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Research Article

An investigation of the behavior of header end-plate connections under monotonic loading

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ABSTRACT

In seismically active regions such as Turkey, the context of the nonlinearity provided by a building is based on the behaviors of structural components; beams, columns and their connections constituting the seismic force resisting system of the structure. Of these members, beam-to-column connections can play a considerably important role even if they have a capability of limited stiffness and flexural strength. Structural steel connections are mainly classified as a pinned or a moment connection. However, some beam-to-column connections having limited stiffness and flexural strength, which are called semi-rigid connections such as header end-plate connections designed so as to transmit only shear forces, can be characterized by moment-rotation relationship. This paper investigates the behavior of header end-plate connections using finite element (FE) modeling. The FE models include material, geometrical and contact nonlinearities. FE modeling technique was first verified through the test results of the experimental research performed by Aggarwal (1990). Then the effect of header end-plate thickness upon moment-rotation relationship was investigated. According to the analyses results, in addition to shear stresses, axial tensile stresses have been observed to occur in the bolts at the tension side and thickness of the header end-plate and beam web play a governing role in the development of initial rotational stiffness and the flexural strength of header end-plate connections.

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1. Introduction

Due to unexpected brittle failure of welded beam-to-column connections during the 1994 Northridge earthquake, researcher have raised investigations on moment resisting beam-to-column connections to sustain their ductility during earthquake. Semi-rigid beam-to-column connections have also been studied since they have exhibited ductile behavior if designed in such a way that plastification due to plate bending has dominated the energy absorption in the connection. Experimental and analytical studies showed that semi-rigid beam-to-column connections could have sufficient rigidity, ductility and strength. Moreover, these connections exhibited more suitable results than rigid welded connections (Chen et al., 2011). In designing a steel framework, it is customary to represent the connection behavior as a fully moment resisting connection or a simple connection. However, this approach leads to an inaccurate prediction of the frame behavior because perfectly rigidity and complete flexibility are idealized forms of connection behavior and cannot be reached in practical connections. In particular, the behavior of lateral force resisting frames with semi-rigid steel beam-to-column connections cannot be accurately estimated under lateral forces when the moment-rotation relationship of the connections has not been considered. Therefore, if the semi-rigid connections are used, effects of the connection flexibility on the frame behavior must be taken into account in the design procedures. The primary parameter of the steel beam-to-column connections is their rotational deformation, θ_R caused by the in-plane bending moment, M (Fig. 1) (Abdalla and Chen, 1995). The shear and axial deformations of the connections are generally ignored.

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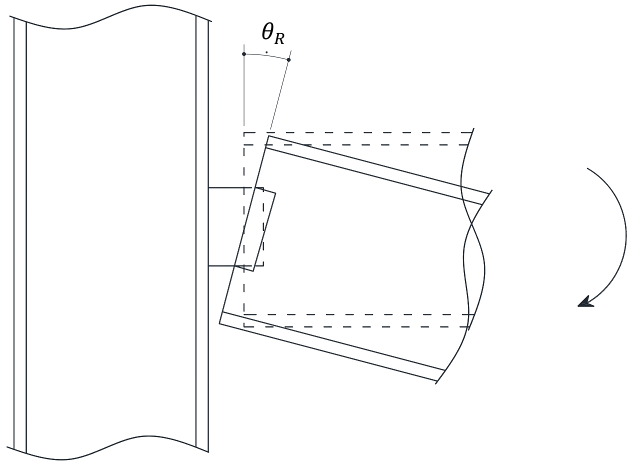

Fig. 1. Rotational deformation of the connection.

In the design of header end-plate beam-to-column connections only shear force has been considered as they are assumed to be pin-connected. However, these types of connections may have a considerable, but limited, flexural strength, and can be rigid enough to provide a contribution in lateral stiffness of the frames, which is influenced by end-plate thickness, bolt diameter and bolt arrangement. In order to account for this contribution for design purposes, moment-rotation relationship of header end-plate beam-to-column connections has to be defined. In fact, tensile stresses in the bolts at the tension side of the connection, in addition to shear stresses, should be taken into consideration during the design procedure (Karasu and Vatansever, 2017).

The aim of this paper is to investigate the behavior of header end-plate beam-to-column connections under monotonically increasing load. These types of connections consist of an end-plate whose height is smaller than the beam depth and bolts arranged as shown in Fig. 2. End-plate (header plate) is welded to the beam web and bolted to the column flange. The importance of these connections in the context of overall structural behavior lies in their rotational stiffness. Actually, they are assumed to be pinned flexible connections during the design procedures although they have pronounced rotational stiffness. Behavior of the header end-plate connections can be defined by moment-rotation relationship obtained by experimental tests, analytical models, numerical models and FE models. The most confidential results can be achieved by experimental tests. However, due to cost of experimental test, one of the other methods alternative to the experimental study can be utilized. In this paper, FE modelling technique which was first validated through a comparison of moment-rotation curves from experimental research performed by Aggarwal (1990) has been used to obtain the behavior of header end-plate connection. By taking the experimental conditions into considerations, such as material properties and contact conditions, the FE model has been developed. The test setup and member details considered are shown in Fig. 3.

