Seismic performance of fire exposed steel welded WUF-W and RBS connections

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Abstract: Strength properties of structural steel are reduced when exposed to fire above temperature 600\(^{\circ}\)C. A widely used general rule is that if the steel members in the building frame are reasonably straight with no visible distortion after a fire event, it can be reused. If these members are accepted for reuse with minor or no repairs, then the performance of steel members with a high-temperature fire exposure under a future earthquake event should be known. This study investigated the cyclic response of fire exposed steel Welded Unreinforced Flange Web (WUF-W) and Reduced Beam Section (RBS) welded connections. The results showed that fire exposed steel connections are vulnerable to earthquake loading if they are reused with minor or no repairs. When the performance of connections was compared, it was understood that the usage of RBS connection provides high safety, whereas WUF-W connection has the high moment capacity and energy dissipation capacity.

1. Introduction
Steel is one of the commonly used building materials. Exposure to high temperatures, as in fire conditions can bring about significant effects like expansion, change in microstructure and lowered elastic and yield strength of steel. Steel buildings may sustain various degrees of damage without collapse. Examples of major steel building fires without collapse are 1991 Credit Insurance Building Fire in Churchill, UK; the 1991 One Meridian Plaza fire in Philadelphia,[10] etc. The initial inspection and assessment of any fire damaged steel structure is visual. This is followed by measurements of the out-of-straightness of linear elements, particularly those that may subject to compression. In the cases of members that are severely deformed, it may be necessary to seek the advice of a metallurgist. However, the damaged structures with deformations within the out-of-straightness limits can be reused without further checking [2]. Even though the deformations may be smaller the strength and stiffness of the steel members may be less due to the fire exposure. The strength degradation of the steel depends on the peak temperature and exposure conditions. Earthquake and fire is a deadly combination and there are various instances where these two occurred and caused destruction. The fires result in strength and stiffness reduction in structural members due to elevated temperatures, which can further exacerbate the seismic damage. It is not known how such a fire surviving heterogeneous steel member will perform under a future earthquake. If the structural elements in the building are reused after a fire event, then the residual load capacity of the fire exposed structure should be known.
The performance of structures in fire can be quantified by “fire resistance” [7] and is measured using standard fire tests. [5] analyzed the high temperature properties of steel for fire resistance modelling of structures. Thermal analysis helps to identify the temperature distribution in the body due to fire exposure. The analyses were based on ASCE manual, Eurocode and Poh models. The results concluded that analysis based on Eurocode predictions came closer to the test results. [8] conducted a numerical analysis of reinforced concrete (RC) beam when exposed to fire using ABAQUS. Temperature dependent properties of concrete and steel and ISO834 curve were given as the input data. [12] conducted an experimental study of residual stress on welded high strength steel (>460 MPa) H shaped section after fire exposure. The finite element package ANSYS was used for the numerical analysis and thermo mechanical analysis method was employed in the software. [6] conducted a study to analyze the stability of steel columns subjected to earthquake and fire loads. A nonlinear stability formulation was developed to assess the response of steel columns under the sequential demand of earthquake and fire loads. The researchers concluded that the inclusion of interstorey drift with uniform longitudinal temperature resulted in the significant reduction in buckling capacity of the columns. [10] investigated the seismic performance of fire exposed steel frame by performing finite element analysis incorporating fire exposed steel material properties using ANSYS software. The frame distortion analysis with a peak fire temperature of 1,000°C revealed that heavy steel frames do not experience significant visible distortion and can be reused. The study demonstrated that fire damaged steel forms soft-storey mechanism under seismic loading.

Steel moment frames are considered to be highly ductile and thus have been used as a major structural system in high-seismic regions. However, the Northridge earthquake in 1994 and the Kobe earthquake in 1995 revealed that brittle failure might occur at beam–column joints or in pre-Northridge connections. In this background after the Northridge earthquake, the Steel Analysis Code (SAC) Joint Venture supported by the Federal Emergency Management Agency (FEMA) in the US conducted research on seismic performance of steel moment frames, and published a series of reports such as FEMA 350 and FEMA 351, which provide feasible information for seismic design of steel beam–column connections. The modified pre-Northridge connections are now referred as post-Northridge connections. In this study the seismic performance of fire exposed post Northridge connections; the WUF-W and RBS connections are investigated. Also, this study investigated the influence of strength degradation of steel due to fire exposure on the seismic performance in case of reuse of fire exposed steel.

