



Behavior Of I-Section Steel Column With Eccentric Lateral Bracing Strengthened By Batten Plates

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Abstract: In the ideal case the connection of the strut member is concentric at the shear centre of the column's web, While in some construction cases, the connection between strut members of a vertical bracing system and the web of a steel column is made eccentrically, which introduces additional instability. The AISC 360-2022 specifications address this condition by providing equations to calculate the critical buckling stress associated with Constrained Axis Torsional Buckling (CATB). This phenomenon arises due to the eccentricity of lateral bracing from the shear center of the section and leads to a significant reduction in the axial capacity of columns, with losses reaching up to 40% in certain cases. This research focuses on improving the axial capacity of steel columns by studying double-sided batten plate configurations applied to the standard IPE profile. The primary objective is to examine the behavior of battened I-section columns with eccentric Lateral bracing, analyze their failure modes, and the enhancement in axial strength achieved using batten plates.

Index Terms - Constrained-Axis-Torsional Buckling (CATB), Finite element analysis, Eccentric bracing, Batten Plates, I-section columns, Lateral Bracing.

I. INTRODUCTION

Steel columns under pure compression can fail in three buckling modes: in-plane Buckling (major axis), out-of-plane Buckling (minor axis), and torsional Buckling (centroid axis), which mainly affect short columns. When vertical bracing with struts and gusset plates is placed at the column midpoint and has adequate stiffness and strength but with no torsional stiffness, it offers ideal lateral support. Under these cases, the out-of-plane buckling factor reaches a value of one, indicating maximum resistance to lateral instability.

In some construction conditions, strut connections in vertical bracing systems are eccentrically placed on the column web (as example in the upper chord of a truss where its connected eccentricity with a strut or in case columns where the strut is shifted so it don't clash with the blockwork), leading to Constrained Axis Torsional Buckling (CATB) a Combination of torsional and flexural buckling as shown in Fig.1. Recent studies on I-section columns with eccentric lateral bracing have shown that eccentrically braced columns experience a reduction in axial capacity when compared to ideally braced columns. As example they found that In column with HEA1000 section the eccentricity can reduce axial capacity by up to 40%.

Enhancing torsional rigidity is a key solution to Improve the axial strength in columns with eccentric bracing. This research examines the effect of double-sided batten plates on I-section profile to mitigate that reduction and return it to the original capacity and improve axial capacity; Previous research has primarily examined the role of batten plates in enhancing the moment capacity of beams. However, there is a noticeable gap in studies addressing their impact on columns with eccentric bracing, highlighting the need for further investigation in this area.

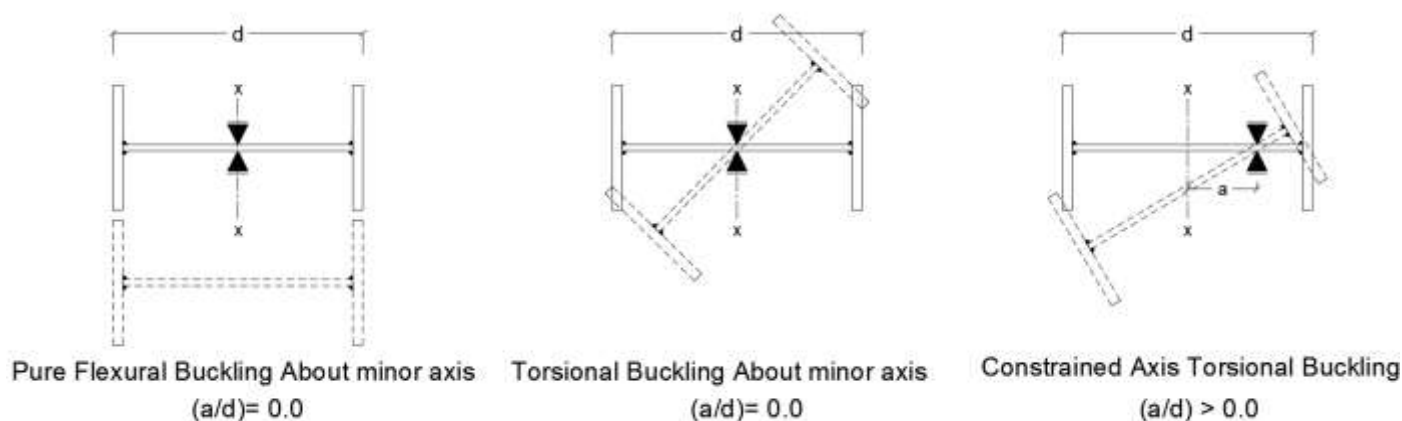


Figure 1: Schematic view for Flexural Buckling about minor axis (FB), Torsional Buckling (TB) and Constrained axis torsional Buckling (CATB).

II. LITERATURE REVIEW

[1] Timoshenko and Gere (1961) developed a quadratic equation to determine the critical torsional buckling load for members with lateral restraints assumed to be infinitely stiff. This equation accounts for Constrained Axis Torsional Buckling (CATB), a combined torsional and flexural buckling mode caused by bracing offset from the shear center.

[2] AISC 360-2022 provided equations to calculate elastic buckling stress for Constrained Axis Torsional Buckling (CATB) of the eccentrically braced columns, it adopted the equation that derived from the Timoshenko and Gere to evaluate critical buckling in doubly symmetric I-shaped members with eccentric bracing along both major and minor axes.

