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Eccomas Proceedia COMPDYN (2025) 4211-4219

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COMPDYN 2025
10th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15-18 June 2025

CONCENTRICALLY BRACED FRAMES CONNECTIONS ACCORDING TO THE SECOND GENERATION OF EUROCODE 8

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Abstract

EN 1998-1:2005 provides only basic design guidelines for bracing connections in concentrically braced frames (CBFs) to ensure ductile behaviour. Currently, the only requirement is a minimum tensile overstrength for brace connections. As a result, neither ductility nor resistance is guaranteed in the pre-buckling and post-buckling phases of the brace, since gusset connections are not designed to accommodate the rotational demands caused by damaged braces. The latest version of Eurocode 8 (prEN 1998-1-2:2022) introduced new detailed provisions for bracing connections in CBFs to enhance the formation of either linear or elliptical yield lines. To assess the efficiency and accuracy of these new prescriptions and to analyse their impact on design, different analytical methods to evaluate forces at gusset plate interfaces have been examined through a parametric study based on finite element (FE) simulations. Initial findings suggest that the method used to distribute forces in the gusset plate, significantly influences the final shape of the connection. However, numerical results indicate that further research is necessary to better understand the realistic force distribution in gusset plate connections.

Keywords: EN 1998, Steel concentrically braced frames, Bracing connection.

ISSN:2623-3347 © 2025 The Authors. Published by Eccomas Proceedia.
Peer-review under responsibility of the organizing committee of COMPDYN 2025.
doi: 10.7712/120125.12729.25244

1 INTRODUCTION

Steel structures are widely used in seismic areas [1-12]. CBFs are widely recognized as an efficient seismic resistance system, provided that brace connections exhibit sufficient ductility to allow displacement and deformation demands. Among the various types of connections, gusset plates (GP) are generally adopted. Their seismic design follows capacity-based principles to ensure that yielding and buckling occur in the braces rather than in the connection itself. In the latest version of Eurocode 8 (prEN 1998–1-2) [13] a new dedicated annex provides detailed design specifications for bracing connections, an aspect not covered in the previous EN 1998–1:2005 [14]. These guidelines are largely derived from North American research, both experimental and theoretical [15-18]. Astaneh-Asl et al. [15] suggested detailing gusset plates to enhance the development of a linear yielding zone. According to this approach, the width of the plastic zone should be between two and four times the plate thickness, though Cochran [16] recommended a minimum of three times the thickness to account for construction tolerances. However, subsequent studies have pointed out that this method can lead to impractical or inefficient designs. An alternative approach, based on an elliptical yield field, has been proposed as a more compact and effective solution [17,18]. Experimental findings indicate that the optimal width of an elliptical yield field should be approximately eight times the thickness of the gusset plate, and a design methodology for this configuration has been developed. Beyond achieving sufficient plastic rotation capacity, the design of gusset plate connections must consider three key aspects: (i) the brace-to-gusset interface, (ii) the gusset plate itself, and (iii) the connection between the gusset plate and the beam/column. While EN 1998–1-2 defines design actions and geometric constraints, it does not prescribe a specific method for determining how forces are distributed from the brace to the beam and column via the gusset plate. Various force distribution models exist [19-24], but the choice of method can significantly influence the final design, leading to variations in structural reliability.

Given the above, this study evaluates different gusset plate connection designs to assess the adequacy of Eurocode provisions and the impact of varying force distribution assumptions. A case study of a six-story office building, designed according to the second-generation Eurocodes 3 and 8, has been analyzed using a detailed Finite Element (FE). This paper is structured as follows: (i) an overview of EN 1998–1-2 requirements for gusset plate connections is summarized, (ii) an explanation of the design methodology is given, (iii) a description of the numerical model used for analysis is reported, and (iv) conclusions are drawn based on key findings from the numerical simulation.

2 GUSSET PLATE DESIGN

2.1 Eurocode 8 part 1-2 prescriptions

The latest version of Eurocode 8 classifies seismic-resistant structures into three ductility classes (DC) based on their ability to dissipate energy and undergo plastic deformations.

