Comparative Study on Seismic Performance of Fire Exposed Steel Structure Under Protected & Unprotected Condition

Anju Ani, Ardra P Nair, Dr.C.Justine Jose

Abstract— Most of the earthquakes in urban areas can cause fire.Such fire can deterio- rate the structure of the building and can make it in repairable, if exposed for a long period of time.During a fire event,the temperature exposed is different along the depth of the beam and column. If the building frame does not deform severely after such a fire event, it is usually rehabilitated for continued occupation. This study investigates the comparative study of seismic performance of a fire-exposed steel frame under protected and unprotected coating by performing finite-element anal- yses incorporating fire-exposed steel material properties. The simulation responses demonstrate vulnerability of fire-exposed steel buildings under seismic loading. Re- sults show that the protected floors offer a higher fire resistance as the temper- ature of the steel section remains within 600C even after 60-minute standard fire exposure. Lower temperatures in steel result in lesser reductions of strength and stiffness, hence, the protected floors undergo lesser deflections and offer higher fire resistance.

Index Terms— Earthquake, Steel, Fire, CFT, In-tumescent Coating, Load Degradation, Soft Storey Mechanism

1 INTRODUCTION

Most of the earthquakes in urban areas can cause fire. As reported by Scawthorn et al.,most of the damage occured during earthquake were by fire. When an earth quake occurs, it may cause structural and fire protection damage, and thus the build- ing is more vulnerable to a fire. In the same time, after an earthquake, the loss of water supply or the low water pressure, combined with multiple independent fires, traffic congestion and the limited resources which are not able to respond promptly to all fires, allow to the fire to spread.These fires result in strength and stiffness reductions in structural members due to elevated temperatures. Steel buildings under fire may sustain various degrees of structural damage without collapse. When a steel structure survives a fire event, reparability and reuse of the structure depend on the extent of structural damage. Even in case of a latter post-earthquake fire , the effect on the structure must be adequately taken into account, since the earthquake-induced damage makes the structure more vulnerable to fire effects than the undamaged one. Modern seismic design accept a certain level of damage of the non-structural and structural members of a steel structure, often non-visible after an earthquake, and only an appropriate post-earthquake expertise may evaluate this aspect.The steel structural members that experienced temperatures over 600 C start to deform noticeable. A study by Wald et al. showed the development of large buck- ling 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.

1.1 ASTM A992 STEEL

The A992 Specification covers W shapes (rolled wide flange shapes) in- tended for use in building framing. The major advantage of A992 is its better material definition. It has an upper limit on yield strength of 65 ksi, a minimum tensile strength of 65 ksi, a specified maximum yield-to-tensile ratio of 0.85 and a specified maximum carbon equivalent of 0.47.

1.1.1 Material Property Degradation Of Fire-Exposed Astm A992 Steel

During a fire event,the temperature exposed is different along the depth of the beam and column. Consequently, upon cooling, material properties of beam and column sections at different locations will be different depending on the subsequent cooling process: slow air cooled or water quenched. A literature review suggests that there are very little data available on the effects of heating and cooling on the mechanical properties of ASTM A992 steel.

1.2 POST EARTHQUAKE FIRE

The possibility of fires following an earthquake has attracted considerable attention from many researchers for over a decade. Scawtorn et al. in his studies of the 1906 San Francisco earthquake and the 1923 Tokyo earthquake, showed that in about 80 of cases, damage was due to the fires following the earthquake rather than the earthquake itself. This phenomenon leads to substantial loss of human life and damage to urban infrastructure and facilities. There are various factors that can cause fires after an earthquake. Prior to the fire, the active and passive fire resistance system may be damaged by the earthquake. The probability of ignition is high because of toppled furniture, electrical malfunction, movement of hot equipment, and damage to fireproofing systems in buildings such as sprinklers and vertical pipes. This may pose a serious threat to the structural integrity of buildings, and be detrimental to the life safety of the occupants and rescue workers.

