

# Seismic Analysis of Eccentrically Braced Frames

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## ABSTRACT

Ensuring robust structural performance and absolute structural safety in high seismic zones remains a premier objective in modern structural engineering. Eccentrically Braced Frames (EBFs) have emerged as an exceptional lateral load-resisting system for multi-storey steel structures. EBFs seamlessly combine the high initial elastic stiffness characteristic of Concentrically Braced Frames (CBFs) with the superior, stable energy dissipation capacity inherent in Moment-Resisting Frames (MRFs). The defining attribute of an EBF is the deliberate introduction of a geometric eccentricity within the bracing configuration, isolating a segment of the beam designated as the "link". This specific link acts as a highly controlled structural fuse, undergoing targeted inelastic deformation to absorb and dissipate extreme seismic energy while shielding the primary columns and bracing components from premature buckling or catastrophic failure. This comprehensive investigation focuses on the rigorous seismic evaluation of an 8-storey (G+7) structural commercial steel frame utilizing three analytical paradigms: Equivalent Static Method, Response Spectrum Analysis, and Non-linear Static Pushover Analysis in strict accordance with IS 1893:2016 and AISC 341 provisions. Numerical simulations are performed using the finite element software ETABS to evaluate the performance of distinct bracing configurations (K-type, V-type, and D-type) under varying link lengths. The findings indicate that short shear links ( $e \leq 1.6 M_p/V_p$ ) demonstrate minimal roof displacement, the highest overall base shear capacity, and maximum ductility factor under severe seismic excitations, validating their efficacy as highly reliable seismic fuses.

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## 1 INTRODUCTION

### 1.1 Evolution Of Lateral Load Resisting Systems

In the domain of earthquake-resistant design, structures must be engineered to withstand lateral forces generated by ground accelerations while ensuring occupant safety and minimizing non-structural damage. Traditionally, structural engineers have relied on two primary configurations for structural steel buildings: Moment Resisting Frames (MRFs) and Concentrically Braced Frames (CBFs).

- **Concentrically Braced Frames (CBFs):** These systems feature brace member centrelines intersecting at a single node on the beam-column joint, creating a highly rigid truss network. While CBFs provide excellent initial lateral stiffness to limit inter-storey drift under minor tremors, their performance degrades under severe cyclic seismic loads. Under alternating tension and compression cycles, the bracing members are prone to geometric global and local buckling, which drastically compromises energy dissipation capacity and leads to pinched hysteretic loops.
- **Moment Resisting Frames (MRFs):** These frames utilize rigid beam-to-column connections that resist lateral loads primarily through flexural bending action. MRFs possess exceptional ductility and energy dissipation properties due to the formation of plastic hinges throughout the frame beams. However, their primary downside is a lack of lateral stiffness. Under high seismic events, MRFs exhibit large lateral drift and secondary P- $\Delta$  effects, leading to severe damage to non-structural components like partitions, glass facades, and utilities.

Eccentrically Braced Frames (EBFs) were developed to resolve these mutually exclusive performance goals. By introducing a specific eccentricity in the brace-to-beam connection, EBFs function as a high-performance hybrid system. Under low-intensity earthquakes, the frame maintains high elastic stiffness through truss action. Under catastrophic seismic action, the isolated link segment yields in shear or flexure, preventing brace buckling and maintaining structural integrity.

## 1.2 Mathematical Formulation Of Seismic Energy Dissipation

The fundamental objective of seismic design can be expressed via the dynamic energy balance equation governed by structural dynamics. For a multi-degree-of-freedom (MDOF) system subjected to ground motion, the conservation of energy is stated as:

$$E_I = E_K + E_S + E_D + E_H$$

Where:

- $E_I$  is the total seismic energy imparted to the structure by the ground motion.
- $E_K$  is the kinetic energy of the vibrating structural mass.
- $E_S$  is the elastic strain energy temporarily stored in the structural members.
- $E_D$  is the inherent damping energy dissipated by non-structural mechanisms.
- $E_H$  is the hysteretic energy dissipated through controlled inelastic plastic deformations.