DC1 structures can be classified as low-dissipative structures that resist seismic loads almost elastically. Consequently, design seismic actions are obtained through an elastic analysis.

DC2 structures are characterized by moderate ductility. Simplified design principles are introduced to allow local deformation and ensure moderate energy dissipation. Nevertheless, such prescriptions are not aimed at guaranteeing a global plastic mechanism. Gusset plates designed to accommodate brace buckling should be verified against forces separately evaluated from Formulas (1), (2) and (3) in the restrained plane:

$$N_{T,j,Ed} = \omega_{rm}\,\omega_{sh}\,N_{pl,Rd} \tag{1}$$

$$N_{C,j,Ed} = \omega_{rm}\,N_{b,Rd} \tag{2}$$

$$M_{j,Ed} = \omega_{rm}\,\omega_{sh}\,M_{pl,Rd} \tag{3}$$

where $N_{pl,Rd}$ is the design plastic resistance of the brace in tension, $N_{b,Rd}$ is the design buckling resistance of the brace in compression, while $M_{pl,Rd}$ is the design plastic moment of the cross section of the brace evaluated in the plane of the frame. In addition, the plastic rotation capacity of the connection should not be lower than the required rotation at the design storey drift. Gusset plates designed to promote the formation of a linear or elliptical field are considered to have sufficient plastic rotation capacity. The width of the linear hinge zone should range between two and four times the gusset plate thickness (t_{gp}), whereas the width of the elliptical clearance should be set to eight times the gusset plate thickness. The tensile and compressive capacity of the gusset plate should be calculated using an effective width (w_{gp}) at the hinge zone, identified by two 30° lines drawn from the starting point of brace-to-gusset connection. The buckling resistance of the gusset plate should be calculated from the buckling length evaluated as:

$$L_{gp,b} = 0.12\,(L_1 + L_2 + L_3) \tag{4}$$

where L_1, L_2 and L_3 are the projections of the effective widths to the edges of the gusset plate. DC3 structures are designed to develop a global plastic mechanism, requiring additional design rules beyond those for DC2. In addition to the compressive force expressed in the equation (2), a shear force acting along the theoretical line of restraint in the buckling direction should be considered. This force can be evaluated through equation (5) where f_y represents the yield stress of the gusset plate material.

$$V_{gp,Ed} = 0.06\,\omega_{rm}\,f_y\,t_{gp}\,w_{gp} \tag{5}$$

2.2 Design example

The study analyses a six-story building that complies with DC2 regulations and is designed according to the second generation of Eurocodes 3 and 8. The structure is subjected to a reference seismic load of $S_\delta = 5.0$ m/s² on Type C ground. The plan dimensions are 31.00 m × 24.00 m, and the story height is 4.00 m on the base and 3.50 m on the upper levels. Two seismic-resistant configurations are examined: cross-concentrically braced frames (X-CBFs) and chevron-concentrically braced frames (Λ-CBFs). The braces are made of cold-formed circular hollow sections, while beams and columns utilize IPE and HE profiles, respectively. Gusset plates are used to connect brace with other structural members; in particular, fillet welds are used for the brace-to-gusset connection, while double clip angles are used at gusset plate interfaces (Figure 1).