There are many factors that could lead to a fire to go out of control. Ground shaking due to a strong earthquake may cause severe damage to buildings and in- frastructure such as, roads, bridges, and life-line systems. A hot dry and windy weather help in the spread of fire, while damaged communication and transportation systems can limit access of fire fighting in the disaster area; and the damage of water supply system can limit the fire control measures. In that case, more effort and extinguisher material like water to control the fire. Therefore, it is essential to set up fire safety objectives for buildings and urban design. It is also necessary to ensure the structural integrity of the affected buildings for certain duration of a PEF event that the emergency resources can be availed.

1.3 IN-TUMESCENT COATING

An in-tumescent is a substance that swells as a result of heat exposure, thus increasing in volume and decreasing in density. In-tumescents are typically used in passive fire protection and require listing, approval and compliance in their installed configurations in order to comply with the national building codes and laws.

2 THERMO-STRUCTURAL ANALYSIS FOR UNPROTECTED CONNECTION

A Beam-to-CFT High Strength Joints with External diaphragm is considered.Finite element analysis four four fire cases are considered and worked out.

2.1 Modal Geometry

We have used the model suggested by Vulcu.The material model was as- sumed to be characterized by a combined isotropic kinematic hardening method. Beam IPE400 was considered.

TABLE 1
BEAM DIMENSIONS

Beam	fy	Ci	ri	Q	b
Beam Flange	390.3	42,096	594.45	60	9.71
Beam Web	423.1	42,096	594.45	60	9.71

TABLE 2
PROPERTIES OF STEEL

1	Density	7850 kgm−3
2	Young's Modulus	2e+11 Pa
3	Poisson Ratio	0.3

TABLE 3
PROPERTIES OF CONCRETE

Density	25000 kgm−3
Young's Modulus	32837 Pa
Poisson Ratio	0.2
Tensile Ultimate Strength	0.2
Compressive Ultimate Strength	0.2

2.2 Cases considered for study

Case I	Without Fire Exposure
Case II	With Fire Exposure on Bottom Left
Case III	With Fire Exposure on Bottom Right
Case IV	With Fire Exposure on top Left
Case V	With Fire Exposure on top Right

2.3 Thermo-structural analysis

2.3.1 Case I Without Fire Exposure

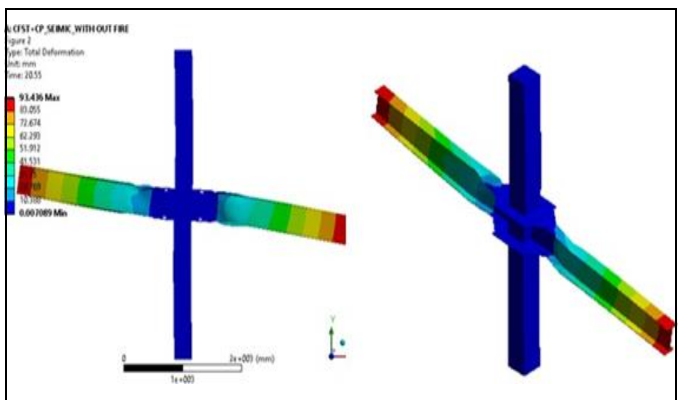

Figure 1: Total Deformation of Joint under without fire exposure

- *Anju Ani is currently pursuing masters degree program in structural engineering in VAST, Thrissur, India. E-mail: parveenhamzapj@mail.com*
- *Ardra P Nair is currently Assistant professor in department of civil engineering in VAST, Thrissur, India.*
- *Dr.C. Justine Jose is currently Professor in department of civil engineering in VAST, Thrissur, India.*