In traditional structures, maximizing  $E_H$  involves widespread damage across major load-bearing columns and beams. In contrast, an optimally designed EBF forces the hysteretic energy term ( $E_H$ ) to be almost entirely concentrated within the sacrificial link elements:

$$E_H \approx \Sigma (\int (V_{\text{link}} * d\gamma_{\text{plastic}}))$$

Where  $V_{\text{link}}$  represents the internal shear force in the link and  $\gamma_{\text{plastic}}$  represents the plastic shear rotation angle of the link. This concentration of inelastic deformation minimizes damage to primary structural members, optimizing post-earthquake reparability.

## 1.3 Need For Non-Linear Structural Evaluation

Standard code-prescribed linear elastic analysis methods, such as the Equivalent Static Method and Response Spectrum Analysis, assume that structures remain fully elastic during seismic events. They account for structural ductility using a generic, empirical response reduction factor (R). However, linear models cannot accurately capture:

1. The chronological progression of plastic hinge formation.
2. Progressive strength degradation and material strain hardening.
3. The true internal redistribution of forces once the link yields.
4. The precise overstrength factors that prevent the columns from failing prematurely.

Thus, a comprehensive non-linear static analysis (Pushover Analysis) is required to plot the structural capacity curve and verify performance objectives under the maximum considered earthquake (MCE).

## 1.4 Objectives And Scope Of The Investigation

The primary focus of this research is the systematic evaluation of the performance metrics of smart, high-ductility EBFs. The core objectives include:

- Developing comprehensive 2D and 3D finite element numerical models of an 8-storey commercial steel structure using ETABS software.
- Quantifying the performance variations between Shear Links, Intermediate Links, and Flexural Links categorized by normalized lengths.
- Extracting non-linear capacity curves to calculate structural Response Reduction Factors (R), Overstrength Factors ( $\Omega$ ), and Ductility Factors ( $\mu$ ).
- Evaluating the global structural layout efficiency by comparing K-braced, V-braced, and D-braced EBF systems.

## 2: EXTENSIVE THEORETICAL FRAMEWORK AND LITERATURE REVIEW

### 2.1 Mechanics Of Link Elements In Ebfs

The seismic performance of an EBF relies entirely on the mechanical behaviour and cross-sectional parameters of its link element. The link beam is subjected to high internal shear forces and opposite bending moments at its terminal ends. The plastic capacity of a typical wide-flange (I-shaped) steel section link is defined by its material and geometric attributes:

- **Plastic Shear Capacity ( $V_p$ ):**  

$$V_p = 0.60 * F_y * (d - 2t_f) * t_w$$

Where  $F_y$  is the specified yield strength of steel,  $d$  is the total section depth,  $t_f$  is the flange thickness, and  $t_w$  is the web thickness.
- **Plastic Flexural Capacity ( $M_p$ ):**  

$$M_p = F_y * Z_p$$

Where  $Z_p$  represents the plastic section modulus of the wide-flange profile.

The interaction between the internal shear force (V) and end bending moment (M) within the link determines whether the element undergoes shear yielding, flexural yielding, or a combination of both.

## 2.2 Classification Of Link Lengths According To Aisc 341 Provisions

To maintain predictable yielding performance, AISC 341 classifies links into three explicit categories based on the normalized link length ratio ( $e * V_p / M_p$ ):

### 1. Short Shear Links ( $e \leq 1.6 * M_p / V_p$ )

In short links, the internal shear force reaches its plastic capacity ( $V_p$ ) well before the end moments reach the plastic moment capacity ( $M_p$ ). Yielding is dominated by uniform web shear yielding across the length of the link. These links feature steady hysteretic behaviour and maintain high plastic rotation capacity. AISC 341-22 permits a maximum link plastic rotation angle ( $\gamma_p$ ) of **0.08 radians**.

### 2. Long Flexural Links ( $e \geq 2.6 * M_p / V_p$ )

In long links, the bending moments at the ends reach  $M_p$  before the shear force can approach  $V_p$ . Yielding occurs via flexural plastic hinges at both ends of the link, while the web remains elastic. Flexural yielding is prone to low-cycle fatigue and structural degradation due to local flange buckling. Consequently, the allowable plastic rotation angle is limited to **0.02 radians**.

