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Numerical Modelling Using ABAQUS Finite Element Software for Two Types of Beam-to-Column Connections and Reduced Cross-Section Connections to Reduce Progressive Damage

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Abstract

If major structural members fail due to accidents, terrorist attacks or similar events, and other members are unable to come to the aid of the failed members, this type of failure will propagate to adjacent members, eventually causing a partial or complete collapse of the structure. The collapse of structures such as the World Trade Centre clearly demonstrates the importance of designing structures to resist abnormal loads. While there has been extensive research in the past on the seismically resistant beam-to-column connections of steel frames, recent studies have focused on the behaviour of these connections against progressive damage. Although extensive research has been carried out on beam-column flexural connections against earthquakes, challenges to the performance of this type of connection against ground shaking remain unresolved. This study, using the numerical finite element method, evaluates the performance of beam-to-column connections in progressive damage and, to improve the performance of these connections, the behaviour of reinforced connections in progressive damage (evaluation of the performance of simple beam-to-column connections, evaluation of the performance of reduced-section connections, evaluation of the performance of simple reinforced beam-to-column connections and evaluation of the performance of reduced-section reinforced beam-to-column connections). As this research is based on numerical and applied modelling, its statistical population is based on data and information from reputable scientific articles and journals. After modelling and outputting the results from the ABAQUS software, the analysis results were entered into Excel, and the graphs were compared and evaluated.

Keywords: Beam-to-column connections, Finite element software, Reduced cross-section, Progressive damage.

1 | Introduction

highlighted the importance of progressive failure design [1]. Although there has been extensive research into the seismic behaviour of beam-column flexural connections in steel frames, there has been little research into

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the performance of these types of connections against progressive failure. However, there are significant differences between the forces applied in these two scenarios. In seismic events, forces are applied to the connections cyclically, resulting in cyclic behaviour, whereas in progressive failure, the applied forces are mostly constant and increase continuously.

Various design methods have been proposed to deal with progressive deterioration, which can be broadly categorised into two approaches: direct and indirect. Direct design methods include Alternative Paths (AP) and the determination of local member resistance and directly examine the behaviour of the structure against the removal of one or more key members. In contrast, indirect design methods increase the overall resistance of the structure to this phenomenon through the use of integrated systems and optimal arrangement of structural components. The AP method is one of the most widely used methods in design codes for assessing the resistance of a structure to progressive damage [2]. In this method, it is assumed that a structural column is removed under a severe abnormal load, and then the ability of adjacent elements to transfer the loads and prevent collapse is examined. In this situation, connections play a very critical role in the transfer of forces and must be able to withstand significant tensile forces and bending moments [2].

Conventional connections in steel buildings are primarily designed to transfer bending forces and are not capable of withstanding large tensile forces resulting from large displacements. The ability of the connections to transfer these forces from one beam to another is a key factor in determining the resistance of the structure to progressive failure. In the single-column removal scenario, the frame beams will undergo significant displacements and behave more like cables in tension than conventional beams. In this case, the connections must be able to transfer the tensile forces resulting from these displacements to prevent progressive failure.

In view of the above, this study investigates the performance of connections in steel frames in a progressive damage scenario [4]. In order to analyse this behaviour in more detail, numerical modelling based on the finite element method is used, and the effect of strengthening connections on increasing the resistance of the structure to progressive damage is evaluated [1].

2 | Research methodology

2.1 | Validation of Numerical Models

The frame considered for validation consists of 2 spans with a distance of 9 metres and 6 columns with a height of 3 metres. As shown in *Fig. 1*, the removal of the central column of the lower floor causes failure. By removing the centre column, the geometry of the frame changes to a span length of 18 metres and the rest of the structure transfers and supports the loads. In this case, the new load is much greater than the initial load, and the beam undergoes plastic deformation. For symmetry reasons, half of the frame is modelled. The beam is of type 457x191UB98, and the columns are of type 305x305UC283. The shell and intermediate plates are used to model the geometry of the beams and columns. A shear plate with a length of 5 cm and a depth of 36 cm is considered for the beam-column connection. Two concentrated loads of 90 kN are considered at a distance of one-third of the span.

For both the retrofitted and original structures, the load has been changed to 135 kN to take account of the dynamic effect with a magnification factor of 1.5. The width is assumed to be equal to the beam flange, and the length and thickness of the cover plates are assumed to be 50 cm and 2.5 cm, respectively. The width and thickness of the column stiffener are assumed to be the same as those used in the retrofit of the beam end flange [5].

Fig. 2 and *Table 1* show the stress-strain curve and its value. The geometric characteristics of the beam and column sections are shown in *Fig. 3*.

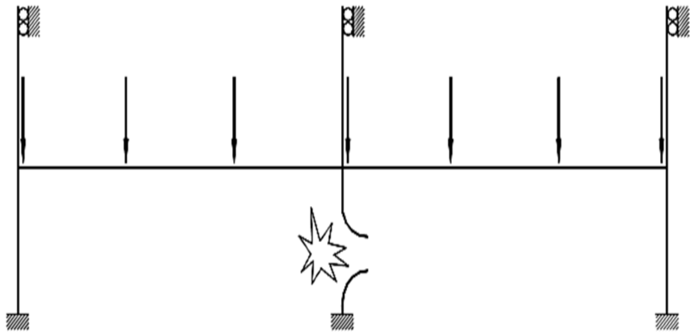


Fig. 1. The frame considered in this analysis.

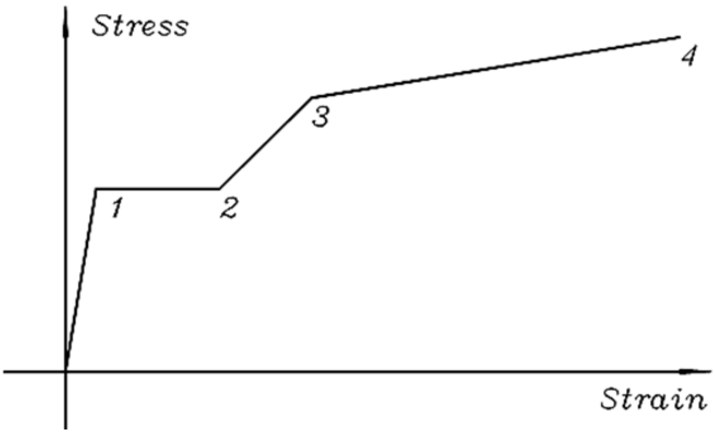


Fig. 2. Stress-strain curve.

Table 1. Values considered in the stress-strain curve in Fig. 2.