[3] Helwig and Yura (1999) found that the position of lateral bracing on a column's cross-section significantly affects its torsional buckling behavior. Using finite element analysis, they determined the torsional stiffness needed for bracing to match the column's minor-axis flexural strength. When bracing prevents both lateral and torsional movement, out-of-plane buckling governs stability. However, if bracing only limits lateral translation (especially when eccentrically placed) torsional buckling becomes the dominant design concern. They derived equations to calculate the torsional buckling capacity of columns rotating about their centroid, highlighting how eccentric lateral bracing affects stability. Their analysis shows that when bracing is offset from the centroid, torsional buckling capacity decreases, with greater offsets (along major or minor axes) leading to more significant reductions.

Figure 2 illustrates that as the lateral bracing moves farther from the column's centroid (increasing distance a), its torsional buckling resistance declines. This emphasizes the critical role of bracing placement in maintaining column stability and the importance of precise structural design.

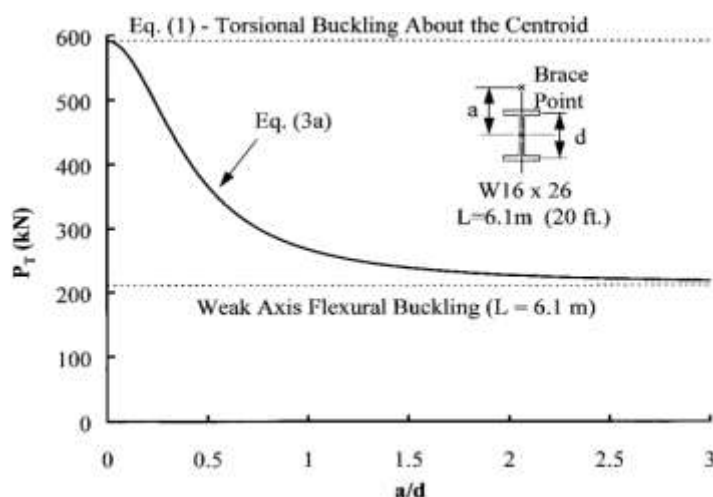


Figure 2: Comparison between torsional buckling about the centroid and about an axis offset from the centroid.

[4] Horne and Ajmani (1969) developed an equation to assess torsional buckling capacity in columns with rigid eccentric lateral bracing and restrained twisting at the ends. Their findings show that eccentric bracing requires higher torsional stiffness to ensure stability, and its effectiveness diminishes as the bracing moves farther from the centroid—underscoring the critical role of precise bracing placement in structural design.

[5] Abokifa et al. (2018) used non-linear FEA with ANSYS 14.5 to study how eccentric vertical bracing affects steel column performance. They analyzed the impact of eccentricity ratio (e/d), cross-section type, bracing location, and steel grade, varying e/d from 0 to 0.5 to assess changes in axial behavior under increasing eccentricity. as illustrated in Fig.3.

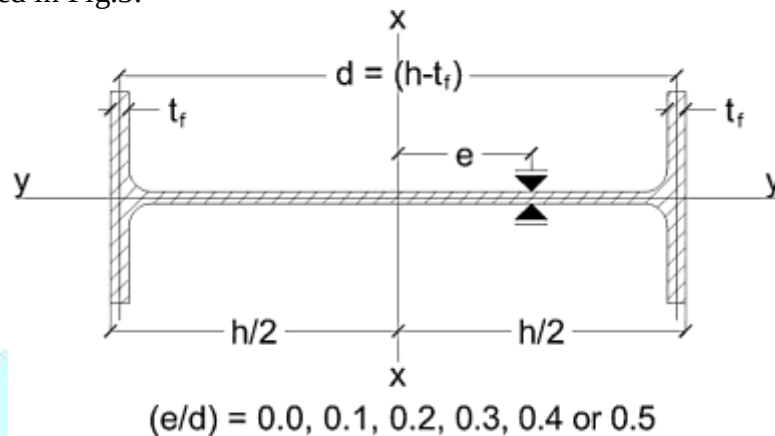


Figure 3: Eccentricity value (e/d) (Abokifa et al., 2018)

The parameter (a/L) indicates the bracing joint's position along a column, affecting its buckling behavior. Values like 0.5, 0.4, and 0.3 represent different locations, with 0.5 placing the brace at mid-height for optimal stability as illustrated in Fig.4.

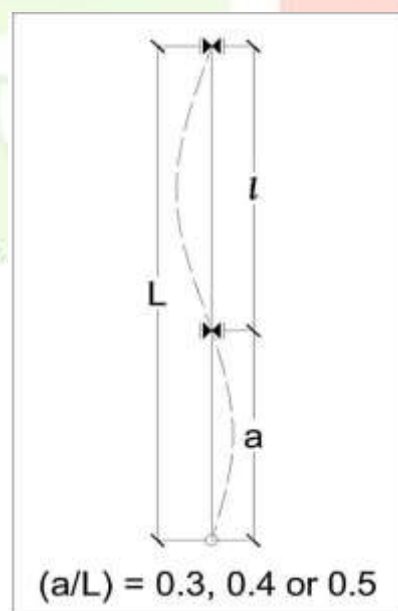


Figure 4: Lateral bracing joint location along column length (a/L) values (Abokifa et al., 2018)

Figure 5 illustrates the variation of the axial capacity ratio $PFEM/P_y$ with the eccentricity ratio " e/d " for IPE 600, IPE 400, and IPE 300 columns at an out-of-plane slenderness ratio of $KL/r_y = 80$. The results indicate that eccentric vertical bracing has a pronounced effect on columns with larger cross-sectional depths. Since the bracing eccentricity " e " is defined as a proportion of the column depth " d ", increasing the column depth inherently increases the bracing eccentricity. This increase in eccentricity leads to a greater reduction in axial load capacity. Consequently, columns with larger depths generally experience a more significant drop in axial capacity when eccentric vertical bracing is used.

