Design of Steel Beam-Column Connections

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Abstract: In this paper a theoretical and experimental research of the steel beam-column connections is presented. Eight types of specimens were being researched, composed of rigid and semi-rigid connections from which 4 connections are with IPE-profile and 4 connections with tube’s section for the beam. From the numerical analysis of the researched models, and especially from the experimental research at the Laboratory for Structures in the Faculty of Mechanical Engineering - Skopje, specific conclusions were received that ought to have theoretical and practical usage for researchers in this area of interest.

Keywords: rigid and semi-rigid connections, beam-column connections, design (structural design), bending moment, rotation, plastic hinge, collapse.

1. Introduction

With the beginning of the third Millennium as the traffic and population increased, so did the need for buildings and motorway structures. The building land in the centers of big towns became compact and its price increased. At the same time the “highest buildings” became an aspiration and prestige of the structural designers. Consequently the use of multi-storey composite structures (steel columns, steel beams, and reinforced concrete slabs) became a necessity. The multi-storey composite structures are used for different types of buildings such as office buildings, bank buildings, industrial buildings, public buildings, high-rise parking buildings, etc. These kinds of buildings are built all around the world, mainly in the highly developed countries depicting their financial and technical power.

The safety and function expressed through control of the mass, stiffness, strength, and ductility at the structural design of the multi-storey composite structures, and mostly of the connections of the elements exposed under cyclic-horizontal loading (such as earthquake, wind loading) in combination with other loadings is of highest priority for this kind of structures (buildings).

See(Fig.1-3): Multi-storey structures (administrative and residential buildings).

Figure 1., High-rise parking building (under construction)
Figure 2., Figure 3. Skyscrapers in Chicago, USA

The behavior of the beam-column connections in the multi-storey frame structures (MSFS) is viewed as a whole and it’s in direct correlation and dependence with the behavior of their main constructive fundamental elements (steel beams, columns and the elements for their connection) [5]. In other words, the way the beams, the columns and their elements of connection behave, that way the MSFS behaves [3].
The behavior of the beam-column connection in the MSFS depends mostly from their constructive solution. However, dominant in this paper is the research of new or modified constructive solutions of the connections, and all with the purpose of improving their loading capacity in conditions of real external loadings. Some constructive solutions can be controlled by the dissipation of energy, meaning, they can be controlled by the stress and deformational distribution in the sections of the elements of the MSFS i.e. the constructive solution of the connection directly influences the appearance of the plastic hinges in some of the sections of the elements, when their loading capacity is exhausted [4].

2. Design criteria for controlled damages and collapse for beam-column connections

In the numerical modeling of the beam-column connection the demands of the new codes [8],[9], are incorporated, which is the well-known concept of the seismic resistant structure that proposes development of plastic hinges in the beams, and rarely in the part of the columns. Consequently, the size of the statical influence that dictates the order of the plastic hinge appearance should be taken under consideration i.e., column bending strength should be larger then beam bending strength. As a result, for each beam-column connection the following equation should be satisfied:

$$\sum M_{R,c} > \sum M_{R,b}$$

(1)

, where $\Sigma M_{R,c}$ is the sum of existent bending moments in the column, and $\Sigma M_{R,b}$ is the sum of existent bending moments of the beams in the connection point [9].

With the alternative of the collapse mechanism (the order of the element’s plastification) and generally the mechanism of energy dissipation, two different approaches exist:

The first approach is based on the contribution of the panel–zone in the energy dissipation with the purpose of its reduction and also accepting a part of the plastic deformations, without excluding the contribution of the columns and the beams.

The second approach excludes the panel zone into the energy dissipation. As a result, in this case, the end parts of the beam should accept the plastic deformations. Accordingly, the beam–column connection should be specified in detail.

