Experimental Investigation of Reduced Beam Section Connections by Use of Web Openings

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Since the 1994 Northridge Earthquake, a wide variety of different moment connections have been developed to provide high ductility and reliable performance. Most of these moment connections incorporate the reinforcement at the connection using cover plates, ribs or haunches (Engelhardt, Sabol, Aboutaha, and Frank, 1995; Engelhardt and Sabol, 1998; Uang and Bondad, 1996; Uang, Bondad, and Lee, 1998; Whittaker and Gilani, 1996). The purpose of the reinforcement at the connection was to force the plastic hinge away from the face of the column, where premature fractures can occur due to potential weld defects, stress concentrations, etc. The reinforced connections generally showed good performance. However, reinforcing the connection may increase its fabrication costs. Moreover, there were some concerns with respect to reliability resulting from the very large weld for reinforcement.

An alternative to reinforcing the connection is weakening of the section of the beam away from the face of the column to force the plastic hinge to form at the weakened location. This type of connection is termed a Reduced Beam Section (RBS) connection, also known as a dogbone connection (Iwankiw and Carter, 1996; Iwankiw, 1997; Chen, Yeh, and Chu, 1996; Plumier, 1997; Zekioglu, Mozaffarian, and Uang, 1997; Tremblay, Tchebotarev, and Filiatrault, 1997; Engelhardt, Winneberger, Zekany, and Potyraj, 1998; Venti, 2000). In most cases, weakening of the beam sections has been accomplished by trimming away portions of the beam flange in the region adjacent to the beam-to-column connection. While showing excellent performance similar to that of the reinforced connections, the dogbone connections also appeared as a more economical and reliable solution.

It has been a common practice to cut openings in steel beam webs for the passage of service ducts and pipes. If it is possible to force the plastic hinge at the web opening location by adjusting the size of the opening, the RBS connection can be constructed without trimming the beam flange leading to a more economical connection. Different methods to improve the performance of high-seismic moment resisting frames using beam web openings have been proposed (Goel, Leelataviwat, and Stojadinovic, 1997; Halterman and Aschheim, 2000; Kim, 2000). The RBS connection with web openings appears to be a preferred choice in the rehabilitation of existing construction because trimming of the beam top flanges where floor slabs are present may be difficult. This paper summarizes the results of a test program on seismic-resistant RBS connections constructed using beam web openings.

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TEST PROGRAM

A total of six large-scale tests were performed on single cantilever type test specimens. The specimens were subjected to slowly applied cycles of loads. The test setup is shown in Figure 1. A wide-flange section of 792 mm (31.18 in.) beam depth × 300 mm (11.81 in.) flange width × 14 mm (0.55 in.) web thickness × 22 mm (0.87 in.) flange thickness was used for the beam. For the column, a wide-flange section of 458 mm (18.03 in.) column depth × 417 mm (16.42 in.) flange width × 30 mm (1.18 in.) web thickness × 50 mm (1.97 in.) flange thickness was used. The beam was of SS 400 steel (equivalent to ASTM A36 steel) and the column was of SM 490 A steel (equivalent to ASTM A572 Gr. 50 steel). The results of tensile coupon tests are listed in Table 1.

While panel zone yielding has been recognized as an efficient way to dissipate seismic energy, excessive panel zone deformations can lead to local kinks in the column flanges, which can contribute to premature fracture at the beam-to-column weld (Krawinkler, 1978; Kawano, 1984; Popov, 1987; El-Tawil, 2000). The main purpose of this study is to demonstrate that removing web material may be effective in creating a ductile fuse in the beam to protect the weld. Therefore, the column size was selected to force most of the inelastic action to occur in the beams with the panel zone remaining essentially elastic.

All the specimens used the same detail for the beam-to-column connection as shown in Figure 2. The connection incorporated the following features:

- Use of a welded flange-bolted web connection. The beam flange was connected to the column flange using complete-joint-penetration groove welds. The connection between the beam web and the column flange was made with a conventional bolted shear tab, which is more economical than a fully-welded web connection.

Table 1.
Measured Tensile Coupon Data

Section		F_y	F_u
WF-792×300×14×22 mm (31.2×11.8×0.55×0.87 in.)	Flange	300 MPa (43.51 ksi)	439 MPa (63.67 ksi)
	Web	368 MPa (53.37 ksi)	478 MPa (69.33 ksi)
WF-458×417×30×50 mm (18.0×16.4×1.18×1.97 in.)	Flange	418 MPa (60.62 ksi)	600 MPa (87.02 ksi)
	Web	396 MPa (57.43 ksi)	577 MPa (83.69 ksi)
Reinforcing plate		370 MPa (53.66 ksi)	541 MPa (78.46 ksi)

- Use of high toughness weld metal. Complete-joint-penetration groove welds between the beam flange and the column flange were made with the gas shielded flux cored arc welding process using the Supercored 70B electrode (similar to E71T-5 electrode) with a minimum specified Charpy V-notch toughness of 20 ft-lbs. at –20°F. Gas shielded flux-cored arc welding process is usually used for field welding in Korea.
- Removal of weld tabs at both the top and bottom beam flange groove welds. Weld runoff tabs were removed after completion of the weld and ground smooth to eliminate notch effects.
- Both the top and bottom beam flange backing bars were left in place, although removal of bottom flange backing is recommended by the SAC Design Criteria (FEMA, 2000). This was done because removal of backing bars is difficult and increases costs. It was believed that the reduced beam section at the web opening would act as a fuse limiting the stress at the beam flange groove welds and allowing the backing bars to remain in place. All groove welds were made by a certified welder and passed the ultrasonic test.

