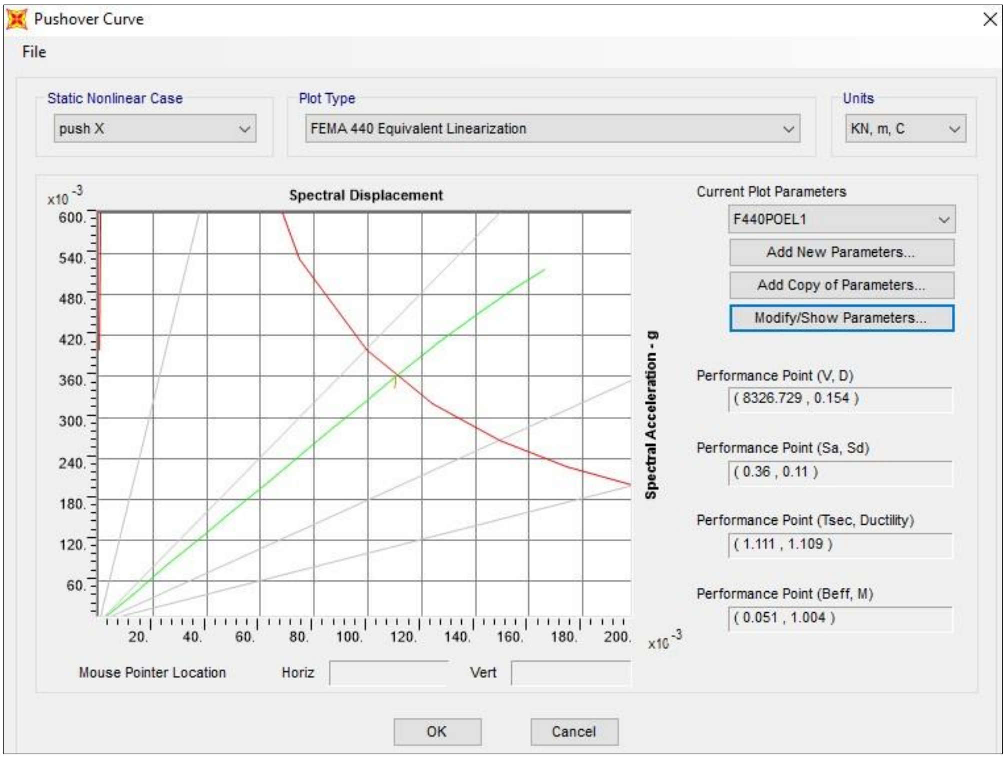


*Figure 12. Static pushover curve for push X case in frame model with single diagonal bracing*