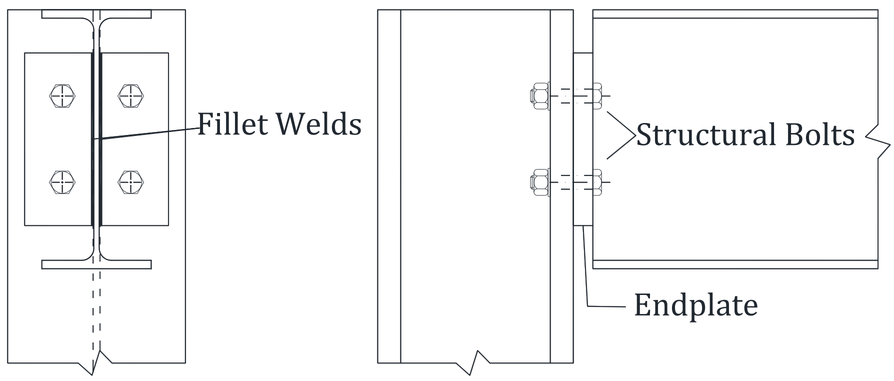

Fig. 2. Components of header end-plate connection.

For the header end-plate connections, collapse mechanism typology is related to the ratio between the flexural strength of the header end-plate and axial strength of the bolts (Faella et al., 2011). Additionally, beam web yielding and beam web local buckling are also limit states for the connection in bending. However, premature web local buckling must be avoided as it causes a brittle failure. This can be ensured that the flexural rigidity of header end-plate should be less than the axial rigidity of beam web. If this is warranted, ductile behavior is also provided.

Tensile stresses in the bolts at the snug tight condition may reach a high level, resulting in a decrease in the shear strength of the bolts. In order to avoid premature bolt failure due to tension forces from bending and prying forces, yield line mechanism should be developed in header end-plate before the strength of the bolts is exceeded. To guarantee the yield line mechanism in the header end-plate, pretension can be applied to the bolts. This paper also investigates the variation of axial stresses in the bolts preloaded up to 70% of their tensile strength. To better understand the yield line configuration, FE analyses have been performed to observe the failure mechanisms of the connection models with different end-plate thicknesses.

The studies on header end-plate connections have concentrated on experimental tests and analytical studies for achieving the moment-rotation relations (Sommer, 1969; Aggarwal, 1990; Pilgr, 2009). In the study conducted by Sommer (1969), twenty tests have been performed to observe the effects of the end-plate thickness, bolt diameter and bolt orientation on the moment-rotation relations. Aggarwal (1990) has carried out some tests on header end-plate connections with two different end-plate configurations. Pilgr (2009) has performed experimental tests to observe moment-rotation relations.