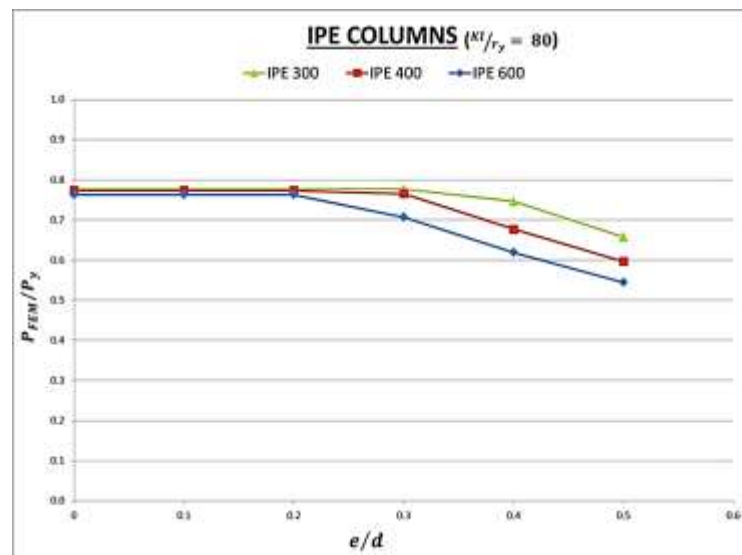
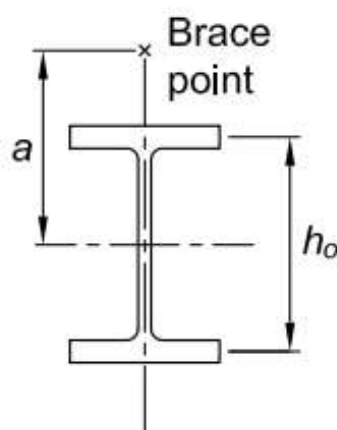


Figure 5: Ultimate load/Yield load “ P_{FEM}/P_y ” versus eccentricity ratio “ e/d ” for set of IPE columns with $(KL/r_y=80)$ and supported at mid-length (Abokifa et al., 2018)

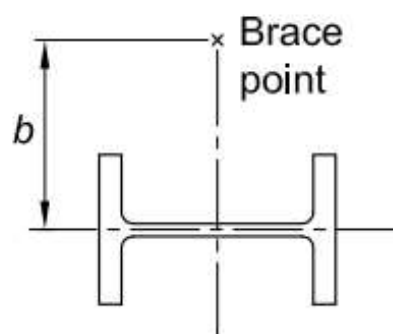
The results show that increasing eccentricity in vertical bracing reduces the axial capacity of steel columns, especially in short columns and deeper sections. The significant reductions were observed at minor axis slenderness ratio ($\lambda=KL/r_y$) equal to 80. Higher steel grade columns show greater reductions in axial capacity under eccentric bracing, even at low eccentricity ratios. To address these challenges, the study proposed simplified design equations for torsional buckling strength tailored to I-section columns.

The recent AISC 360-2022 introduced design equations for doubly symmetric I-steel columns considering constrained axial torsional buckling (CATB) with eccentric lateral bracing offset from the shear center. Prior to this, Previous versions of the code did not address such eccentric bracing conditions. The nominal compressive strength (P_n) is determined by the lowest value among flexural, torsional, flexural-torsional, and CATB limit states.

This offset can occur along either the major or minor axis, potentially affecting the column's stability and increasing susceptibility to torsional buckling as shown in Fig.6.



(a) Bracing offset along minor axis



(b) Bracing offset along major axis

Figure 6: Bracing details resulting in an offset relative to the shear center.(AISC 360 ,2022)

When lateral bracing is offset from the section's shear center, it can trigger constrained-axis torsional buckling (CATB), a failure mode that compromises the member's stability by introducing torsional stresses along the constrained axis.

III. FINITE ELEMENT MODEL

The finite element models are developed using the commercial software (ANSYS, 2019) [7] which is branded as an excellent finite element software with easy use and good accuracy. This study employs Shell 181 which is a four-node finite element used for analyzing thin to moderately thick shell structures. It supports both membrane and bending behavior, with six degrees of freedom per node (three translations and three rotations). Linear analysis as well as large rotations and large strain non-linear applications, accounts for shell thickness variations, and uses five integration points per layer for accurate modeling. The geometry, node locations, and coordinate system for this shell element are illustrated in Fig.7.

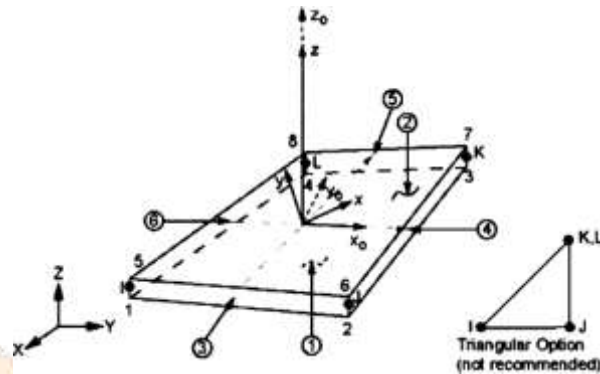


Figure 7: Element Geometry (SHELL181) (ANSYS, 2019)

Meshing divides the 3D model into shell elements. A Sensitivity study tested mesh sizes from $100 \times 100 \text{ mm}^2$ to $10 \times 10 \text{ mm}^2$, finding that $25 \times 25 \text{ mm}^2$ offers the best balance between accuracy and computational efficiency.