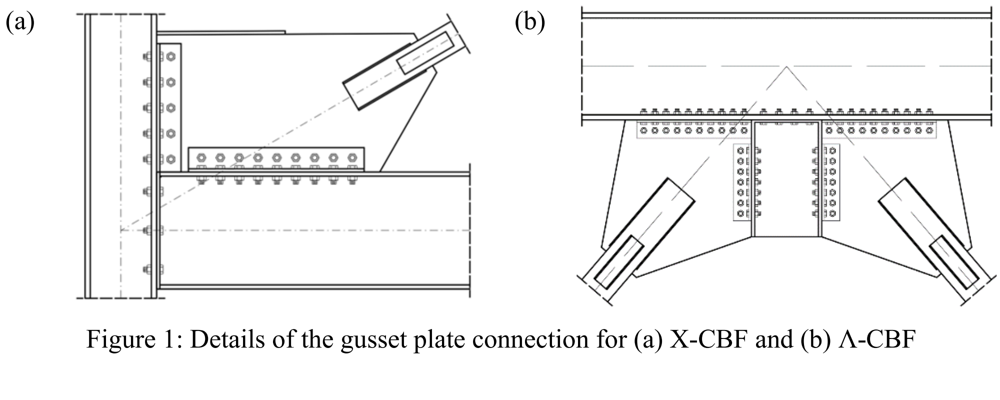


Figure 1: Details of the gusset plate connection for (a) X-CBF and (b) Λ-CBF

3 FINITE ELEMENT ANALYSIS

3.1 Modelling strategies

Finite Element Models (FEM) were developed using ABAQUS [25]. The analyzed part of the frame has been identified by considering the connection zone and the relative structural elements up to the null point of the bending moment diagram which arise under lateral loads (Figure 2). Appropriate boundary conditions have been imposed to ensure structural continuity. Analysis computational time has been reduced by considering 2D elements for beam and column outside the gusset plate zone, and 3D elements for the remaining parts. Timoshenko beam elements (B31) and 8-node linear brick elements (C3D8R) have been used to mesh elements. A sweep technique has been adopted to generate the mesh, while the required approximal mesh size has been identified through a sensitivity analysis. Contacts between components were explicitly modeled, with "Coulomb friction" and "Hard Contact" applied to represent tangential and normal behavior, respectively. The “Deformation Plasticity” model was assigned to the brace, while other elements followed an elasto-plastic hardening law with a yield stress of 355 MPa. The Young’s modulus and Poisson’s ratio were set to 210 GPa and 0.3, respectively. Material behavior was defined using the Von Mises yield criterion and isotropic hardening. Bolts were simplified following a commonly used approach in the literature [26], and preloading was simulated via a “Bolt Load.” Welds were represented with solid elements, using an elasto-plastic material with a 460 MPa yield stress, and were connected to adjacent parts via "tie constraints." To capture brace instability, a scaled buckling mode shape obtained from an eigenvalue analysis has been imposed. The influence of fatigue is not considered in this model [27-30]. Both models were subjected to a progressively increasing displacement.

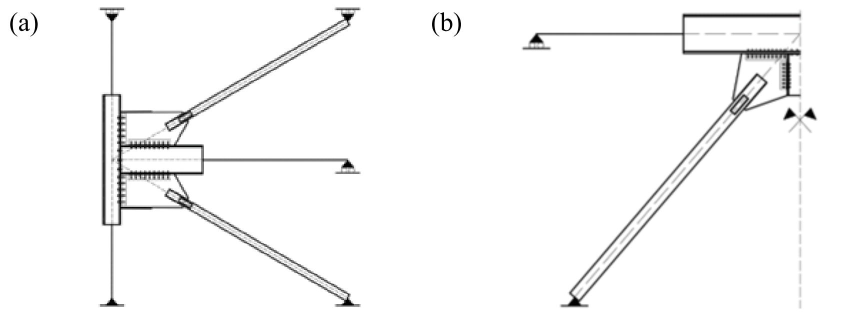


Figure 2: FEM scheme and boundaries conditions of (a) X-CBF and (b) Λ-CBF

3.2 FE analysis outcomes

FE analyses were conducted to study the forces and moments evolution at the gusset plate connections in both structural configurations, from the elastic phase to post-critical behavior. The X-CBF model was subjected to incremental displacement up to 2.5% of the story height, while the Λ-CBF model reached 1%. The deformed configuration along with the stress distribution for the X-CBF and Λ-CBF model is reported in Figure 3 and 6 respectively. For visualization purposes, plasticized areas are represented in gray. As expected, both cases exhibit a linear yield zone, with tension stress distribution following an approximate 30° pattern in the gusset plate. In the X-CBF model (Figure 3), portions of the beam and column also enter the plastic range due to the gusset plate’s considerable size, which increases joint stiffness and leads to plasticization of structural members. Additionally, the stress distribution is asymmetric due to torsional and minor-axis bending moments caused by the brace buckling. Conversely, in the Λ-CBF model (Figure 6), no plasticization is observed in the beam, though the stress distribution remains asymmetric due to local effects.