- Design of beam-column connection without contribution of the panel-zone into the energy dissipation

Supposing that the shearing stress is equally distributed in the panel-zone (part of the web of the column between the two beams) shearing stress developed in the panel – zone is presented as follows:

$$\tau_p = \frac{V_p}{(d_c - 2 \cdot t_{sf}) \cdot t_p}$$

(2)

, where $V_p$ is the shearing force into the panel-zone at the steel beam-column connection.

$$V_p = \frac{\sum M_b}{(d_b - t_{sf})} - V_c$$

(3)

In the beam-column composite connection $V_p$ is the shearing force in the columns designed through the assumption that zero-moment is located into the middle part of the column section [6].

$$V_p = \frac{M_b^1}{(d_b - t_{sf})} + \frac{M_b}{d_b} - V_c$$

(4)

Consequently, taking into consideration the balance of $\Sigma M_b = \Sigma M_c$, the result is as follows:

$$V_p = \frac{\sum M_b}{(d_b - t_{sf})} \cdot \left(1 - \frac{d_b - t_{sf}}{H - d_b}\right)$$

(5)

, the middle stress of shearing in the panel-zone is as follows:

$$\tau_p = \frac{\sum M_b}{t_p \cdot (d_c - 2 \cdot t_{sf}) \cdot (d_b - t_{sf})} \cdot \left(1 - \frac{d_b - t_{sf}}{H - d_b}\right)$$

(6)

3. Types of researched models-numerical design

For nonlinear numerical analysis of the eight types of models (SP1, SP2, SP3, SP4, SP5, SP6, SP7 and SP8) the software packages DRAIN-2DX and ANSYS Workbench were used [10] [11].
**MODEL-SP1** (Fig.4) is a modification of the standard and most commonly used rigid beam-column connection with end-plate welded with fillet welds at the end of the beam. The modification used consists of additional triangular webs at the end of the beam that are welded to its upper and lower level with the end-plate, so that the stiffness in the end of the beam is enhanced [3]. This is the hot rotated beam (according to DIN1025) IPE200 [7], with height h=200 mm, width of the level b=100 mm, thickness of the web t_r=t=5.6 mm, and thickness of the levels t_p=t=8.5 mm. The material of the beam is S275 JO+M according to EN10025-2/2004 with the following mechanical characteristics: \(f_y = \sigma_T = R_{Eh} = 338\) (MPa), \(f_u = \sigma_m = R_m = 464\) (MPa), \(A = 33.1\%\) [5].

The connection between the steel beams (designed by the valid standards MKS U.E7.140 correlated with EC3) was realized by the modified beam-column connection using end-plate connection that was welded to the end of the beam. At the end-plate 2 holes were made (d=17mm.), deployed in 4 rows, total 8 holes in one plate i.e. on one beam-column connection. For connecting the beams through the end-plates with the columns, high valued bolts M16 class 10.9 were used, pretensioned with the right moment (Mu = 253Nm).

![Figure 4. The stress distribution in MODEL-SP1](image)

In figure 4 the stress distribution is shown and it is clearly visible that the influence of the welded triangular webs is big i.e. the maximal stress is moved from the end-plate through the middle of the beam i.e. from the end of the beam to the top of the triangular webs. Due to the stability problem of lateral buckling, the plastic hinge appearance is expected near the location of the maximal stress in the pressed part of the beam [MKS U.E7.101 –for calculating the lateral buckling of pressed flange; (MKS U.E7.121– calculating buckling of pressed web) [1].

**MODEL-SP2** (Fig.5, Fig.6) is a designed rigid beam-column connection with end-plate welded with angular welds to the end of the beam [4]. Same as the previous, in this connection 2 triangular webs are used at the end of the beam that are welded to the upper and the lower part of the tubular beams are used at the end of the beam that are welded to the upper and lower part of the end-plate, so that the stiffness of the end of the beam is enhanced, as the loading of the welded beam-column connection in the plate. The beam in the SP2 model is a hot shaped hollow profile with rectangular cross-section (according to DIN59411, MKSC.B5.213) [7] 200.100.5, with height h=200 mm, width of the level b=100 mm, thickness of the walls in the section= s= 5 mm. The material of the beam is S355J2H according to EN10219-1/2006/EN10021 with the following mechanical characteristics: \(f_y = \sigma_T = R_{Eh} = 437\) (MPa), \(f_u = \sigma_m = R_m = 554\) (MPa), \(A = 24.8\%\).