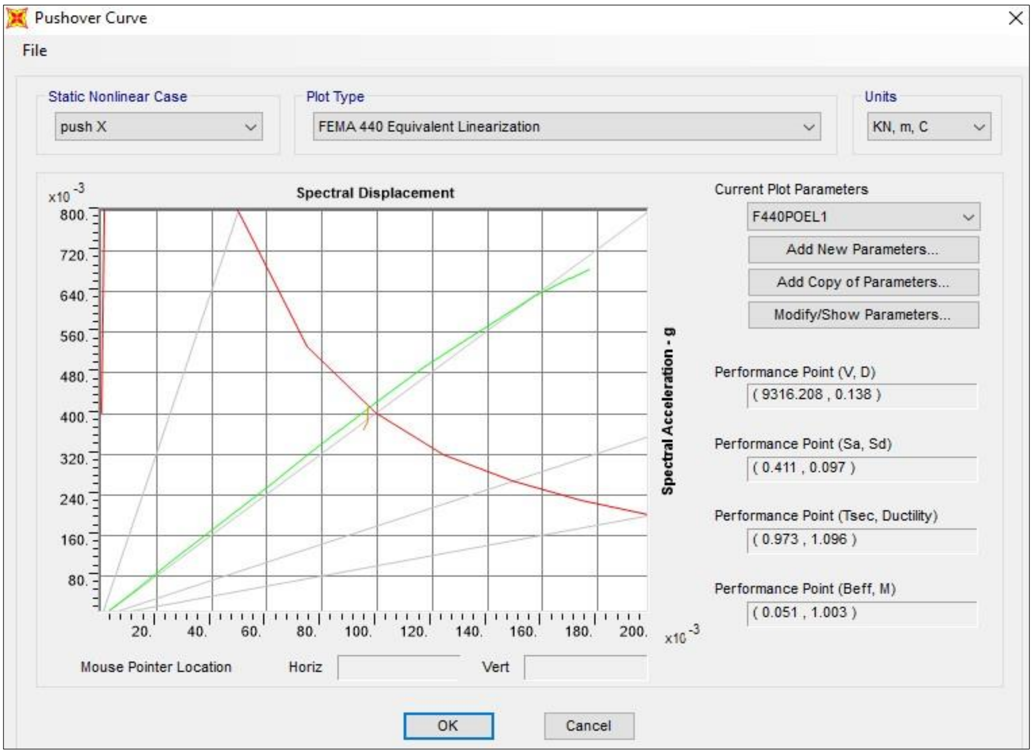


*Figure 13. Static pushover curve for push X case in frame model with X-bracing*

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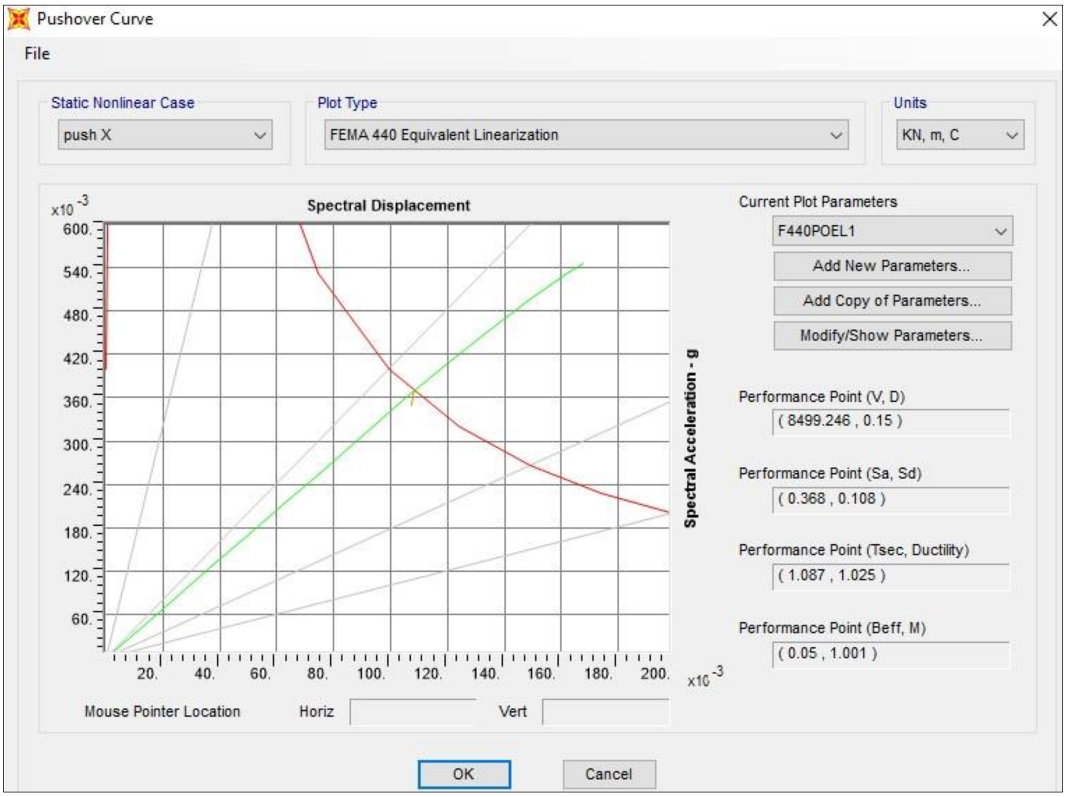