Steel grade S275 is used, with ultimate strength $F_u = 450 \text{ N/mm}^2$ and yield strength $F_y = 275 \text{ N/mm}^2$. A bilinear stress-strain model is applied, using linear elastic properties ($E = 2 \times 10^5 \text{ N/mm}^2$, $G = 79 \times 10^3 \text{ N/mm}^2$, $\nu = 0.3$, $E_t = 0.05E$) where (E_t is the steel tangential modulus of elasticity) and strain hardening for nonlinear finite element analysis as shown in Fig.8.

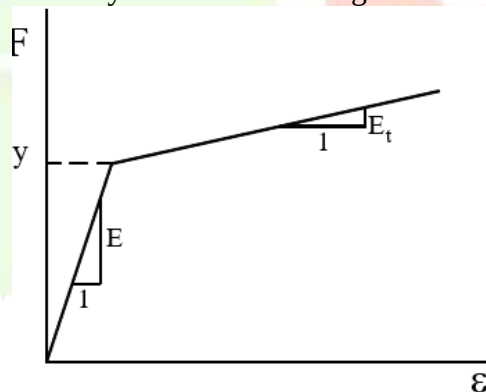


Figure 8: Constitutive material model

The column's upper end is restrained in horizontal translations (U_x , U_y) and rotation (R_z), but allows axial movement (U_z). The lower end is fully fixed in all translations and R_z rotation. A coupling constraint links the column surface to the support node for stability, and 25 mm thick endplates are added at both ends to prevent local buckling as shown in Fig.9.

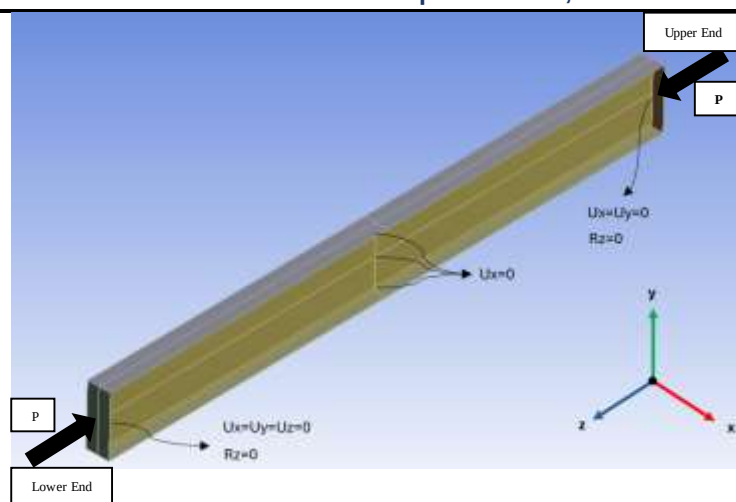


Figure 9: Boundary conditions in FE models at three locations of Lateral bracing

An axial compressive load is applied at the column's Top end plate. The load was uniformly distributed over the area of the plate to avoid stress concentration. The load increases gradually until failure evaluates the column's axial capacity and response.

In the finite element analysis, structural instability is defined as the condition in which the model is subjected to deformation-controlled loading until a pronounced and continuous increase in deformation occurs with little or no corresponding increase in resistance, indicating a loss of stiffness and the onset of instability. At this stage, the structure is considered to have reached its failure condition, and the reaction force obtained from the analysis is taken as the axial load-carrying capacity of the member. This definition enables the capture of instability and post-buckling behavior without premature numerical termination and provides a consistent basis for evaluating axial capacity.

The numerical model employs two types of analysis. An eigenvalue buckling analysis is first performed using linear material properties and linear geometry to identify the critical buckling mode shape, which is subsequently used to initiate the nonlinear analysis. The nonlinear analysis incorporates a bilinear stress-strain relationship and geometric nonlinearity, with initial geometric imperfections introduced in accordance with the provisions of AISC 360 (2022) [2].

The initial imperfections scaled to 1/1000 of the column unsupported length taken as the total column length for Torsional Buckling (TB) and Constrained Torsional Buckling (CATB) and half the column length for Flexural Buckling (FB) as shown in Fig.10. This geometry is then used in a non-linear analysis to predict axial capacity.

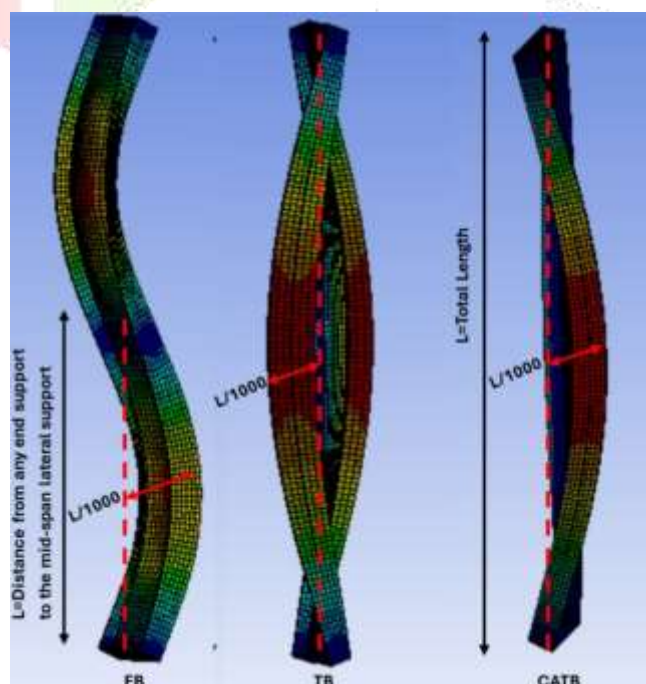


Figure 10: Initial geometric imperfections shapes

IV. VERIFICATION OF THE FINITE ELEMENT MODEL

The results of the finite element model were verified using the AISC 360 (2022) [3] design equations for axial compression in Chapter E, mentioned in section (2.5) to confirm accuracy. It involved linear buckling analysis to define initial imperfections, followed by non-linear analysis.

