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Experimental study of beam-column connections with web opening in a low-rise steel frame

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Abstract. Steel frame structures have been widely used in multi-storey and high-rise buildings and the connections in these structures are critical. In the Northridge and Kobe Earthquake, beam-column connections suffered damage due to brittle fracture. According to seismic design codes, ductility of the beam to column connection is also necessary. A study on the behavior of a beam to column connection with the aim of improving ductility as well as preventing brittle failure was carried out. In order to control the position of a plastic hinge on the beam, a connection with a hole in the beam web was developed. Five specimens with different parameters under cyclic load were assessed. The results are presented in terms of the stress distribution of the beam, hysteretic behavior, and ultimate capacity. Furthermore, the finite element method was also used to analyze the model, and the results were compared with those obtained from the experiment. It is shown from the analysis and experimental results that this type of connection is effective in terms of improving ductility for a beam to column connection in low-rise buildings.

Keywords: steel frame; beam to column connections; position of plastic hinge; experiments study; finite element analysis.

1. Introduction

Steel frame structures have been widely used in multi-storey and high-rise buildings (Wang and Shen 2004). Steel structures are considered to offer consistently reliable anti-seismic performance,

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given that steel has good plasticity behavior. However, numerous brittle failures on beam-column connections occurred during the earthquakes at Northridge in U.S.A. and Kobe in Japan. The plastic deformation of the steel material and structure can not enough developed. It means that fracture of the connection takes place before plastic deformation of the structure. Although local failure of beam to column connections does not necessarily cause collapse of the whole structure (SAC Interim guidelines 1995), repairs to the structure after an earthquake would be difficult (Yao *et al.* 2002). In order to make good use of steel material, the seismic design code suggests the concept of 'strong column weak beam'. Hence, when the plastic hinge of a steel beam of the frame is established at the appropriate position, plastic deformation occurs. Accordingly, many researchers have focused on anti-seismic behavior of the beam-column connection in steel frames (Song *et al.* 2001, Yang *et al.* 2001). At present, methods to improve the anti-seismic performance of the connection mainly focus on two aspects: improving the stress distribution on the connection region and reducing the stress concentration (Chen and Tu 2004, Chen *et al.* 2001).

According to theoretical and experimental study, methods such as removing the welding base plate, improving welding technology (Shi *et al.* 2002), and replacing some members can evidently reduce the stress concentration on the connection region. Another method is to change the failure position from the connection to the beam according to the principle of 'strong column weak beam', and to form a plastic hinge at the appropriate position on the beam. Other common methods include strengthening the connection using a cover plate or a weak beam. For example, in the typical "dog bone" connection, the upper flange of the beam has been weakened (as shown in Fig. 1). (Chen *et al.* 2001).

The plastic hinge is formed on the weakened position of the beam and plastic deformation can develop. Meanwhile, this kind of connection can effectively enhance the ductility of the joint (Uang and Fan 2001). However, it will reduce the rigidity and loading capacity of the beam due to the weak beam flange.

In recent years, research on reducing the beam strength by providing an opening on the beam web has been conducted in Korea, Taiwan, and the United States. The Engineering Journal of AISC published research results and FEMA also included this in the FEMA-350 report. However, details are lacking. In order to study the behavior for this kind of connection and to improve ductility as well as the capacity to prevent brittle failure, a beam connection that is weakened by way of an opening on the beam web (as shown in Fig. 2) is researched further in this paper. The position of the plastic hinge is controlled by the position and size of the opening on the beam web.

Five specimens with different parameters under cyclic loading were tested. The stress distribution

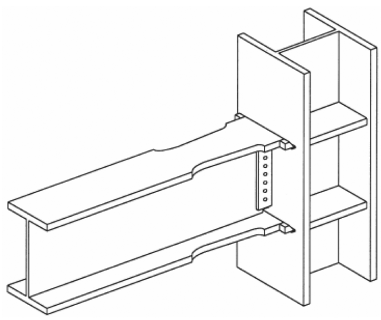


Fig. 1 "dog's bone" connection

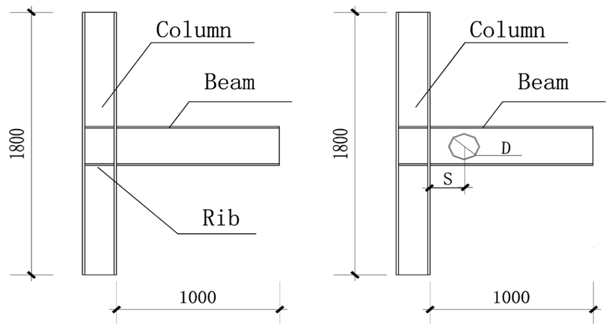


Fig. 2 Connection with hole on beam web

on the beam, hysteretic behavior, and ultimate capacity are discussed. Furthermore, the finite element method is also used to analyze the model of the specimens and the results are compared with those obtained from the experiment. Theoretical calculations and test results show that this type of connection is effective in terms of improving the ductility of the beam column connection in low-rise buildings. In addition, it is very simple to construct.