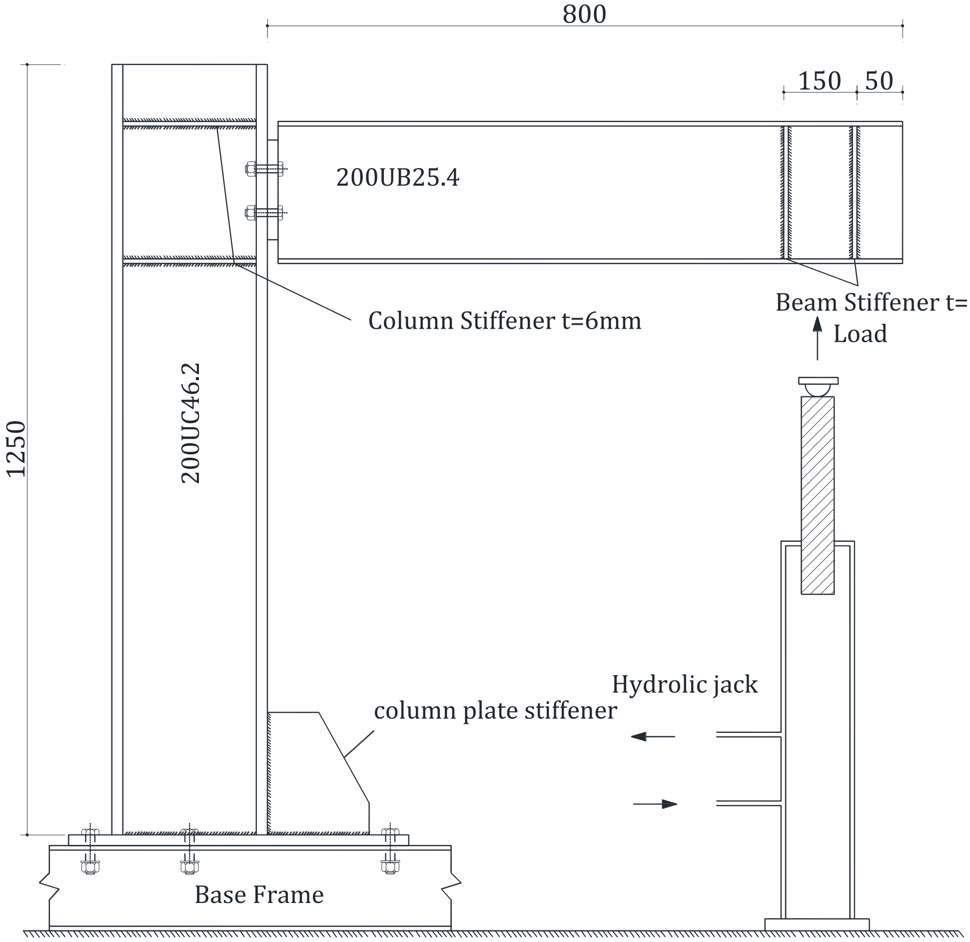

Fig. 3. Test setup for specimen M3 (adopted from Aggarwal, 1990).

2. Verification of Finite Element (FE) Modelling

Before FE models of the header end-plate connections considered in this study, FE modelling technique has been verified through the results of experimental research conducted by Aggarwal (1990). For the verification, the results from the test of the specimen M3 has been taken into account and its FE model has been developed. The model includes material and geometrical nonlinearities. Initial imperfection corresponding to appropriate buckling mode associated with the loading has also been accounted for with the scale value of $L_{\mathrm{BEAM}}/1000$ (Ismail et al., 2016). Linear eigenvalue analysis is used to estimate the suitable buckling mode. Therefore, application of the imperfection becomes possible to trigger buckling of the beam web within the connection. Riks method is used for nonlinear analysis of the connection.

In order to simulate the test setup, the column is assumed to be fixed at the bottom and the lateral displacement of the column is prevented. A point load was applied to tip of the beam which is allowed to deflect upward in the same manner as in the test. Lateral displacement of the beam was restrained to prevent any possibility of premature failure caused by lateral torsional buckling.

The specimen considered consists of a beam 200 UB 25.4 with 12mm thick end-plate attached to the beam web with fillet welds and connected to the flange of the column 200 UC 46.2 with four M20-10.9 bolts. All the bolts are pretensioned to the minimum specified pretension force defined as 70% of the bolt tensile strength. The yield stress considered for the material of beam, column and end-plate is 250 MPa. The yield and ultimate stress for the bolts were taken as 900 MPa and 1000 MPa, respectively. Joint details are given in Fig. 4. Each material for each member was defined by true stress-true strain curve. Therefore, engineering stress-strain values were converted to true stress-strain values (Kaufmann et al., 2001).

2.1. Element properties

3d FE model corresponding to the specimen of M3 is given in Fig. 5. The connection model is developed by using a general commercial program ABAQUS. The model includes material and geometrical nonlinearities, and also nonlinear interaction between the surfaces in contact to each other. All bolts in the connection are tightened with the sufficient pretension. For more reliable performance, the eight-node brick elements C3D8I with incompatible modes are used. Since having additional degree of freedoms, C3D8I has a capability to capture bending behavior better. By using this element type, shear locking which leads to observe stiffer behavior for bending behavior due to getting shear deformations instead of flexural deformations, is removed and volumetric locking is much reduced.

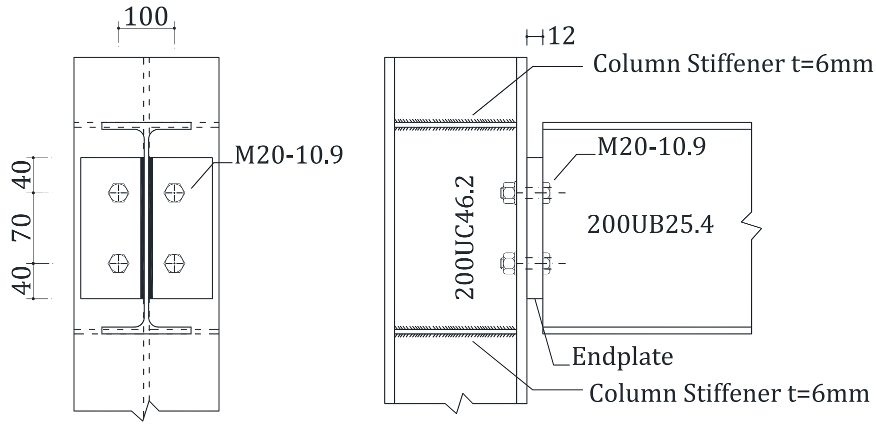

Fig. 4. Details of the header plate connection.