Eight pin-ended columns with slenderness ratios of 60, 70, 80, and 140 were analyzed using FEM for IPE600 profile. Lateral support was applied at mid-span, with horizontal translation (U_x) restrained. Two bracing positions were studied: concentric ($e/d = 0.0$), eccentric ($e/d = 0.45$) as illustrated in Fig.11. The selected eccentricity value of “0.45” is based on practical considerations. At this value, the edge of the bracing strut aligns directly with the flange, meaning that both elements share the same edge line.

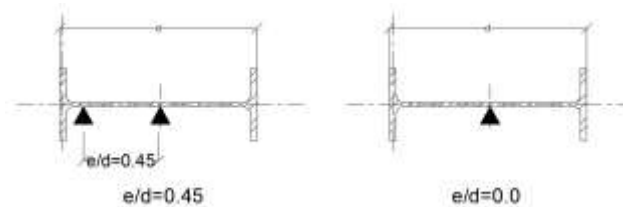


Figure 11: Application points of vertical lateral bracing.

For pure axial loading, the FEM was used to determine the maximum compressive strength (P_{FEM}) and compared with the nominal strength (P_n) from Chapter E of the AISC 360 (2022) [3], considering various buckling modes including major/minor-axis, torsional, flexural-torsional, and constrained axis torsional buckling.

Table 1 compares finite element model results with AISC 360 (2022) design provisions, demonstrating how closely the simulations align with code predictions and confirming the accuracy and reliability of the FE method for assessing structural performance.

Table 1: Comparison of FEM and nominal strength equations for the column in the AISC 360-22 specification.

Section	Kl/r_y	e/d	P_{FEM} (KN)	P_n (KN)	INTERACTI ON RATIO
IPE600	60	0.00	3600.00	2912.46	1.24
IPE600	70	0.00	3100.00	2528.74	1.23
IPE600	80	0.00	2350.00	2265.14	1.04
IPE600	140	0.00	1375.00	1418.76	0.97
IPE600	60	0.45	2750.00	2515.77	1.09
IPE600	70	0.45	2090.00	1948.81	1.07
IPE600	80	0.45	1675.00	1545.46	1.08
IPE600	140	0.45	1275.00	1143.79	1.11

Table 1 shows that for columns with zero eccentricity and low slenderness ratios, the FEM predicts slightly higher failure loads than AISC 360 (2022) due to thick end plates at both ends, which restrain warping and enhance load capacity.

V. PARAMETRIC STUDY

A parametric study is performed to determine the effect of batten plates on the axial compressive capacity of I-section steel columns with eccentric vertical bracing with respect to their webs. Different parameters are introduced and investigated, with a total of 36 specimens studied. In this work, the main parameters investigated are the batten plate dimensions, which are discussed in the following sections.

In this study, batten plate thicknesses were selected as $0.5t_w$, t_w and $2t_w$, (where t_w represents the web thickness of the I-section) and batten plate depths were selected as $0.5d$, d and $1.5d$, (where d represents the depth of the I-section) as shown in the Fig.12. The location of the batten plate is at the top and the bottom of the column at both sides. The investigation was conducted on IPE600 profile. All specimens were modeled with pin-ended boundary conditions, and lateral bracing was positioned at the mid-span of the column height. The range of out-of-plane slenderness ratios (KL/r_y) was considered, specifically 60, 80, 100, and 140. The material used throughout the study was structural steel grade S275, characterized by a yield strength of 275 N/mm².

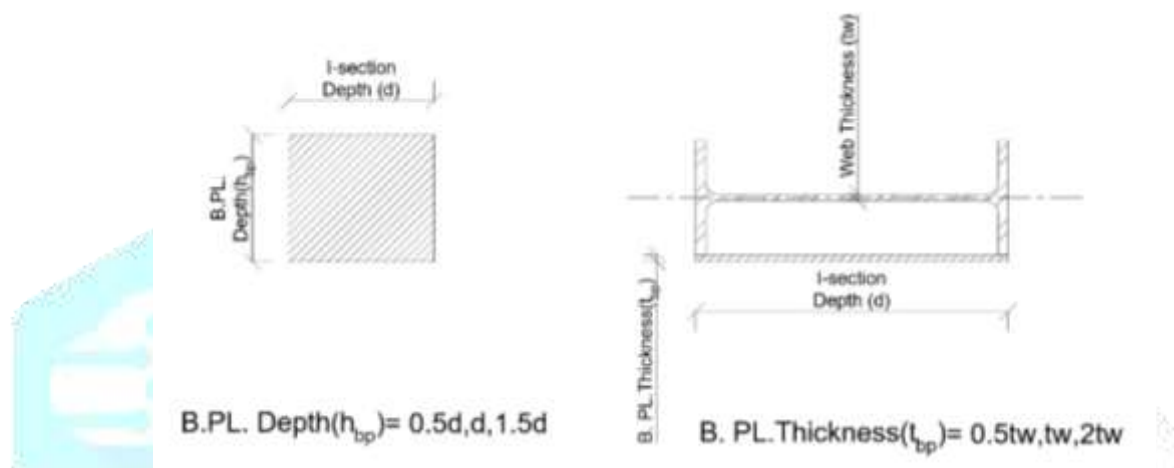


Figure 12: Batten Plate Dimension

VI. RESULTS AND DISCUSSION

Due to the eccentricity in lateral bracing, the axial capacity of columns decreases. Therefore, the main objective of the study becomes restoring the column's axial strength to reach the original capacity with concentric lateral bracing.