2. Experimental set-up

2.1 Specimens detail

Five T-shape specimens were tested in this study. Four specimens were designed with an opening on the beam web with different diameter and position. The fifth specimen is a common connection for comparison. A bolt was adopted to connect the beam web and column flange and welding was used to connect the beam and column flange for all specimens (as shown in Fig. 3).

Both the beam and column section are comprised of welded H-shape steel. Plates should be welded to column flanges using a complete joint penetration (CJP) groove weld. At the top and bottom flange, the weld backing is removed and backgouged, and a 5/16” minimum fillet weld is

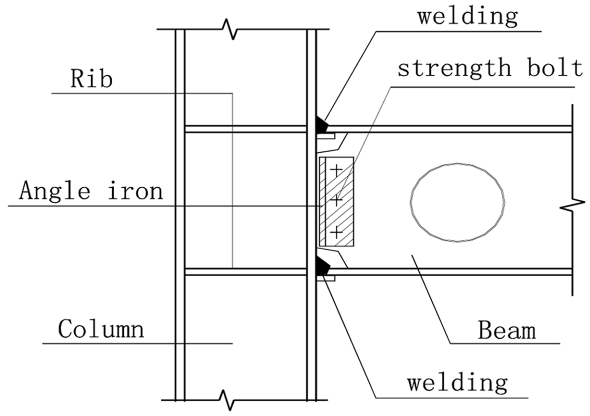


Fig. 3 Details of specimen

Table 1 Parameters of specimens

Parameters	Length/ mm	Flange width /mm	Flange Thickness /mm	Web height /mm	Web thickness /mm	Sectional area /mm^2	Inertial moment /10^4 mm^4	Compression stiffness /kN	Bending Stiffness /kN · m^2
Beam	1000	150	10	350	6	4980	10466.85	1025880	21561.71
Column	1800	250	12	250	8	7808	9266.15	1608448	19088.26

Table 2 Connection parameters of specimens

Item \ Specimens No	FR-1	SP-2	SP-3	SP-4	SP-5
Hole diameter on the beam web of the Specimens /mm	0	100	150	100	200
Distance from the center of the hole to column flange /mm	0	200	200	400	400
Type of angle iron connected specimens			L90X10		
Grade of bolt connected specimens			grade 8.8		
Diameter and number of bolt			3^{M16}		

added. The size of the beam and column are $150 \times 350 \times 6 \times 10$ and $250 \times 250 \times 8 \times 12$ respectively. Some of the sectional parameters of the beam and column are shown in Table 1. Angle steel was used to connect the beam web with the column using high strength bolts and welding was adopted to connect the beam flange and the column. The connection parameters of the specimens are shown in Table 2. The material properties of the specimens were obtained from three groups standard tension test. The average values of the ultimate yield strength and elastic modules are 422 MPa and 210 GPa, respectively, and the Poisson ratio is about 0.3.

2.2 Equipment for the test

In order to simulate the real structure conditions of the beam to column connection, the parts of the frame with rigid connection were adopted (Dubina and Ciutina 2000). Hence, a jack could provide axial force on column. The test set-up is shown in Figs. 4 and 5 (Wang and Liang 2003).

A hydraulic servo-actuator was used to induce cyclic load in the test. During the test, the load applied by the hydraulic jack was controlled using a load sensor and the displacements were measured using a vertical sensor. The load, displacements, and strains of the test points were recorded using a data line connected to a computer. A photograph of the test set-up is shown in Fig. 5. The detailed layout of the strain gage position on the specimens and displacement sensors on the beam and column is shown in Figs. 6 and 7.

2.3 Loading mode

In order to research the behavior of force and deformation of the connection under earthquake motion, cyclic load mode was selected. The loading mode shown in Fig. 8 was applied on the beam using a jack with a 20 kN load increment for each load step.