Point	1	2	3	4
Stress (MPa)	235	235	360	490.8
Strain	0.001175	0.01328	0.03653	0.4

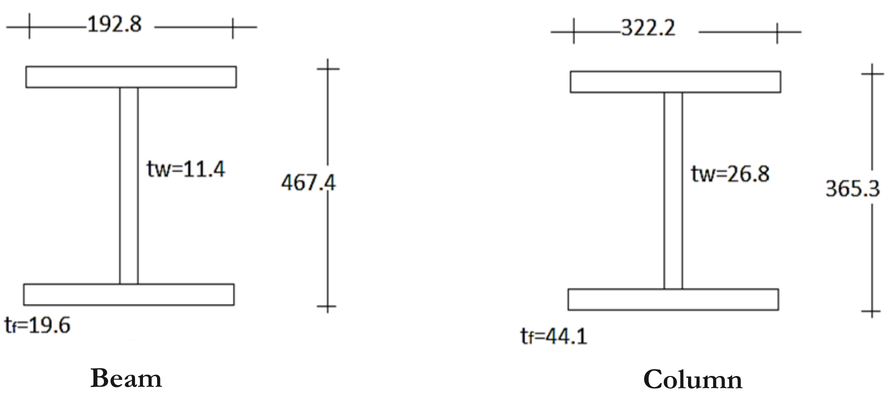


Fig. 3. Beam and column cross-sections in millimeters.

In the original structure, the beam-column connection is assumed to be an articulated connection for the purpose of comparing the original and retrofitted structures because the bending stiffness in the simple connection is assumed to be less than that of the beam and column. Therefore, the beam-column connection is changed from a simple connection to a rigid connection for the purpose of retrofitting. However, it is not possible for a structure to replace itself in Abaqus software.

To solve this problem, an alternative method is presented to model the modified structure by changing only the boundary conditions at different stages. The modelling process is carried out in three stages. The first stage is the zero bending moment simulation of the beam-column connection, which takes into account the loads and boundary conditions specified for the structure, which are two concentrated loads of 90 kN at one-third of the span. For simplicity, only the horizontal displacements are assumed to be constrained, and the two ends of the beam are allowed to rotate freely because the stiffness of the concrete slab is small in the out-of-plane direction and very high in the in-plane direction. The ends of the column are clamped at both the top and the bottom and specific values of the boundary conditions are given to the ends of the column. If the displacements of the column ends are equal to these specific values, there will be no bending moment in the columns. The rotations at the top and bottom of both ends of the column will be equal to the ends of the beams. The absolute value of the rotation at the end of the column is calculated as follows:

$$\theta = \frac{PL^2}{9EI} = \frac{90\times10^3\times9^2}{9\times2\times10^{11}\times4.752\times10^{-4}} = 0.008523. \quad \textbf{(1)}$$

$$d = \theta h = 0.008523\times3 = 0.02557\text{m}. \quad \textbf{(2)}$$

In the second step, the two concentrated loads are changed from 90 kN to 135 kN. In the third step, displacement is applied. The boundary conditions in this step are the same as in the first step, but the vertical displacement values of 1 m and 1.1 m are applied to the right end of the two considered structures. *Fig. 4* shows the example considered for modelling. The results in *Fig. 5* show a comparison of the two considered specimens and the numerical model in the case of a frame without reinforcement.

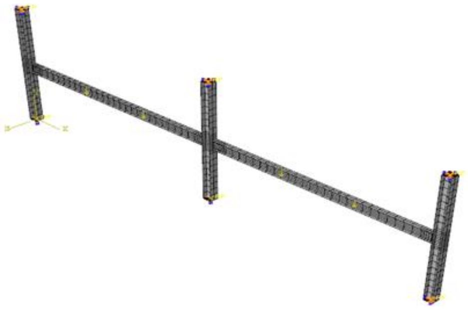


Fig. 4. The frame considered for modeling.

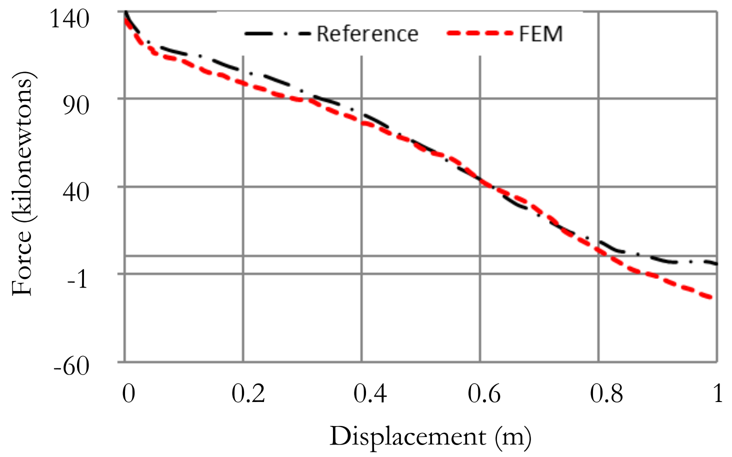


Fig. 5. Comparison showing the force-vertical displacement of the initial frame.

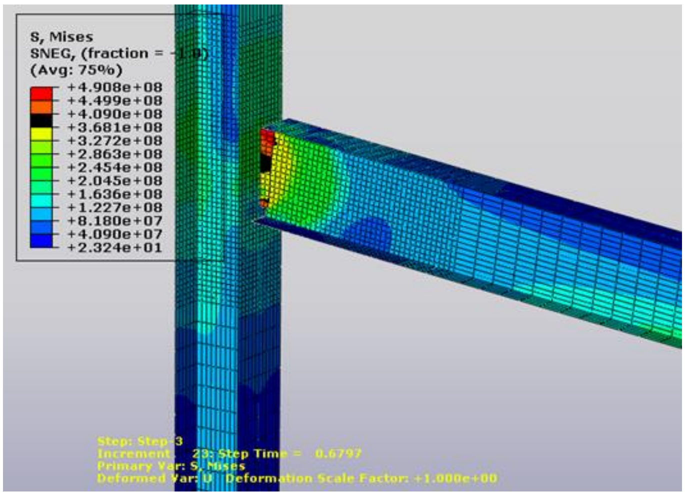


Fig. 6. Stress gauge of the unreinforced model in Abaqus software.

Fig. 6 shows the stress gauge of the unreinforced model. From the figure, it can be seen that when the right displacement of the beam reaches 6.797 cm, the von Mises stress will reach 8.490 MPa, which is the ultimate stress of the materials used. According to *Fig. 7* in the article, the displacement to reach this stress limit is 0.6748 m, which indicates that the simulation using Abaqus software is correct and has an acceptable agreement with the case considered in the article.

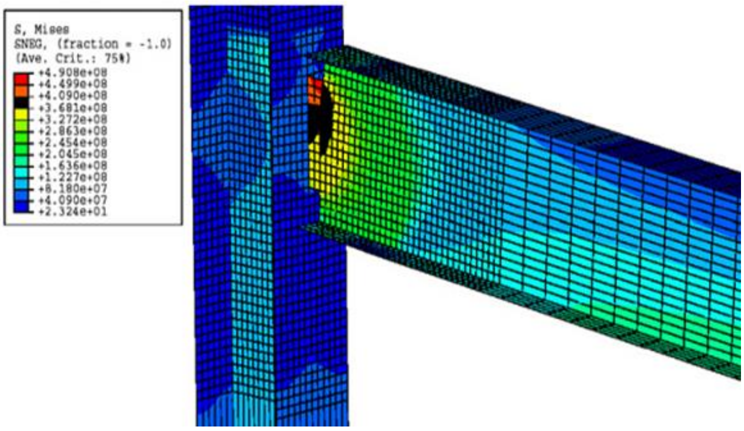


Fig. 7. Stress contour for the unreinforced model in the article at a vertical displacement of 6.748cm.