*Figure 14. Static pushover curve for push X case in frame model with V-bracing*

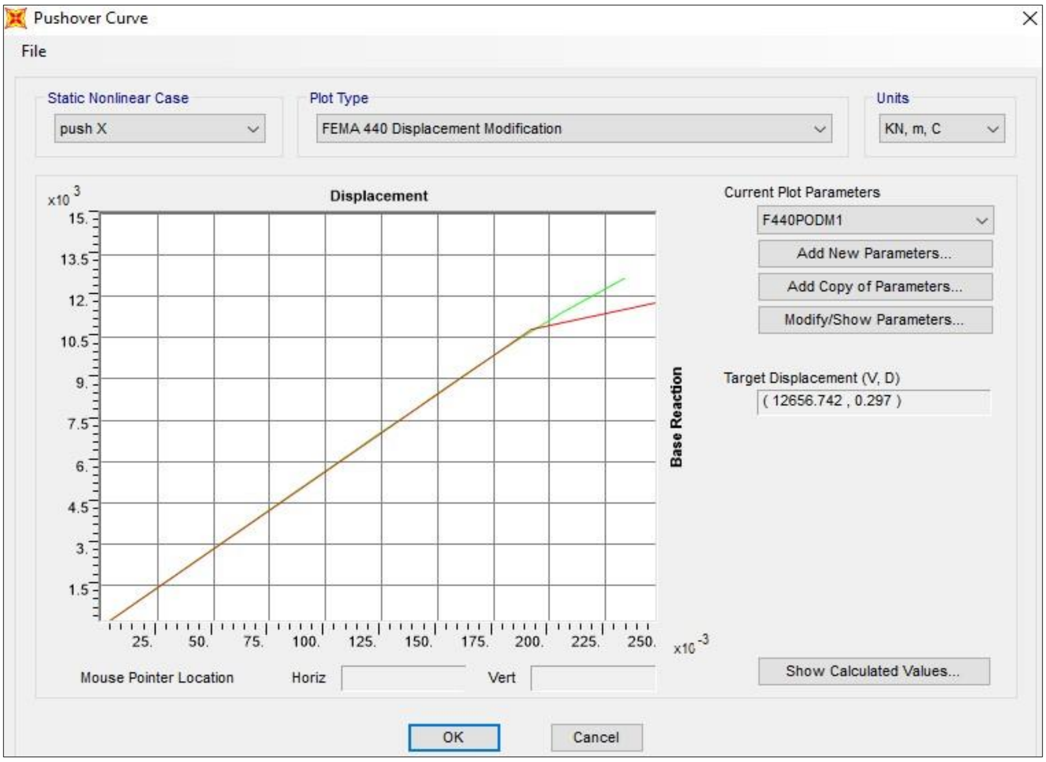


*Figure 15. Static pushover curve for push X case in frame model with inverted V-bracing*

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### 4.2 Hinge Results

Hinge formation with incremental load application result in following outcome for both directions, interpretation shall be done considering force deformation behaviour shown in Figure 5.

*Table 2. Hinge result table for push X case*

| Type of Frame | Step | A to B | B to IO | IO to LS | LS to CP | CP to C | C to D | D to E | Beyond E |
|---|---|---|---|---|---|---|---|---|---|
| Without bracing | 83 | 344 | 126 | 102 | 43 | 29 | 34 | 4 | 2 |
| Single Diagonal Bracing | 73 | 316 | 143 | 4 | 0 | 0 | 4 | 0 | 0 |
| X- Bracing | 96 | 271 | 157 | 0 | 6 | 2 | 14 | 0 | 0 |
| V- Bracing | 105 | 190 | 152 | 4 | 0 | 0 | 4 | 0 | 0 |
| Inverted V- Bracing | 116 | 210 | 132 | 2 | 2 | 0 | 4 | 0 | 0 |