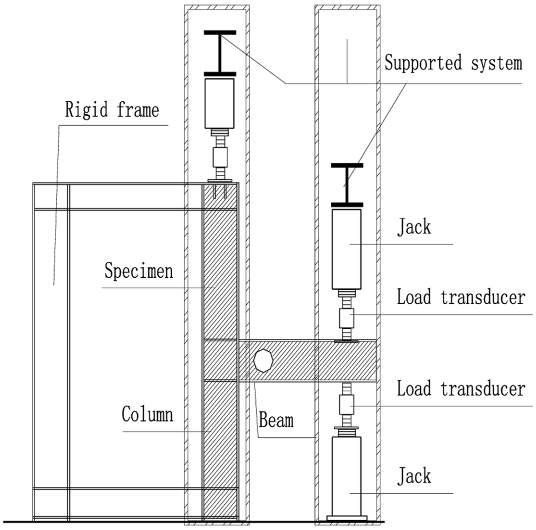


Fig. 4 Test setups

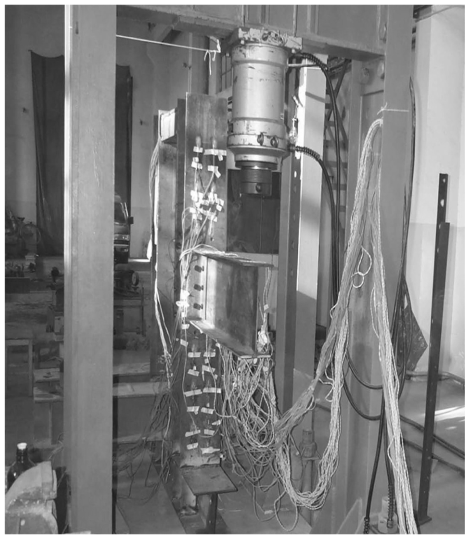


Fig. 5 Photo of the test setups

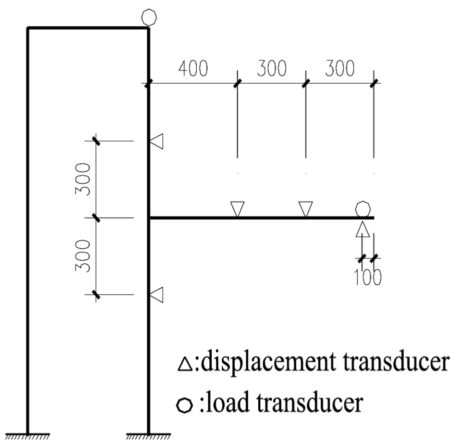


Fig. 6 Position of the sensor

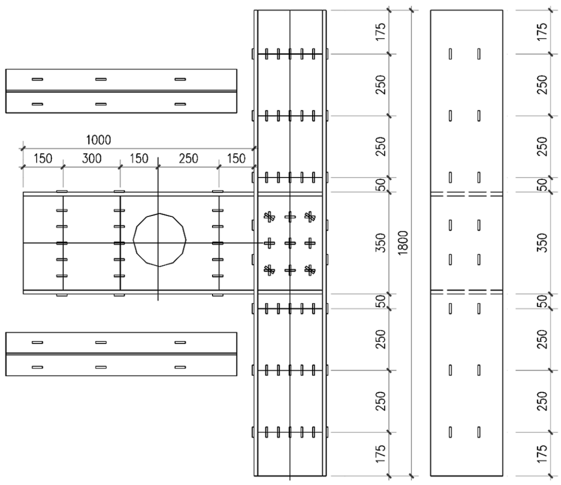


Fig. 7 Strain gage position on the specimen

3. Test results

The stress distribution on the cross-section of the beam, the maximum displacement at the end of the beam, the rotation angle of the section, and the ultimate load were obtained from tests. In addition, the hysteretic curves of the specimens were also obtained. From these results, the anti-seismic behavior of the connection between the beam and column is discussed.

3.1 Deformation of the joint

From tests, some phenomena could be observed. Local curvature of the beam with the hole has been changed on the top of the beam. This deformation is shown in Fig. 9. Moreover, the

