COMPARATIVE DAMAGE ASSESSMENT OF STEEL BEAM-TO-BOX COLUMN CONNECTIONS UNDER FIRE AND SUBJCTED TO EARTHQUAKE SHAKING

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SUMMARY

In this paper, the result is presented of a comparative study on fire and seismic behaviors of steel beam-to-column connections without and with beam-web openings. The technique of finite element analyses (FEA) was first applied to simulate the results of seismic tests on full-scale steel beam-to-column connections. Then the FEA model was used to study the connection behavior at elevated temperature. The von Mises stress, equivalent plastic strain and rupture index were calculated and compared in detail. Both the test and analysis results indicate that both the connections may have performance meeting the AISC 2016 seismic requirements. For the connection without beam-web openings, at the elevated temperature of 800ºC, the constraints of thermal expansion can cause serious thermal stress and strain near the column face, and may increase the potential of fracture to a high level, just like a great earthquake. In comparison, the added beam-web openings can greatly reduce the potential of connection fracture, mitigating both the seismic and thermal damage.

Keywords: *steel connections; web openings; FEA; earthquakes and fire; potential of fracture.*

INTRODUCTION

The seismic application of reduced web section (RWS) connections has recently attracted much attention, especially regarding steel moment frames in existing buildings. A beam flange and the adjacent beam web with an opening form a T-section. This T-section needs to take shear force and bending moment simultaneously, which can cause the beam flange to undergo local plate buckling and even fracture failure. The premature buckling failure of the beam flange can severely affect the strength development of an RWS connection. To eliminate this effect, a modified design was developed and verified numerically, in which two horizontal long openings were created and the centerline of the openings were located at a distance of one beam depth from the column face (Hedayat and Celikag, 2009). Recently, the results were reported about the design modifications and experimental verificaiton on the RWS connections with two horizontal long openings, as to further enhance the seismic application with general sections (Chang et al., 2022).

This paper presents the result of a comparative study on fire and earthquake damage to steel beam-to-column connections with and without two horizontal long openings. The technique of finite element analyses (FEA) has

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been applied to simulate the results of seismic tests on two full-scale steel beam-to-column connections (Lai et al., 2021; Chang et al., 2022). The FEA model is also used to study the fire behavior of the two connections. The von Mises stress, equivalent plastic strain and rupture index are calculated and compared in detail for the connections with and without beam-web openings, and for fire and earthquake damage.

SIMULATION OF SEISMIC TESTING

Specimen Details

Fig.1 shows the RWS connection with two horizontal long openings. The connection is composed of a SN490B steel box-750×750×28 (mm) column and H-800×300×14×25 (mm) beam. The distance between the beam end and column centerline is 4500 mm. An A572 Gr.50 steel 640×210×22 (mm) plate is used as a shear tab to connect the column face using double fillet welds. Sixteen M24 S10T high-strength bolts are then connected to the shear tab and beam web. The surfaces of the shear tab and beam web are sandblasted to achieve a contact friction coefficient of 0.45 (AIJ 2012). The comparion connection has almost the same details but using 12 bolts without beam-web openings. The effects of high strength bolts were studied and confimred not to affect the following comparion (Chang et al., 2021).

Seismic Testing

The beam displacement increased gradually as per the quasistatic cyclic loading protocol of AISC 2016. Figs. 2 and 3 show the experiment photos of the two connections. The test of the RWS connection was terminated after the first loading cycle with a drift angle of 5% rad, because of severe deterioration in the connection strength and stiffness.The test of the comparison connection was terminated right after the loading cycle with a drift of 4% rad, beause of the occurnece of fracture in the bottom beam flange near the column face.

