Parametric FEM analysis of steel beam-to-column connection with extended end-plate and diagonal stiffeners

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Abstract. Connection behaviour is the most important factor in the steel structures behaviour subjected to cyclic actions. To avoid formation of plastic hinges in the column, it is stiffened with additional diagonal stiffeners. This paper presents an FEM analysis of extended end-plate beam-to-column connections with diagonal stiffeners subjected to the cycling loading proposed by FEMA350/SAC protocol. Models were analysed with the ANSYS finite element analysis program. The study analyses the energy dissipation capacity, the stress and deformation state. The parameters are the direction of singular diagonal stiffeners, influence of crossing diagonal stiffeners and lack of stiffeners. The design of the connections is based on the strong column-weak beam principle.

1. Introduction
Earthquake are natural, complex geological phenomena, which are exposed by disordered earth crust movements, resulting from the sudden release of energy in Earth’s crust. This movements, are with relatively short duration, and have rapid variations of directions, speed and acceleration of soil. Vibrations result either from tectonic plate movements, or from geological layers failure. Earthquakes can act on the environment by altering the state of equilibrium of superficial ground structures; subsidence and liquefying of saturated sand deposits. Earthquakes can produce negative effects on constructions and assemblies, such as: the total or partial destruction of vulnerable buildings, the destruction of structural or non-structural elements.
Seismic hazard is the imminent probability of an earthquake occurring in a certain area and with an intensity exceeding a certain threshold. The Romanian design rules of the structures provide for the probabilistic determination of seismic hazard using the mean of recurrence period (IMR). In Romania, the risk of a major earthquake is very high, so greater attention should be paid to the behaviour of structures subject to seismic action.
Steel structures are very widespread due to their proper behaviour in seismic actions, the relatively simple process of execution and maintenance. In the case of degradations, they can be brought back into operation in a shorter time, simpler and cheaper workmanship than the case of reinforced concrete or masonry structures [1]. Thus, beam-to-column joints are regarded as a critical area of the structure because in the joint or the associated area, the plastic hinges which are formed can lead to collapse of the entire structure. To improve the behaviour of the structure, it is necessary to improve the performance of the joint. In actual practice, the proper design of the joint is a problem in ensuring the strength and stability of the structure.
There is a dependency relationship between the stiffness, the strength, the ductility of the joint and the response of the structure to the seismic action [2]. Steel structures are very common in areas with high seismic risk due to ductile behaviour, efficient energy dissipation, and the relatively low complexity of execution [3]. Stress concentrations, weld defects and material imperfections are some of the reasons why fragile breaking of welded joints occurred [4].
To improve the behaviour of steel beam-to-column joints, a process of research and innovation has begun with the main purpose of preventing the failure of steel structures subjected to seismic actions. Considering the requirement of ductile yield and energy dissipation of steel structures, namely the possibility of plastic deformation without major loss of strength. The design codes require the use of seismic force reduction factors, which means the significant reduction of seismic design forces, at the same time, it was intended to create new types of joints with an increased degree of ductility and energy dissipation capacity, respecting the principle of “strong column – weak beam” [5], according to which the plastic joint develops exclusively in the beam end and at the column base.

The major degradations that occurred in structures following the (1994) Northridge and (1995) Kobe earthquakes have shown that the beam-to-column joints fail brittle [6] [7].

Numerous studies have been performed to analyze the behavior of the joint under cycling loading, determined by the Northridge and Kobe earthquakes [8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19]

2. Analysed models

The ductile behaviour of the joint is provided by directing the formation of the yielding mechanism in the end plate or beam end [20] The main objective of the presented analyses is to study the state of stress and deformation of the components of beam-to-column joints with end plate and stiffeners on the column flanges. End plate joints where the rigidities of the column and the beam are approximately the same, show some yielding phenomena at the column flanges, phenomena which should be avoided.

The considered nodes have as a common element the column – HEA300, the IPE360 beam and the bolts M22, group 10.9. The steel grade in beam, column, end plate, and stiffeners is S235. The profiles chosen for the beam and column have similar rigidities.

Node 1 has 20mm thick end plate, M22 bolts, 12mm stiffeners on the column at the beam flanges and 10mm diagonal upward stiffeners. Node 2 is similar to node 1 except the direction of diagonal stiffeners, that are applied in downward direction. Node 3 is similar to nodes 1 and 2, but diagonal stiffeners are intersected. The nodes are shown in figure 3. For an analysis of the connections under cycling loading, it has been developed a model using the ANSYS finite element software.