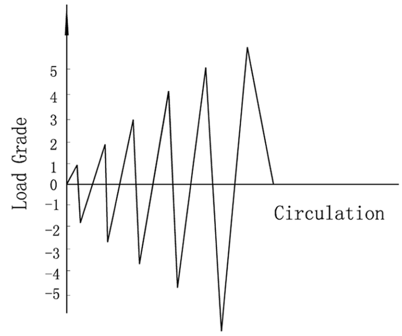


Fig. 8 The loading mode

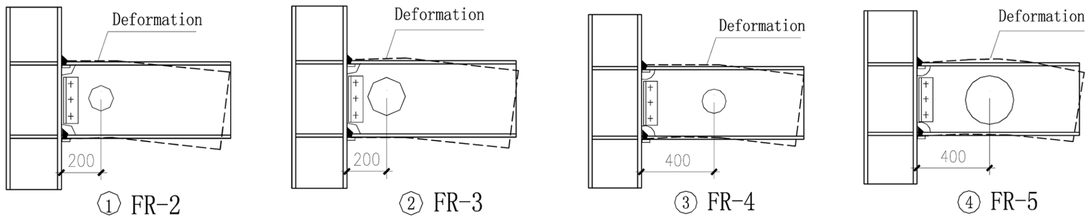


Fig. 9 Deformation of beam with web opening

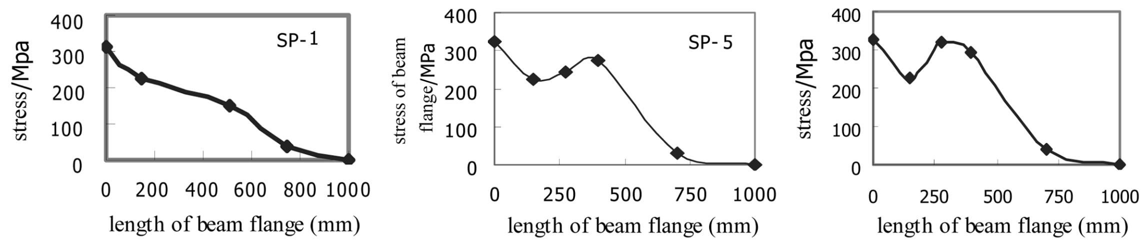
deformation is more pronounced when the hole diameter of the beam is increased. In addition, the local curvature is greater when the hole of the beam is located closer to the flange of the column. These phenomena indicate that the deformation increases due to the hole located on the beam web.

The deformations of the beam and ultimate load of the connection are shown in Table 3. Based on these results, it is found that as the diameter of the hole is increased the ultimate load of the connection decreases and the displacement of the beam increases.

Theoretical calculations carried out according to the size of the cross-section of the beam and column as well as the plastic moment of the beam and column reveal 226.98 kNm and 175.8 kNm, respectively. The maximum shear capacity of the high strength bolt used to connect the column flange and the beam web is 336 kN and 405 kN, respectively, and the shear capacity of the net cross-section at the position of the hole is 256.05 kN. Therefore, the concentrated load at the end of beam should be less than 218.8 kN. However, the results show that the capacity of the member does not reach the calculated values. The average rotational angle from Table 3 is the average value

Table 3 The test results

No	Maximum displacement of the beam /mm	Average value of rotation stiffness /KN·M/rad	Rotation angle /rad	Ultimate load of the connection /kN
SP-1	11	16632	0.01294	213
SP-2	13	14364	0.01529	192
SP-3	15	10766	0.01765	183
SP-4	10.5	15623	0.01235	160
SP-5	16	8688	0.01882	203



(a) Specimen SP-1 (b) Specimen SP-3 (c) Specimen SP-5

Fig. 10 Stress distribution on the flange along the beam

of the relative rotational angle of the beam-column.

From the test results, specimen SP-1 is found to have the highest capacity. Specimen SP-4 failed before yielding of the beam due to the low quality of the welding. The capacities of other specimens decrease gradually as the hole diameter increases except for SP-5. These results show that the position and the diameter of the hole on the beam are critical parameters.

3.2 Stress distribution

3.2.1 Stress distribution on the beam flange

The stress distribution on the flange along the beam is shown in Fig. 10. Usually, the maximum stress of the beam appears at the fixed end of the beam, and the stress decreases to the free end side of the beam. This is confirmed by the test results of specimen SP-1. However, for other specimens, the distribution on the flange along the beam is different from that for specimen SP-1. There are two points of peak stress on the beam flange. The first is located at the fixed end of the beam and the other is close to the position of the hole. Specimens SP-3 and SP-5 show this tendency more clearly than the other specimens, as shown in Fig. 10. From these results, it appears that by weakening the beam web locally the plastic hinge position can be controlled. Of course, the size and position of the hole also should be properly chosen.

3.2.2 Stress distribution on the beam flange in cross section

Generally, it is assumed in an analysis that the stress distribution on the beam flange in the cross-section is uniform for the I-shaped beam. However, from the test results, the strain and stress distribution on the beam flange at the fixed end of the beam show non-uniform distributions, as demonstrated from specimen SP-5. The stress distributions on the flange parallel to the beam cross-section of specimen SP-5 are shown in Fig. 11.

The stress of the flange section at the beam varies with the change of load. When the load level is 120 kN, the stress at the middle point of the flange is higher compared with the stresses located at the end side of the flange. It is also shown that, when the value of the load reaches 160 kN, the stress distribution appears to be uniform. Moreover, as the load increases, the stress of the flange also increases, but the distribution is not uniform (load level 180 kN). This phenomenon indicates that yield of the beam occurs on the flange of the beam section. Additionally, the stress distribution on the beam flange becomes uniform at locations farther from the free end of the beam.

