Post-fire Seismic Performance of Beam to Concrete Filled Tube Column Joints

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Abstract— This paper presents the post-fire seismic behaviour of two types of moment resisting joints in multistory frames of concrete-filled high-strength rectangular hollow sections (RHS) columns and mild carbon steel beams. Two joint typologies: with reduced beam section (RBS) and with cover plates (CP) are studied. The seismic behaviour of these joints after exposure to the ISO-834 fire standard has been numerically investigated using finite element software ANSYS Workbench 16.1. temperature distribution, failure mode and moment-rotation curve are studied for both the joints. From the results obtained it was clear that beam column joint with cover plate gives better post-fire seismic performance.

Index Terms— Concrete filled tubes, Dual-steel structural system, Finite element analysis, High strength steel, Post-fire, Seismic performance, Temperature distribution, Welded beam-to-column joints

1 Introduction

Fires and earthquakes are accidental actions and are generally treated in a traditional single-objective design as independent events. Fire hazards continue to occur in civil engineering structures such as buildings, tunnels, and bridges because of various reasons and cause catastrophic consequences. In such scenarios, buildings get exposed to fire for a long period of time, resulting in damage of spray-on-fire resistive materials. Consequently, steel frame temperature may reach as high as the maximum fire temperature, which subsequently cools slowly to ambient temperature. If the building frame does not deform severely after such a fire event, it is usually rehabilitated for continued occupation. The performance of such a fire-exposed steel building during a future earthquake is not known [1].

A study by Wald et al. (2006) showed the development of large buckling at temperatures over 1,000°C; they also showed that the distortion of structural members not only depends on the peak temperature, but also on the fire exposure time. Most of the fire performance studies on structural members cited previously were generally performed on smaller beam and column section; consequently the beam column specimens distorted considerably at a relatively lower temperature range (600°C-800°C).

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Seismic-resistant building frames designed as dissipative structures must allow for development of plastic deformations in specific members, whose behavior is expected to be predicted and controlled by proper calculation and detailing. Members designed to remain elastic during an earthquake, such as columns, are characterized by high strength demands. Dual-steel structural systems, for which high-strength steel (HSS) is used in predominantly elastic members while mild carbon steel (MCS) is used in dissipative members, can be reliable and cost efficient.

Columns fabricated from steel hollow sections can be filled with concrete with the aim to combine the properties of the two materials, the final element being characterized by higher stiffness, strength, and ductility, as well as enhanced fire resistance in comparison with bare steel configurations. A particular advantage of the composite solution is related to the reduction of the column cross-sectional area, and by using steel tubes as permanent formwork, the construction speed is increased. It is to be highlighted that, regardless of the various advantages, a particular attention should be given to the joining aspect between beams and columns from the point of view of the connection technology and detailing. [5]

Concrete filled steel tube (CFST) members demonstrate better fire resistance than those with hollow steel tubes as the concrete infill can delay the temperature increase in the outer steel tube by helping to absorb the heat , and the composite action between core concrete and its steel tube can be partially maintained under high temperatures.[12].

Framed structures consisting of concrete filled steel tubular columns and steel beams have been widely used in modern construction. In framed structures, beam-column joints are critical elements for ensuring the load transfer among different components and maintaining the integrity of structural systems in fire[12]. Studies on post-fire seismic performance of such beam column joints are still very limited.

Vulcu et al. (2017) developed an experimental program on beam-to-column joints considering: full-strength and rigid joints, hot rolled MCS beams, cold-formed HSS rectangular hollow section columns filled with concrete, and two welded joint typologies: with reduced beam section (RBS), and with cover plates (CP). Furthermore, the connection solution of beams and columns, for both RBS and CP joints, was based on the use of stiffening plates welded around the steel tube in the shape of an external diaphragm. These joints are considered in the current study.

This paper investigates the post-fire seismic performance of welded beam-to-column joints, with columns realized as concrete-filled tubes (CFT) with rectangular hollow sections (RHS) with two joint typologies (RBS and CP).

2 Finite Element Analysis

2.1 General

In order to study the post fire seismic behavior of I beam to column joints with concrete filled rectangular hollow section (CF-RHS) column, three dimensional finite element models were modeled in ANSYS Workbench 16.1. Two joint typologies (RBS and CP) are analyzed in this study. The column length is 2.9m and beam length is 2.8 m from the centre of the column. Fig.1 and fig.2 shows details of joint configurations adapted in this study. Beam column joints consist of I beam, CF-RHS column, external diaphragm, cover plate and stiffeners.