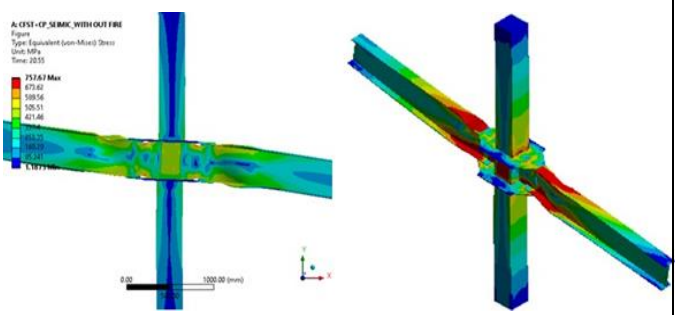

Figure 2: Equivalent Stress under without fire exposure

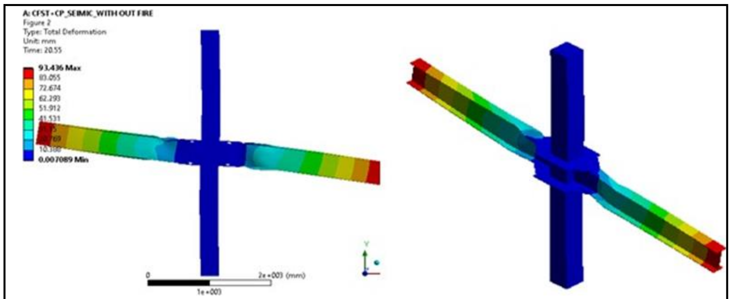

Figure 5: Total Deformation of Joint under with fire exposure

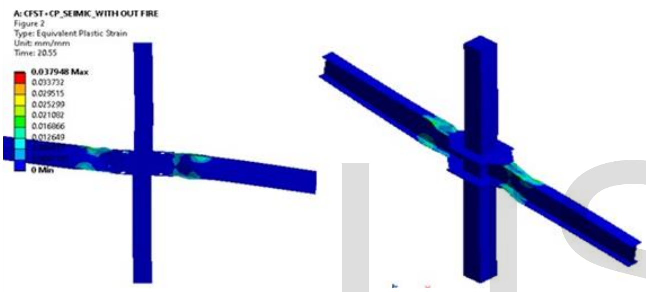

Figure 3: Equivalent Plastic Strain for without fire exposure

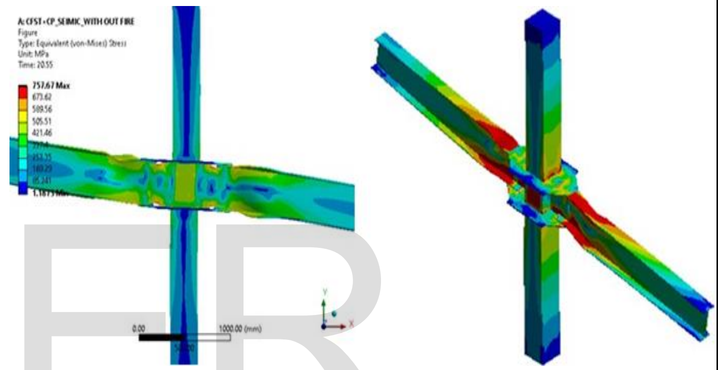

Figure 6: Equivalent Stress under without fire exposure

2.3.2 Case II With Fire Exposure on Bottom Left

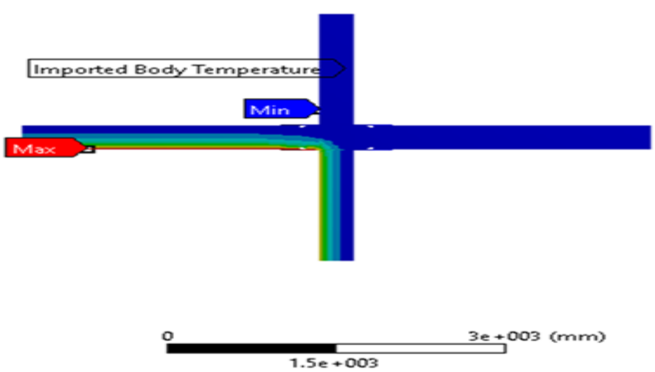

Figure 4: Fire imported on Bottom Left

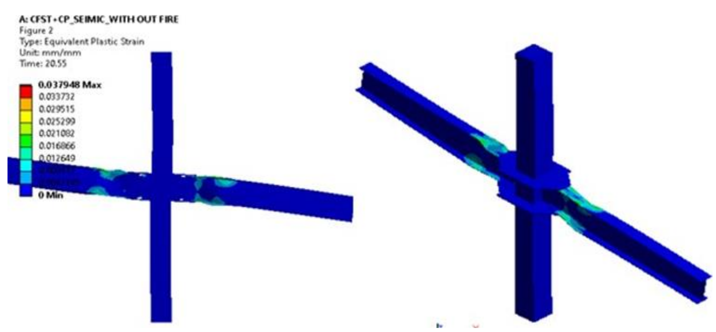

Figure 7: Equivalent Plastic Strain for without fire exposure Case

2.3.3 Case IV With Fire Exposure on top left

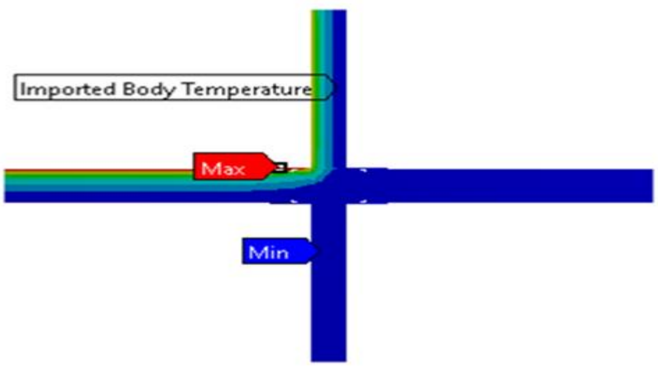

Figure 8: Fire imported on Top Left

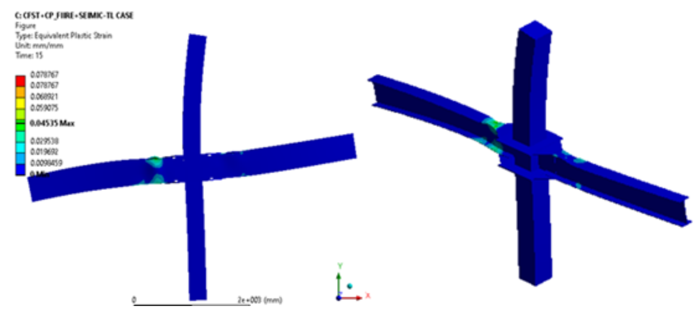

Figure 11: Equivalent Plastic Strain for without fire exposure Case

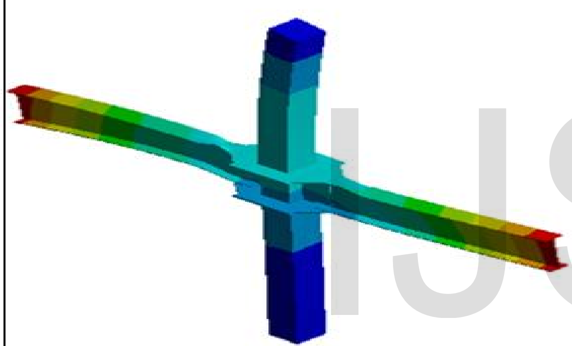

Figure 9: Total Deformation of Joint under without fire exposure