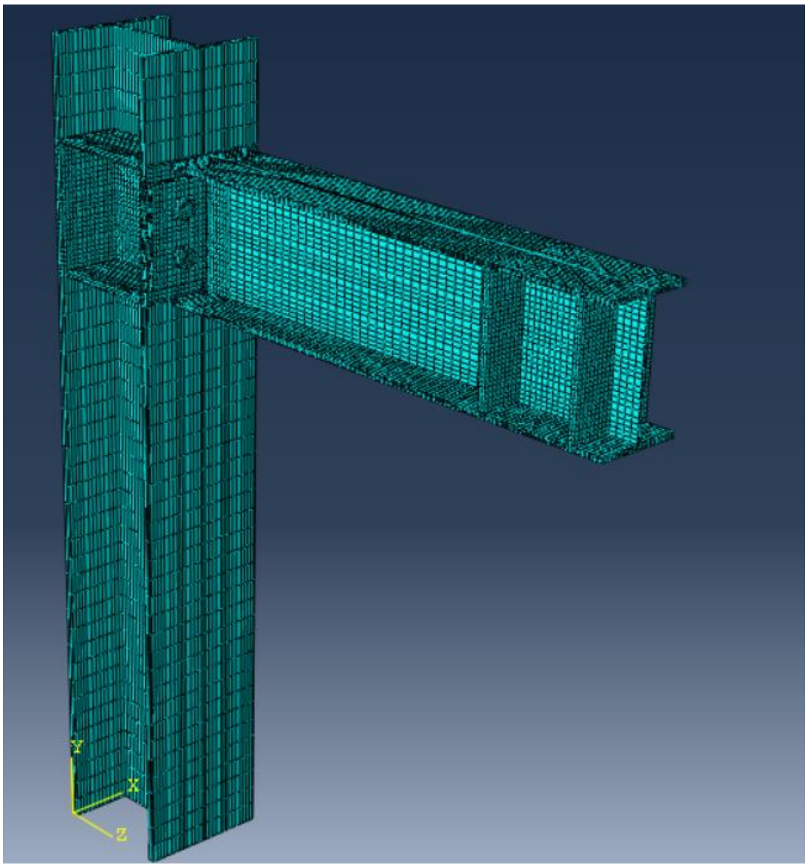

Fig. 5. 3D FE model in ABAQUS.

2.2. Contact interaction properties

One of the most important process for the FE modelling is to reflect the contact interaction properties into the models properly. If this type of interaction is improperly modeled, it is not possible to simulate the behavior of the connection in a realistic manner. Modelling the contact interaction; between the outer surfaces of the column flange and header end-plate and between the bolt head/nut and column flange/end-plate requires two different interaction properties to define. First one is 'hard contact' property assuming that when the surfaces are in contact, any contact pressure can be transmitted between surfaces. However, if the contact pressure is zero, the surfaces are separated. The second one is tangential behavior of the contact. In this definition, friction between the contact surfaces at the connection is modelled using the classical Coulomb model where the friction coefficient is 0.30 (Ismail et al., 2016) by using penalty stiffness formulation. It is also allowed to small sliding effects at the interface of the contact surfaces. Welding connection between the header end-plate and beam web is modelled by tie constraint.

For the simulation of header end-plate connection, in addition to static and implicit solver, explicit dynamic solver is also suggested due to complicated contact problems. For the static and implicit analyses for each load increment, the continuity of the contact state is checked first and if there is any change on the contact state, general stiffness matrix is calculated on the new changed contact state. Therefore, it will be difficult to find the changed contact state when there are too much contact interactions in the model that leads to convergence difficulties for the both static and implicit solver. In the quasi-static explicit dynamic solver, kinematic contact was used. According to this algorithm, at first model predicts the configuration of the deformed shape without contact conditions. Then the equilibrium state is determined to calculate time increment and resultant force to remove the penetration of the slave nodes on the master surfaces. In the quasi-static explicit analyses, the ratio of kinetic energy to internal energy must be smaller than 1% throughout all loading steps (Hongxia et al., 2007). This can be ensured by reducing the mass scaling, but which increases the computational time.

2.3. Meshing

Another important issue in the FE modelling is element meshing that governs the accuracy of the analysis results. For accurate results with less computational time, different mesh sizes have been assigned to each FE model part, i.e. a finer mesh in the vicinity of the connection where high stress and strain gradients are assumed to take place was used, while the coarser mesh was employed in the areas far from the connection zone. Also, to get acceptable results under flexural moment, at least four layers were formed through the member thickness. Meshing of each component considered in the model are shown in Fig. 6.