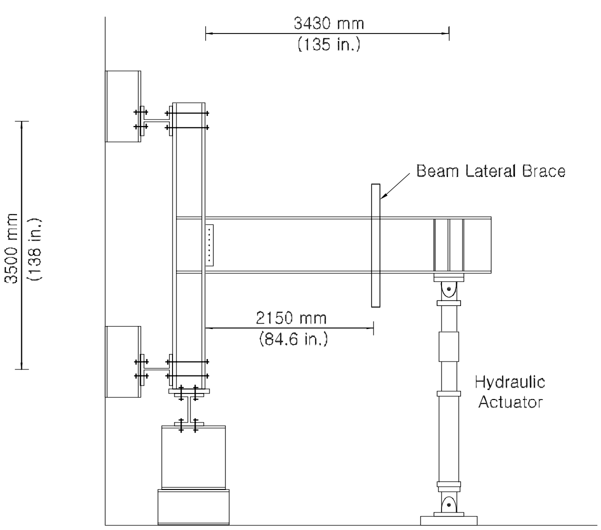

Fig. 1. Test setup.

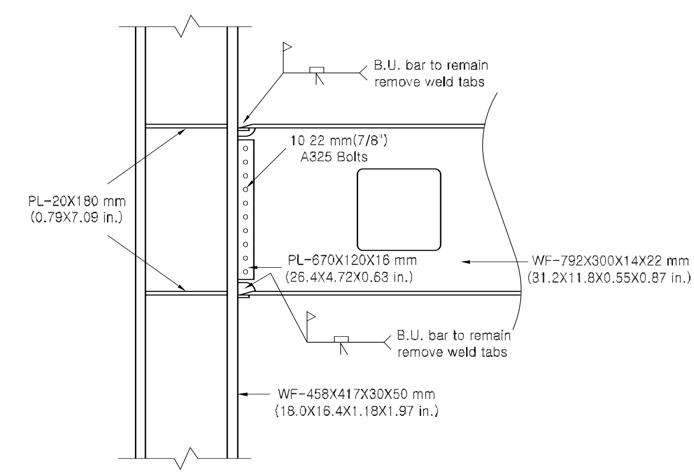

Fig. 2. Typical connection detail B1.

Spec.	Opening Type	Opening Size	$M_{n\text{-}RBS}$	M_F	M_F/M_p
B1	Rectangular	400 x 400 mm (15.75 x 15.75 in.)	1605 kN-m (1184 k-ft)	2086 kN-m (1538 k-ft)	0.93
B2	Rectangular	400 x 500 mm (15.75 x 19.69 in.)	1489 kN-m (1098 k-ft)	1940 kN-m (1431 k-ft)	0.87
B3	Rectangular	500 x 200 mm (19.69 x 7.87 in.)	1477 kN-m (1089 k-ft)	1924 kN-m (1419 k-ft)	0.86
B4	Reinforced Rectangular	400 x 900 mm (15.75 x 35.43 in.)	1540 kN-m (1135 k-ft)	2220 kN-m (1737 k-ft)	0.99
B5	Circular	560 mm (22.05 in.)	1343 kN-m (990 k-ft)	1751 kN-m (1291 k-ft)	0.78
B6	Circular	510 mm (20.08 in.)	1594 kN-m (1176 k-ft)	2037 kN-m (1502 k-ft)	0.91

Table 2. Description of Test Specimens

M_p = plastic moment of beam based on tensile coupon test = 2243 kN-m (1654 k-ft)
$M_{n\text{-}RBS}$ = nominal bending strength at opening based on tensile coupon test
M_F = beam moment estimated at face of column when moment at opening reaches $M_{n\text{-}RBS}$

The specimens, designated as B1 through B6, are described in Table 2. All openings were centered at the mid-depth of the beam. The rectangular openings were first drilled at the corners and then flame cut by hand. After cutting, the edges were finished by grinding to have a surface roughness of no more than 500 µin. (AWS C4.1-77 sample 4). Specifications require that the corner radius not be less than $2t_w$ (t_w is web thickness) or $^5/_8$ in., whichever is greater (Darwin, 1990; SEI/ASCE, 1999). Based upon these criteria, $2t_w$ is 28 mm (1.1 in.) for the specimens tested in this program. However, the corner radius adopted for the rectangular openings was 25 mm (0.98 in.), because this was the largest drill size available in the laboratory. It should be noted that Specifications do not recommend openings in members subjected to significant cyclic or fatigue loading (Darwin, 1990; SEI/ASCE, 1999). Openings were located at a distance of one beam depth away from the face of the column to protect the beam-to-column connection weld from undesirable influence of yielding at the opening. Figure 3 shows the details of the reinforced opening of Specimen B4. The center of the reinforced opening of Specimen B4 was at 1050 mm (41.34 in.) away from the face of the column.

The sizes of openings were determined to force the plastic hinge to occur at the openings. The nominal bending strength at the openings was estimated using the cubic interaction equation adopted by the American Institute of Steel Construction and the American Society of Civil Engineers/Structural Engineering Institute (Darwin, 1990; SEI/ASCE, 1999). When the moment at the openings reached the nominal bending strength, the calculated moment at the face of the column varied from 0.78 to 0.99 times the plastic moment of the full beam section, M_p. The bending moments used in the determination of the opening sizes are presented in Table 2. Each specimen was instrumented as specified by the SAC Joint Venture (Clark, Frank, Krawinkler, and Shaw, 1997).