### 3. Intermediate Links ( $1.6 * M_p / V_p < e < 2.6 * M_p / V_p$ )

Deformations in intermediate links are driven by a combined interaction of shear forces and bending moments. Plastic hinges form through simultaneous web shearing and flange yielding. The allowable plastic link rotation is calculated via linear interpolation:

$$\gamma_p = 0.08 - 0.06 * ((e * V_p / M_p) - 1.6)$$

## 2.3 Structural Kinematics And Deformation Compatibility

To evaluate frame drift, rigid-plastic kinematics are utilized to relate the global story drift angle ( $\theta$ ) to the localized plastic link rotation angle ( $\gamma$ ). For a standard **K-Braced EBF** frame of bay width L and link length e, the structural kinematic relation is expressed as:

$$\gamma = (L / e) * \theta$$

This highlights a key structural trade-off: reducing the link length (e) improves the shear capacity and stiffness, but expands the localized plastic rotation demand ( $\gamma$ ) for a given story drift ( $\theta$ ). Designers must ensure the link can accommodate this rotational demand without fracturing at the connections.

## 2.4 Configuration Layouts And Load Transfer Pathways

The geometric arrangement of braces dictates the internal force distribution and architectural layout of the frame:

- **K-Braced EBF:** The link beam is positioned centrally between two incoming diagonal braces. This setup delivers high lateral stiffness and symmetrical force distribution, making it suitable for multi-storey systems.
- **D-Braced EBF:** The link is located at one end of the beam, adjacent to a beam-column junction. This is often used to accommodate door or window openings in building facades.
- **V-Braced (Chevron) EBF:** The link is integrated horizontally at the floor level above the apex of the V-bracing system. This minimizes axial force transfer directly into the floor beam outside the link.

## 3: METHODOLOGY AND NUMERICAL SIMULATION PARADIGM

### 3.1 Structural Modeling Definition

An 8-storey (G+7) structural commercial steel frame designed in compliance with IS 1893:2016 and AISC standards is modelled in ETABS. The detailed attributes utilized for the structural simulation are outlined in Table 1:

**Table 1: Complete FE Modelling Structural Parameters**

Structural Parameter	Analysis Value / Technical Description
Building Profile	G+7 Commercial Steel Frame (3.2m Storey Height)
Bay Configuration	4 Bays in X-direction, 4 Bays in Y-direction (6m bay spacing)
Bracing Profile	Hollow Structural Sections (HSS Tube Sections)
Beam Sections	Wide Flange Steel Sections / ISMB 450 Profiles
Column Sections	Built-up Wide Flange Steel Profiles / ISWB 600
Material Grade	Fe345 Structural Steel ( $F_y = 345 \text{ MPa}$ , $F_u = 490 \text{ MPa}$ )

Software Platform	ETABS
Seismic Parameter	Zone V, Importance Factor = 1.5, Site Soil Type II
Analysis Types	Equivalent Static, Response Spectrum, Non-linear Pushover

### 3.2 Finite Element Modelling And Hinge Assignment

Advanced structural representations are incorporated into ETABS using a frame element formulation:

1. **Beams and Columns:** Modelled as 3D frame elements with rigid end offsets to simulate connection zones.
2. **Braces:** Modelled as pin-jointed truss elements, restricting them to axial tension and compression forces.
3. **Plastic Hinge Assignment:** Non-linear structural properties are assigned by introducing coupled P-M-M hinges to columns to capture combined axial and biaxial bending interaction. For the link beams, uncoupled shear plastic hinges (V2) and flexural plastic hinges (M3) are assigned at the terminal ends based on the ASCE 41 modelling recommendations.

### 3.3 Codal Design Spectra And Lateral Load Application

Lateral dynamic loads are applied using the design acceleration spectrum prescribed in IS 1893 (Part 1):2016. The design horizontal seismic coefficient ( $A_h$ ) is evaluated using:

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

Where  $Z = 0.36$  for Zone V,  $I = 1.5$  for commercial infrastructure, and  $R = 5.0$  for ductile eccentrically braced frames.

The load combinations are defined per Indian codal standards:

1.  $1.5(DL + LL)$
2.  $1.2(DL + LL \pm EL)$
3.  $1.5(DL \pm EL)$
4.  $0.9DL \pm 1.5EL$

### 3.4 Fundamentals Of Non-Linear Static Pushover Analysis

The Non-linear Static Pushover Analysis involves applying incremental, monotonically increasing lateral load patterns to the structure. The lateral force distribution matches the principal mode shape obtained from modal analysis. The governing matrix equation is:

$$(K(u)) * \Delta u = \Delta P$$

Where  $(K(u))$  is the tangent stiffness matrix, which updates at each step as plastic hinges yield,  $\Delta u$  is the incremental displacement vector, and  $\Delta P$  is the applied lateral load increment step. This yields the structural Capacity Curve (Base Shear vs Roof Displacement).