The outcomes of the parametric study are presented through graphical relationship illustrates the relation between the ratio

" P_{SP-E-L} / P_{SP-C} " and the batten plate thickness " t_{bp} " at different batten plate depths " h_{bp} ". The definitions and descriptions of these parameters are provided in the following sections.

- P_{SP-C} : maximum nominal strength of the columns with concentric lateral bracing obtained from non-linear FEA.
- P_{SP-E} : maximum nominal strength of the columns with eccentric lateral bracing obtained from non-linear FEA.
- P_{SP-E-L} : maximum nominal strength of the columns with eccentric lateral bracing strengthened with batten plate at the top and the bottom of the column at both sides obtained from non-linear finite element analysis.
as shown in **Fig.13**.
- KL/r_y : is the out-of-plane slenderness parameter of the column.
K: is the effective length factor and its value is taken equal to unity,
L: is half the column length,
 r_y : is the column cross-section minor radius of gyration.

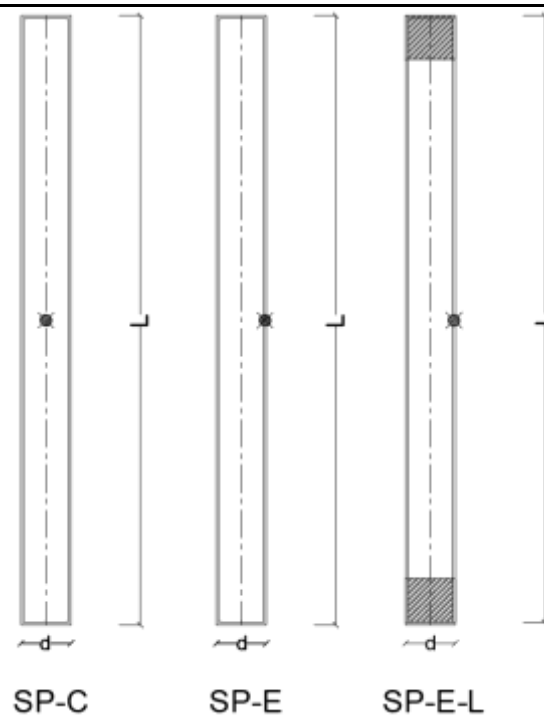
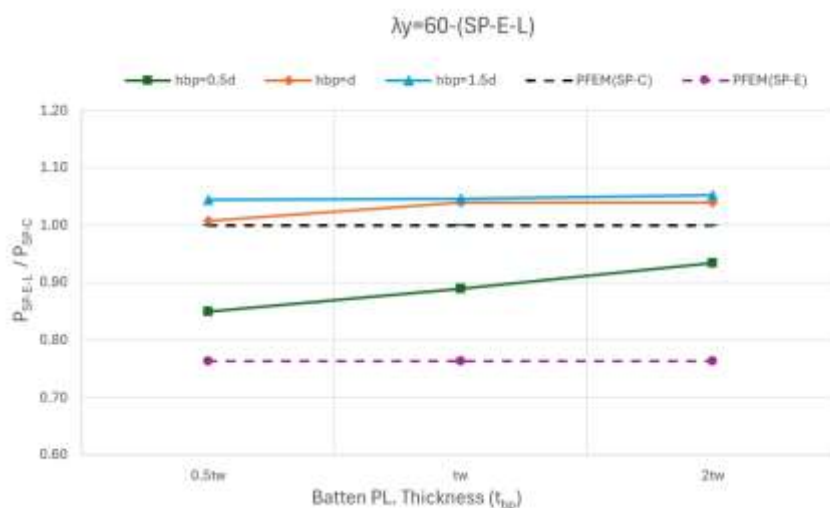


Figure 12: Different Columns Configurations

Figure .14 illustrates the relationship between “ $P_{FEM} / P_{FEM(SP-C)}$ ” and the batten plate thickness “ t_{bp} ” for slenderness ratios 60,80 and 100 for an IPE600 column supported at its mid-length. three graphs are presented, each corresponding to a different batten plate depth: “0.5d”, “d”, and “1.5d” at batten plate spacing “L”. In all cases, the Lateral bracing has eccentricity equal to “ $e/d=0.45$ ”. The steel grade used throughout this study is S275.