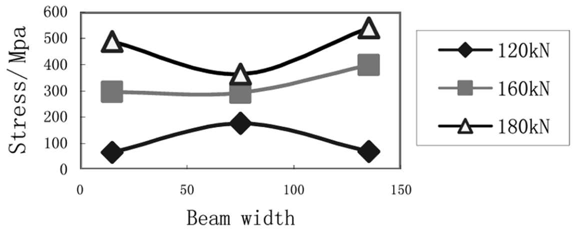


Fig. 11 Stress distributions on the flange parallel to the beam cross-section

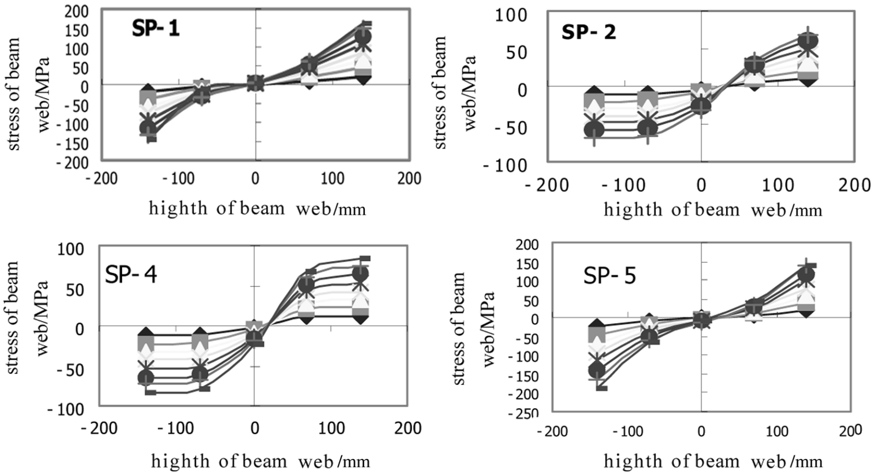


Fig. 12 Stress distribution of beam web

3.2.3 Stress distribution on beam web

Stress distributions on the beam web are shown in Fig. 12. For the entire beam the stress distribution follows the basic assumption of a linear stress distribution. In addition, the neutral axis of the beam with a hole is slightly shifted due to the hole position on the beam web. Also, the lateral stiffness of the beam might be decreased because of the hole. Consequently, it leads to local torsions and a nonlinear stress distribution. Therefore, in actual conditions, the lateral stiffness of the beam must be guaranteed and a rib can be used to strengthen the beam in the lateral direction.

3.3 Analysis of hysteretic behavior

The load displacement curve under cyclic load is very important for the beam column connection in a steel frame. It can demonstrate comprehensive properties such as the ductility, ability of absorbing energy, the strength, the stiffness, etc. Experimental load-displacement hysteretic curves of the specimens are shown in Fig. 13.

From these results, it was found that:

(1) The specimens with a hole in the beam web showed much better ductility compared with other specimens. From Fig. 13, specimen SP-5 shows the largest envelope line area. This also illustrates that the ductility of this specimen is better than that of the other specimens. This result indicates that the ductility of this connection is also related to the position and the size of the hole on the beam web. Additionally, when the parameters of the specimens are the same except for the