Figures 4 and 5 illustrate the force evolution on vertical and horizontal angles for the X-CBF model, while Figures 7 and 8 show the corresponding results for the Λ-CBF model. Solid and dashed lines differentiate between preloaded and non-preloaded bolts, respectively. Forces and moments were calculated at the geometric centroid of the angles, acting within the plane of the frame. To enable comparison, forces were normalized against the diagonal's tensile capacity, while moments were scaled by the plastic moment of the bracing arm. In each plot, the relative value obtained from the following four analytical methods is reported: KISS [19-20], PARALLEL [21], TRUSS [22], UFM [24]. The numerical results reveal significant variations in force distribution across different behavioral phases (elastic, onset of instability, and post-instability). Moreover, the force patterns at the connections differ considerably between the two structural configurations due to variations in gusset plate geometry and frame deformation mechanisms. None of the analytical methods accurately estimate the forces at the gusset interface, highlighting the need for further research to refine force distribution assessments in bracing connections and mitigate potential failure risks.

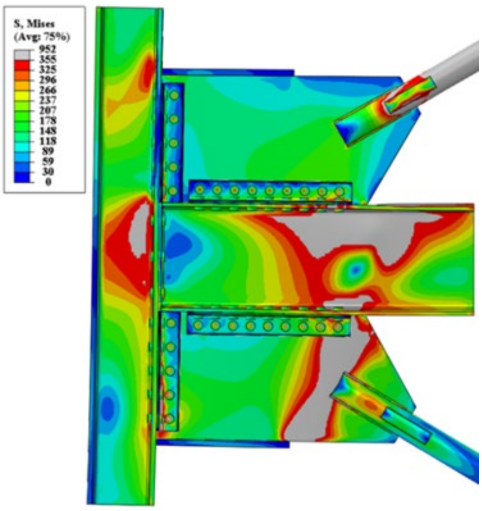
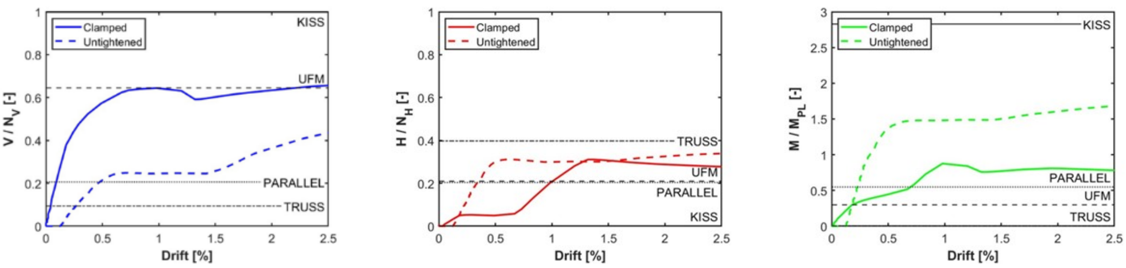
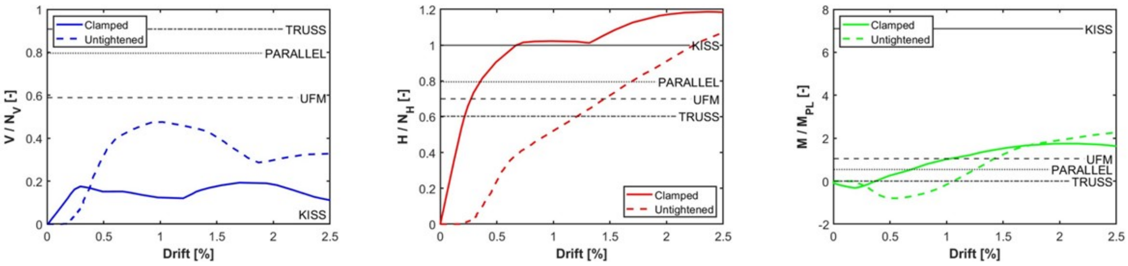


Figure 3: X-CBF stress distribution at 2% drift



(a) (b) (c)

Fig. 4: Forces on vertical connection (X-CBF): (a) vertical, (b) horizontal, (c) moment



(a) (b) (c)