*Table 3. Hinge result table for push Y case*

| Type of Frame | Step | A to B | B to IO | IO to LS | LS to CP | CP to C | C to D | D to E | Beyond E |
|---|---|---|---|---|---|---|---|---|---|
| Without bracing | 70 | 263 | 40 | 82 | 34 | 29 | 176 | 2 | 2 |
| Single Diagonal Bracing | 121 | 213 | 129 | 4 | 2 | 0 | 6 | 0 | 0 |
| X- Bracing | 129 | 179 | 142 | 15 | 4 | 0 | 16 | 0 | 0 |
| V- Bracing | 132 | 156 | 186 | 4 | 0 | 0 | 5 | 0 | 0 |
| Inverted V- Bracing | 142 | 144 | 226 | 54 | 2 | 0 | 13 | 0 | 0 |

### 4.3 Lateral Displacements

Storey wise maximum lateral displacement for all the different models is shown below.

*Table 4. Storey wise maximum displacements for Push-X case*

| Storey | Steel frame without bracing (m) | Steel frame with Single Diagonal Bracing (m) | Steel frame with X Bracing (m) | Steel frame with V Bracing (m) | Steel frame with inverted V Bracing (m) |
|---|---|---|---|---|---|
| **1** | 0.04 | 0.021 | 0.024 | 0.022 | 0.021 |
| **2** | 0.058 | 0.044 | 0.046 | 0.047 | 0.044 |
| **3** | 0.070 | 0.070 | 0.070 | 0.074 | 0.070 |
| **4** | 0.097 | 0.096 | 0.097 | 0.102 | 0.097 |
| **5** | 0.189 | 0.123 | 0.125 | 0.130 | 0.125 |

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| | | | | | |
|---|---|---|---|---|---|
| **6** | 0.387 | 0.150 | 0.154 | 0.158 | 0.152 |
| **7** | 0.541 | 0.175 | 0.183 | 0.185 | 0.177 |
| **8** | 0.650 | 0.197 | 0.209 | 0.208 | 0.200 |
| **9** | 0.716 | 0.216 | 0.234 | 0.229 | 0.220 |
| **Roof** | 0.750 | 0.231 | 0.254 | 0.244 | 0.234 |

***Table 5. Storey wise maximum displacements for Push-Y case***

| Storey | Steel frame without bracing (m) | Steel frame with Single Diagonal Bracing (m) | Steel frame with XBracing (m) | Steel frame with VBracing (m) | Steel frame with inverted VBracing (m) |
|---|---|---|---|---|---|
| **1** | 0.076 | 0.021 | 0.021 | 0.023 | 0.032 |
| **2** | 0.090 | 0.043 | 0.042 | 0.048 | 0.063 |
| **3** | 0.124 | 0.068 | 0.065 | 0.075 | 0.098 |
| **4** | 0.255 | 0.094 | 0.092 | 0.105 | 0.135 |
| **5** | 0.264 | 0.121 | 0.120 | 0.134 | 0.174 |
| **6** | 0.237 | 0.148 | 0.149 | 0.163 | 0.211 |
| **7** | 0.369 | 0.173 | 0.177 | 0.191 | 0.247 |
| **8** | 0.457 | 0.196 | 0.204 | 0.215 | 0.278 |
| **9** | 0.506 | 0.216 | 0.229 | 0.236 | 0.305 |
| **Roof** | 0.529 | 0.232 | 0.250 | 0.251 | 0.325 |

### 4.4 Time History Analysis

The time history method shall be based on an appropriate ground motion (preferably compatible with the design acceleration spectrum in the desired range of natural periods). It shall be performed using accepted principles of earthquake structural dynamics. For this study, the Time History acceleration data of the five Indian earthquakes has been adopted. The ground motion time histories were downloaded from the cosmos website (https://strongmotioncenter.org/vdc/scripts/default.plx). The details of the ground motions considered are given in Table 1. The effect of site conditions was not explicitly studied in the present work. Five different earthquakes were considered for the study, with two components in each. The two components of the earthquake were treated as independent ones. Based on the time history analysis on the selected frames the results are listed below.