The details of the reduced beam section are as follows: a=90mm, b=260mm, c=35mm, R=260mm (radius) shown in fig.1. The details of column and external diaphragm are bcp=500mm, bed=150mm, bc=300mm and bf=180mm. The section dimensions for IPE 400 beam are: height=400mm, flange width=180mm, web thickness=8.6mm and flange thickness=13.5mm. Fig.2. Shows details of connection with cover plate.

In these models high strength steel (S460) is used for column, external diaphragm and stiffeners. Mild steel (S355) is used for IPE400 beam and cover plate. C30/37 concrete is used to fill inside the RHS. Table 1 shows details of joint components considered in this study. In addition, tables 2, 3 and 4 present the input data for the plastic behavior of the steel parts from each joint assembly.

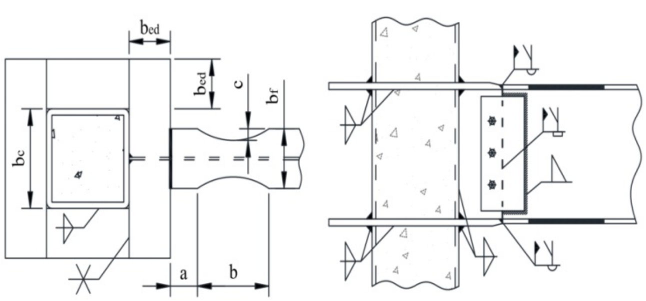

Fig. 1. RBS joint configuration

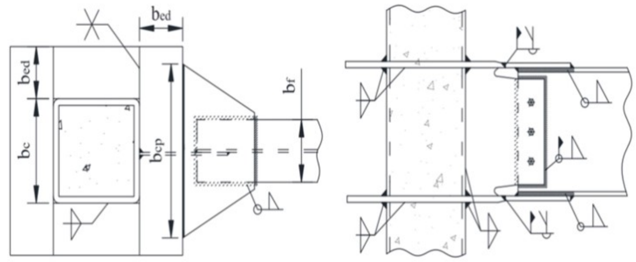

Fig. 2. CP joint configuration

TABLE 1
Details of Joint Components

Joint	Column-S460	Stiffeners-S460	Cover plate-S355	Beam
RBS	RHS 300x300x12.5mm	150x20mm	-	IPE 400-S355
CP	RHS 300x300x12.5mm	150x20mm	500x15mm	IPE 400-S355

TABLE 2
Stress Strain Values for Steel Parts

Stress Strain Pair no:	COLUMN (RHS)		IPE 400 BEAM	
	True Stress N/mm²	True Strain mm/mm	True Stress N/mm²	True Strain mm/mm
1	497.8	0	390.3	0
2	533.9	0.0087	402.6	0.0273
3	557.5	0.0219	465.4	0.0477
4	569.7	0.0365	503.9	0.0702
5	593.8	0.0679	567.7	0.1386
6	619.8	0.1139	590.2	0.1804
7	775	0.4	710	0.4
8	780	0.6	825	0.65
9	790	1	850	0.95

TABLE 3
STRESS STRAIN VALUES FOR STEEL PARTS

Stress Strain Pair no:	COVER PLATES		STIFFENER	
	True Stress N/mm²	True Strain mm/mm	True Stress N/mm²	True Strain mm/mm
1	414.7	0	467.2	0
2	437.7	0.0393	481.8	0.0197
3	462.6	0.0481	519.3	0.0237
4	501.1	0.0688	584	0.041
5	548.4	0.1164	680.3	0.0939
6	564.1	0.139	709.6	0.1272
7	572.6	0.1532	755	0.2
8	665	0.313	890	0.4
9	775	0.65	965	0.8

TABLE 4
STRESS STRAIN VALUES FOR STEEL PARTS

Stress Strain Pair no:	EXTERNAL DIAPHRAGM	
	True Stress N/mm²	True Strain mm/mm
1	451	0
2	459	0.0154
3	494.2	0.021
4	564.6	0.0377
5	595.6	0.048
6	643.2	0.0705
7	694.6	0.1148
8	716.4	0.1464
9	755	0.2

2.2 Material Properties and Thermal Parameters

Table 5 shows material properties and thermal properties of the steel and concrete respectively.