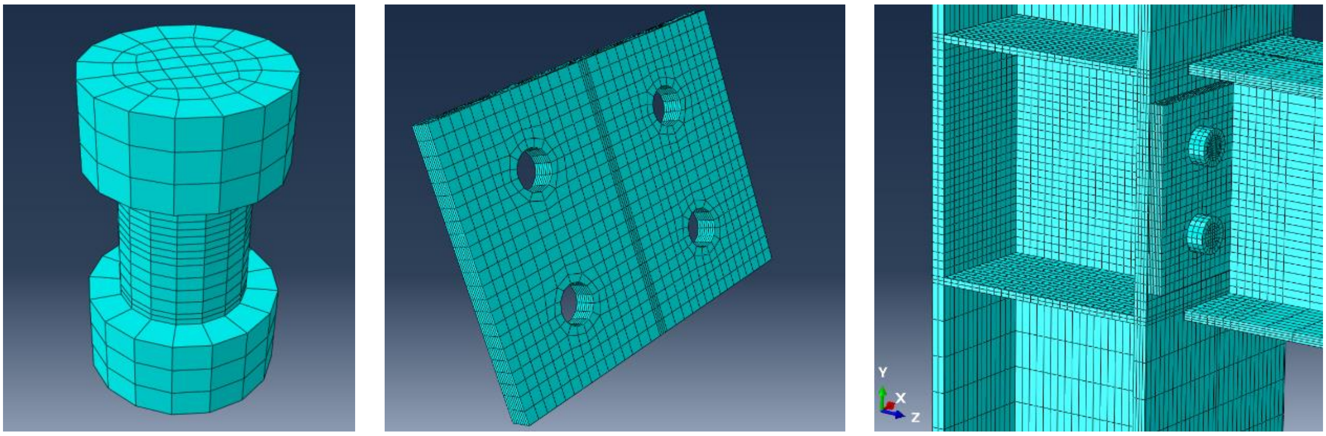

Fig. 6. Meshing of different components in the model.

2.4. Loading

The FE analyses of the model involved two load steps. The first load step was used to apply pretension forces to the bolts by applying displacements to the ends of the bolt heads. The prescribed bolt displacements corresponding to the axial force of 172 kN (TCDCSS, 2016) which is defined as the minimum pretension force for M20-10.9 bolts were calculated considering axial rigidity of the bolts. The second load step was employed to define monotonic loading path applied by imposing upward displacement to the free end of the beam.

2.5. Parametric study

A parametric study was carried out to examine the effect of header end-plate thickness upon the response of the four-bolt header end-plate connections. The thicknesses were chosen to vary from 8 to 16 mm.

3. Comparison of the Moment-Rotation Curves

Analysis results from two different models were compared with experimental data in terms of moment-rotation characteristics, load carrying capacity and the failure mode of the connection. As shown in Fig. 7, the analysis results are in good agreement with those from the experimental test. Both results from experimental test and FE analyses support that the end-plate flexural deformation and yielding of beam web govern the flexural moment capacity of the joint. Deformed shape of the connection and plastic regions occurred at the end of analyses are shown in Fig. 8. The load carrying capacity which is obtained as 32 kN (Chen et al., 2011) also estimated with good accuracy.

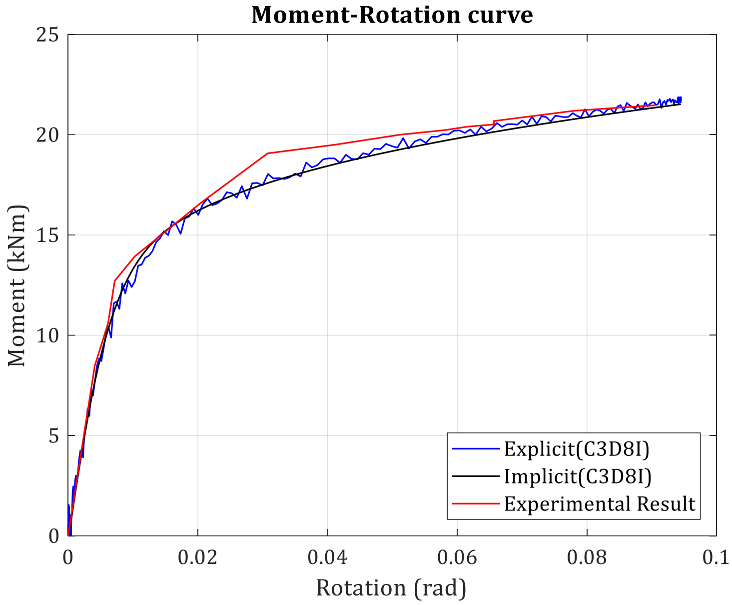

Fig. 7. Moment-rotation relationship.