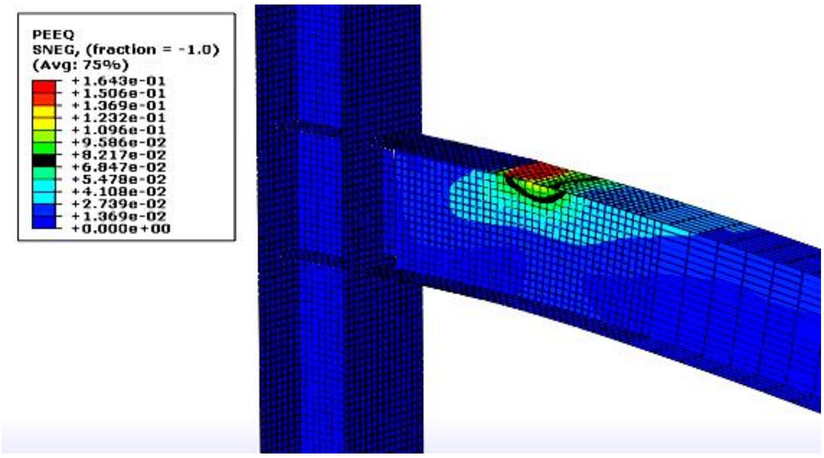


Fig. 8. Shows the plastic strain gauge of the reinforced model when the displacement on the right side of the beam is 1.1 m.

Considering that the plastic strain value is 1-10×643.1, which is slightly different from the value of 1-10×651.1 in *Fig. 9*, which is the plastic strain considered in the article, it indicates the accuracy of the modelling.

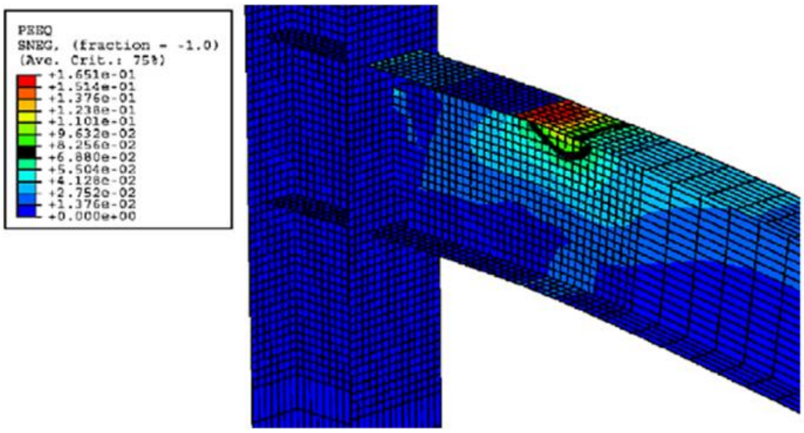


Fig. 9. Plastic strain contour of the reinforced model at a vertical displacement of 1.1 m considered in the paper.

2.2 | Numerical Model

In order to compare the results, all connections are of the same type of beam with the same characteristics. Columns The ratio of the moment of inertia of the column to the length of the column to the ratio of the moment of inertia of the beam to the length of the beam $((Ic/Lc)/(Ib/Lb))$ is assumed to be 0.5, 1 and 2 in three cases: very weak, weak and strong, respectively.

Each of these three cases was simulated in two normal cases and also with a weakened section in the flange beam. In addition, in order to evaluate the effect of strengthening the columns in preventing structural failure, the columns are continued with the help of a plate with the characteristics of a continuous plate to be strengthened above the connection and all around the column. In all three cases, the vertical connection of the beam to the column was considered to be rigid by groove welding the flanges to the column and connecting the beam web to the column using a 10×70×200 mm plate and a bolted connection of the connecting plate to the beam web and a welded connection of the connecting plate to the column flange.

A simple connection in the case of a very weak column and considering a stiffness ratio of the column to beam of 0.5: In this case, the column characteristics are two 300×10 mm plates for the two wings and one 150×8 mm plate for the column web.

A simple connection in the case of a weak column and considering a stiffness ratio of the column to beam of 1: in this case, the column characteristics are considered to be two 300×10 mm plates for the two wings and one 210×8 mm plate for the column web.

A simple connection in the case of a strong column and considering a column-to-beam stiffness ratio of 2: in this case, the column characteristics are considered to be two 300×10 mm plates for the two wings and one 300×8 mm plate for the column web.

In order to strengthen the frames, the columns are reinforced. Also, a comparison of the connections in two conditions of weak and strong columns, as well as with the help of a scarf plate and a header to strengthen the connections, was made and compared in the two conditions. *Fig. 10* The frame is specified considering the boundary conditions, meshing and very weak column.

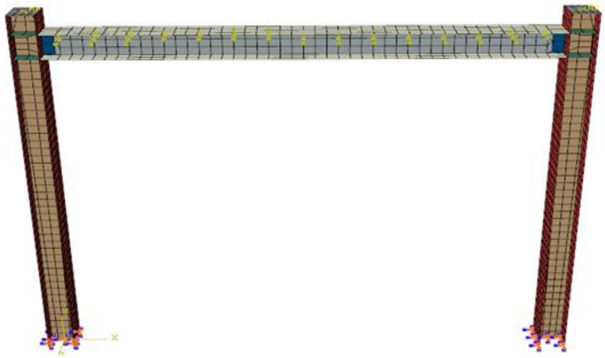


Fig. 10. Flexural frame with simple beam and very weak column.

Fig. 11 clearly shows that the plastic hinge has occurred at the link spring. The load-displacement curve is also shown. Also, in all the frames, the beam section was made of two 300×10 mm plates for the two wings and one 210×8 mm plate for the web. The free span (the internal distance between the two columns) is three metres, and the net length of the beam is three metres. The material and loading specifications are the same as in the validation case. Two design loads were applied to the beam at a distance of one-third of the span, each equal to 105,000 N. Only half of the frame was modelled and analysed due to symmetry. This model is half of a beam with a length of one and a half metres and a full column with a height of three metres. The load was applied in three stages:

I. A concentrated load of 105,000 N was applied at a distance of half a metre from the free edge of the beam.

II. The beam support was changed from a fully clamped to a roller-clamped condition.

III. A concentrated load of 157500 N is applied at a distance of half a metre from the free edge of the beam.

A stiffening plate is used to prevent local crushing of the beam flange at the point of load application, as shown in *Fig. 11*.