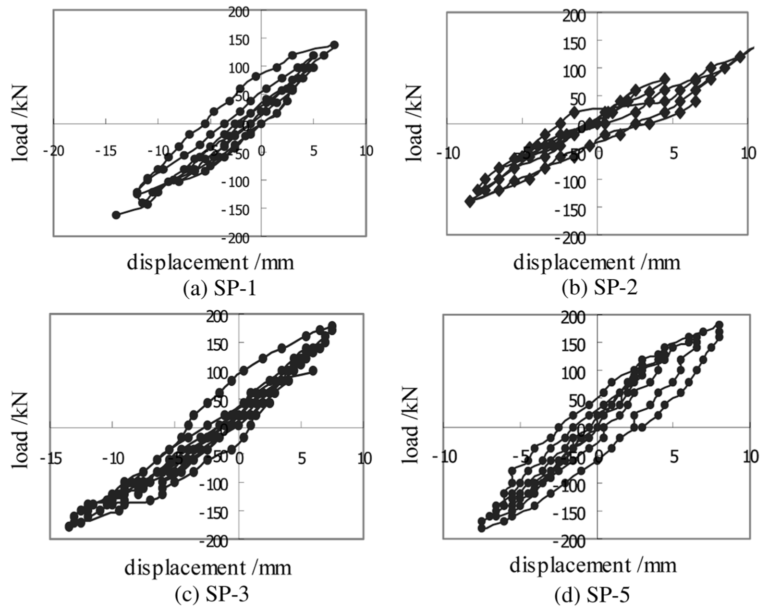


Fig. 13 Load-displacement curve of beam

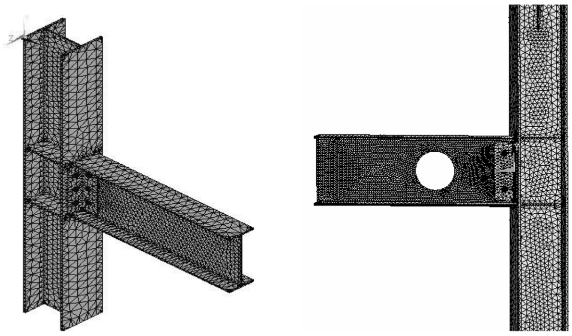
hole size, as the hole diameter increases, the ductility of the beam-column connection also increases. Of course, the capacity of a beam under load can be satisfied. Moreover, the position of the hole on the beam is also very important to enhance anti-seismic performance. According to the present results, the suggested diameter of the hole on the beam web is $D = 0.55h\text{~}0.75h$, where h is the height of the beam section.

Furthermore, when the distance of the hole is considered as a variable in order to observe the ductility of the specimens, the area of the hysteretic curve also can provide valuable information. The suggested distance value from the center of the hole to the column flange is $s = 0.75h\text{~}1.15h$. Specimen SP-5, with $S = 400$ mm and $D = 200$ mm (D is the hole diameter), shows the best hysteretic behavior among the specimens, as well as acceptable capacity of the connection.

(2) Deformation taking place in the connection zone strongly influences the hysteretic behavior of the beam column joint. The test results indicate that plastic rotation of specimens SP-1, SP-2, and SP-3 is mainly caused by rotation of the connection zone. In the case of specimen SP-5, the plastic rotation is relatively large because of weakening of the beam flange. The rotation of the connection zone considerably affects the plastic rotation with a large quantity. Furthermore, the test results demonstrate that plastic deformation on the connection zone occurred before that on the beam flange, and that sufficient plastic deformation occurred.

4. Finite element analysis and discussion

In order to confirm the test results analytically, the finite element method was used to analyze specimens tested in the experimental study. Stress distribution on the section, hysteretic behavior, ultimate capacity, and displacement were also obtained from the analysis and comparisons between the test results with calculated values revealed relatively good agreement.



(a) Specimen FR-1 (b) Specimen FR-5

Fig. 14 Analysis model and meshwork of specimens

4.1 Analysis of finite element models

The finite element analysis presented in this study was conducted using available commercial software (ANSYS) and a three-dimension non-linear analysis was also applied. Three types of elements were selected in the finite element model: (1) three-dimension solid elements (SOLID92) used for the beam, column, angle shape connector, and rib; (2) contacted elements (TARGE170 and CONTA174) used for the contact area between the angle shape connector and the beam or column, and the connection between the bolt and angle shape connector; and (3) pre-tension elements (PRETS179) used for bolts with pre-stress. The finite element models used in this study are shown in Fig. 14. The Von-Mises yielding criterion and the plastic flow law were applied in the analysis. Furthermore, the multi-linear hardening criterion was adopted to describe the initial yield criterion followed by the increment of plastic strain. Both material and contact non-linearity were also considered in the analysis.

4.2 Calculation results

The displacement at the free end of the beam and the ultimate capacity of the connection obtained from the analysis are shown in Table 4.

It is shown that as the hole in the beam web increases, the ultimate load decreases and the maximum displacement of the beam increases. The deformation and the stress distribution on the flange along the beam and those parallel to the beam cross-section were also obtained from analysis. The deformation and stress distribution of specimen SP-5 are shown in Fig. 15. It is shown

Table 4 Calculation results

No	Maximum displacement of the beam /mm	Average value of rotation stiffness /KN·M/rad	Rotation angle /rad	Ultimate load of the connection /kN
SP-1	11.82	16632	0.0131	196.6
SP-2	13.46	14364	0.0150	204.35
SP-3	17.65	10766	0.0196	198.02
SP-5	20.08	8688	0.0235	183.14