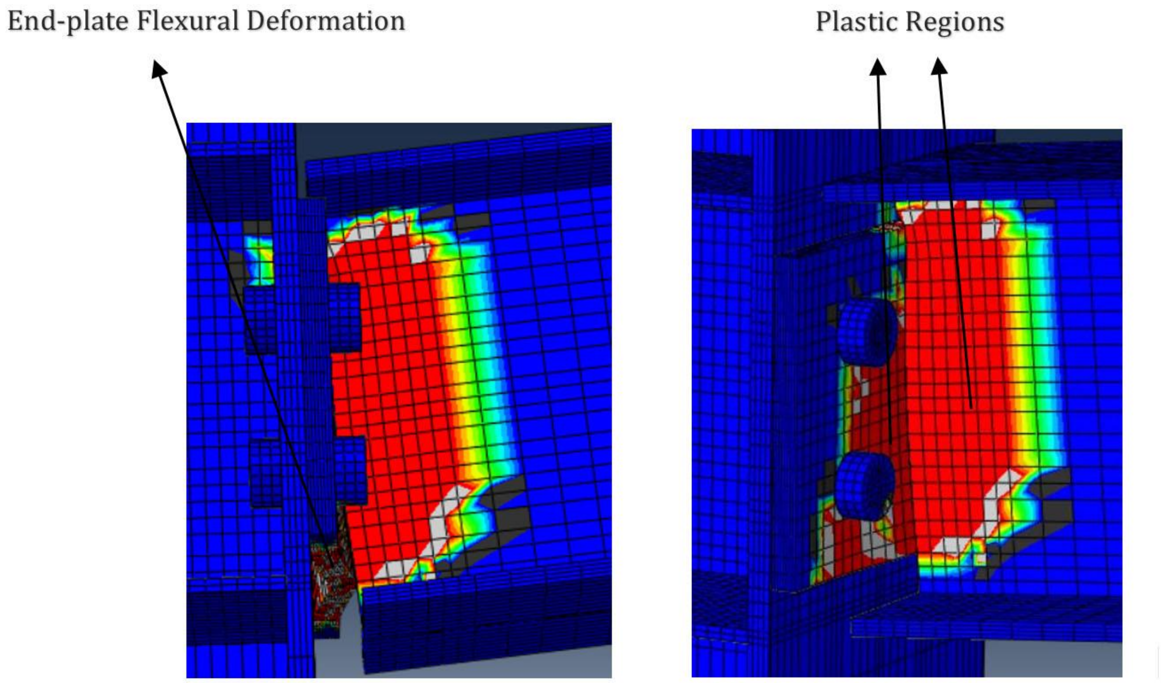

Fig. 8. Deformed shape and plastic regions of the connection.

4. Analysis Results

Analysis results have shown that pretension applied to the bolts influenced the initial response of the connection. For the explicit dynamic analyses, it was observed the minor vibrations in the elastic and inelastic part of the moment-rotation curve due to loading speed effect. Depending on the bolt diameter and the thicknesses of header end-plate and column flange, bolts may exhaust their strength if the tensile stresses in bolts reach a sufficiently high value before a yield mechanism in the header end-plate develops (Jaspart and Demonceau, 2008). To observe the variation of the axial stresses (S33) in the bolt shank is therefore shown in Fig. 9. Based on the implicit analyses results, since the most stressed region was quarter part of the cross-section at the mid-length of the bolt shank, this region was considered to obtain the variation of axial stress shown in Fig. 10. In this figure, axial stresses were calculated by taking the average of axial stresses on the nodes within the region. It was observed that the axial stresses decreased about 300 MPa that is equal to 30% of the tensile strength of the bolt. Nevertheless, clamping force, which is provided by pretension applied to the bolts, to keep the header end-plate attached to the column flange was not beaten. Furthermore, this allows the yield mechanism to develop in the header end-plate or the column flange, whichever is the governing.