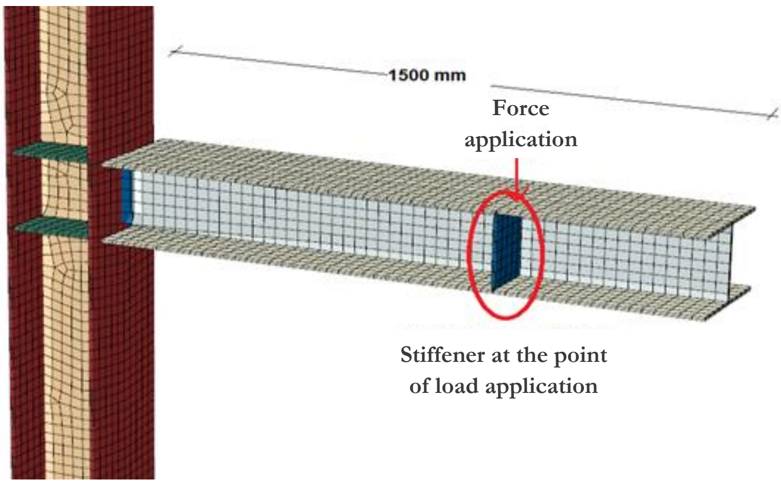
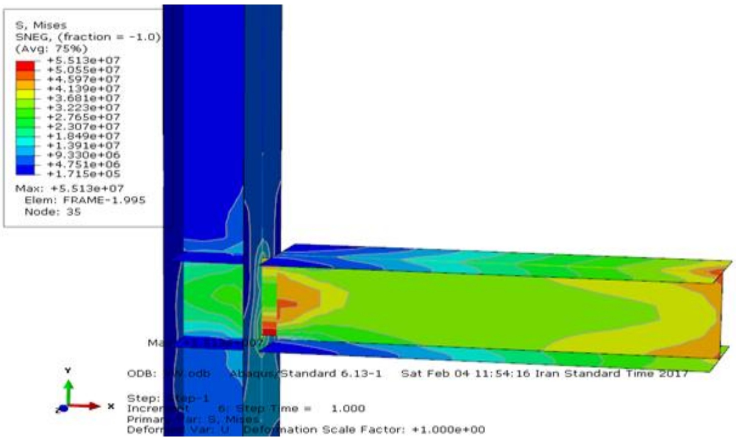


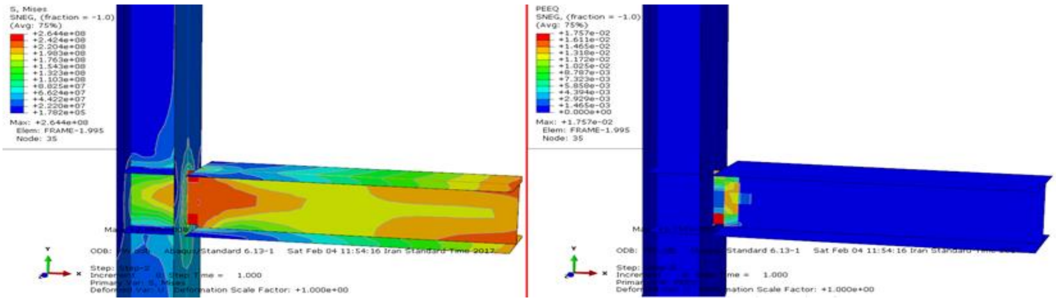
Fig. 11. Location of stiffeners in the beam web and force input.

2.3 | Examples of no reinforcement

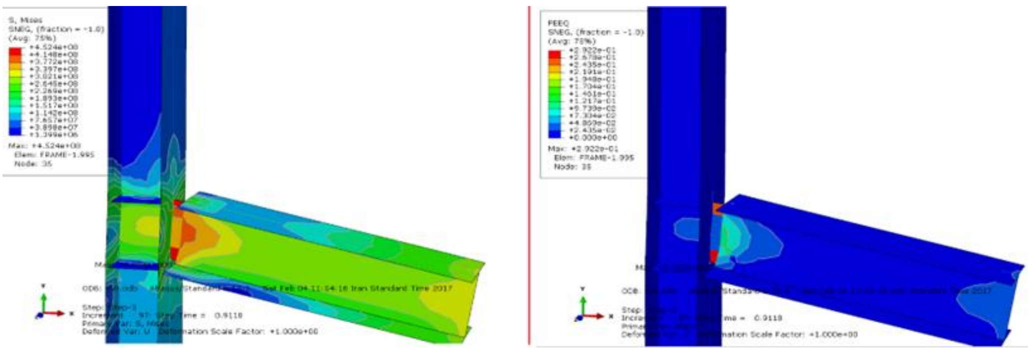
In this section, a very weak column is loaded with a conventional beam. The stress distribution at the end of the first stage of loading shows that the maximum stress developed in the connection is approximately 55 MPa, which is less than the yield stress of the steel, and no buckling is observed in the members. At the end of the second stage, when the gravity load is fixed, it is seen that the connection cannot remain in the linear stage beyond the yield point, as shown in the stress and strain distribution. The strain developed is 0.0176, where the steel material of the coupling enters the post-hardening stage. The steel strain at the end of the steel hardening stage, according to *Table 1,* is 0.01328 [6]. The maximum stress developed under load at this stage is 264 MPa, which is slightly higher than the yield stress of the steel. In the third stage, the dynamic effect of the load increase is applied at a factor of 1.5 times the gravity load, which indicates that the structure could not be analysed to the end during the analysis, which could be due to the high divergence problems that occurred during the problem-solving. However, it is predicted that at the end of this stage, the steel will reach its ultimate stress and experience a strain greater than the ultimate strain. At the end of this stage, the analysis was stopped, and the result was that it reached a plastic strain of approximately 0.3. As a result, it can be concluded that the sample evaluated cannot be used against the progressive failure phenomenon. In this case, at the end of the third stage, the maximum displacement of the beam end is 26 cm.



a.



b.



c.

Fig. 12. Flexural frame with weak column and simple beam; stress and strain distribution through three stages: a. first stage, b. second stage, c. third stage.

The stress distribution at the completion of the first stage loading is shown for the weak column and simple beam conditions [7]. The maximum stress developed in the connection is approximately 77 MPa, which is less than the yield stress of the steel and, in addition, no buckling effect is seen in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield stress, as indicated by the stress and strain distribution. The strain developed is

0.088, where the steel material of the coupling enters the post-hardening stage. The steel strain at the end of the steel hardening stage, according to Table (1), is 0.01328. The maximum stress obtained under load at this stage is 264 MPa, which is slightly higher than the yield stress of the steel. In the third stage, the dynamic effect of load increase is applied at a factor of 1.5 times the gravity load, which shows that the plastic strain reaches approximately 0.43, which is 7.5% more than the ultimate plastic strain of the steel, which can be concluded that the section has failed and is damaged. By considering the progressive failure mode force and applying boundary conditions, it can be concluded that the frame has failed and the connecting spring has reached its ultimate stress. Therefore, the weak column in this frame cannot prevent progressive failure. In this case, the maximum displacement of the beam end at the end of the third stage is 5.33 cm.