TABLE 5
MATERIAL PROPERTIES AND THERMAL PROPERTIES

Material	Steel	Concrete
Young's modulus	E=21000MPa	E=32837MPa
Poisson's ratio	0.3	0.2
Density	7850kg/m³	2500kg/m³
Specific heat	Shown in Fig.3.	Shown in Fig.5.
Conductivity	Shown in Fig.4.	Shown in Fig.6.
Coefficient of linear thermal expansion	12x10-6	10x10-6

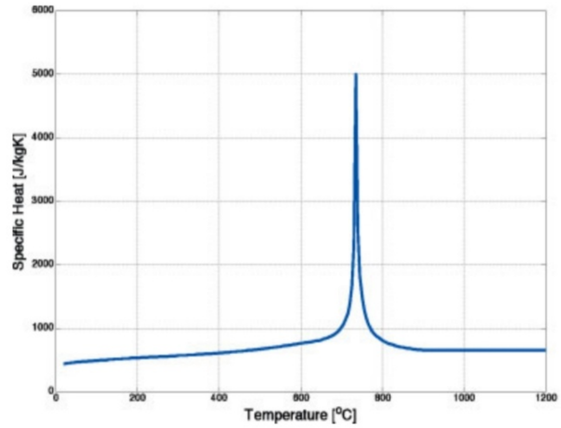

Fig. 3. Specific heat of steel

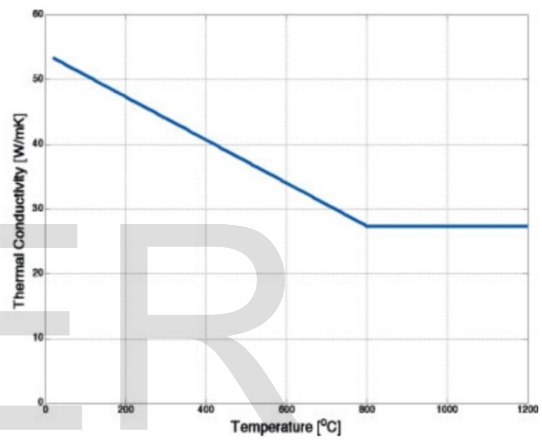

Fig. 4. Thermal conductivity of steel

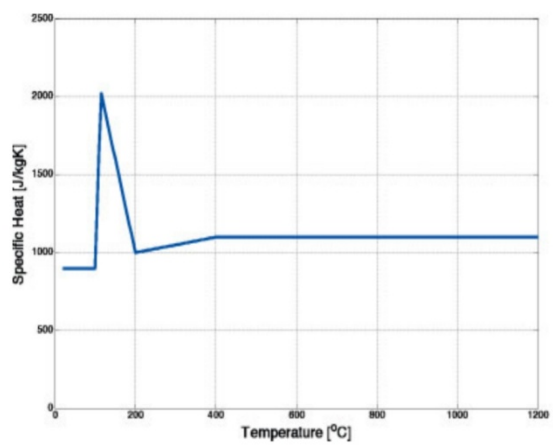

Fig. 5. Specific heat of concrete

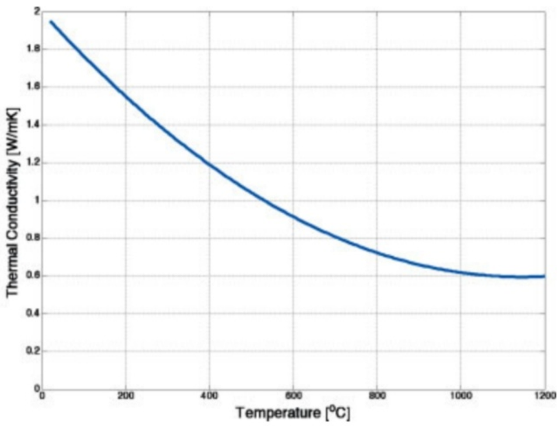

Fig. 6. Thermal conductivity of concrete

2.3 Modeling and Analysis

In this study, there were two analysis steps involved in the numerical simulation- thermal analysis and thermo mechanical structural analysis-and both steps used the same element meshing and node numbering.
In thermal analysis, the elements in the model are discretized using SOLID 90. It is a higher order version of the 3-D eight node thermal element. The element has 20 nodes with a single degree of freedom, temperature, at each node. The 20-node elements have compatible temperature shapes and are well suited to model curved boundaries. The 20-node thermal element is applicable to a three-dimensional, steady-state or transient thermal analysis.
Subsequent to thermal analysis, thermo mechanical structural analysis was performed by changing the element types from thermal solids (SOLID90) to structural solids (SOLID186). SOLID 186 is a higher order 3-D 20-node solid element. The element is defined by 20 nodes having three degrees of freedom per node: translations in the nodal x, y and z directions. The element supports plasticity, hyper elasticity, creep, stress stiffening, large deflections and large strain capabilities. Fig.7 and fig.8 shows the ANSYS models of the two joint typologies (RBS and CP).