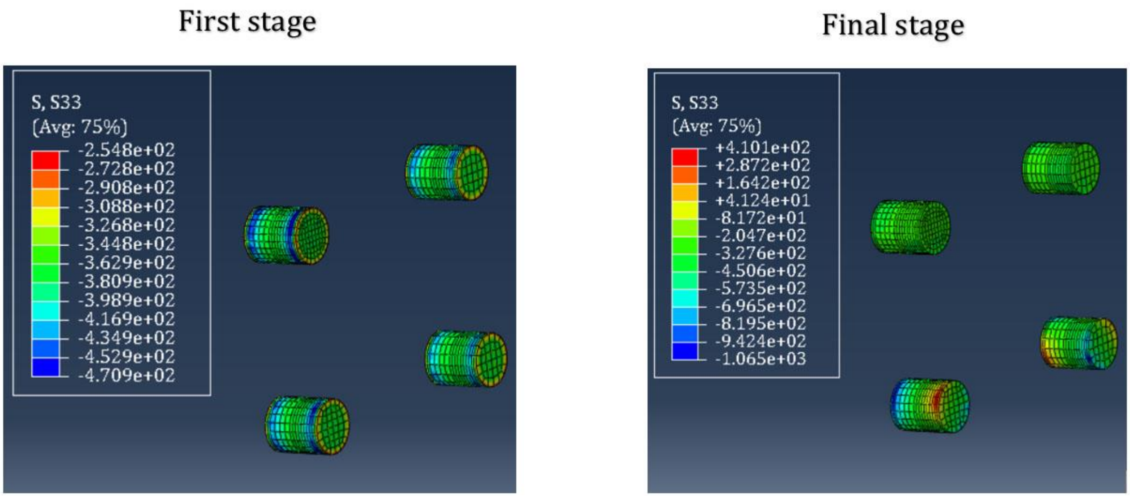

Fig. 9. Bolt axial stresses S33 for the first and final stage of the loading.

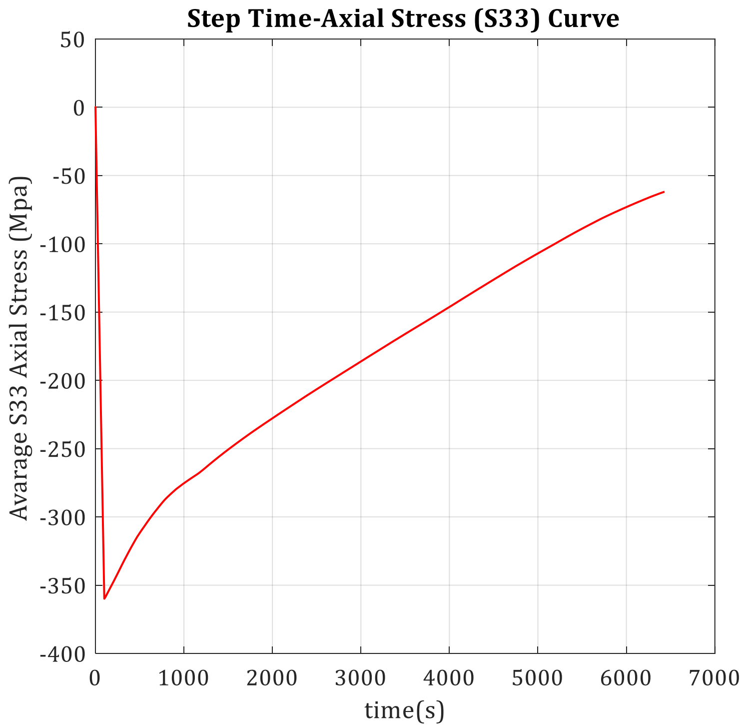

Fig. 10. Variation of axial stresses S33 in the bolt shank (avg.).

Analysis results show that the header end-plate thickness has an important role in determining the behavior of the connection. The plastic regions (indicated by red color) of the beam side for different thicknesses were given in Fig 11. The header end-plates with the thickness larger than 16 mm can be considered as a non-energy dissipative element in the context of this study because they have no a considerable contribution to the energy dissipation. Instead, the beam web yielding and its post-buckling behavior seem to be pronounced. Moreover, it is clear that thicker end plates, which means the plates having higher rigidity, increase the initial rotational stiffness and cause less or no praying force when the Figs. 11 and 12 are examined.

5. Conclusions

A series of 3D FE analyses was conducted to investigate the behavior of header end-plate connection using a general purpose of FE software, ABAQUS. In the study, header end-plate thickness was selected as a parameter. According to the analyses results, the connections with relatively thin end-plates have reached the flexural capacity with the plastification of end-plates and beam webs while the post-buckling of the beam web together with yielding was dominant in the others. Since the post-buckling behavior of beam web causes the strength degradation this phenomenon should be avoided. Based on the analysis results only, the ductility of the connection was found to be acceptable for all connections. However, an experimental research is still needed for further evaluation on the ductility. As long as the thinner end-plates were employed, praying forces which increase the tensile stresses in the bolts became more pronounced. Also, the bolts remained elastic during the analyses. Nevertheless, based on the variation of the axial stress in bolts, it should be taken into account in the design of the bolts. Finally, although header end-plate connections have a limited flexural stiffness and strength, they seem to be promising to provide additional energy dissipation which is essential for better performance of seismic force resisting systems.