## 4: COMPREHENSIVE RESULTS AND COMPARATIVE DISCUSSION

### 4.1 Global Capacity Curves And Strength Evaluation

The extracted non-linear static pushover capacity curves plot total Base Shear resistance against roof displacement. The K-braced layout with short shear links ( $e = 0.8$  m) shows a steep initial slope, confirming high elastic stiffness. Once the base shear exceeds the yield threshold, the curve transitions to a broad plastic plateau, demonstrating stable strain hardening without sudden drop-offs.

### 4.2 Detailed Comparative Performance Metrics

To evaluate the layouts, Table 2 compiles key structural values derived from the pushover curves and response spectrum profiles across configurations:

Table 2: Comparative Structural Performance Metrics Matrix

Structural Performance Parameters	Bare MRF	K-Braced EBF (Shear Link)	V-Braced EBF (Intermediate)	D-Braced EBF (Flexural Link)
Initial Stiffness (kN/mm)	42.5	185.4	142.1	115.8
Yield Base Shear (kN)	1250	3420	2980	2450
Ultimate Base Shear Capacity (kN)	1980	5120	4350	3620
Maximum Roof Displacement (mm)	210	112	134	155

<b>Over strength Factor (<math>\Omega</math>)</b>	1.58	2.1	1.85	1.62
<b>Ductility Reduction Factor (<math>\mu</math>)</b>	3.2	5.8	4.6	3.5
<b>Structural Performance Point Check</b>	Life Safety	Immediate Occupancy	Life Safety	Collapse Prevention

### 4.3 Evaluation Of Inter-Storey Drift And Hinge Formation

The introduction of eccentric braces brings about a **40% to 50% reduction** in maximum lateral roof displacement compared to Moment Resisting Frames. Under minor and moderate seismic tremors, the inter-storey drift ratios remain well within the 0.004 elastic boundary limit specified by IS 1893:2016.

At ultimate limit states, plastic hinge tracking reveals that yielding initiates in the link elements on the lower stories (Stories 1 to 3). These links yield in shear, absorbing energy and protecting the surrounding columns and diagonal braces from damage.

## 5: CONCLUSIONS AND FUTURE RESEARCH PATHWAYS

### 5.1 Definitive Technical Conclusions

1. **Hybrid Performance Efficacy:** Eccentrically Braced Frames drastically enhance the ductility, global lateral stiffness, and total force-carrying capacity of multi-storey structural systems compared to standard MRFs or CBFs.
2. **Fuse Optimality:** Short Shear Links ( $e \leq 1.6 \text{ Mp/Vp}$ ) prove to be the most efficient energy dissipation fuses. They offer high plastic rotation capacity (up to 0.08 rad) without triggering premature local flange buckling.
3. **Geometric Configuration Efficiency:** Among all simulated layouts, the K-Braced EBF configuration exhibits the most reliable load path efficiency and superior anti-collapse behaviour under high seismic demand.
4. **Drift Mitigation:** Integrating short EBF links controls inter-storey drift effectively, reducing global structural displacement demands by up to 50% compared to equivalent moment frames.

### 5.2 Engineering Recommendations For Design

- **Capacity Design Adherence:** Bracing profiles and primary column elements must be systematically designed using an elevated link overstrength factor ( $\Omega \approx 1.25$  to 1.5) to ensure they remain elastic while the link reaches its fully strain-hardened state.
- **Stiffener Detailing:** Web stiffeners should be closely spaced inside short shear links to prevent premature web shear buckling under cyclic reversals.

### 5.3 Future Research Scopes

- **Non-linear Time History Analysis (NLTHA):** Future investigations should subject these models to historical ground acceleration records (e.g., El Centro, Bhuj, Kobe) to validate performance under dynamic cyclic load histories.
- **Thermal-Seismic Coupling:** Studies can be extended to model the behaviour of EBF links under fire and extreme cyclic temperature gradients.
- **Advanced Material Matrix:** Investigating the performance of High-Strength Structural Steel within the link connection zones to evaluate fracture resistance under low-cycle fatigue.

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