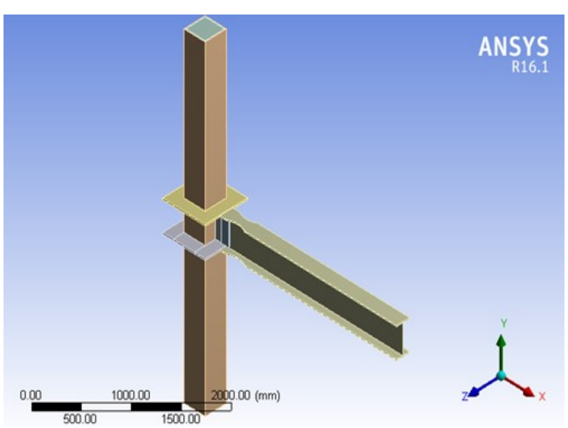

Fig. 7. ANSYS model of RBS joint

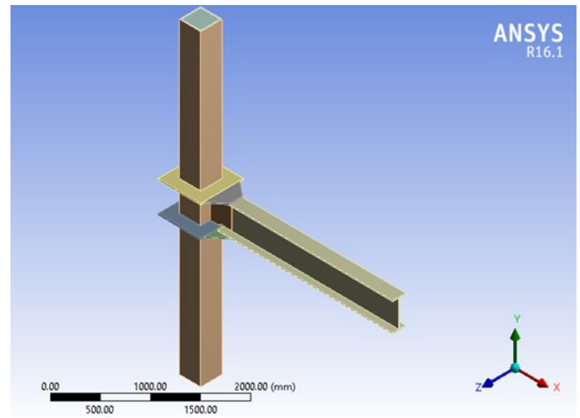

Fig. 8. ANSYS model of joint with CP

2.4 Thermal Analysis

The standard ISO 834 fire curve was applied to the lower compartment of the beam column joint as thermal load, through convection heat transfer mechanism. Bottom flange of the beam and column face below the beam were exposed to fire. The fire curve drawn based on the details given in table 6.

TABLE 6
STANDARD FIRE TIME TEMPERATURE CURVE

TIME (sec)	TEMPERATURE °C
0	22
600	500
900	700
1200	780
2400	900
3600	950
4800	1000
6000	1050
7200	1075
8400	1100

2.5 Thermo Mechanical Structural Analysis

Structural analysis of beam column joint is done by using static structural in ANSYS. The column was supported at both ends using pinned connections. In this analysis, temperature history determined from the thermal analysis was prescribed as the input load. Cyclic loading was applied at the tip of the beam. A smooth cyclic loading pattern was used, which was characterized by one cycle for each of the following amplitudes: 10, 20, 30 ,40 ,50 and 60 mrad. The loading conditions for cyclic loading are given in table 7. Cyclic loading pattern is shown in fig. 9.

TABLE 7
CYCLIC LOADING CONDITIONS

Cyclic loading	
Rotation (mrad)	Displacement (mm)
10	28
20	56
30	84
40	112
50	140
60	168

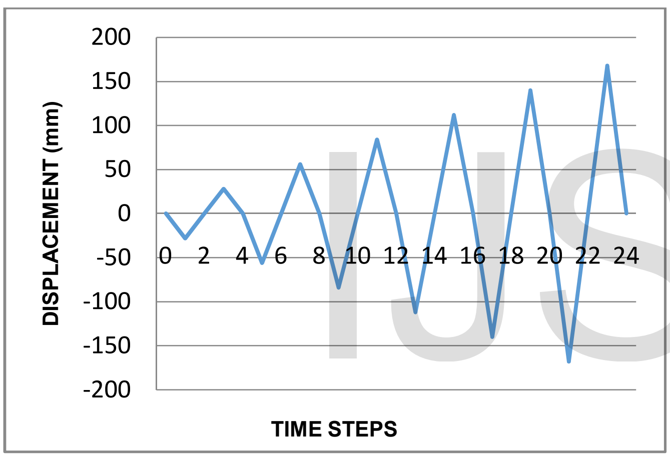

Fig. 9. Cyclic loading pattern

3 RESULTS AND DISCUSSIONS

3.1 General

The main objective of this research work is to understand the post fire seismic response of two types of moment resisting MCS beam to CF-RHS column joints. Temperature distribution, failure mode and moment-rotation curve of the beam column joints are studied.