#### 4.4.1 Base Shear

Maximum base shear for all time-history load cases is shown in Figure 16, it is clear that the Uttarkashi earthquake leads to the largest value of base shear in both directions, which is reasonable as this earthquake is of greater magnitude than the others used for the analyses.

The results in x- and y-direction are generally similar, which is reasonable as the base shear highly depends on the mass of the building when the ground conditions are equal, and the mass is the same in both directions. For

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the India-Burma earthquake, which is of lowest magnitude, the base shear is lowest in the both the direction, while for the two larger earthquakes (Bhuj and Uttarkashi) the base shear is reasonably high. The larger earthquakes may have periods closer to the period of mode 2, which is in x-direction, further causing the largest impact in x-direction.
Figure 16 shows the base shears for all of the models at a scaled PGA of 0.24g. Base shear is an element of mass as well as stiffness of the structure. However, due to increase in seismic weight of structure due to bracing, there is an increase in the base shear. The braced structure's base shear increases as the structure's mass increases. Least base shear is observed in regular building with no bracing.

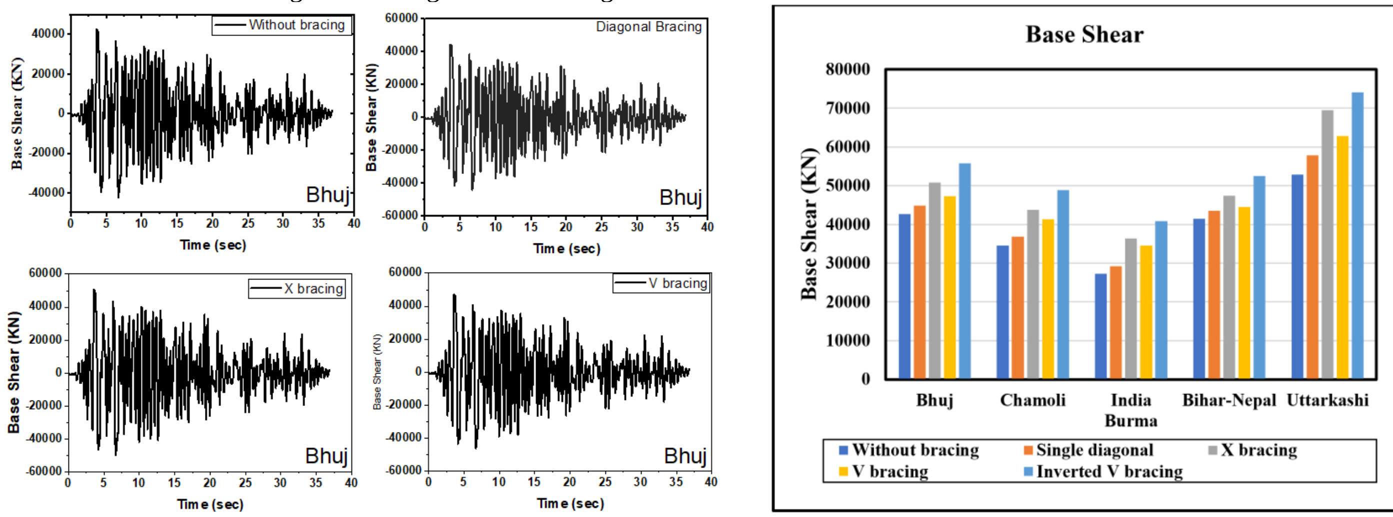


*Figure 16. Static Base shear time history for Bhuj earthquake for different frame model and comparison of base shear under various earthquakes*