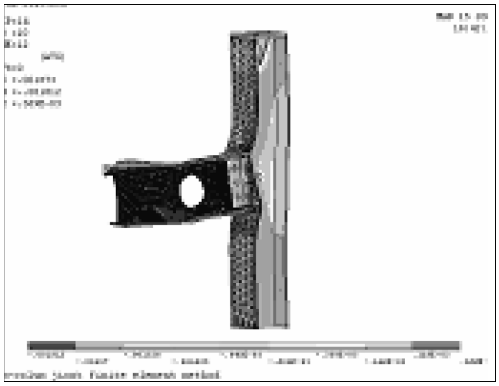
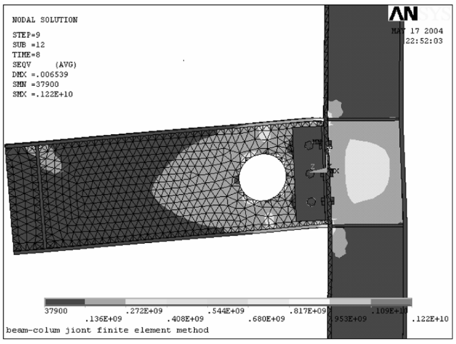
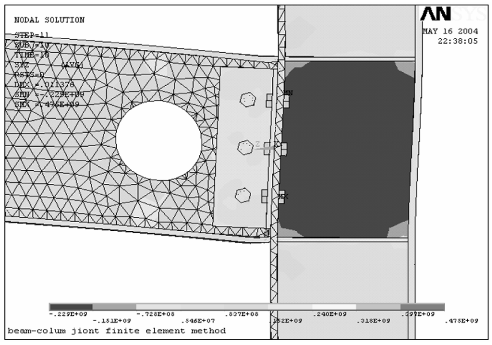


Fig. 15 Deformation of specimen SP-5



(a) P=140kN



(b) P=180kN

Fig. 16 Stress distribution on panel zone

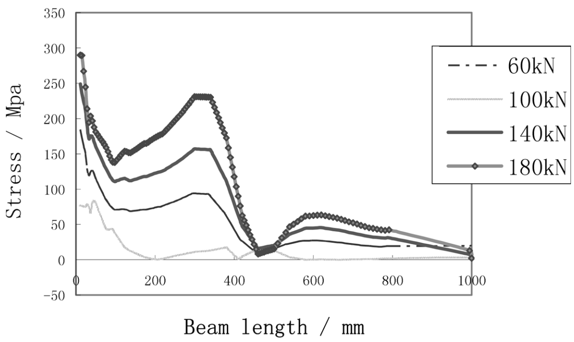


Fig. 17 Stress distribution on the flange along the beam

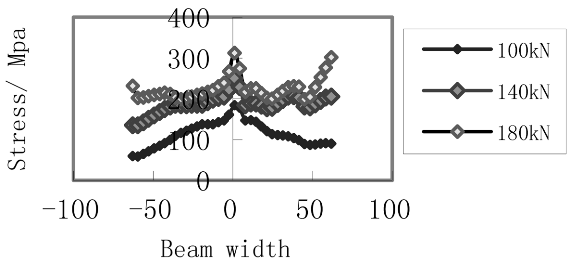


Fig. 18 Stress distribution on the flange parallel to the beam cross-section

that the highest stress level is located on the connection zone, and the flange of the beam located near the hole shows buckling.

In addition, the load-displacement hysteretic curve obtained from the analysis was compared to that obtained from the test. Based on this, the following is noted.

(1) The deformation characteristics from the analysis are similar to those obtained from experiments. The maximum stress takes place on the connection zone. The stress distribution on the connection zone is shown in Fig. 16. This clearly shows that a plastic area has developed on the connection zone and the side of the hole in the beam web.

(2) From the stress distribution on the flange along the beam, two points of maximum stress were obtained, which can effectively improve the stress concentration at the fixed end of the beam and also improve the connection ductility. These results are identical to those obtained from the experimental test (as shown in Fig. 17).

(3) The stress distribution on the flange parallel to the beam cross-section is also close to the test results, as shown in Fig. 18. The analysis shows that when the load is less than 100 kN, the stress distribution of the flange at the fixed end of the beam is not uniform and the stress in the connection of the web and flange is relatively higher than that at both sides of the flange. However, when the load is more than 140 kN, the stresses of the flange become uniform and agree with the assumption. This indicates that the hole in the web develops a stress concentration. When the load level is 180 kN, the stress distribution changes greatly, indicating that the fiber on the beam flange has yielded and plastic hinges has occurred in the section due to the large rotation.

(4) The stress distribution along the web of the beam section is shown in Fig. 19. This figure shows a non-linear stress distribution on the beam section with a hole. At a load less than 60 kN, the stress distribution along web of the beam section is linear. These results show good agreement with the assumption of plane-section. When the load approaches 140 kN, the neutral axis begins to diverge to some extent, and the top and bottom edges of the web begin to yield, which is consistent with the test results. This stress distribution is attributed to the hole in the beam web. Furthermore, the distribution will vary according to the diameter of the hole.