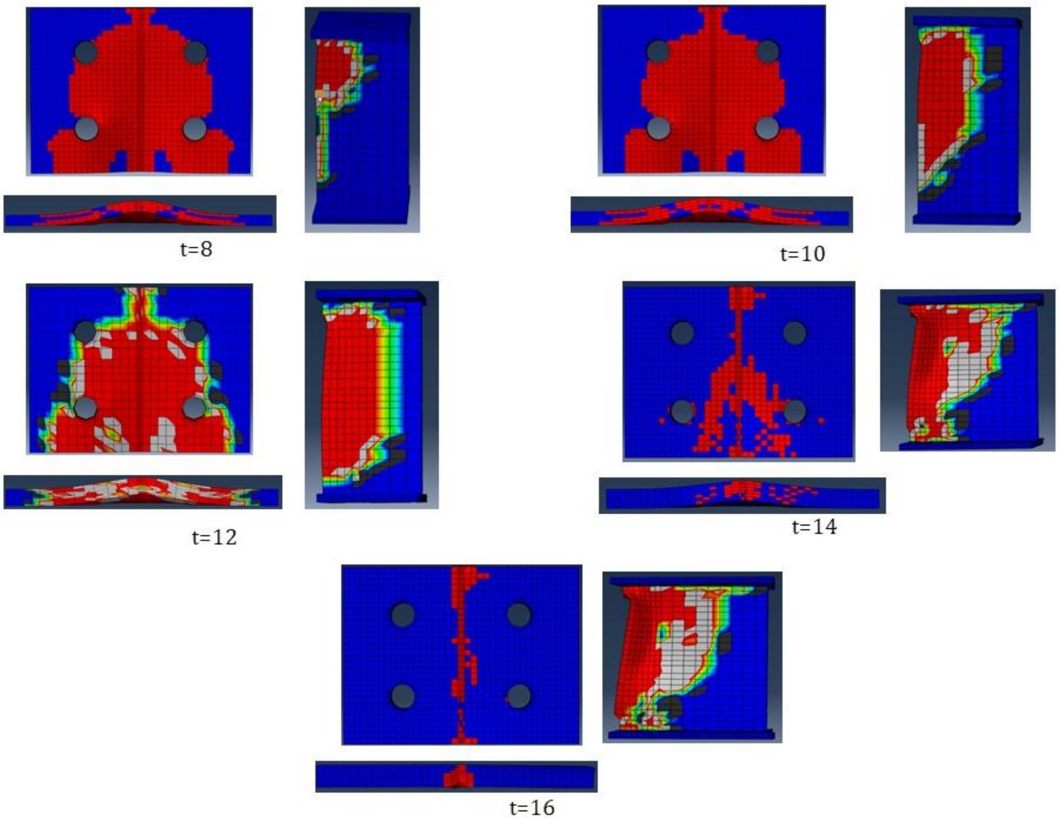

Fig. 11. Plastic region propagation in header end-plate and beam web.

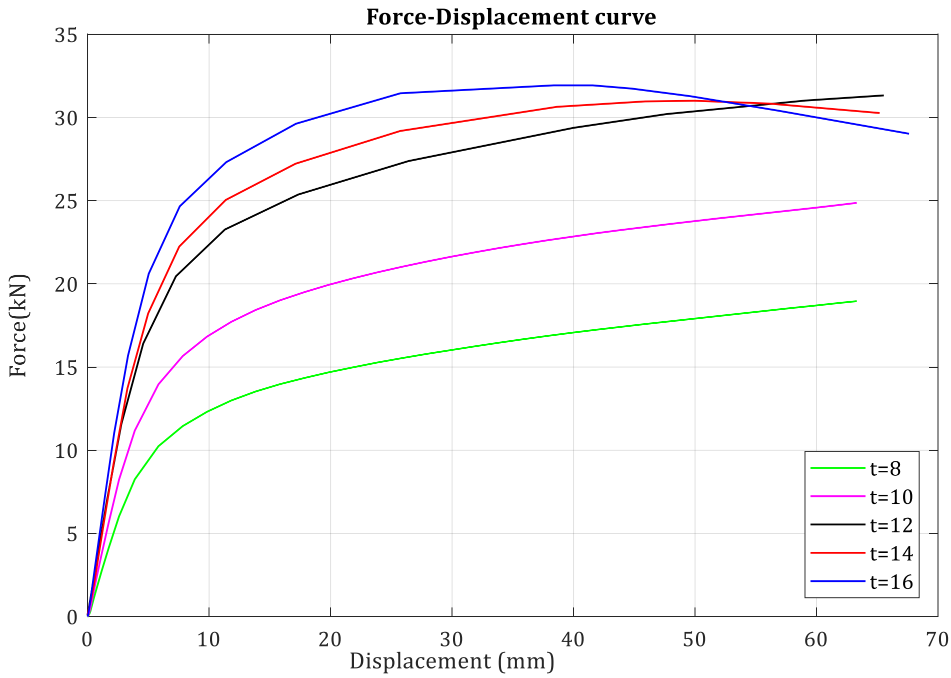

Fig. 12. Effect of header end-plate thickness on the connection behavior.

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