Our valid standards MKS U.E7.140 were used for the design of the beam, adapted for the shape of the beam. Here, same as the previous model, end-plate connection is used with an end-plate that has a bigger height than the one of the SP1 model, as a result to the specific shape of the beam (rectangular cross-section), also welded to the end of the beam. At the beginning, on the plate 2 holes were made (d=17mm), deployed in 4 rows. That is total of 8 holes on one plate i.e. on one beam-column connection for connecting the beams through the end-plates with the columns, with bolts M16 class 10.9.

![Figure 5, Figure 6. The stress distribution in MODEL-SP3](image)
The stress distribution and the influence of the welded triangular webs are shown in Fig. 6. The maximal stress is moved from the end-plate to the middle of the beam i.e. at the end of the beam to the top of the lower triangular web [1].

**MODEL-SP3** (Fig.7) is a designed rigid beam-column connection with an ending joint that is welded to the dismantling connection on the beam with the column. The joint is made by welding 3 separate plates under an angle of $90^\circ$ to the background end-plate, where holes are made for the connection with the beam and the connection with the column. The design of the joint is made so that its loading is bigger than the loading at the end of the beam so that the reallocating of the eventual plastic hinge will be on a bigger distance from the column, different from the previous 2 analyzed models. The loading of the joint depends from: $W_x$ – section modulus receiving the bending moment, $A_r$ – surface of the web receiving the cutting (shearing) and the mechanical characteristics of the adopted material.

Holes are made on the beam IPE200 i.e. 2 in the levels and 3 on the web, total 7 bolts for the connections on the beam with the joint. For bigger beams, the number of bolts in the levels and webs can increase. Important for the design of this model is the loading capacity of the bolts from the flange connection that experience shearing. Considering their loading and the loading of the beam connection, the mechanism of collapse can be predicted.

**Figure 7.** The stress distribution in MODEL-SP3

In this model the maximal stress, appears in the horizontal angular welds of the joint, but these, same as the elements of the joint, are designed with bigger static sizes at the end of the beam, so that they are not a danger for connection collapse. Relatively smaller are the stresses, but still big enough in the place of the connection in the levels of the beam, with the joint, same as the connection between the web and the vertical web in the joint. The forces from the bending moment in the end of the beam are expected to cause stretching in the tensioned level and an eventual buckling in the pressed level, if appeasement does not occur in the leveled screws that are exposed to cutting (shearing). The further development can go into a direction of defining the exact correlations of the geometrical and static characteristics at the end of the beam, the number and the diameter of the means of connection (bolts) and the dimensions of the joint, everything in manner of getting a controlled collapse mechanism of the connection [1].

**MODEL-SP4** (Fig. 8) is a designed rigid beam-column connection. The connection of the beam (IPE200) with the column is made only by the levels of the beam, through previously made joints, one for each one of the levels of the beam. The joints can be made from hot rolled or welded L-profiles depending on the needed thickness, with welded triangular web in the middle of its strengthening. The connection of the joint with the level of the beam is made with 4 bolts, or total of 8 bolts on the two of the levels of the beam. The joints are designed so that their loading capacity is bigger than the loading at the end of the beam and the eventual collapse would appear in the assets of the connection. The connection in each of the joints with the column is made with 4 high valued bolts class 10.9, strained at the desirable moment. These bolts are designed so that they must not release during the most extreme loadings in the connections.

**Figure 8.** The stress distribution in MODEL-SP4

In the MODEL SP4 the maximal stress appears in the vertical fillet welds in the connection between upper angular stiffener and the plate. Because the joint (same as in the SP3 model and the rest of the models where a joint is used) is designed with big static sizes in the end of the beam, the maximal stress doesn’t represent a danger for collapse of the connection. The forces from the bending moment
at the end of the beam are expected to cause tension in the upper set of bolts or eventual buckling of the pressed parts of the beam. In this model, different from the previous SP3 model, this influence is emphasized as a result to the connection that was made only through the levels of the beam, and not through the web that was the case in the SP3 model [1].