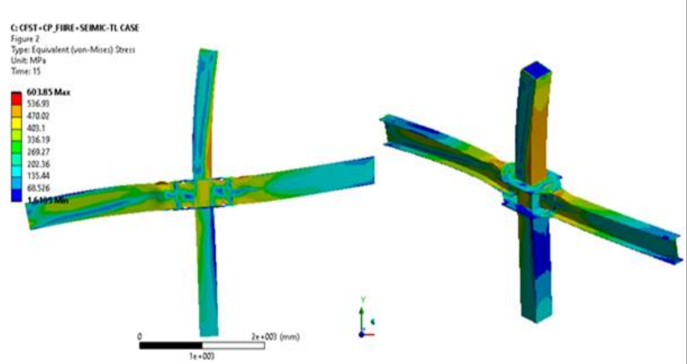

Figure 10: Equivalent Stress under without fire exposure

2.4 Result from study

TABLE 4
STUDY FROM VARIOUS CASES

Case No	CASE		RIGHT BEAM				LEFT BEAM		
			TIME	DISPLACEMENT	LOAD		TIME	DISPLACEMENT	LOAD
I	Without Fire Case	+	18.7	98	2.58E+05		20.55	92.4	2.65E+05
		-	20.55	-92.4	-261280		18.7	-98	-261760
		AVG	19.63	95.2	259.53				
II	Fire at bottom Left		18.48	66.5	265250		16.75	105	247270
			20.6	-100.8	-267410		14.67	-75.04	-233410
						AVG	15.71	90.02	240.34
III	Fire at bottom right	+	14.85	95.2	2.51E+05		16.7	98	258940
		-	16.55	-77	-237760		18.55	-77	-259580
		AVG	15.7	86.1	244.145	AVG			
IV	Fire at top left		18.7	98	268040		16.6	84	240090
			20.6	-100.8	-273090		15	-112	-253310
						AVG	15.8	98	246.7
V	Fire at top right		10.7	58.8	227300		16.7	98	259080
			16.7	-98	-244180		18.56	-78.4	-260920
		AVG	13.7	78.4	235.74				

2.5 Inference

The Force-Displacement indicates the connection load carring capacity degradation that start after a 4.94% with loading cycles.The lateral strength degradation for case II is 5.44% while for case III is 9.55% for case IV is 2.94% and for case V is 17.64%. It indicates that the Top left bay when exposed to fire has more load carrying capacity.This indicates that this bay can resist more load compared to others of the bay and can perform well during a seismic performance.It Can be also found that in case V the structure behave like a soft storey mechanisim. Resulting in complete failure of the structure during seismic analysis.

3 THERMO-STRUCTURAL ANALYSIS FOR PROTECTED CONNECTION

A Beam-to-CFT High Strength Joints with External diaphragm is consid- ered.Finite element analysis four four fire cases are considered and worked out.

3.1 Intumenscent Coating

A 1200µm layering coating is applied. The specific heat of the coating is 1200kg^-1C^-1 .

3.2 Loading Protocol

SAC loading protocol is a graphical representation of defor- mation against cycle on y-axis is the peak deformations and on x axis cycles are plotted.