TEST RESULTS

Each of the specimens was subjected to an identical displacement controlled loading sequence specified by the SAC Joint Venture (Clark et al., 1997). The loading sequence is listed in Table 3. The hysteretic response of each specimen is shown in Figures 4 through 9. These plots show beam moment versus total rotation. The beam moment was calculated as the beam tip load multiplied by the distance between the beam tip and the face of the column. The total rotation was calculated as the beam tip displacement divided by the distance between the beam tip and the centerline of the column.

Most of the inelastic action occurred within the beam in all specimens. Although the beam flange near the face of the column showed some yielding, the most intense yielding was observed at the corners of the opening. Only slight

Table 3. Loading Sequence		
Load Step #	**Peak deformation θ**	**Number of cycles, *n***
1	0. 00375	6
2	0.005	6
3	0. 0075	6
4	0.01	4
5	0.015	2
6	0.02	2
7	0.03	2
8	0.04	2
Continue with increments in θ of 0.01, and perform two cycles at each step		

yielding was observed in the column panel zone in all specimens. The maximum shear strain obtained from the rosettes provided at the center of the panel zone was on the order of 0.23 to 0.27 percent reflecting the slight yielding observed in the test. The shear deformation of the panel zone was measured by means of a pair of displacement transducers running diagonally across the panel zone. The flexural deformation of the column was measured using two displacement transducers placed on the column flange at the top and bottom flange of the beam. Deformation components were computed from the displacement measurements using the method proposed by Uang and Bondad (Uang and Bondad, 1996). Calculation of deformation components indicated that the rotation developed by the column including the panel zone yielding was less than 10 percent of the total rotation in all specimens. Figure 10 shows the beam moment versus column rotation (including panel zone shear deformation) curve of Specimen B3, which developed the highest peak strength.

Specimen B1

Specimen B1, which was provided with a 400 mm (15.75 in.) × 400 mm (15.75 in.) rectangular opening, showed good performance in the early inelastic loading cycles. At the opening corners, yielding was first observed in the loading cycles of 0.0075 rad, and then increased and spread during subsequent loading cycles. In the loading cycles of 0.015 rad, yielding was observed in the beam flange at the opening end close to the column. However, sudden fracture

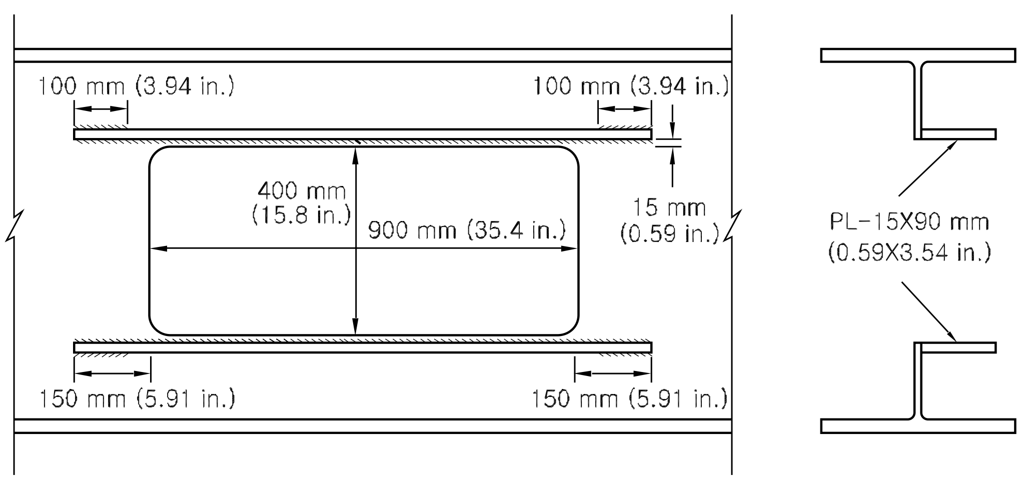

Fig. 3. Reinforced opening detail.

occurred at the beam bottom flange weld during the first loading cycle of 0.03 rad. Although it was not clear that the backing bar caused the fracture, removal of the backing bar might have helped identification and correction of weld root flaws. No local buckling or cracking was observed at the opening corners. The maximum moment developed at the column face was 1.01 times the plastic moment of the full beam section. Figures 11 and 12 show photos of Specimen B1 after testing.

Specimen B2

Based on the test result of Specimen B1, it appeared that better performance might be possible by increasing the size of the opening to force the plastic hinge to occur at the opening location. Specimen B2 was provided with a 400 mm (15.75 in.) × 500 mm (19.69 in.) rectangular opening. This specimen showed good performance. At the opening corners, yielding was first observed in the loading cycles of 0.005 rad, and then increased and spread during subsequent loading cycles. In the loading cycles of 0.01 rad, yielding

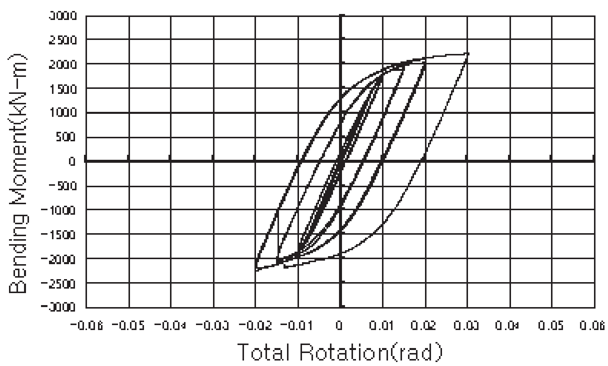

Fig. 4. Hysteretic response of Specimen B1.