3.2 Temperature Distribution

The temperature distribution of beam column joint with RBS is shown in fig.10. The maximum temperature at 8400s is 1027.7°C and minimum is 22°C. The temperature distribution of beam column joint with CP is shown in fig.11. The maximum temperature at 8400s is 1032.7°C and minimum is 22°C. For both the joints, the maximum temperature is distributed at the bottom flange of the beam and at the interior face of the lower column and the minimum temperature is distributed at the top flange of the beam and at the upper column. This is expected because the bottom flange of beam was directly exposed to 1100°C, whereas the top flange remained at the ambient temperature.

3.3 Failure Mode

Fig.12. and fig.13. show the accumulated equivalent plastic strain contour of RBS and CP joints respectively. In case of joint with RBS connection, yielding was initiated in the beam flanges within the RBS zone which was followed by large plastic deformations-local buckling of flanges and web. In case of joint with CP connection, yielding was initiated in beam flanges near cover plate which was followed by local buckling of flanges and web.

In both the joints, the beam-column assemblies developed significant buckling in the beam flanges and web, which resulted in out-of-plane deformation of the beam. No damage was observed in the external diaphragm, cover plate and column panel.

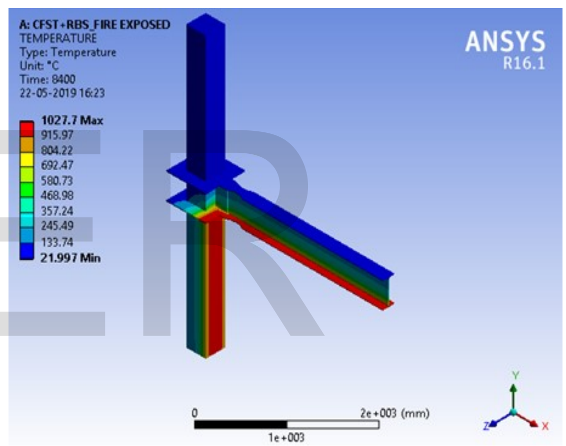

Fig. 10. Temperature distribution in RBS joint

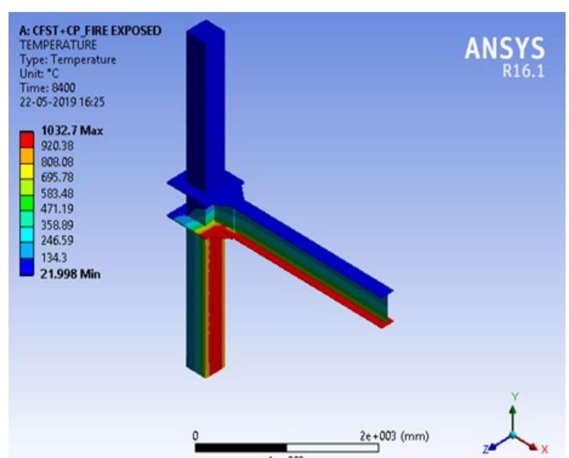

Fig. 11. Temperature distribution in CP joint

3.4 Moment Rotation Curve

Moment-rotation curve of joints with RBS and CP are shown in fig.14. and fig.15. respectively. Fire exposure changes material properties of the connection to be heterogeneous, which leads to an unsymmetrical moment-rotation hysteresis response. Maximum moment obtained is 592.64kNm in RBS joint and 712.57kNm in CP joint. Comparing these magnitudes of moment, it can be observed that the moment capacity of CP joint is 20.23% more than that of RBS joint.