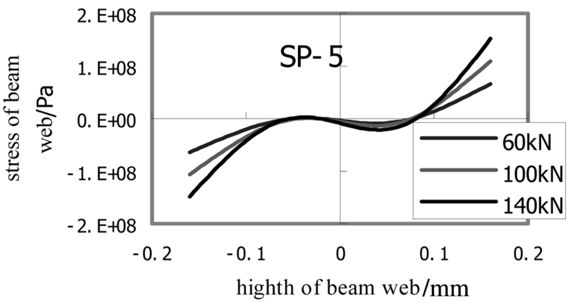


Fig. 19 Stress distribution on the web

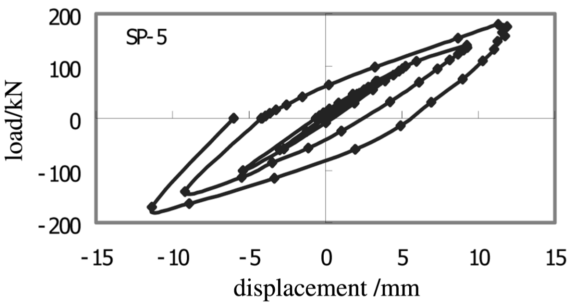
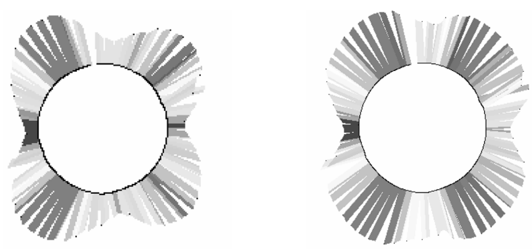


Fig. 20 The hysteretic curve of specimen SP-5



(a) P=100kN (b) P=180kN

Fig. 21 Stress distribution on the side of the hole

Table 5 Compare of results

No	Maximum displacement of the beam /mm		Ultimate load of the connection /kN	
	Test	Analysis	Test	Analysis
SP-1	11	11.82	213	196.6
SP-2	13	13.46	192	204.35
SP-3	15	17.65	183	198.02
SP-5	16	20.08	203	183.14

(5) In order to obtain the hysteretic curve, the cyclic load was applied in the analysis. The representative results of specimen SP-5 are shown in Fig. 20. Based on these results, it is found that the connections with holes exhibit good ductility. In addition, local buckling of the beam web is the main cause the failure of the beam column joint connection, as verified by the test results.

(6) The stress distributions on the side of hole under different load levels are shown in Fig. 21. The stress on the side of the hole is not uniform and the maximum value is in the direction of 45^{o}. Moreover, the yielding zone becomes larger as the load increases.

4.3 Comparison between analysis and test

Comparisons between the test and analysis results show good agreement. In particular, the character of the stress distribution on the beam flange along the beam length and width, as well as the hysteretic curve, correspond well with the test results. Table 5 presents a comparison between the maximum deformations and the capacity of the connection. The results show that the finite element analyses yield results that are in good agreement with those of the analysis. In addition, the capacity of the connection is reduced by less than 8%, indicating that this type of connection, i.e., weakening the beam web, can be utilized if the hole of optimum diameter is located at the appropriate location. It is also should be noted that the shear capacity of the beam must be guaranteed.

5. Conclusions

The current required plastic rotational angles of beam-to-column connections for seismic application are generally at least 3% rad. However, the findings presented here indicate that the method of beam-column connections with a web opening is effective. It should be noted, however, that this study adopted small scale specimens and used an angle for the shear connection, which is only applicable for low-rise steel frames. Further research will be carried out. Experimental and analytical studies have been conducted in order to investigate the behavior of a new type of beam column connection under cyclic load. Based on these, the following conclusions have been obtained:

(1) The position of the plastic hinge can be controlled through weakening of the beam section under certain conditions. It is shown from the analysis and experimental results that this type of connection is effective in terms of improving the ductility of the beam column connection.

(2) From the analysis and test results, the position of the beam web opening is determined to be important with regard to the formation of a plastic hinge on the section of the beam. According to

the test results, the suggested value of the hole diameter is $D = 0.55h$~$0.75h$ (where h is the height of the beam section) and the distance from the center of the hole on the beam web to the column flange is $s = 0.75h$~$1.15h$.

(3) Deformation of the connection panel zone strongly affected the hysteretic behavior. Hence, in order to realize the position of the plastic hinge take place on the beam, it is necessary to guarantee the stiffness of the connection on panel zone.

(4) The capacity of the connection can be acceptably reduced if the position and diameter of the hole on the beam web are appropriate. From the analysis and test results, the capacity of the connection was reduced by less than 8%.

(5) Specimen SP-4 failed due to low quality welding with a maximum load lower than the design load. Thus, the quality of welding must be assured in practice. Also, it might be necessary to use appropriate welding techniques in order to reduce the stress concentration of the connection.

(6) In the test, all of the bolts slid to some degree. Hence, the use of appropriate bolts is suggested so as to prevent such sliding and to ensure that the connection between the beam web and column is adequate.

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