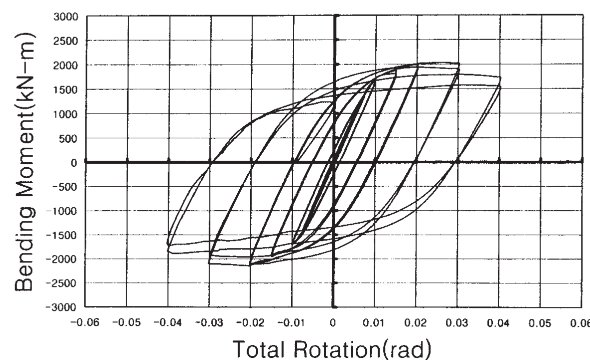

Fig. 5. Hysteretic response of Specimen B2.

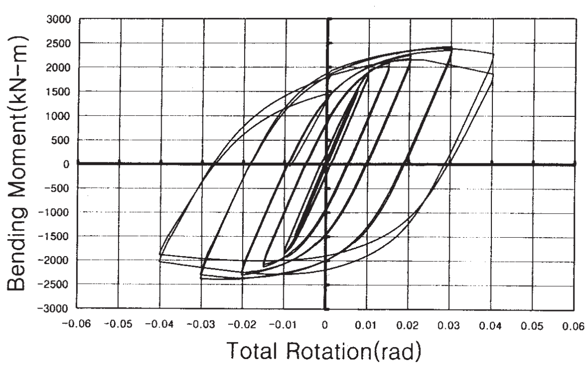

Fig. 6. Hysteretic response of Specimen B3.

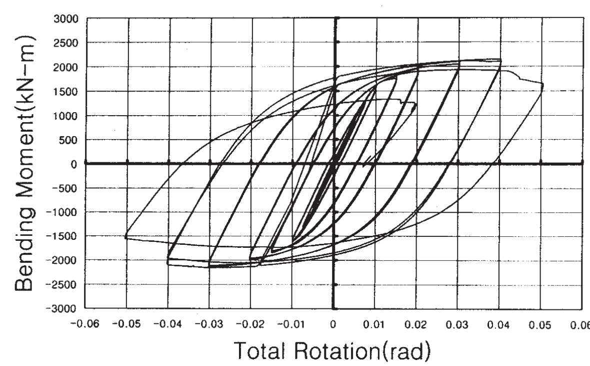

Fig. 7. Hysteretic response of Specimen B4.

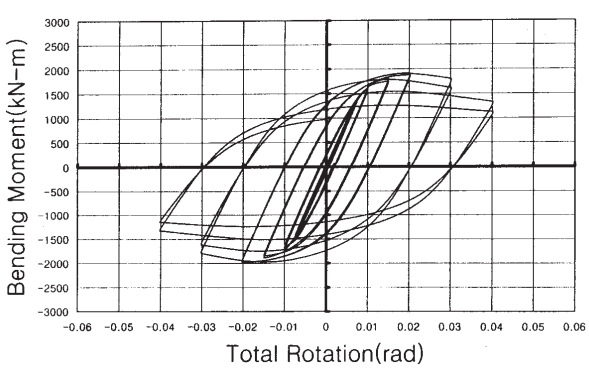

Fig. 8. Hysteretic response of Specimen B5.

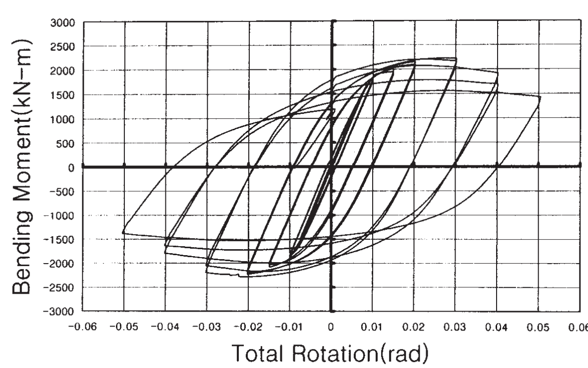

Fig. 9. Hysteretic response of Specimen B6.

was observed in the beam flange at the opening end close to the column. In the loading cycles of 0.03 rad, a gradual deterioration of strength occurred due to local buckling of the beam web at the corners of the opening. During the second loading cycle of 0.04 rad, a fracture developed at the bottom corner of the opening close to the column. The test was stopped after the completion of the loading cycle. The maximum moment developed at the column face was 0.96 times the plastic moment of the full beam section. No crack was observed in the beam flange groove welds and at the weld access holes. Figures 13 and 14 show Specimen B2 at the end of the test.