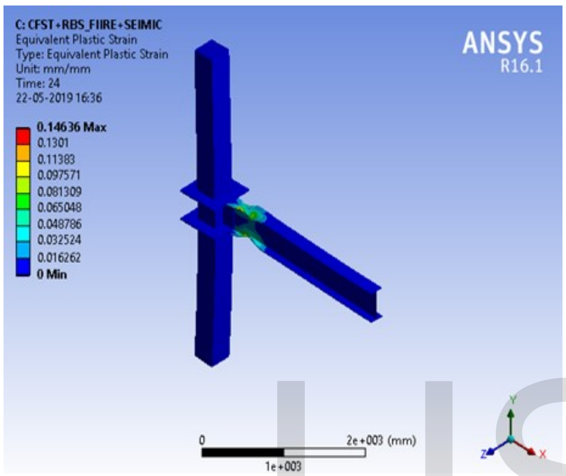

Fig. 12. Equivalent plastic strain contour of RBS joint

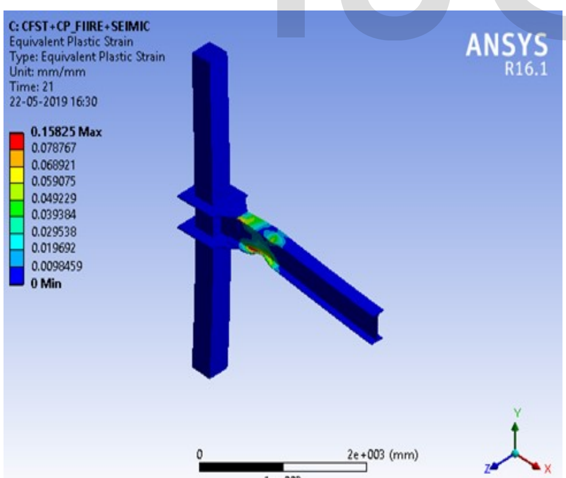

Fig. 13. Equivalent plastic strain contour of CP joint

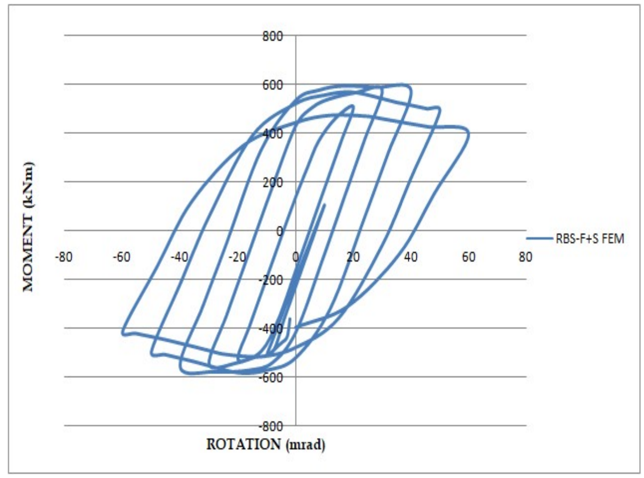

Fig. 14. Moment-rotation curve of RBS joint

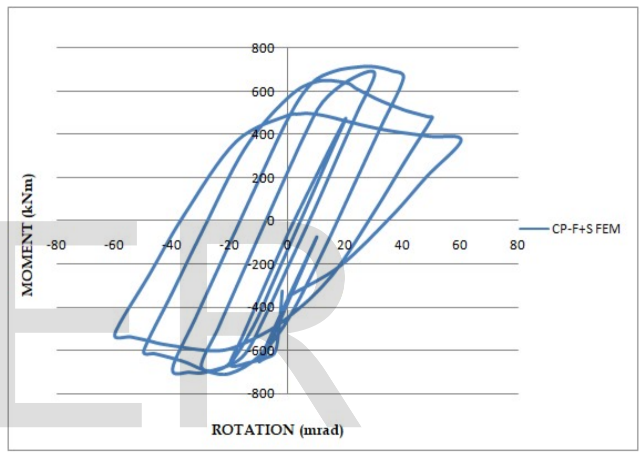

Fig. 15. Moment-rotation curve of CP joint

4 Conclusion

This study provides data on post-fire seismic behavior of welded steel I beam to concrete filled RHS column with two joint typologies (RBS and CP). The following observations and conclusions can be drawn based on the limited research reported in this paper.

1) For both the joints, the maximum temperature is distributed at the bottom flange of the beam and at the interior face of the lower column after the application of ISO 834 fire curve.
2) For RBS and CP joints, main plastic deformations occurred in the beam. The beam-column assemblies developed significant buckling in the beam flanges and web, which resulted in out-of-plane deformation of the beam.
3) After the application of cyclic load subsequent to fire, no specific damage was observed in external diaphragm, cover plate and column panel.
4) Moment capacity of CP joint is 20.23% more than that of RBS joint which indicates better post fire seismic performance of CP joint.

5 References

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