**4.4.2 Drift Ratio**
Figure 17 illustrates the variation in drift for all time-history load cases as a function of time. For all five earthquakes that are scaled to 0.24g the results are similar in both directions. The results show that the maximum roof drift occurs during the Bhuj earthquake, which is reasonable considering previous results that also shows peak values during this earthquake. Generally, the maximum roof drift shows a larger value in y-direction, which may be due to irregularities in the geometry of the structure. It can be seen for the smaller earthquakes that there is a sudden change in the slope in the 7th to 9th story, which is in the region where the lower part of the building ends. This results in a sudden change in stiffness, and thus the result is reasonable. The peak values are varying between this region and at the upper stories, which is also reasonable as the displacements are higher at the top story.
Figure 17 shows the highest storey drift at each storey level derived using ESM and RSM for the various configurations. In all methodologies, without bracing frame models indicate maximum storey drift in top storeys. Models with single, X and V bracing performs well in terms of storey drifts. The value of storey drift is minimum for X braced frames.

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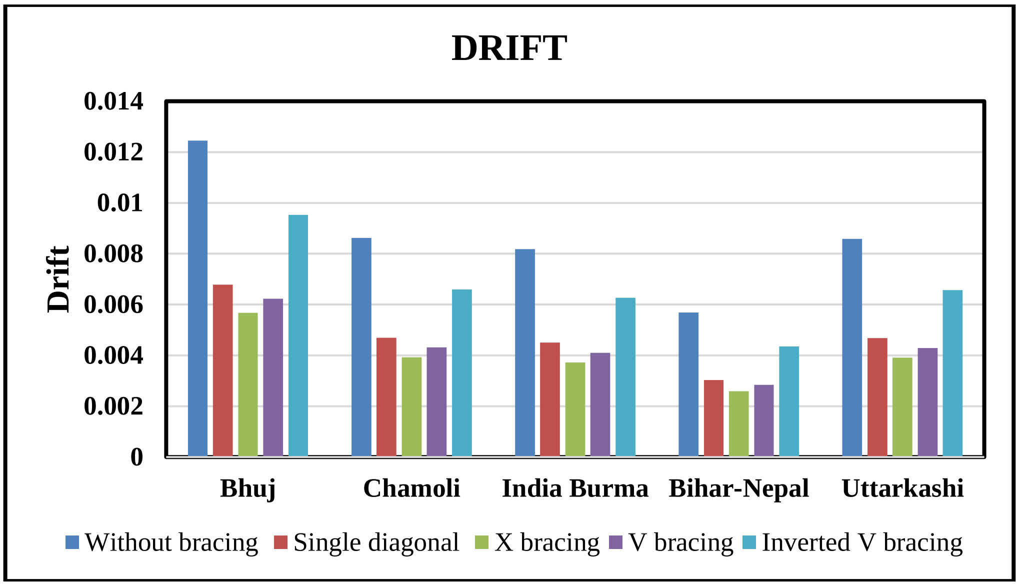


*Figure 17. Comparison of drift ratio under various earthquakes*

### 4.4.3 Storey Displacement

Figure 18 display the results of storey displacement at scaled PGA of 0.24g for the time-history load cases for Bhuj. As can be seen, the graphs display the variation in displacement with time of each time-history load case. The graphs indicate similar response in both directions for each of the earthquakes.