Specimen B3

Specimen B3 was provided with a 500 mm (19.69 in.) × 200 mm (7.87 in.) rectangular opening to investigate the effect of the opening length. This specimen showed good performance. At the opening corners, yielding was first observed in the loading cycles of 0.0075 rad, and then increased and spread during subsequent loading cycles. In the loading cycles of 0.02 rad, local buckling was observed in the beam flange at the opening end close to the column. In the loading cycles of 0.03 rad, slight local buckling of the beam web was observed at the corners of the opening. In the first loading cycle of 0.04 rad, deterioration of strength occurred due to the local buckling of the beam flange and web, and a small crack was observed at the top corner of the opening far from the column face. In the second loading cycle of 0.04 rad, the crack grew rapidly, and the peak strength was reduced significantly. The test was stopped after the completion of the loading cycle. The maximum moment developed at the column face was 1.08 times the plastic moment of the full beam section. No crack was

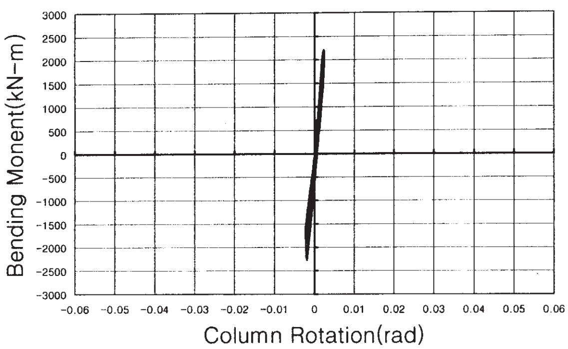

Fig. 10. Moment vs. Column Rotation of Specimen B3.

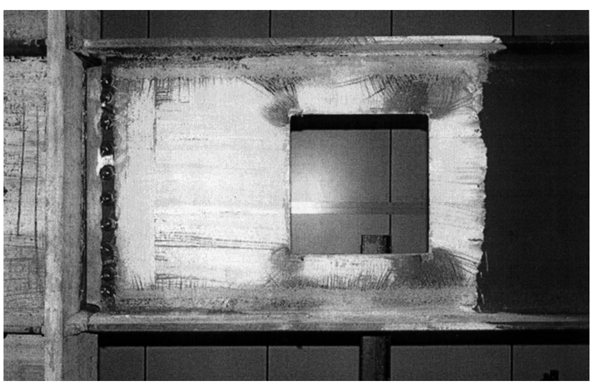

Fig. 11. Specimen B1 after testing.

Fig. 12. Fracture of bottom flange (Specimen B1).

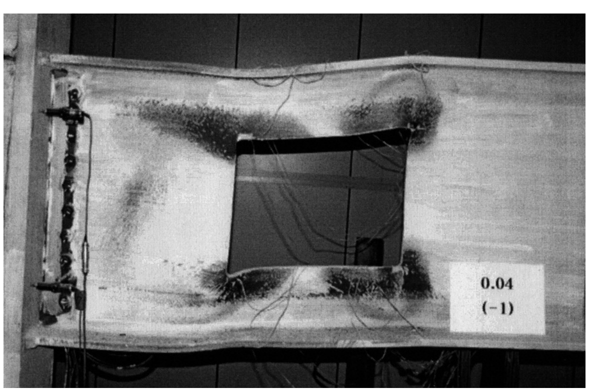

Fig. 13. Specimen B2 after testing.

Fig. 14. Crack at opening corner (Specimen B2).

observed in the beam flange groove welds and at the weld access holes. Figures 15 and 16 show Specimen B3 at the end of the test.

Specimen B4

Specimen B4 was provided with a 400 mm (15.75 in.) × 900 mm (35.43 in.) rectangular opening reinforced with one-sided horizontal plates. It was believed that the reinforcing plates might reduce the local buckling at the opening corners leading to better performance. As expected, Specimen B4 showed more stable performance than the other specimens with unreinforced rectangular openings. At the opening corners, yielding was first observed in the loading cycles of 0.0075 rad, and then increased and spread during subsequent loading cycles. In the loading cycles of 0.01 rad, yielding was observed in the beam flanges and the reinforcing bars at the opening end close to the column. In the loading cycles of 0.03 rad, local buckling of the beam web and the reinforcing plate occurred simultaneously at the opening corners. During the loading cycles of 0.04 rad, local buckling of the beam web and the reinforcing plate became more severe, and small cracks were observed at the corners of the opening. However, strength deterioration was not noticeable. In the first loading cycle of 0.05 rad, these cracks grew rapidly past the reinforcing bars, and the peak strength was reduced significantly. The test was stopped after the completion of the first loading cycle. The maximum moment developed at the column face was 0.96 times the plastic moment of the full beam section. No crack was observed in the beam flange groove welds and at the weld access holes. Figures 17 and 18 show Specimen B4 at the end of the test.

Specimen B5

Specimen B5 was provided with a circular opening of 560 mm (22.05 in.) diameter. This specimen showed high ductility. However, the peak strength was somewhat smaller than other specimens due to the large reduction of the web. Web yielding was first observed in the loading cycles of 0.005 rad at the opening about 30° from the vertical through hole center, and then increased and spread during subsequent loading cycles. In the loading cycles of 0.01 rad, yielding was observed in the beam flange at the location of the web yielding close to the column. In the loading cycles of 0.02 rad, slight local buckling occurred in the beam flange and web simultaneously at the location of the web yielding close to the column face. In the loading cycles of 0.03 rad, a gradual deterioration of strength occurred due to local buckling of flange and web combined with lateral-torsional buckling. Consequent strength degradation was apparent in the next loading cycles. The strength degraded to less than 80 percent of the peak strength in the loading cycles of 0.04 rad and testing was stopped after completion of the second loading cycles of 0.04 rad. The opening size of Specimen B5 was close to the maximum value (0.7 times the beam depth) permitted by the Specifications (Darwin, 1990; SEI/ASCE, 1999). However, Specimen B5 could not satisfy the acceptance criteria of code provisions due to severe strength degradation (FEMA, 2000). No fracture was observed at either the connection or the opening. The maximum moment developed at the column face was 0.88 times the plastic moment of the full beam section. Figure 19 shows Specimen B5 at the end of the test.