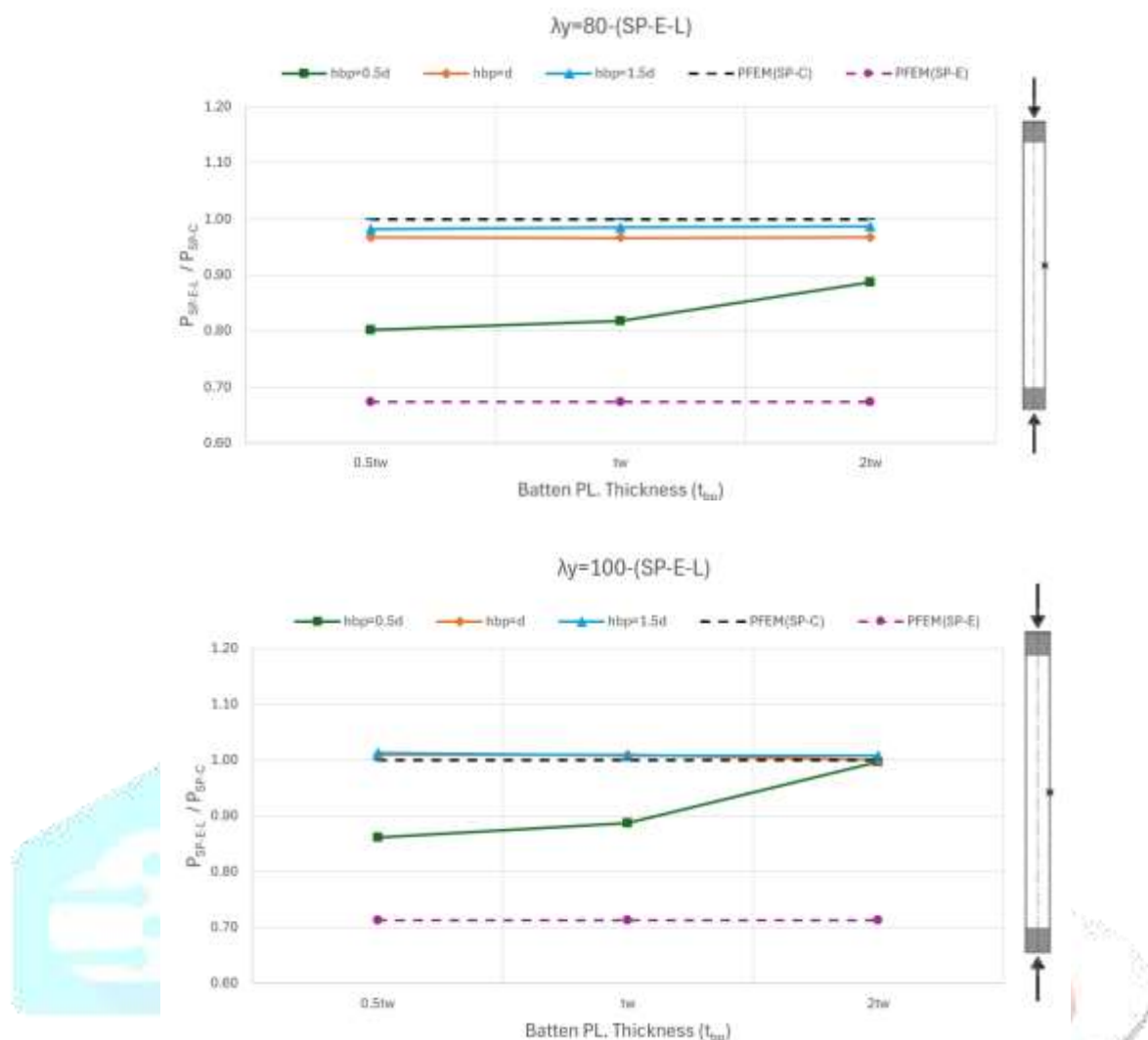


Figure 14: The relation between the ratio “ P_{SP-E-L} / P_{SP-C} ” and the batten plate thickness “ t_{bp} ” at different batten plate depths “ h_{bp} ” for slenderness ratios 60, 80 and 100

- For slenderness ratios of 60, 80, and 100:

The effect of batten plate thickness varies depending on the depth. When the batten plate depth is maintained at d or $1.5d$, increasing the thickness from $0.5t_w$ to $2t_w$ does not affect the ultimate axial capacity. In these cases, the columns reach their original capacity at the minimum tested thickness of $0.5t_w$, indicating that additional thickness offers no structural advantage. However, when the batten plate depth is reduced to $0.5d$, the axial capacity becomes sensitive to thickness. Here, the capacity increases progressively with thickness and only reaches the original value (column with concentric bracing) at $2t_w$. This suggests that reduced batten depth compromises lateral restraint, which must be compensated by increased thickness to restore performance.

Figure .15 illustrates the relationship between “ $P_{FEM} / P_{FEM(SP-C)}$ ” and the batten plate thickness “ t_{bp} ” for slenderness ratio 140 for an IPE600 column supported at its mid-length. three graphs are presented, each corresponding to a different batten plate depth: “ $0.5d$ ”, “ d ”, and “ $1.5d$ ” at batten plate spacing “ L ”. In all cases, the Lateral bracing has eccentricity equal to “ $e/d=0.45$ ”. The steel grade used throughout this study is S275.