2. Methodology
ANSYS version 19 – finite element software was used for the numerical assessment. The connections were analyzed with and without fire exposure to study the influence of fire.

2.1. Design of Connections
The weld access hole, shear tab and radius cut for the RBS were designed according to the recommended seismic design provisions of [3] and [1]. The connections were made up of ASTM A992 steel. The connections were given with same material properties, beam spans and beam and column sections so as to make the comparisons possible.

Figure 1 shows the reference special moment frame with the exterior connection and the connection designed for this moment resisting frame. The support conditions are also shown in the figure. The exterior connection is either WUF-W or RBS. The exterior connections are constructed with beam section W36 X150 and column section W14 X 257.
2.2. Finite element modelling

A two dimensional model (2-D) was developed for the connections using the commercial finite element software ANSYS. The model was discretized by 2-D element shell 181 and the mesh size was selected based on the mesh convergence study and an optimized mesh size was chosen. The connections are made up of ASTM A992 steel. The multilinear kinematic hardening model was used in the finite element study. The temperature dependent properties of ASTM A992 steel were collected from [9]. The temperature dependent stress strain curve of the steel was taken from [4]. The properties of steel include Density, Conductivity, Specific heat, Coefficient of thermal expansion, Poisson’s ratio and Young’s Modulus of Elasticity.
In this study, both welding and geometrical imperfections were neglected. To simplify modelling and numerical computation time, the welds in the connections were not modelled since it is not having any effects on the global performance of the connections. Since welding was not considered, the effects of initial residual stress were also ignored. Geometrical nonlinearities are not going to have any effect on non-linear static or dynamic responses and hence neglected in the study. To simplify the modelling the contacts considered near the beam and column connections were bonded contact type. Bonded contact is a linear type of surface contact provided by the software.

2.3. Fire analysis of the connections
The thermal analysis of the connections was done by applying a fire time-temperature curve representative of a real fire scenario taken from [11] and is given in figure 3. It is selected because the curve represents both the cooling and heating phases while the traditional fire curves like ASTM A119 and ISO-834 curves only have the heating phases. The fire curve was applied inside the column flanges and beam bottom flanges for both connections and is shown in figure 3. For the ease of computation, steel protection coating is not used in the study. The connection is directly exposed to fire for 3-4 hours and the protective coating will fail by this time. Hence there is no need to model the protective coating since they are not going to have any effect on the global response of the connections. It is assumed that the steel was air cooled after the heating phase. The air cooling in the thermal analysis was simulated by allowing the free heat transfer through convection and radiation. This simulation was done by using appropriate heat transfer coefficients.

2.4. Seismic analysis of the connections
After the thermal analysis of the connections, thermo mechanical analysis was done using the finite element general purpose software ANSYS. The temperature history or distribution from the thermal analysis was imported and applied as the thermal load in the subsequent mechanical analysis. Along with the thermal load, the simulated cyclic load was also applied at the free end of the half beam. Both the ends of the column of the connection are assumed to be fixed supports and are shown in figure 1.

A cyclic vertical displacement history is taken from [3] and is shown in figure 4. The graph is the number of cycles (N) versus story drift angle (θ). Storey drift angle is obtained by dividing the displacement at the beam end (ΔCL) by the distance from the loading point to the column centreline (LCL) as indicated in the figure 4. Therefore, required displacement values at the loading point are computed by multiplying θ by LCL. The displacement controlled quasistatic loading protocol includes

![Figure 3. Time–temperature curve and fire curve applied in the connection](image-url)

a)WUF-W b)RBS
six cycles of 0.375\%, 0.5\%, 0.75\% storey drift followed by 4 cycles of 1\% drift and two cycles each of 1.5\%, 2\%, 3\%, 4\%, 5\%, 6\% and 8\% drifts.

![Figure 4. Vertical Displacement (a) cyclic loading history (b) angular rotation](image)

3. Results and discussions

The WUF-W and RBS connections were analysed with or without fire under a simulated cyclic vertical displacement. First the mechanical analysis of the connections was done (without any fire) and then thermo mechanical analysis of the connections (with fire) was done. The temperature distributions, moment rotation responses, plastic strain along the beam flange and the failure mode of the connections were compared for with fire and without fire cases.