Fig. 5: Forces on horizontal connection (X-CBF): (a) vertical, (b) horizontal, (c) moment

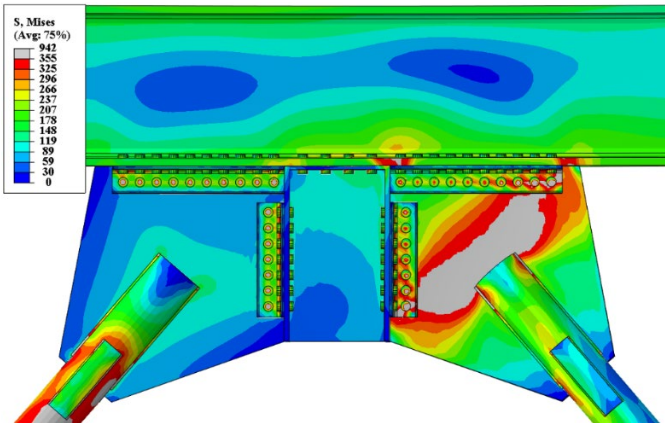
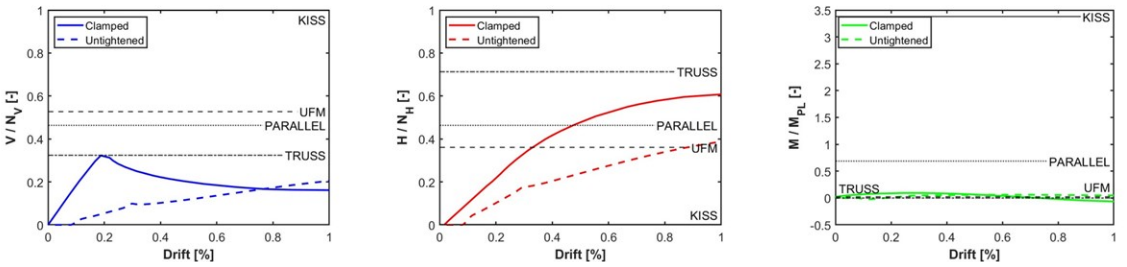
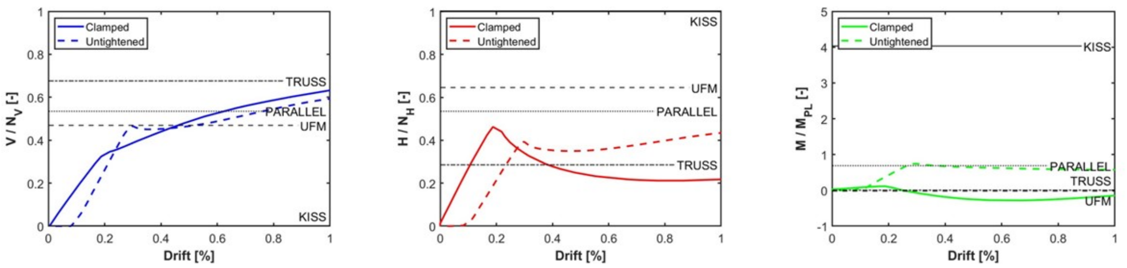


Figure 6: Λ -CBF stress distribution at 1% drift



(a) (b) (c)

Fig. 7: Forces on vertical connection (Λ-CBF): (a) vertical, (b) horizontal, (c) moment



(a) (b) (c)

Fig. 8: Forces on horizontal connection (Λ-CBF): (a) vertical, (b) horizontal, (c) moment

4 CONCLUSIONS

This study explores the design provisions outlined in the last version of Eurocode 8 for gusset plates used in concentrically braced frame (CBF) structures. Initially, connections in two seismic-resistant systems (X-CBF and Λ-CBF) have been designed based on the guidelines of the new Annex E, using a structural archetype. The impact of different analytical force redistribution methods on the final connection configuration has been examined. Subsequently, finite element models have been developed to assess key parameters. The study highlights the force distribution at gusset plate interfaces and compares them with values obtained from established literature methods. The findings emphasize the need for further research to evaluate ductility demands, stress concentration, and failure mechanisms for different gusset plate configurations.

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