Specimen B6

Based on the results for Specimen B5, it appeared that better performance might be possible by decreasing the size of the opening to increase the peak strength. Specimen B6 was provided with a circular opening of 510 mm (20.08 in.) diameter. This specimen showed better performance than Specimen B5. Web yielding was first observed in the loading cycles of 0.0075 rad at the opening about 30° from the vertical through hole center, and then increased and spread during subsequent loading cycles. In the loading cycles of 0.01 rad, yielding was observed in the beam flange at the

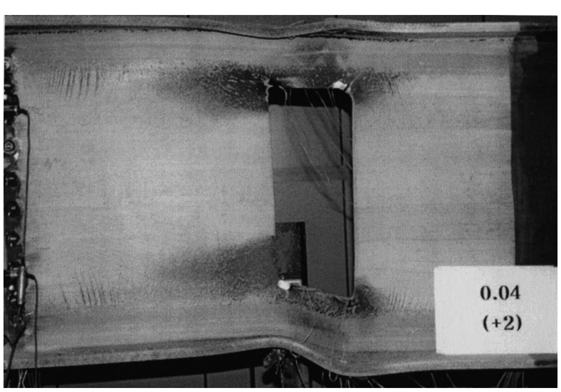

Fig. 15. Specimen B3 after testing.

Fig. 16. Crack at opening corner (Specimen B3).

location of the web yielding close to the column. In the loading cycles of 0.03 rad, slight local buckling occurred in the beam flange and the web close to the column simultaneously. The peak strength was higher than that of Specimen B5. Strength deterioration due to local buckling combined with lateral-torsional buckling started later in the loading cycles of 0.04 rad. Specimen B6 could develop 0.05-rad peak deformation, but the test was stopped after the first cycle of 0.05-rad peak deformation, since the strength degraded to less than 80 percent of the peak strength. No fracture was observed at both the connection and the opening. The maximum moment developed at the column face was 1.02 times the plastic moment of the full beam section. Figure 20 shows Specimen B6 at the end of the test.

Summary

Table 4 lists the total rotation and the plastic rotation achieved by each specimen at the end of the test. Plastic rotations were obtained by removing the elastic rotation from the total rotation. New proposed code provisions (FEMA, 2000) have changed the acceptance criteria from 0.03 rad of plastic rotation to 0.04 rad of total rotation (elastic plus plastic). Table 4 shows that all specimens except Specimen B1, which failed by fracture of the beam flange near the weld, developed the target level of total rotation.

Table 4 also presents the magnitude of bending moments developed in the specimens. The maximum moment developed at the center of the opening ranged from 0.97 to 1.26 times the nominal bending strength calculated using the cubic interaction equation (Darwin, 1990; SEI/ASCE, 1999). The values greater than 1.0 account for strain hardening at the opening. The equation gives satisfactory predictions except for Specimen B3, which has an unusually short opening length. The maximum moment developed at the face of the column ranged from 0.88 to 1.08 times the plastic moment of the full beam section.

In the specimens with rectangular openings, fractures occurred at the opening corners due to stress concentrations. Strain gauges were provided at the corners of the rectangular openings and at the locations about 30° from the vertical line through the center of the circular openings. The maximum strain was on the order of 9 percent for the rectangular openings and 3 percent for the circular openings, respectively. The stress concentrations increase as the shear force at the opening increases. Therefore, circular openings may be the ideal configuration for short spans. On the other

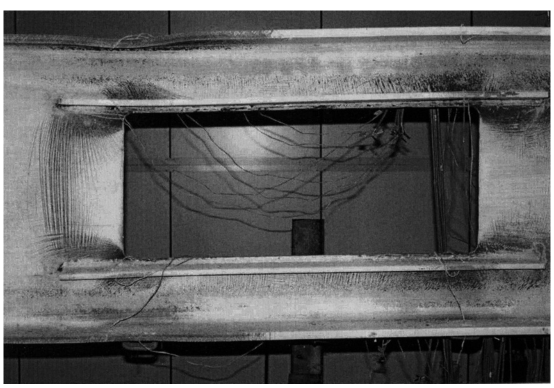

Fig. 17. Specimen B4 after testing.

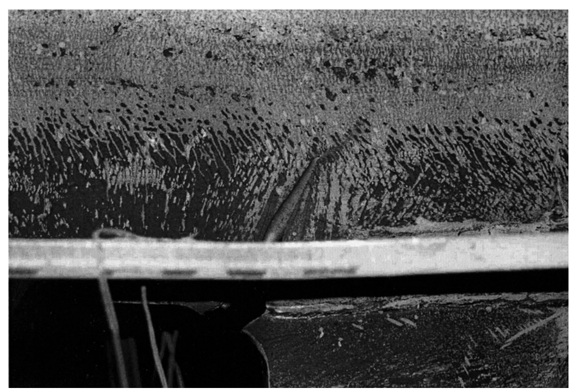

Fig. 18. Crack at opening corner (Specimen B4).

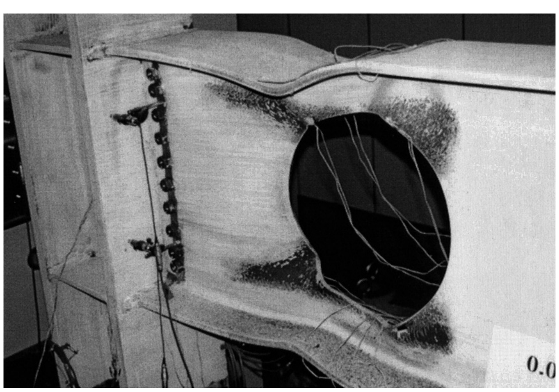

Fig. 19. Specimen B5 after testing.