The bending frame with weak column and simple beam and the stress and strain distribution at a) completion of the first stage, b) completion of the second stage, and c) completion of the third stage shows the stress distribution at the completion of the first stage loading for the case of strong column and simple beam. The maximum stress generated in the connection is approximately 79 MPa, which is lower than the yield stress of the steel and, in addition, no buckling effect is seen in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point, which is shown in the stress and strain distribution. The strain generated is 0.085, where the steel material of the coupling enters the post-hardening stage. The steel strain at the end of the steel hardening stage, according to *Table 1,* is 0.01328. In the third stage, when the dynamic effect of increasing load is applied at a factor of 1.5 times the gravity load, which shows It reaches a plastic strain of approximately 0.42, which is 0.42 percent higher than the ultimate plastic strain of steel, which is 5 percent higher than the ultimate plastic strain of steel. It can be concluded that the section will fail, and the connecting spring will reach the ultimate stress of its material in this case.

Therefore, the strong column cannot prevent progressive failure in this case. In this case, the maximum displacement of the beam end at the end of the third stage is 32 cm.

Flexural frame with strong column and single beam and stress and strain distribution at: a) end of first stage b) end of second stage c) end of third stage *Fig. 13* shows the force-displacement diagram according to the paper used to validate the numerical models when a displacement of one metre is produced at the end of the beam.

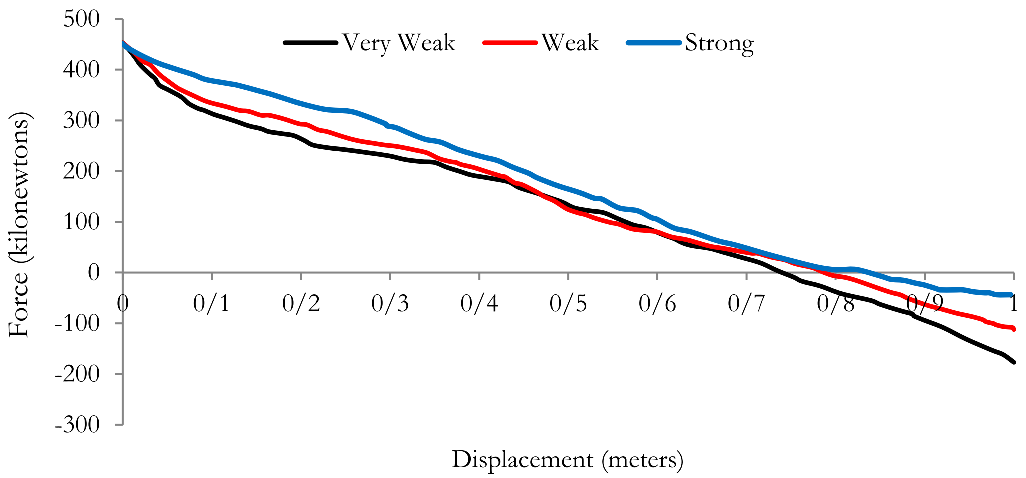


Fig. 13. Force-versus-displacement diagram.

2.4 | Cases with Head and Scarf Plate Reinforcement

Based on the previous results in the simple connection condition, a plastic hinge is formed at the beam-column connection. Therefore, this area needs to be reinforced. Therefore, we evaluate the performance of the connection reinforced with scarf and header plates. The stress distribution at the end of the first stage of loading is investigated for a very weak column reinforced with header and scarf plates and a simple beam.

The maximum stress developed in the connection is approximately 96 MPa, which is less than the yield stress of the steel, and there is no evidence of buckling in the members. At the end of the second stage, when the gravity load was fixed, the stress and strain distribution showed that the connection could not remain in the linear stage beyond the yield point. The strain developed is 0.025, where the steel material of the connection enters the post-hardening stage.

The steel strain at the end of the steel hardening stage, according to *Table 1,* is 0.01328. In the third stage, when the dynamic effect of load increase is applied at a factor of 1.5 times the gravity load, the plastic strain reaches approximately 0.365, which is 8% higher than the final plastic strain of the steel. This increase indicates that the section has failed, and in this case, the connection spring has reached its ultimate material stress. Therefore, a very weak column, even with the reinforcement of the beam flanges with headers and scarf plates, cannot prevent progressive failure in this frame. In this case, the maximum displacement of the beam end at the end of the third stage is 3.23 cm. Therefore, it can be concluded that with these reinforcements, the failure and yielding have been transferred from the beam-column connection plate to the connection spring between the flange and the column stiffening plates.

The comparison results show that strengthening with head and scarf plates is not beneficial in the conditions of very weak and weak columns. This strengthening method is effective for strong column conditions and can prevent failure and ultimate yielding due to the progressive failure phenomenon. The rotational capacity of the connection is increased, which, in the examples discussed, leads to a reduction in the vertical displacement of the beam end caused by changing from the simple to the reinforced state with headers and scarf plates.

The connection spring moment for the six cases evaluated in this chapter is given in *Table 2*. Regarding the naming of the cases, it should be noted that VW indicates the case with a very weak column, W indicates a weak column, and the letter S indicates a strong column. The letter F after these letters indicates the reinforcement plate under the header and the scarf located on the beam flange.

Table 2. Rotational capacity of two different connection modes.

	VW	W	S	VW-F	W-F	S-F
Rotation (degrees)	12	11	8	9	8	5

2.5 | Reinforced Column Cases with Global Plates

In order to increase the bending capacity of the system and to transfer the plastic zone from the connection string to points further away from the spring, the column was reinforced with global plates of 1 cm thickness.

The stress distribution at the end of the first stage shows the load for a very weak column reinforced with global plates and a simple beam. The maximum stress generated in the connection is approximately 67 MPa, which is less than the yield stress of the steel, and there is no evidence of buckling in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point, as indicated by the stress and strain distribution. The strain generated is 0.084, where the steel material of the coupling enters the post-hardening stage. The steel strain at the end of the steel hardening stage, according to *Table 1,* is 0.01328.

In the third stage, the dynamic effect of load increase is applied at a factor of 1.5 times the gravity load, which shows that the plastic strain reaches approximately 0.412, which is 3% higher than the ultimate plastic strain of steel, which indicates that the section has failed and is damaged. Considering the application of boundary conditions and the progressive failure force, it can be concluded that the frame in question has failed, and the connecting spring has reached its ultimate material stress. In this case, the maximum displacement of the beam end at the end of the third stage is 31.7 cm.

The stress and strain distribution of a very weak column reinforced with universal plates and a simple beam at a) the end of the first stage, b) the end of the second stage, c) the end of the third stage shows the stress distribution at the end of the first stage of loading for a weak column reinforced with universal plates and a simple beam. The maximum stress developed in the connection is approximately 76 MPa, which is less than the yield stress of the steel, and there is no evidence of buckling in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point, as indicated by the stress and strain distribution. The strain developed is 0.085, where the steel material of the coupling enters the post-hardening stage. The steel strain at the end of the steel hardening stage, according to *Table 1,* is 0.01328. In the third stage, when the dynamic effect of increasing load is applied at a factor of 1.5 times the gravity load, which shows that it reaches a plastic strain of approximately 0.415, which is 3.5% higher than the ultimate plastic strain of steel, the maximum stress generated is 418 MPa, from which it can be concluded that the section has failed and failed. Considering the application of boundary conditions and the progressive failure force, it can be concluded that the frame in question failed to prevent progressive failure. In this case, the maximum displacement of the beam ends at the end of the third stage is 31.6 cm, which is only a 6% reduction in displacement compared to the very weak column with full-column reinforcement.