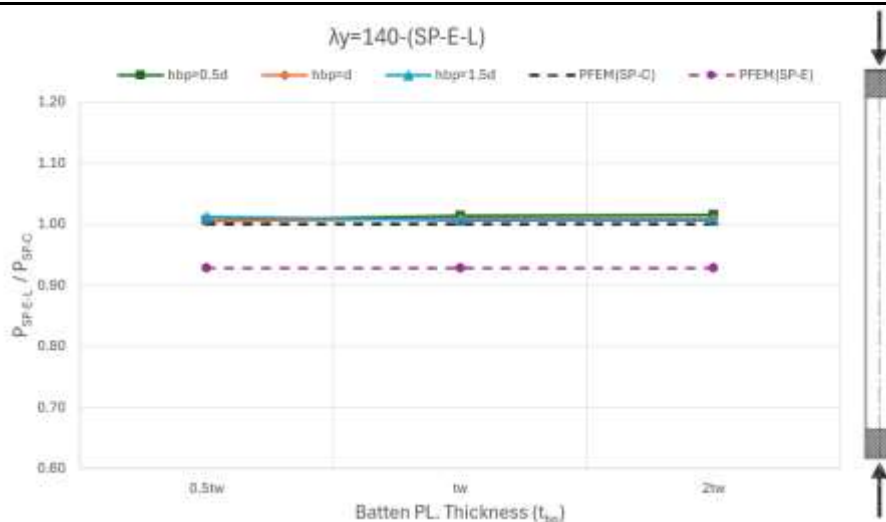
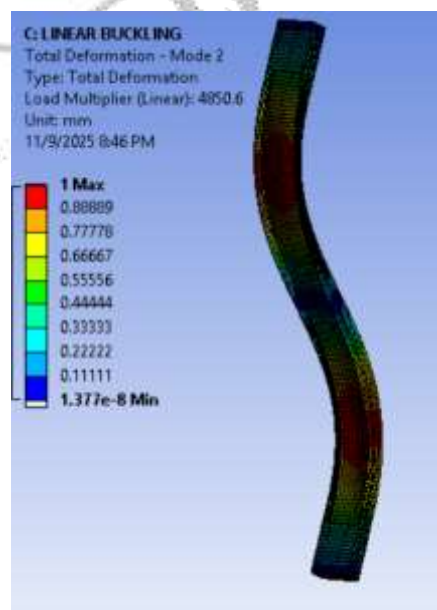
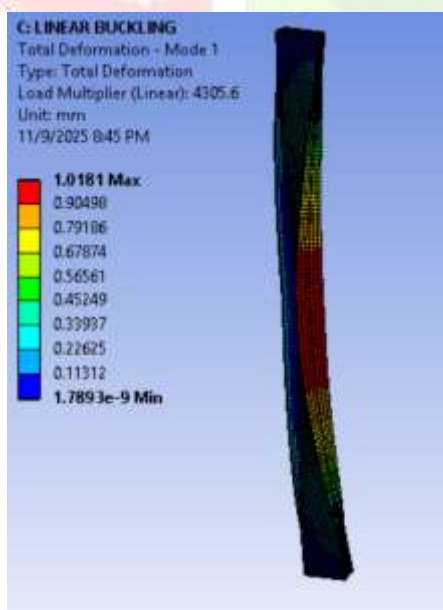


Figure 15: The relation between the ratio “ P_{SP-E-L} / P_{SP-C} ” and the batten plate thickness “ t_{bp} ” at different batten plate depths “ h_{bp} ” for slenderness ratio 140

- At a slenderness ratio of 140.

The behavior is markedly different. Regardless of batten plate, whether 0.5d, d, or 1.5d, the ultimate axial capacity remains unchanged across the thickness range from 0.5tw to 2tw. The columns consistently achieve their original capacity at the minimum thickness of 0.5tw. This implies that at high slenderness levels, the global buckling behavior dominates, and local variations in batten plate geometry have minimal impact on axial strength.

This behavior can be attributed to the eccentricity of the lateral bracing, which promotes constrained-axis torsional buckling (CATB) as the critical failure mode. The eccentric load path induces torsional effects that reduce the column's axial capacity. By incorporating batten plates, the torsional rigidity of the column is enhanced, effectively restraining the torsional deformation. In certain configurations, this added stiffness is sufficient to shift the dominant buckling mode in linear analysis from CATB to out-of-plane flexural buckling, which is generally less critical. This transition in buckling behavior is illustrated in Fig.16., which illustrates the constrained-axis torsional buckling and out-of-plane buckling modes, respectively.



(a) Constrained-axis torsional buckling (CATB) mode (SP-E-L) (b) Flexural out of plane buckling mode (FB) (SP-C)

Figure 15: Transition in Buckling mode from CATB to FB in linear analysis.

The use of batten plates has a noticeable effect on columns with eccentric vertical bracing, especially in short columns those with low out-of-plane slenderness ratios (KL/r_y). In these cases, batten plates help increase the column's strength by resisting twisting caused by the eccentric bracing. This is because constrained-axis torsional buckling (CATB) is the main failure mode in short columns, and batten plates improve torsional stiffness.

However, in long columns those with high out-of-plane slenderness ratios (KL/r_y), this effect is much less noticeable. The dominant failure buckling mode is flexural buckling about the minor axis, which is not significantly influenced by the presence of batten plates. As a result, adding batten plates does not meaningfully enhance the axial capacity of long columns with eccentric lateral bracing.

VII. CONCLUSIONS:

1. The ultimate axial capacity of column with eccentric lateral bracing is enhanced by using batten plate.
2. The Enhancement effect of using batten plate in short column is significant than long columns.
3. As the depth of batten plates increases the enhancement effect of the ultimate axial capacity of the column increases.
4. The Effective depth of the batten plate is “d” and with thickness “ $0.5t_w$ ”.
5. Batten plate's thickness is effective while using batten plate with low depth and has no significant effect while using batten plate with large depth.
6. The location of batten plate at the top and bottom of the column gives good enhancement to the column compressive capacity.

REFERENCES

- [1] Timoshenko, S. and Gere, J., (1961), “Theory of Elastic Stability”, New York: McGraw-Hill.
- [2] AISC Specifications for structural steel buildings, Load and Resistance Factor Design (LRFD) (2022)
- [3] Todd A. Helwig and Joseph A. Yura, “Torsional Bracing of Columns”, Journal of Structural Engineering, Vol.125, No. 5, May 1999.
- [4] Horne, M. R., Ajmani, J. L. (1968), “Stability of columns supported laterally by side rails”, International Journal of Mechanical Sciences, Vol. 11, 1969, pp. 159-174.
- [5] Mohamed AboKifa, Sherif M. Ibrahim, Abdelrahim K. Dessouki (2018) “BUCKLING OF STEEL COLUMNS WITH ECCENTRIC BRACING”, International Journal of Scientific and Engineering Research.
- [6] ANSYS standard user's manual, Version 19.2.