**MODEL-SP5** (Fig.9) is a designed rigid beam-column connection. The connection between the beam (200.100.5) and the column is realized through 2 already prepared and welded horizontal plates to the end of the upper and the lower part of the beam that are additionally strengthened with 2 vertical webs. The dimensions of the joints are same as the ones in the SP4 model, however in this model are used exterior (the furthest) holes on the joints for the connection with horizontal plates at the end of the beams; the joints can be made from hot rolled or welded L-profiles depending on the needed thickness, with a welded triangular web in the middle of its strengthening. The connection between the joint and the plates that are at the end of the beam is made with 4 bolts, or total of 8 bolts for one beam. The joints are designed so that their loading is bigger than the loading of the beam and the eventual collapse would happen in the assets for the connection between the joints and the beams after some plastic deformations at the end of the beams. The connection between every joint and column is made with 4 high valued bolts class 10.9, strained at the desirable moment. These bolts are designed so that they mustn’t release during the most extreme loadings in the connections i.e. the collapse of the connection would take place at the end of the beam, in the joint.

**MODEL-SP6** (Fig.10) is a designed semi-rigid beam-column connection. The connection between the beam and the column is made only through the web in the beam (3 bolts) and the final welded joint. The welded T-joint is connected to the column with total of 4 high valued bolts 10.9. The joint is made by welding the 2 plates under an angle of $90^\circ$, and connecting them with bolts to the web of the beam and the web of the column. The design of the connection implies that under extreme loading the plastic hinge will appear at the end of the beam. On the beam IPE200 are made 3 holes on the web for the T-joint. For bigger beams, the number of bolts on the web can increase.

**MODEL-SP7** (Fig.11) is a designed semi-rigid beam-column connection. The connection between the beam and the column is made through the temporary end of the beam and the final welded T-joint with the help of total 3 bolts. The welded T-joint is dismantling connected to the column with a total of 4 high valued bolts class 10.9. The joint is made by welding the 2 plates under an angle of $90^\circ$. 

The forces from the bending moment at the end of the beam are expected to cause shear of the bolts in the tightened level and an eventual buckling of the pressed beam web. In this model this influence is emphasized as a result to the connection that was made only through the levels of the beam, and not through the web that was the case in the SP3 model [1].
where previously were made holes for the connection in the web of the beam and the connection with the column. The design of the joint is made so that its loading is big enough that the eventual collapse would appear at the end of the beam i.e. the beam-column connection. On the beam (isten 200) a steel plate is welded with transverse web on which are made 3 holes for the connection with the T-joint. For bigger beams, the number of bolts on the web can increase.

This design of the connection would be an answer to the previous model (SP6) in case where for the beams a rectangular hollow section would be used instead of an open I-section. This is a quashing connection because it takes a specific bending moment followed by movements. The eventual collapse would appear first in some of the ending bolts that are the most loaded and where a superimposition of the cutting forces emerges from the transversal shearing force and the forces from the receiving bending moment.

**Figure 11. The stress distribution in MODEL-SP7**

In the model, the maximal stress appears in the web of the beam, next to the holes for the screws and accordingly to the vertical welded plate of the joint. Also, big stress appears in the welds from the joint same as at the end of the beam and its upper and lower part next to the welded plate. The collapse of the connection is expected in the part of the beam as a result of the stretching and cutting of the vertical part of the plate next to the hole in the upper screw or as a result to the cutting of one of the external (upper or lower) bolts [1].

**MODEL-SP8** (Fig.12) is a designed semi-rigid beam-column connection. The connection between the beam and the column is realized with 3 long bolts between the end of the beam and the final welded U-joint. The welded U-joint is dismantling connected to the column with the total of 4 high valued bolts class 10.9. The joint is made by welding the 3 plates under an angle of 90\(^\circ\), where previously were made holes for the connection with the web of the beam and the connection with the column. The design of the joint is made so that its loading is big enough that the eventual collapse would appear in the end of the beam i.e. the beam-column connection. Three holes are made on the beam (isten 200) for the connection with the U-joint. For bigger beams, the number of bolts on the web can increase.