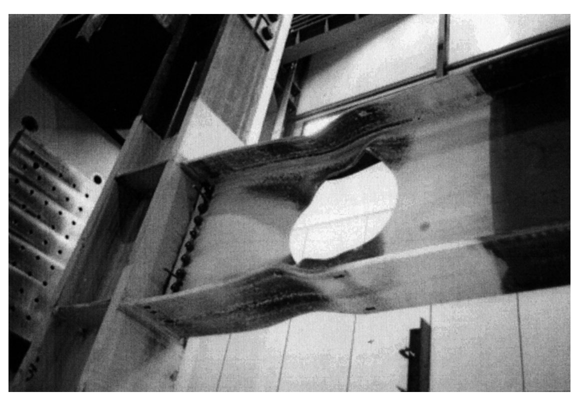

Fig. 20. Specimen B6 after testing.

Table 4. Summary of Test Results						
Spec.	$M_{max\text{-}RBS}$	M_{max}	$\frac{M_{max\text{-}RBS}}{M_{n\text{-}RBS}}$	$\frac{M_{max}}{M_p}$	Total Rotation (rad) (Plastic Rotation)	Failure modes
B1	1731 kN-m (1277 k-ft)	2258 kN-m (1665 k-ft)	1.08	1.01	0.02 (0.010)	Fracture of beam bottom flange near weld
B2	1645 kN-m (1213 k-ft)	2146 kN-m (1583 k-ft)	1.11	0.96	0.04 (0.031)	Fracture at the corner of the opening
B3	1858 kN-m (1370 k-ft)	2422 kN-m (1786 k-ft)	1.26	1.08	0.04 (0.028)	Fracture at the corner of the opening
B4	1492 kN-m (1100 k-ft)	2151 kN-m (1586 k-ft)	0.97	0.96	0.05 (0.040)	Fracture at the corner of the opening
B5	1515 kN-m (1117 k-ft)	1977 kN-m (1458 k-ft)	1.13	0.88	0.04 (0.033)	Strength degradation due to combined local and lateral torsional buckling at the opening
B6	1752 kN-m (1292 k-ft)	2286 kN-m (1686 k-ft)	1.12	1.02	0.05 (0.042)	Strength degradation due to combined local and lateral torsional buckling at the opening

M_{max} = maximum moment developed in beam at face of column
$M_{max\text{-}RBS}$ = maximum moment developed at opening

hand, rectangular openings with circular ends may be the ideal configuration for long spans.

DESIGN IMPLICATIONS

The number of tests conducted in this program is limited and insufficient to develop general design guidelines for RBS connections by use of web openings. However, based on the test results, some suggestions can be made for preliminary guidelines that may be useful until further analytical and experimental data are available. The key features of the connections tested in this program are the geometry of the web opening, the use of high toughness weld metal, and leaving of bottom flange backing bars. The bottom backing bars were not removed since the primary inelastic demand was shifted to the RBS. However, one specimen suffered sudden fracture at the bottom flange weld. Therefore, the writers suggest removal of bottom flange backing bars until further research is available.

Key dimensions for the web openings are the distance from the face of the column to the centerline of the opening, the length of the opening, and the depth of the opening. As the opening moves close to the column, the opening becomes more effective in limiting the maximum beam moment at the face of the column. However, the writers recommend the distance from the face of the column to the centerline of the opening not be less than the beam depth to protect the beam-to-column connection weld from undesirable influence of yielding at the opening. Opening depth is typically limited to 70 percent of the beam depth and the length-to-height ratio of the opening is limited to 3.0 (Darwin, 1990; SEI/ASCE, 1999). However, the writers recommend that the depth of the opening not exceed 0.65 times the beam depth to avoid severe strength degradation based on the test results of this program.

Once the dimensions of the opening have been selected, a capacity design procedure is implemented following the various steps as follows:

1. The nominal strength of the RBS at the centerline of the opening is determined using the cubic interaction equation (Darwin, 1990; SEI/ASCE, 1999).

$$\left(\frac{M_n}{M_m}\right)^3 + \left(\frac{V_n}{V_m}\right)^3 = 1 \tag{1}$$

where

M_n and V_n = nominal moment and shear strengths under combined bending and shear

M_m = maximum moment capacity based on zero applied shear

V_m = maximum shear capacity based on zero applied moment

The nominal shear strength can be expressed from the desired plastic mechanism in ductile moment resisting frames (Figure 21) as

$$V_n = \frac{2M_n}{L'} \tag{2}$$

where L' is the distance between interior plastic hinges (opening centers).

The nominal bending strength can be computed from Equation 1 and 2 as

$$M_n = M_m \left[\left(\frac{2M_m}{V_m L'} \right)^3 + 1 \right]^{-1/3} \tag{3}$$

2. The maximum moment developed at the centerline of the opening (M_{RBS} in Figure 22) is assumed to be equal to 1.1 times the nominal bending strength at the opening (M_n of Equation (3)) considering strain hardening. Figure 22 shows the linear moment diagram along half of a beam span under the usual assumption that the point of inflection occurs at mid-span.
3. The maximum beam moment (M_F in Figure 22) can be calculated by projecting the moment at the opening to the face of the column as

$$M_F = M_{RBS} \left[1 + \frac{2e}{L'} \right] \tag{4}$$

   where e is the distance from the face of the column to the centerline of the opening.
4. The maximum beam moment at the face of the column is limited to approximately 90 to 100 percent of the beam plastic moment. While it is desirable to limit the maximum beam moment as low as possible to reduce the possibility of sudden fracture in the beam flange groove weld, the maximum beam moment smaller than 90 percent of the beam plastic moment is expected to require excessively large openings. If the calculated value of the maximum beam moment is out of the limited range, the above procedure should be followed repeatedly with a different choice of the dimensions of the opening until it comes into the limited range.
5. Strong column–weak beam conditions and the adequacy of the shear strength of the column panel zone should be checked in accordance with the SAC Design Criteria (FEMA, 2000).