A bending frame with a weak column reinforced with universal plates and a simple beam, stress and strain distribution at a) the end of the first stage, b) the end of the second stage, c) the end of the third stage shows the stress distribution at the end of the first stage of loading for a strong column reinforced with universal plates and a simple beam. The maximum stress developed in the connection is approximately 84 MPa, which is less than the yield stress of the steel, and there is no evidence of buckling in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point, as indicated by the stress and strain distribution. The strain developed is 0.0853, where the steel material of the connection enters the post-hardening stage.

The steel strain, according to *Table 1,* at the end of the steel hardening stage is equal to 0.01328. In the third stage, when the dynamic effect of load increase is applied at a factor of 1.5 times the gravity load, which shows that the plastic strain is approximately equal to 0.416, which is less than the final plastic strain of the steel, the maximum stress generated is equal to 490.8 MPa, from which it can be concluded that the section has not failed or has failed. Considering the application of boundary conditions and the progressive failure force, it can be concluded that the frame in question failed to prevent progressive failure. In this case, the maximum displacement of the beam ends at the end of the third stage is 31.6 cm, which is only a 6% reduction in displacement compared to a very weak column with full-column reinforcement.

One of the features of static analysis in Abaqus software is that the existing stress becomes greater than the ultimate stress when the specimen is loaded beyond its strength, but it only specifies the maximum stress defined by the user; the strains are not limited to the program input values. In other words, it is possible to specify strains greater than the defined input values. Considering the boundary conditions and the progressive failure force, it can be concluded that a strong column with total reinforcement of the column flange and web by means of 10 mm thick plates cannot prevent progressive failure. In this case, the maximum displacement of the beam end at the end of the third stage is 31.6 cm.

A strong column frame reinforced with universal plates and a simple beam and stress and strain distribution at a) the end of the first stage, b) the end of the second stage, and c) the end of the third stage. The important point is that the strengthening by the universal strengthening method of the column has very little effect on the flexural capacity in this case. By looking at the comparison between the universal strengthening method of the column and the joint strengthening method, the effect of joint strengthening can be understood. By evaluating the flexural capacity of the connection spring by 73, 55 and 141 percent for the strong, weak and very weak columns in the joint strengthening method with the scarf and header plates, respectively, it increases. However, the flexural capacity in the connection spring decreases by 55 percent in the very weak column and increases by 1.5 and 5 percent in the strong and weak columns, respectively.

Therefore, joint strengthening is much more effective than the universal strengthening of the column. The rotational capacity of the connection spring for the three cases is presented in *Table 3*.

Table 3. The rotational capacity of simple and reinforced specimens connected with column cross-section plates.

	VW-C	W-C	S-C
Rotation (degrees)	11	10	8

2.6 | Reduced-Flange Beam Cases

In this section, we will evaluate the performance of reduced-flange beam connections. Given that the size, shape, and section of the RBS affect the connection efficiency, there are 3 types of sections for RBS connections that have been extensively tested by researchers, including:

I. With a constant cross-section.

II. With a trapezoidal cross-section.

III. With a circular cross-section.

a=Distance on the column to the start of the cut
b=Cut length
c=Cut depth

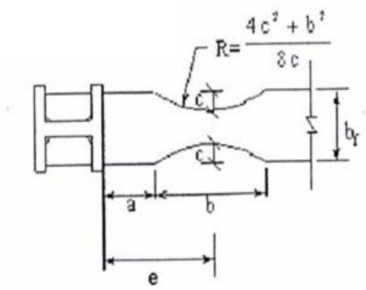


Fig. 14. Types of RBS cuts.

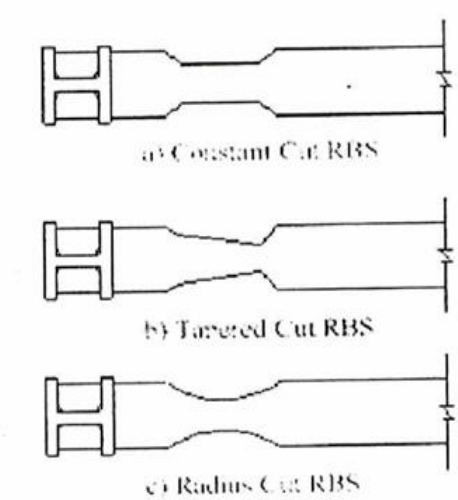


Fig. 15. Dimensions of circular cuts.

Various factors influence how to choose the parameters a and b, which are obtained through the relationship $e=a+b/2$.

To minimise the increase in a moment from the centre of the section to the column, both parameters should be considered small enough, and to make the stress flow uniform from the start of the section to the column, a should be considered large enough [8]. Also, to have a sufficient area for plastic behaviour, parameter b should be considered large enough. On the other hand, the larger the value of a, b, the further away from the column, the start of the reduction in the cross-sectional area will be, and the effect of limiting the moment on the connecting column will be reduced. Furthermore, if these parameters are considered to be small, this will result in a high strain concentration in the column or at the point of section reduction. Evaluation of the results of various tests shows that a and b have been chosen based on the values recommended by FEMA 350. In order to control the bending capacity of the RBS area, which is a function of the maximum moment on the column, so that the moment transferred to the column is between 85% and 100% of the full section plastic moment of the beam, the parameter C must be chosen to achieve this condition. Based on FEMA350 [7].

$$d = \theta h = 0.008523 \times 3 = 0.02557\text{m}. \tag{3}$$

$$0.65d \le b \le 0.85d. \tag{4}$$

$$c \le 0.25b_f. \tag{5}$$

where is b_{fb} the beam flange width and d_b the beam cross-section depth. The percentage reduction in the cross-sectional area of a beam flange is calculated as follows:

$$\frac{2c}{b_f} \times 100. \tag{6}$$

Based on the above relationships, the maximum allowable percentage reduction of the beam flange cross-section is 50%. Therefore, the values of a, b, and c are assumed to be 200 mm, 180 mm, and 60 mm, respectively. Now, we will evaluate these unreinforced cases.

The stress distribution at the end of the first stage loading is very weak for the RBS beam and the column. The maximum stress developed in the connection is about 7.57 MPa, which is lower than the yield stress of the steel, and in addition, there is no evidence of buckling in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point, which is indicated in the stress and strain distribution. The strain developed is 0.098, where the steel material of the coupling enters the post-hardening stage. The steel strain corresponding to the end of the steel hardening stage is 0.01328. In the third stage, when the dynamic effect of load increase is applied at a factor of 1.5 times the gravity load, it is shown that the plastic strain reaches approximately 0.376, which is 6% less than the final plastic strain of steel. However, due to divergence in the solution of the third stage problem, the third stage was not completed, and the data was plotted near the end of the third stage. In this case, it is estimated that the plastic strain is greater than the ultimate strain. Considering the boundary conditions and the progressive failure force, it can be concluded that the frame has failed and the link spring has reached its ultimate material stress, so it cannot prevent progressive failure. In this case, the maximum displacement of the beam end at the end of the third stage is 30.5 cm.