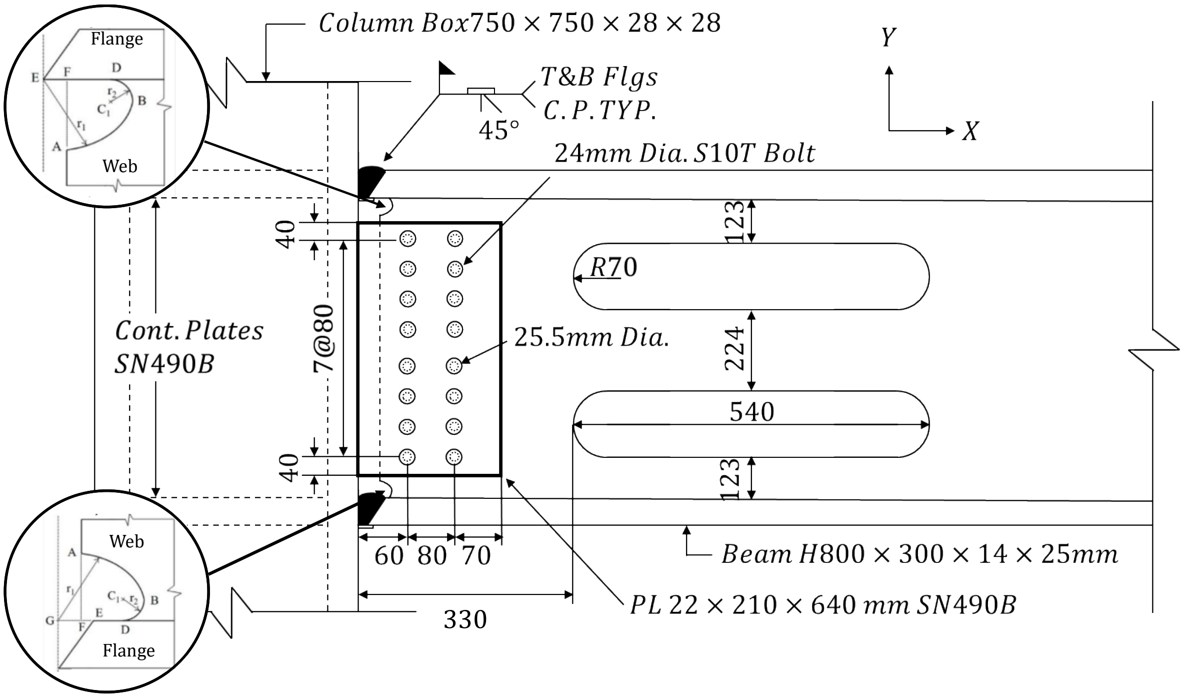

Fig.1 Details of the studied RWS connection with two horizontal long openings (Chang et al., 2022)

Hysteresis Loops

Fig.4 shows the simulated hystereis loops of the two connections (in orange) and compares to those obtained from seismic testing (in blue). Both the connections had a moment capacity more than 80% of the beam nominal plastic moment after completing one loading cycle with a drift angle of 4%. The connection performance met the requirements specified by the AISC seismic provisions. The FEA simulation has been made via ANSYS©. For the RWS connection, the FEA simulation slightly underestimated the degradation in connection strength and stiffness. For the comparison connection, the FEA simulation well agreed with the test result.

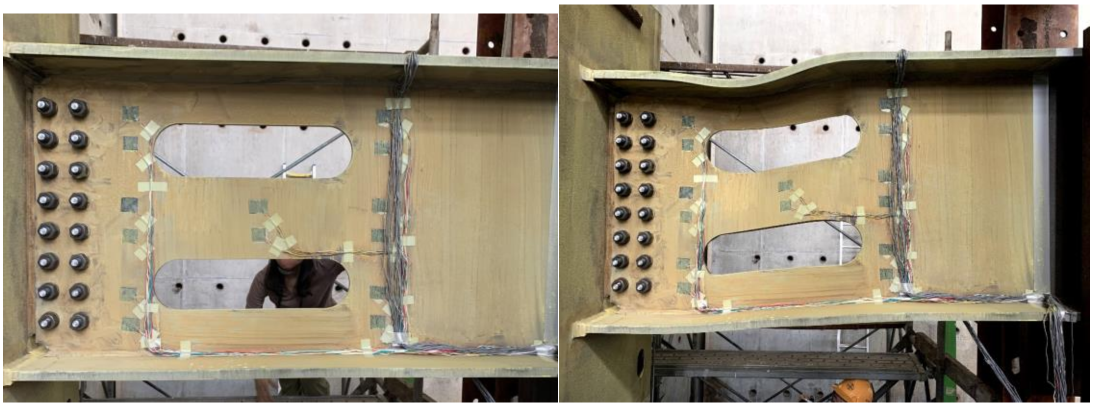

Fig.2 Photos of the RWS connection (T2); no obvious deformation at 2% drift (left), and buckling deformation in the upper beam flange and out of plane deformation of the reduced web section at 4% drift (right)