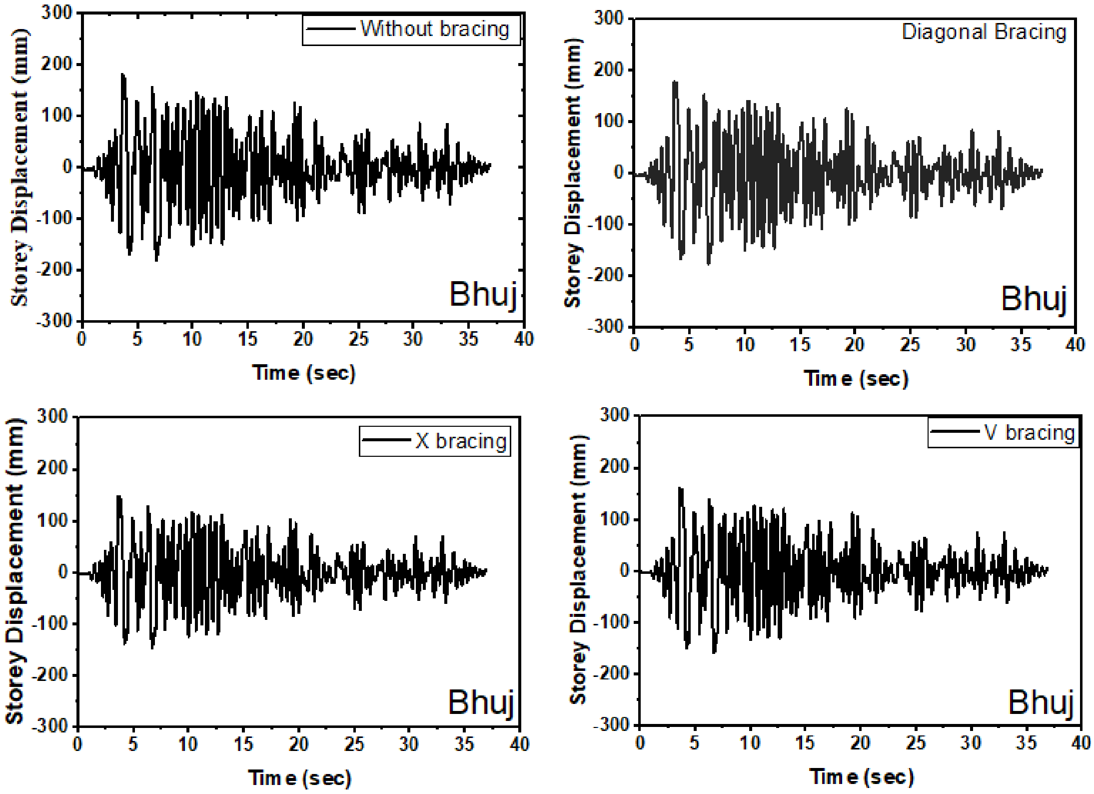


*Figure 18. Storey displacement time history for Bhuj earthquakes for different building types*

Table 6 lists the maximum storey displacements occurring during each earthquake. The results are similar in both directions of the two smaller earthquakes, while for the Uttarkashi and Bhuj earthquake the displacement

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considerably larger than Chamoli, India-Burma and India-Nepal. This may be because of the difference in stiffness of the building due to the irregularity, as the building has lower stiffness. This difference is seemingly magnified as the magnitude of the earthquake increase.

***Table 6. Comparison of storey displacements***

| Storey Displacement (mm) | | | | | |
|---|---|---|---|---|---|
| **Earthquake** | **Without bracing** | **Single diagonal** | **X bracing** | **V bracing** | **Inverted V bracing** |
| **Bhuj** | 184.674 | 175.4403 | 147.7392 | 141.2756 | 162.2066 |
| **Chamoli** | 144.618 | 137.3871 | 115.6944 | 110.6328 | 130.1562 |
| **India Burma** | 108.396 | 102.9762 | 86.7168 | 82.92294 | 97.5564 |
| **Bihar-Nepal** | 130.602 | 124.0719 | 104.4816 | 99.91053 | 117.5418 |
| **Uttarkashi** | 159.606 | 141 | 127.6848 | 122.0986 | 143.6454 |

Models with bracings showed excellent displacement control. Steel braced are proved more beneficial in displacement control than ordinary frame. When earthquake forces are applied across the direction, the top storey displacement is much higher than when they are applied along the direction. The storey shear for various earthquake with different frame are presented below for comparison.