A flexural frame with a very weak column and RBS beam and the stress and strain distribution at a) completion of the first stage, b) completion of the second stage, and c) completion of the third stage. The stress distribution at the completion of the first stage loading for the case of weak column and RBS beam is shown. The maximum stress developed in the connection is approximately 70 MPa, which is less than the yield stress of the steel, and there is no evidence of buckling in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point.

It is given as stress-strain distribution. The strain generated is 0.0853, where the steel material of the coupling enters the post-hardening stage. The steel strain corresponding to the end of the steel hardening stage is 0.01328. In the third stage, where the dynamic effect of increasing the load is applied with a factor of 1.5 times the gravity load, it is shown that the plastic strain reaches approximately 0.412, which is 3.5% higher than the final plastic strain of the steel. Considering the boundary conditions and the progressive failure force, it can be concluded that the frame has failed and the connecting spring has reached the ultimate stress of its material, so the weak column cannot prevent the progressive failure. In this case, the maximum displacement of the beam ends at the end of the third stage is 3.33 cm, which does not change significantly with a very weak column with a beam with a reduced wing section.

The bending frame with weak column and RBS beam and the stress and strain distribution at a) completion of the first stage, b) completion of the second stage, and c) completion of the third stage shows the stress distribution at the completion of the first stage loading for the case of strong column and RBS beam. The maximum stress generated in the connection is approximately 6.81 MPa, which is lower than the yield stress of the steel, and in addition, no buckling effect is seen in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point, which is indicated by the stress and strain distribution. The strain generated is 0.0856, where the steel material of the coupling enters the post-hardening stage. The steel strain corresponding to the end of the steel hardening stage is 0.01328. In the third stage, when the dynamic effect of increasing load is applied at a factor of 1.5 times the gravity load, the plastic strain reaches approximately 0.417, which is less than the ultimate plastic strain of steel. At this stage, the maximum stress generated is greater than 490.8 MPa. In this case, the strains are larger than the defined input value. Considering the application of boundary conditions and the progressive failure force, it can be concluded that the section has not failed and has failed finally. Therefore, the strong column with an RBS beam cannot prevent progressive failure. In this case, the maximum displacement of the beam end at the end of the third stage is 32 cm.

Flexural frame with strong column and RBS beam and stress and strain distribution in, a) completion of the first stage, b) completion of the second stage, c) completion of the third stage. Rotational bending capacity of the connection for three states of the column with RBS beam, all three states showing the same behavior.

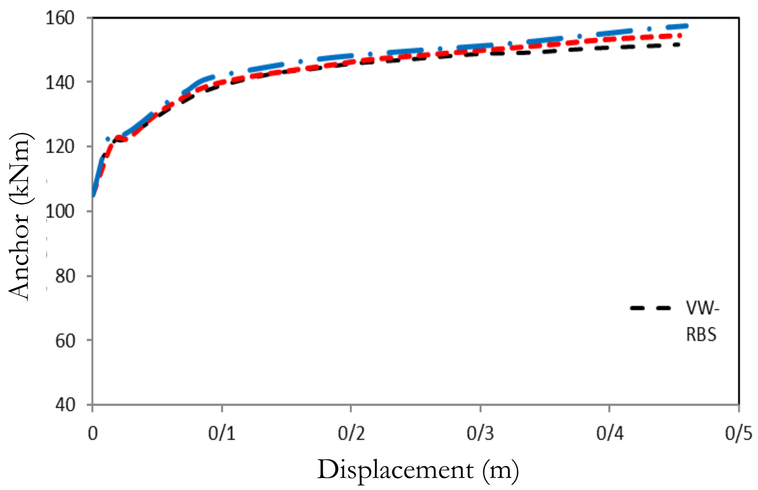


Fig. 16. Moment variation curve displacement at the spring joint location of RBS beam modes.

2.7 | Case with RBS Beam and Beam Flange Reinforced with Head and Flange Plates

The stress distribution at the end of the first stage of loading is characterized by the case of a very weak column reinforced with header and flange plates and an RBS beam. The maximum stress developed in the connection is about 125 MPa, which is lower than the yield stress of the steel, and its location is at the intersection of the compression edge of the beam and the column flange. In addition, no evidence of buckling is seen in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point, which is indicated by the stress and strain distribution. The strain developed is 0.447, where the steel material of the coupling enters the post-hardening stage. The steel strain corresponding to the end of the steel hardening stage is 0.01328. In the third stage, when the dynamic effect of load increase is applied at a factor of 1.5 times the gravity load, which shows

It shows that the plastic strain reaches about 0.6, which is much higher than the ultimate plastic strain of steel. Considering the boundary conditions and the progressive failure force, it can be concluded that the frame in question has failed, and the connecting spring is at the ultimate stress of the material. Therefore, even with the reinforcement of the beam, the very weak column cannot prevent progressive failure. In this case, the maximum displacement of the beam ends at the end of the third stage is 24.4 cm, which is 25% less than the case without reinforcement with the RBS beam. As can be seen at the end of the third stage, the plastic zone covers a large part of the entire connection string, and buckling occurs at the connection of the head and flange plates to the column.

A very weak column reinforced with header and flange plates and RBS beam and stress and strain distribution at a) completion of the first stage, b) completion of the second stage, and c) completion of the third stage. The stress distribution at the completion of the first stage loading for the weak column reinforced with header and flange plates and RBS beam is shown. The maximum stress generated in the connection is about 5.58 MPa, which is lower than the yield stress of the steel, and its location is at the intersection of the compression edge of the beam and the column flange. In addition, no effect of buckling in the members is seen. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point. The strain generated is 0.025, where the steel material of the coupling enters the post-hardening stage. In the third stage, when the dynamic effect of increasing load is applied at a factor of 1.5 times the gravity load, the plastic strain is about 0.268, which is much smaller than the ultimate plastic strain of steel. Considering the boundary conditions and the progressive failure force, it can be concluded that the frame has performed well, but the connecting spring has reached the yield point and needs to be strengthened. In this case, the maximum displacement of the beam end at the end of the third stage is 18.7 cm.

Flexural frame with weak column reinforced with head and flange plates and RBS beam and stress and strain distribution at: a) completion of the first stage, b) completion of the second stage, c) completion of the third stage. The stress distribution at the completion of the first stage loading for the strong column reinforced with head and flange plates and RBS beam is shown. The maximum stress generated in the connection is about 78 MPa, which is lower than the yield stress of the steel, and its location is at the intersection of the compression edge of the beam and the column flange. In addition, no effect of buckling is seen in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point, which is indicated by the stress and strain distribution. The strain generated is 0.044, where the steel material of the coupling enters the post-hardening stage. In the third stage, when the dynamic effect of increasing load is applied at a factor of 1.5 times the gravity load, which shows The plastic strain reaches approximately 0.314, which is much lower than the ultimate plastic strain of steel. Considering the boundary conditions and the progressive failure force, it can be concluded that the strong column with an RBS beam reinforced with headers and scarf plates can prevent progressive failure. In this case, the maximum displacement of the beam end at the end of the third stage is 2.14 cm.