Fig.3 Photos of the comparison connection (UR); cracks initiated from weld access hole at 3% drift (left), local bucking in the bottom beam flange at -4% drift in the 1st loading cycle (middle), and fracture in base metal surrounding the beam groove weld (right)

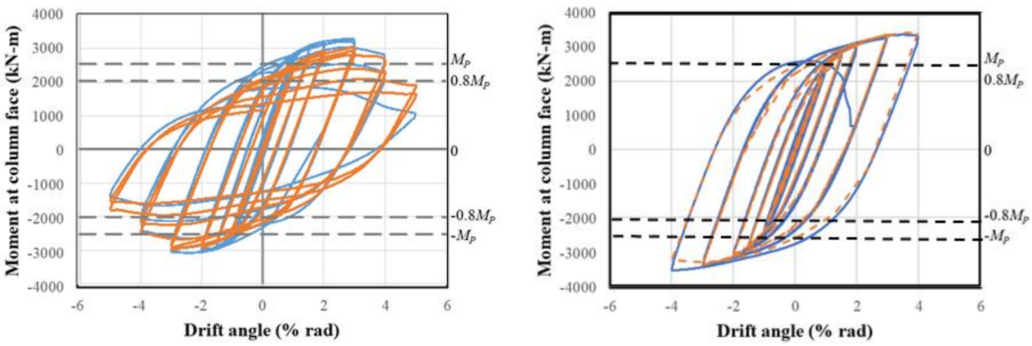

Fig.4 Hysteresis loops of the RWS connection (left) and comparison connection (right) obtained from FEA simulation (in orange) and from seismic testing (in blue)

Potential of Fracture

In the following, ε_{ij} is the vector component of plastic strain in the direction of i and j, ε_y is the yield strin of steel material, and σ_1, σ_2, and σ_3 are the principal stresses. The potential for connection fracture can be measured in terms of the rupture index (RI), which is composed of the equivalent plastic strain (PEEQ) index and stress triaxiality ratio ST (Boresi et al., 1993; El-Tawil et al., 2000).

$$\mathrm{RI} = \frac{PEEQ\ index}{\exp(-1.5ST)} \tag{1}$$

$$\mathrm{PEEQ} = \sqrt{\frac{2}{3}\varepsilon_{ij}\varepsilon_{ij}} \tag{2}$$

$$\text{PEEQ index} = \frac{PEEQ}{\varepsilon_y} \tag{3}$$

$$\text{ST} = \frac{\sigma_m}{\sigma_v} \tag{4}$$

$$\sigma_m = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3) \tag{5}$$

$$\sigma_v = \sqrt{\frac{1}{2}[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]} \tag{6}$$

Fig. 5 shows the distribution of rupture index (RI) in the studied RWS connection (designated as T2, in blue) in comparison to the identical connection without beam-web openings (designated as UR, in orange). The figure also show the position for evaluating the potential of fracture. The distances from the column face of the connection were 25 mm, 43.55 mm and 470 mm for the WF, AHF and BFF lines, respectively. The RI values were not symmetrically distributed with respect to the centerline of the beam flange as the drift angle increases to 4% rad. For the studied RWS connection, the maximum RI occurred in the beam flange adjacent to the beam-web opening (i.e., the T2_BFF case). For the comparison connection without beam-web openings, the maximum RI value of 183.3 occurred in the beam flange near the weld access hole (i.e., the UR_WF case). It was previously mentioned that the comparison connection had carack initiation from the weld access hoel and finally underwent fracture in the beam flange near the column face (see Fig. 3). The RI values well agreed with the experimental observations.