3.1. Temperature distribution in the connection

The temperature distribution in the body is the result obtained in the thermal analysis. The temperature distribution after the fire exposure for 50 minutes for both the connections is shown in figure 5. It is clear from the picture that interior column flanges and beam bottom flanges are having high temperature since they are directly exposed to fire. From the analysis results, it can be understood that the temperature decreases from the bottom flange to the upper flange for the beam. The temperature ranges from 1000\(^{\circ}\)C at the bottom flange to the room temperature at the top flange and at the mid depth of beam temperature reaches about 450\(^{\circ}\)C. In the column also temperature decreases from inner flange to the outer flange. Temperature ranges from 1000 \(^{\circ}\)C to 500\(^{\circ}\)C at the mid depth. The same pattern of temperature distribution is obtained for both the connections. In the fire cases, based on the temperature distribution in the body the material properties and corresponding distortions in the body are computed in the subsequent seismic analysis phase.

![Figure 5. Temperature distribution in the connection a) WUF-W b) RBS](image)
3.2. Moment rotation hysteresis response

Moment at the face of the column was found out and moment for each drift was plotted in the hysteresis response. Area of the hysteresis response gives the energy dissipated in the connection. Figure 6 shows the moment rotation hysteresis response of the fire exposed steel WUF-W connection and the steel connection without fire. The moment rotation hysteresis response of the steel connection without fire shows that the strength degradation of the steel occurs after 2% drift and continued with the loading cycles. After 2% drift the decrease in peak moments is 10% for each loading cycle under positive bending and the strength degradation is 10% for each loading cycle after 4% drift under negative bending.

For the fire exposed steel the mechanical strength degradation started after 1.5% drift and is clear from the curve. The steel connection with fire exposure is having a moment capacity 23% less than that of the steel connection without fire. A symmetrical moment rotation hysteresis curve is obtained in this case. However strength degradation after 1.5% drift is 22% for each loading cycle under both positive and negative bending.

![Figure 6. Moment rotation hysteresis response WUF-W connection](image)

For the fire exposed RBS connection peak moment capacity is 48% less than that of the steel connection with no fire exposure. The decrease in peak moment under positive bending was by 20%, 34%, 44%, 49% at 3%, 4%, 5%, 6% drifts respectively after 2% drift. The strength degradation at 4% drift for the fire exposed steel was very larger compared to the value in steel connection without fire exposure. Material strength deterioration and distorted curve was very significant in the hysteresis curve for the fire exposed condition. By comparing both the curves, it is clear that the steel connection without fire exposure dissipates more energy than the fire exposed steel under simulated cyclic loading.
Figure 7. Moment rotation hysteresis response RBS connection
a) without fire exposure  b) fire exposed steel

ANSI/AISC 358-16 [1] accepts mechanical strength degradation of 20% up to 4% drift for the connections. WUF-W and RBS connections without fire exposure achieve the seismic provision requirements for special moment frames since the strength degradation was less than 20%. But the fire exposed steel connections exhibited strength degradation more than 20% and they have failed to pass the recommended criteria. These seismic analyses explain the vulnerability of fire exposed steel under seismic situations.

3.3. Plastic strain along the beam flange

Figure 8 shows the equivalent plastic strain values at the beam bottom flange along the beam length of the WUF-W connection. This was plotted to investigate the location of plastic hinge zone. It is observed that the plastic hinge location for WUF-W connection occurs near the column face in both the cases (with and without fire). For the fire exposed steel maximum plastic strain value is 26% higher than that of the steel connection with no fire exposure at 6% drift. Considering the safety aspect, it is not desirable to have high plastic strain values.

Figure 9 depicts the equivalent plastic strain values at the beam bottom flange along the beam length of the RBS connection. The maximum equivalent plastic strain value of the fire exposed steel is 20% higher than that of the connection with no fire at 6% drift. The plastic hinge is formed along the radius cut of the RBS connection in both cases. The plastic hinge formation takes place away from the column face for radius cut RBS connection.