The bending frame with a strong column reinforced with head and flange plates and RBS beam and the stress and strain distribution at: a) completion of the first stage, b) completion of the second stage, c) completion of the third stage of connecting spring rotation for 6 different cases are presented in *Table 4*.

Table 4. The rotational capacity of single and reinforced specimens with head and scarf sheets.

	VW-RBS	W-RBS	S-RBS	VW-F-RBS	W-F-RBS	S-F-RBS
Rotation (degrees)	12	11	11	11	8	5

2.8 | Case with RBS Beam and Column Reinforced with Plates Along the Column Length

In this section, the evaluation of columns with full-thickness reinforcement is performed. The stress distribution at the end of the first stage of loading is shown for the very weak column case with full-thickness plates and RBS beam. The maximum stress developed in the connection is approximately 4.66 MPa, which is lower than the yield stress of the steel, and its location is at the intersection of the compression edge of the beam and the column flange. In addition, there is no evidence of buckling in the members. At the end of the second stage, when the gravity load was fixed, it was found that the connection could not remain in the linear stage beyond the yield point. The developed strain is 0.0842. The corresponding steel strain after the steel hardening stage is 0.01328.

In the third stage, the dynamic effect of load increase is applied at a factor of 1.5 times the gravity load, which shows that the plastic strain reaches about 0.447, which is larger than the ultimate plastic strain of steel. Considering the boundary conditions and the progressive failure force, it can be concluded that the frame has failed, but the link spring has reached its ultimate material stress. In this case, the maximum displacement of the beam end at the end of the third stage is 34.6 cm.

A very weak column reinforced with universal plates and RBS beam and the stress and strain distribution at a) completion of the first stage, b) completion of the second stage, c) completion of the third stage. The stress distribution at the end of the first stage loading is shown for the weak column reinforced with universal plates and RBS beam. The maximum stress generated in the connection is approximately 3.77 MPa, which is lower than the yield stress of the steel, and in addition, there is no evidence of buckling in the members. At the end of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point.

The strain generated is 0.0854, indicating that the steel joining materials have entered the post-hardening stage. In the third stage, the dynamic effect of increasing load is applied at a factor of 1.5 times the gravity load, which shows that the plastic strain is approximately 0.415, which is less than the ultimate plastic strain of the steel. Considering the application of boundary conditions and progressive failure force, it can be concluded that the failed section has reasonable performance, but the connecting spring has reached the yield stress and requires reinforcement. In this case, the maximum displacement of the beam end at the end of the third stage is 31.64 cm.

Flexural frame with weak column reinforced with universal plates and RBS beam and stress and strain distribution at: a) completion of the first stage, b) completion of the second stage, c) completion of the third stage shows the stress distribution at the completion of the first stage loading for the case of strong column reinforced with universal plates and RBS beam. The maximum stress generated in the connection is approximately 8.86 MPa, which is lower than the yield stress of the steel, and in addition, no effect of buckling is seen in the members. At the completion of the second stage, when the gravity load was fixed, it was seen that the connection could not remain in the linear stage beyond the yield point, which is indicated by the stress and strain distribution.

The strain generated is 0.0856, indicating that the steel materials of the coupling have entered the post-hardening stage. In the third stage, when the dynamic effect of increasing load is applied at a factor of 1.5 times the gravity load, it is shown that the plastic strain reaches approximately 0.416, which is greater than the ultimate plastic strain of the steel. Therefore, it cannot prevent progressive failure in this case. In this case, the maximum displacement of the beam end at the end of the third stage is 31.6 cm. The bending frame with a strong column reinforced with universal plates and RBS beam and the stress and strain distributions at a) the end of the first stage, b) the end of the second stage, c) the end of the third stage of connecting spring rotation for three cases are shown in *Table 4* and *Table 5*.

Table 5. Connection Capacity.

	VW-C-RBS	W-C-RBS	S-C-RBS
Rotation (degrees)	12	11	10

In all cases of strengthening, using a very weak column is not effective in controlling the progressive failure phenomenon. However, in a strong column, using head and tail plates to strengthen simple beams is effective in controlling the progressive failure phenomenon. In the case of a simple beam, this phenomenon could not be controlled even if the column was completely strengthened. To prevent buckling and fracture of the column flange plate, total plates can be used on all four sides of the column. However, the small weakness that existed in the case of a weak column, which was the formation of a plastic zone inside the connecting spring, requires strengthening.

3 | Conclusion

3.1 | Single Beam Connection

Since the lowest ultimate strength of the members is assigned to the beam web-column flange connection plate, this connection should be strengthened to increase the ability of the structure to withstand the overall failure. The axial force in the beam flanges can be transferred to the column after the connection is strengthened. Therefore, an elastic flexural connection can be used in such a reinforced beam-column connection. This study was conducted to investigate the performance of the system after the failure of a member. Two column cases, weak and strong, were used in this study [10].

The weak columns cannot prevent progressive failure in this frame, which is why the frames with weak columns failed, and the connection spring reached the ultimate stress. Therefore, this area needs to be reinforced in the case of a simple connection because the plastic hinge occurred at the beam-column connection.

Comparing the previous case with the case without the presence of scarf and head plates, because the column is weak, the plastic hinge did not occur at the connection, and the plastic hinge was shifted toward the column and the connection spring. The mechanism of failure in the connector spring is shear. The rotation capacity of the connection was increased by changing from the simple case to the case reinforced with header and scarf plates, which resulted in a decrease in the vertical displacement of the beam end.

For weak and strong columns, the flexural capacity of the connector spring has increased by 55 and 73 percent, respectively, in the connector strengthening method with scarf and header plates.

For weak and strong columns, the flexural capacity of the connection spring has increased by 5 and 1.5 percent, respectively, in the cases reinforced with global plates around the column. The rotational capacity of the connection increased slightly from the simple case to the case reinforced with global plates, which resulted in a very small decrease in the vertical displacement of the beam end.

3.2 | Connection with RBS Bar

Only in the case of a weak column, it could not prevent the formation of a plastic zone in the connecting spring and the final failure in the case of connections reinforced with head and scarf plates with a beam of reduced cross-section.

In the case of using an RBS beam, only reinforcing the beam with header and scarf plates in the case where the column is very weak, which causes total failure, but a weakness is seen in the weak column, which is that the plastic zone inside the connection spring requires reinforcement with compression plates.

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Author Contribution

Amir Hossein Hosseini handled the conceptual development, numerical modeling, data analysis, and writing of the manuscript. The author has assessed and consented to the final version of the manuscript.

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Data Availability

The data supporting the conclusions of this research can be obtained from the corresponding author upon reasonable request.

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