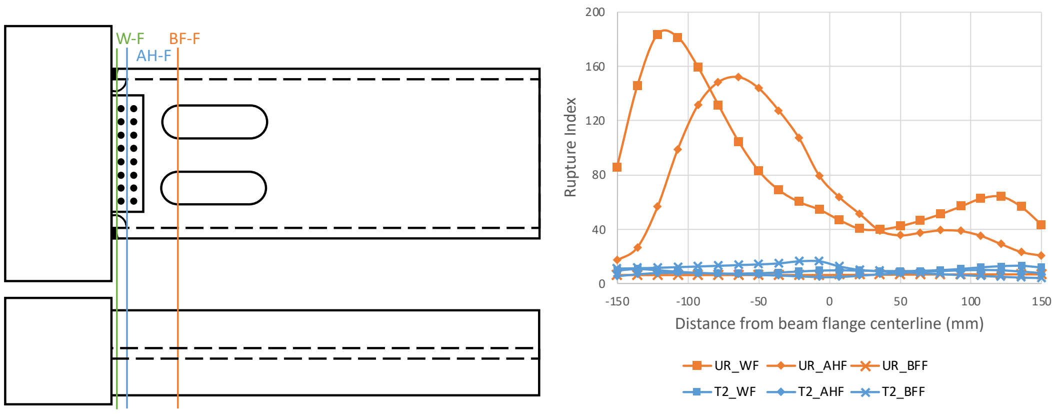

Fig. 5 Distribution of rupture index in the studied RWS connection (designated as T2, in blue) and comparison connection without beam-web openings (designated as UR, in blue) subjected to cyclic loading with the maximum drift angle of 4% rad

SIMULATION OF FIRE BEHAVIOR

Assumptions

Fig. 6 shows the location of fire source and boundary condition of the connection. The fire behavior of a steel connection also varies depending on the loading and boundary conditions. In this study, the fire source was assumed to locate below the bottom flange of the beam and near the column face of the connection. The surfaces of the beam flange and column web were directly heated. The boundary condition and constraints were set to be the same as those in the seismic test. Namely, the column was vertically placed and fixed at the both ends. The beam was horizontally placed and well braced without lateral torsion buckling. A uniformly distributed load of 1.96 kN/m was added in the top flange of the steel beam, as to consider the self-weight of the structure. The transient response of the steel connection was simulated using ANSYS Fluent and the standard temperature-time curve ISO 834.

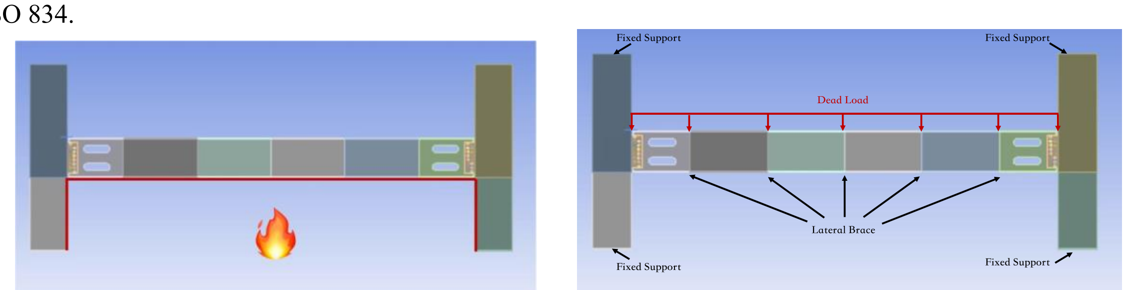

Fig. 6 Location of fire source and boundary condition of the connection

Analysis Results

Fig. 7 shows the distribution of equivalent plastic strain (PEEQ) in the two connections at elevated temperature of 800 ºC. Both the connections had the maximum plastic strain at the root of weld access hole in the bottom beam flange. For the studied RWS connection, greater strain occurred at the corners of the openings and wildely distributied from the column face to the the bottome beam flanges adjacent to the lower web-opening. For the comparison connection, in contrast, strain concentrated in the bottom beam flange near the column face.