**Figure 12. The stress distribution in MODEL-SP8**

In the model the maximal stress is in the welds from the U-joint for the connection between the beam and the column (reactivity wall), but the same are designed so that a danger of collapse does not exist. Also, the bigger stress appears in the holes for the longer bolts, same as in the bolts themselves. Because the joint is designed with bigger static characteristics in the end of the beam, the collapse of the connection is expected in the part of the beam as a result of the cutting of vertical part of the beam next to the hole in the upper bolt or as a result of the cutting of one of the external bolt [1].

4. **Experimental research**

The experimental research of the 8 designed models is completely done in the Laboratory for Structures in the Institute for Welding and Welded Structures at the Faculty of Mechanical Engineering in Skopje. As a final result from the experimental research are the characteristic diagrams: F-\(\Delta L\) (force-displacement) and M-\(\Phi\) (bending moment-rotation in the end of the beam) (Fig.21) [1].

The equipment for adding and transferring the force contains one axial press (piston) that is placed vertically on the steel frame in the experimental desk and the same through the force sensor presses the end of the consol omission, causing negative bending moment and shearing in the connection at the end of the beam (Fig.14).
For every model (SP1, SP2, SP3, SP4, SP5, SP6, SP7 and SP8), a loading diagram was previously made, through adding the displacements. The process was made by adding the displacements, and not adding force, because in nonlinear conditions, when it comes to reducing the force, leading an experiment by adding force is practically uncontrolled and it may come to sudden collapse of the model. The loading of the models is applied by controlled displacement at the free end of the console. The research is presented in the following images for each of the models in some of the research phases:

Figure 13. Model SP1: Appearance of the plastic hinge (stability problem of lateral buckling).

Figure 14. Model SP2: Appearance of plastic hinge (stability problem of buckling in the lower pressed part of the beam).

Figure 15. Model SP3: Connection collapsing due to cutting of the second row bolt in the connection of the upper part of the beam.

Figure 16. Model SP4: Appearance of plastic hinge (stability problem of lateral buckling of the lower pressed part of the beam).
Figure 17. Model SP5 Appearance of the plastic hinge (stability problem of buckling in the lower pressed part of the beam).

Figure 18. Model SP6: Connection collapsing due to cutting of lower bolt in the connection of the rib of the beam.

Figure 19. Model SP7: Connection collapsing due to cutting of the upper bolt of the connecting ribs.

Figure 20. Model SP8/ Collapse of the connection due to plastic deformations of cutting at the end of the beam in the upper long bolt.
5. Conclusion

During design of connections it is important to control the weight, stiffness, strength and the ductility of the material of the elements. This is due the fact that their behavior depends on the mentioned parameters of the elements in the connections [2].

From the comparative experimental analysis of all of the 8 researched models it can be concluded that the capacity of the carrying of the bending moment of the models with rigid connection (SP1, SP2, SP3, SP4 and SP5) is significantly bigger than the one with the semi rigid connections (SP6, SP7 and SP8), what was already expected; if the rigid connections are separately analyzed (SP1, SP2, SP3, SP4 and SP5), the model SP1 has the biggest capacity to carry the banding moment over the others, but on the other hand the smallest rotating capability (plastic deformability) over the other 4 i.e. the 7 models counting the models with the semi rigid connections (SP6, SP7 and SP8); the model SP3 has the biggest rotating capability over the other 4 models with the rigid connections (SP1, SP2, SP4 and SP5); the deformation (rotation) capability in the researched models is inversely proportional to the capacity of carrying of the beam i.e. the semi rigid connections have bigger plastic deformability and possibility for bigger rotation during the use of their total capacity to carry, but they have smaller capacity to carry comparing to the ones with rigid connection.

The main idea and direction for further researches is obtaining one "ideal" beam-column connection that at the same time will have relatively high strength and deformational characteristics i.e. capacity to carry bigger banding moments with bigger rotating capability in the same time.

References

[8] EUROCODE 1
[10] ANSYS Workbench

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