The static scheme of the analysed models is presented in figure 1. The load applied to the models is the alternate displacement, according to FEMA/SAC loading protocol (FEMA 350, 2005) [21] is shown in figure 2, it should be noted that the maximum displacement applied corresponds to the rotation $\theta$ of 0.1 radians.

![Figure 1. The static scheme.](image1)

![Figure 2. Loading protocol FEMA/SAC.](image2)
Figure 3. Analysed nodes.
The FEMA/SAC loading protocol assumes that lateral displacements of the top and bottom of the column are prevented (figure 4).

\[ \Theta = \frac{\Delta}{L} \]

**Figure 4.** Rotation $\Theta$ of the joint in terms of the beam end displacement.

The nonlinear behaviour of steel was modelled using the simple bilinear characteristic curve is shown in figure 5.

**Figure 5.** Rotation $\Theta$ of the joint in terms of the beam end displacement.

Analyses were performed on half of the structure using the symmetry option in order to reduce the number of nodes and finite elements, and to reduce the actual time of computation.
3. Results
The hysteretic curves are presented in figure 6, figure 8, figure 10, and figure 12.

![Figure 6. Force-displacement curve for node 1.](image1)

![Figure 7. Moment of yielding of joint elements for node 1.](image2)
**Figure 8.** Force-displacement curve for node 2.

**Figure 9.** Moment of yielding of joint elements for node 2.
Figure 10. Force-displacement curve for node 3.

Figure 11. Moment of yielding of joint elements for node 3.
Figure 12. Force-displacement curve for nodes 1, 2, and 3.

Figure 13. Displacement at which joint elements are yielded.

The stresses that develop in the end plates are shown in figure 14. In figure 15 and figure 16 are presented the stresses appearing in the flanges of the column. These stresses are recorded at the maximum values of the load.
Figure 14. Von-Mises stress in end plate at maximum load imposed according to the FEMA/SAC loading protocol.

Figure 15. Von-Mises stress in middle column flange at maximum load imposed according to the FEMA/SAC loading protocol.

Figure 16. Von-Mises stress in column flange at maximum load imposed according to the FEMA/SAC loading protocol.
In figure 12 it is observed that the joints have almost identical behaviour. In figure 14 it is observed a stress concentration in the end plate, at the level of beam flanges for all analysed models. At nodes 1 and 2 intensity of the stresses is lower than those at node 3, but differences are insignificant. In figure 15 it is observed that the distribution of stresses on the column flanges is uneven. In the case of node 3, the stress distribution is lower than in case of node 1 and 2. In figure 16 is a rather uniform distribution of the stress at the level of the column. Node 3 shows a higher intensity of tension in the column flange; here plasticizing phenomena are present.

4. Conclusions
According to the results presented in figure 12, it is observed that the direction of application of the diagonal stiffeners does not have a significant influence on the energy dissipation capacity, or the increase of the stiffness of the joint. Instead, the presence of these stiffeners significantly reduces the energy dissipation capacity compared to nodes without diagonal stiffeners. In figure 17 it can be noticed that the node without diagonal stiffeners represents a hysteretic curve significantly larger than the nodes with diagonal stiffeners, but the resistance is lower.

The stresses distribution in the end plate is uniform but at the level of the column flanges the stresses distribution is not as uniform. In all three nodes dissipation capacity is reduced compared to the node without diagonal stiffeners, but the strength of these types of joints is significantly higher.

![Figure 17. Force-displacement curve for node 3 and node without diagonal stiffeners.](image-url)
The similar distribution of stresses in the end plate and almost identical hysteretic behavior of nodes 1, 2, and 3 are due to the almost identical make-up of the nodes, and the position of the single diagonal direction is insignificant in the case of bidirectional cycling loading. Supplying of additional diagonal stiffeners or even stiffeners as conformal to the node 3 lead in the increased end plate stiffness and a reduction of stresses in the column. The similar stiffness between the beam and the column is avoided, but the phenomenon of yielding in the column are not avoided, the column being much more stressed, the end plate being highly demanded and susceptible to failure.

![Figure 18. The force corresponding to the maximum displacement.](image)

5. References

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