Fig.8 shows the distribution of rupture index (RI) in the studied RWS connection (designated as T2, in blue) in comparison to the identical connection without beam-web openings (designated as UR, in orange) at elevated temperature of 800 ºC. The beam-web openings reduced the potential of connection fracture to a low level. In contrast, the identical connection without openings had a high risk of fracture at the root of weld access hole in the bottom beam flange.

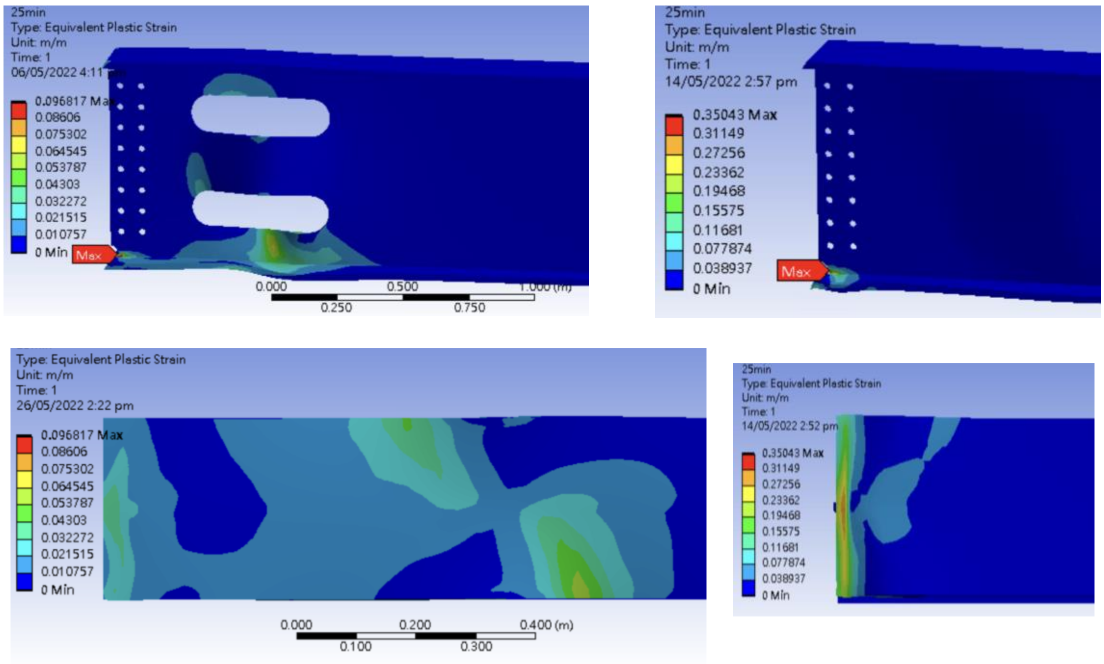

Fig. 7 PEEQ distribution of the studied RWS connection and the identical connection without openings