Equation 2 and Figure 22 neglect the influence of gravity loads assuming that gravity load moments are small compared to seismic load moments. If a moment frame is subjected to substantial gravity loads, the nominal shear strength at the opening and the maximum beam moment at the face of the column can be calculated using the methods presented in the SAC Design Criteria (FEMA, 2000).

Web openings will reduce the elastic lateral stiffness of steel moment frames. In general, the effect of a single web opening on the deflection of a beam under gravity loads can be considered negligible (Darwin, 1990; SEI/ASCE, 1999). A finite element analysis of the specimens tested in this program was performed by the writers using the computer program ANSYS (ANSYS, 1999). The elastic deflection at the beam tip of each specimen was compared with that of a plain model without an opening. The increase in deflection was about 18 percent for the specimen with reinforced rectangular opening (Specimen B4). For the other specimens, the increase in deflection was on the order of 4 to 8 percent. If the reduction in stiffness is a concern, the effects of the opening on lateral stiffness of the steel moment frame should be considered using a refined structural model.

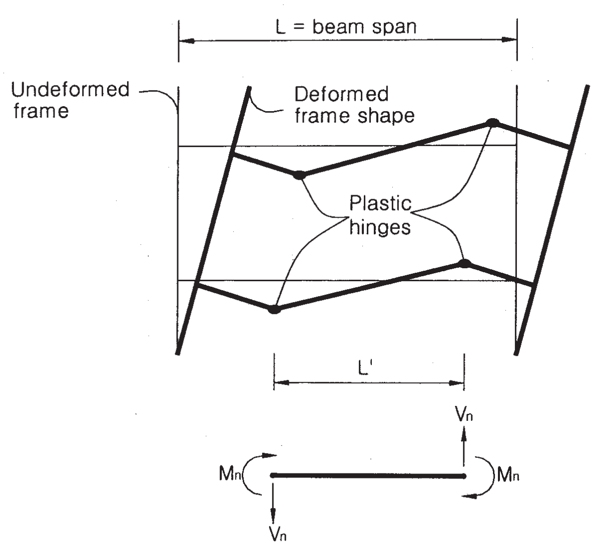

Fig. 21. Plastic mechanism of frames with hinges in beam span.

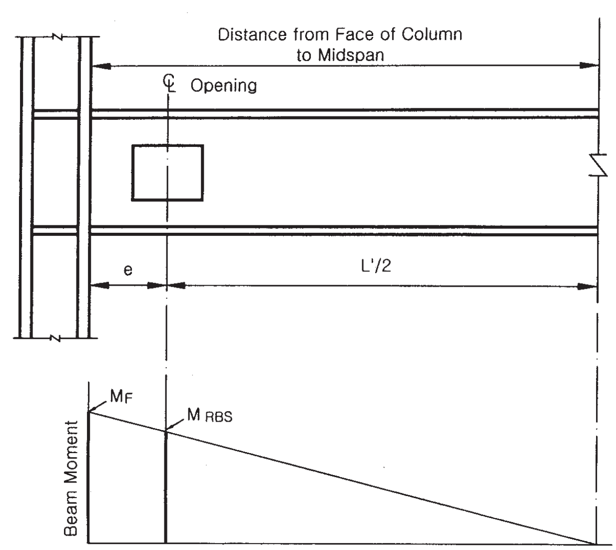

Fig. 22. Beam moment diagram.

CONCLUSIONS

A test program was conducted on Reduced Beam Section connections by use of beam web openings. The test specimens showed good performance overall, although yielding occurred at lower levels of rotation and strength degradation was worse than for reduced beam flange type connections (dogbone connections). Based on the test results, it appears that RBS connections by use of web openings is a promising connection concept for seismic-resistant steel moment connections for new structures and for the retrofit of existing buildings. For new structures, the beam web openings for the passage of service ducts can be used for the reduction of the beam section, if adequate provisions are made to protect the service ducts from the distortion of the beam. RBS connection by use of web openings also appears to be a preferred choice in the rehabilitation of existing construction because trimming of the beam top flanges where floor slabs are present may be difficult.

From the test results, the following additional conclusions can be reached:

1. In connections with rectangular openings, cracks at the opening corners are the major cause for strength degradation. Increase in the corner radius or use of circular ends may improve the performance.
2. In connections with circular openings, local buckling at the opening combined with lateral-torsional buckling is the major cause for strength degradation. Addition of reinforcing plates to the circular openings may assist control of local buckling. Composite concrete floor slabs may also improve the performance.

Subjects that deserve further investigation are the following: (i) the ratio of the moment to the shear at the opening (beam span and opening location), (ii) type and aspect ratio of the opening, (iii) minimum corner radius of rectangular openings for low cycle fatigue, (iv) the effects of composite concrete slabs, (v) use of low toughness weld metal (retrofit of existing buildings).

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