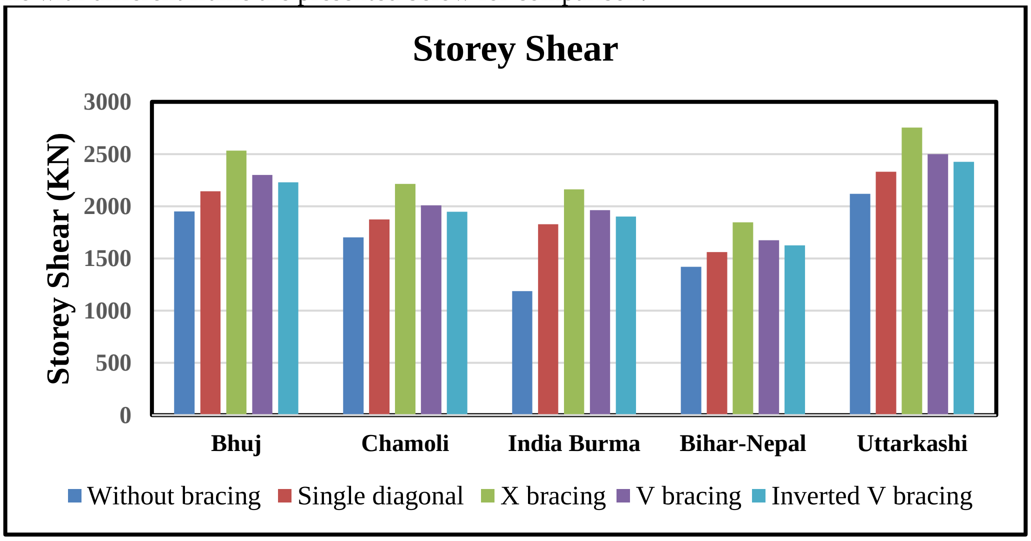


**Figure 19. Comparison of storey shear under various earthquakes**

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## 5. CONCLUSION

The seismic performance of a multi-story steel-framed building is designed following the provisions of the Indian code (IS 800-2007). The structure's ductility can be enhanced by incorporating steel bracings into the structural system. In this study, a typical G+9 multi-story steel frame building is designed with and without different types of bracings according to IS 800-2007. The bracings considered include single diagonal, X, V, and inverted V frames. Each frame's performance is analysed through nonlinear static analysis (pushover analysis) using SAP-2000 software. The deformed shapes, hinge results, lateral displacements, both with and without bracings, are compared. Pushover curves and performance points for different frames with and without bracing systems are also compared to determine the relative performances of the various frames.

The study investigates the enhanced seismic performance of the retrofitted building frame with stiffness bracings by considering the effects of earthquakes. The G+9-story steel building frame is retrofitted with cross bracings using different techniques. If the response exceeds the limit, the next story is retrofitted. The performances of retrofitted and un-retrofitted building frames are compared, with an optimal retrofitted bracing configuration identified. Few outcomes are,

- The performance point study of structures with and without bracing revealed that structures with bracing achieve performance points at less vulnerable damage states compared to structures without bracing.
- Comparing the results of structures with and without bracing, the base shear versus displacement curve shows that braced structures perform significantly better than those without bracing. It also indicates that the capacity curve becomes more linear for structures with bracing.
- The study found that hinges in structures without bracing were more vulnerable to damage, leading to severe collapses, suggesting that bracing effectively reduces damage extent.
- The study shows that braced steel frames significantly reduce lateral displacements, have a shorter modal period, and have higher frequencies compared to unbraced frames.
- The analysis results show that the base shear at the performance point is higher for building frames with bracing compared to those without bracing.
- In the retrofitted frame, inelastic deformation damage remains below the immediate occupancy level during earthquakes, with most damage confined to the bracings.
- Damages caused by Bhuj and Uttarkashi earthquakes—evidenced by base shear, story displacement, and the number of plastic hinges—can be significant and should be considered in the seismic design of structures; current codes do not explicitly include this provision.

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***Principal author: Shekhar Ghildyal** is a post graduate structural engineering student at Al-Falah University, Haryana, India. His current interests lie in non linear pushover analysis and time history analysis in steel multistorey buildings. Phone no: +91 9868048526 Email: shekharghildyal@gmail.com
**Mr. Mirza Aamir Baig** is assistant professor in department of civil engineering, Al-Falah University, Haryana, India.
Phone no: +91 8130958258 Email: aamirmirzagarri@gmail.com