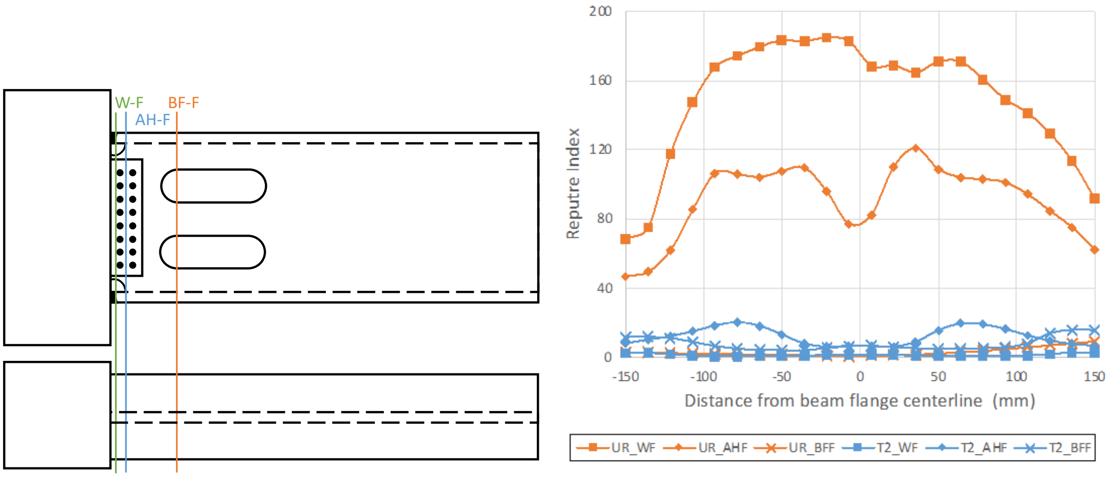

Fig. 8 Distribution of rupture index in the studied RWS connection (designated as T2, in blue) and comparison connection without beam-web openings (designated as UR, in blue) at elevated temperature of 800 ℃

Comparative Assessment

Table 1 lists the RI values of the RWS connection and identical connection without beam-web openings, under fire and subjected to earthquake shaking. The maximum RI value was 185 and 183.3 respectedly when the connection was under fire and subjected to earthquake shaking, respectively. It was previously mentioned that the comparison connection had carack initiation from the weld access hoel and finally underwent fracture in the beam flange near the column face (see Fig. 3). The similar RI values suggest that the 800ºC high temperature can increase the risk of connection fracture to the same level as that induced by the cyclic loading with a drift angle of 4% rad. In contrast, the maximum RI value was much lower for the studied RWS connection with two horizontal long openings, either when the connection was subjected to severe earthquake shaking, or when the connection was at the elevated tempeartature of 800ºC.

Table 1 Max RI values of the studied two connections subjected to cyclic loading and at elevated tempearture

Max RI values	Comparison Connection (UR)			RWS connection (T2)		
	WF	AHF	BFF	WF	AHF	BFF
At temperature of 800 ºC	185	121	9	3	13	16
At the drift angle of 4% rad	183	152	7	20	10	17

SUMMARY AND CONCLUSION

In this paper, the result is presented of a comparative study on fire and seismic behaviors of steel beam-to-column connections without and with beam-web openings. The results have illustrated the necessity of further studying the thermal effect, especially the constraints of high-temperature thermal expansion near the column face of the connection. In comparison with earthquake damage, fire damage hasn't received enough attention. But the 800ºC high temperature can increase the potential of connection fracture like the great earthauke shaking. The results have also shown the benefits of improving connection ductility. The studied RWS connection had better performance not only when subjected to earthauek shaking, but also when affected by a fire and high temperature. The relevant impact is recommended for further study, as to be accurately reflected in the structural design and earthquake retrofit of steel construction.

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REFERENCES

Chang, H. Y., Liao, C. T., Kang, S. Y., Ho, S. Y., and Lai, C. M. (2022), "Seismic performance of RWS moment connections to steel box-columns and H-beams with general sections," *Journal of Contructional Steel Research* (under re-review)

Hedayat, A.A., and Celikag, M. (2009) Post-Northridge connection with modified beam end configuration to enhance strength and ductility, *Journal of Construction Steel Research*, Vol. 65, 1413–1430.

AISC, *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, 2016.

AIJ, *Recommendation for Design of Connections in Steel Structures*, Architectural Institute of Japan Tokyo, Japan, 2012. (in Japanses)

Lai, C.M., Yeh, C.Y., Kang, S.Y., and Chang, H.Y. (2021). "Effects of shear tabs and high-strength bolts in seismic performance of steel moment connections," *Buildings*, 11, 415.

Boresi, A.P., Schemidit, R.J., Sidebottom, O.M. (1993). *Advanced Mechanics of Materials*, Wiley, United Kingdom.

El-Tawil, S., Mikesell,T., Kunnath, S. K. (2000). "Effect of local details and yield ratio on behavior of fr steel connections," *Journal of Structural Engineering*, ASCE, Vol. 126, 79–87.

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