While comparing the WUF-W and RBS connections it is observed that the chances of local buckling of beam are high in RBS connection. The plastic hinge location for WUF-W and RBS connections occur approximately 90 mm and 390 mm away from the column flange. In WUF-W connection the plastic hinge occurs at the column root which can lead to the failure of column later. The same trend was observed for the fire exposed steel connections also. However the plastic strain values were higher in fire exposed condition because of the material strength degradation.
3.4. Failure mode of the connections
Failure mode in the deformed shape of the WUF-W and RBS connection without fire exposure at the end of 6% drift is shown in figure 10. The same pattern of failure mode is observed in the fire exposed steel also. Failure of the connection occurred in the beam due to the local buckling of beam flanges and web. It can be seen that the failure of the WUF-W connection occurred near the column face and in the RBS connection buckling occurred in the radius cut of the beam.
4. Conclusion
Post earthquake forensics reveals that earthquakes often result in fire breakout. Temperature has a bearing on the properties of steel. Since a large amount of structural steel is reused in construction, it is essential to investigate the effects of fire exposure on the performance of structural steel. In this study, fire exposed WUF-W and RBS steel connections were analysed to assess the cyclic performance of fire exposed post-Northridge welded connections.

The key conclusions drawn from the numerical studies are summarized below:

- For the fire exposed RBS connection peak moment capacity was 48% less than that of the steel connection with no fire exposure, whereas in WUF-W connection the reduction was only 23%. From the moment-rotation response, it is clear that the moment capacity of the fire exposed steel is less when compared to the steel connections without any fire exposure. This indicates the strength degradation of the structural steel.
- The connections without any fire exposure managed to go beyond 4% drift without more than 20% strength degradation. So these connections passed the seismic provision requirements as per ANSI/AISC 358-16 [1]. However, fire exposed steel connections failed to meet the codal requirements.
- Failure of the investigated connections occurred in the beam because of the local buckling of web and flanges for both fire exposed and connections without any fire exposure.
- It should be mentioned that the plastic hinge was formed in the radius cut for RBS connections. That is, in RBS connections the plastic hinge formation takes place far from the column face. Whereas in WUF-W connection plastic strain formation started from the root of the column face. The pattern of plastic hinge formation mentioned above was same for connections with and without fire exposure. However, the value of plastic strain was high in fire exposed connections.
- It is observed that WUF-W connection is having better performance than RBS connection in terms of moment capacity, whereas the usage of RBS connection provides more safety in connection.

The results from the analysis of the connections showed that the fire exposed steel connections with heavy beam-column sections are vulnerable to earthquake loading if they are being reused after minor or no repairs. Thus, it can be concluded that the fire exposed steel connections with heavy beam-column sections can only be reused after proper repair works.

References
[1] ANSI/AISC 358-16 2016 Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications (Chicago: American Institute of Steel Construction).
[2] BS 5950 Part 82000 Code of Practice for Fire Resistance Design (London: British Standard Institution).

[3] FEMA – 350 2000 Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings (Washington, D.C.: Federal Emergency Management Agency).

[4] Hu G, Morovat M A, Lee J and Engelhardt M D 2009 Elevated Temperature Properties of ASTM A992 Steel Proceedings of the 2009 structures congress – Don’t mess with structural engineers: expanding our role (Austin, TX, United States) 1067-76.

[5] Kodur V, Dwaikat M and Fike R 2010 High-temperature properties of steel for fire resistance modelling of structures Journal of Materials in Civil Engineering 22 423-434.

[6] Memari M, Mahmoud H and Ellingwood B 2018 Stability of steel columns subjected to earthquake and fire loads Journal of Structural Engineering 144.

[7] Mostafaei H, Sultan M A and Benichou N 2009 Recent developments on structural fire performance engineering – state-of-the art report (Canada: National Research Council).

[8] Patade H K and Chakrabarti M A 2013 Thermal stress analysis of beam subjected to fire International Journal of Research and Applications 3 420-424.

[9] Quayyum S and Hassan T 2017 Initial residual stresses in hot-rolled wide-flange shapes: a computational technique and influence on structural performances Journal of Structural Engineering 143.

[10] Quayyum S and Hassan T 2018 Seismic performance of a fire exposed moment resisting frame, Journal of Structural Engineering 144.

[11] Quiel S E and Garlock M E M 2008 Modeling High-Rise Steel Framed Buildings Under Fire, Structures Congress (Vancouver, British Columbia- Canada).

[12] Wang W Y, Li G Q and Ge Y 2015 Residual stress study on welded section of high strength Q460 steel after fire exposure Advanced Steel Construction 11 150-164.