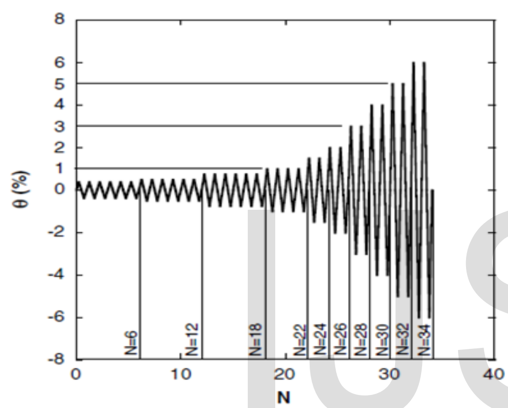

Figure 12: Loading Protocol

3.3 Thermo-structural analysis

3.3.1 Case II With Fire Exposure on Bottom Left

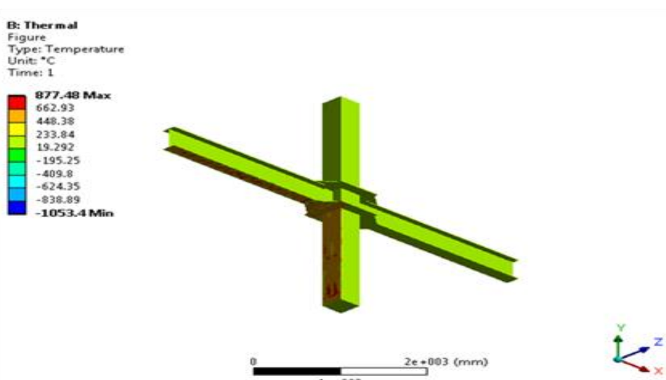

Figure 13: Fire imported on Bottom Left

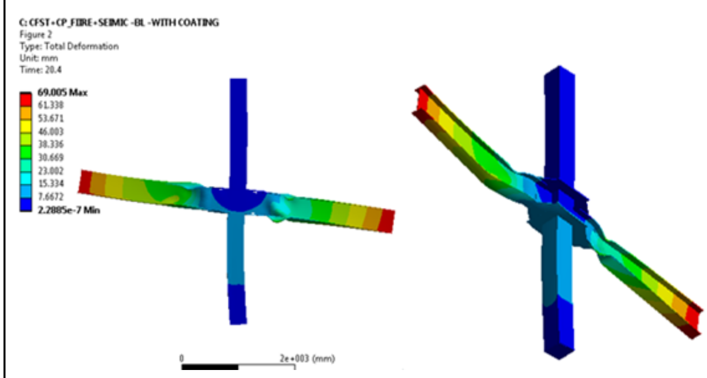

Figure 14: Total Deformation of Joint under with fire exposure

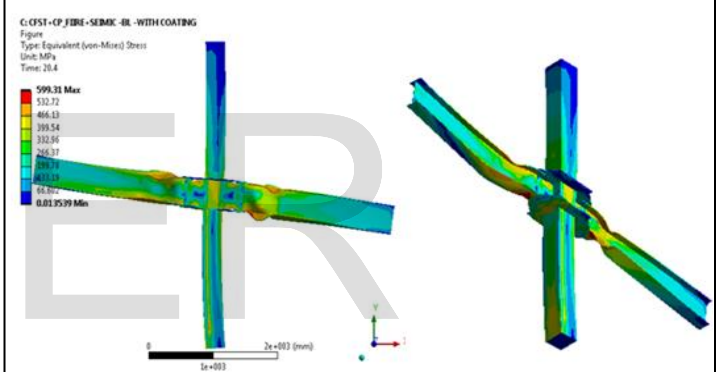

Figure 15: Equivalent Stress under without fire exposure

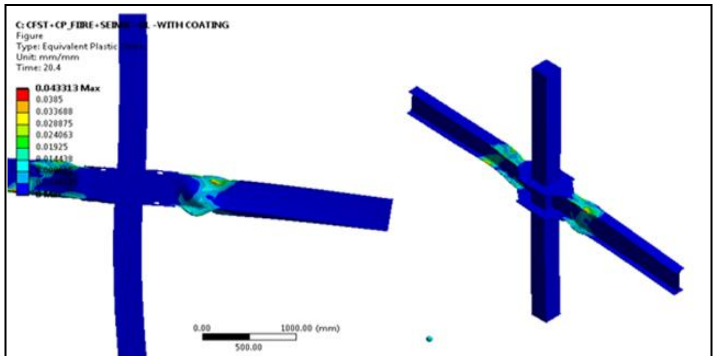

Figure 16: Equivalent Plastic Strain for without fire exposure Case

3.3.2 Case IV With Fire Exposure on Top Left

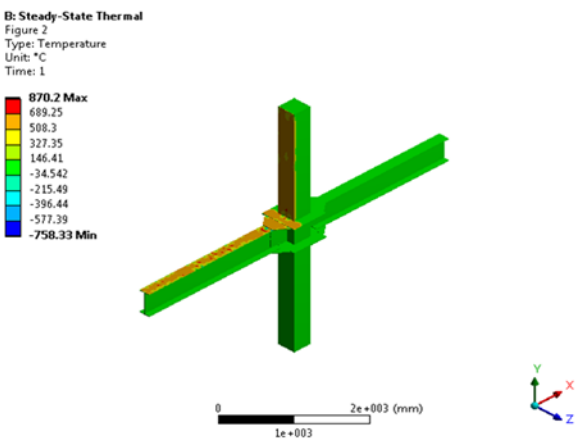

Figure 17: Fire imported on Top left

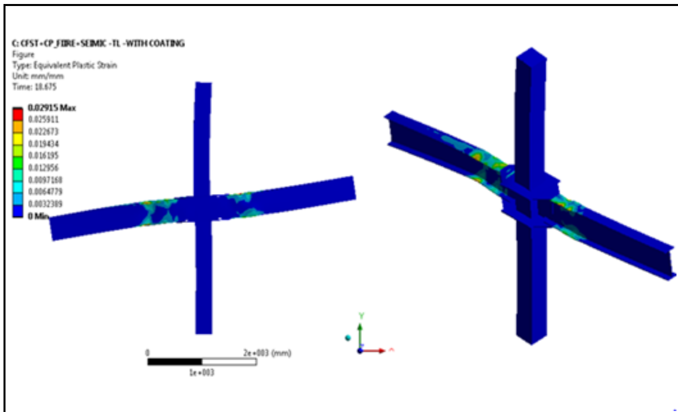

Figure 20: Equivalent Plastic Strain for without fire exposure Case

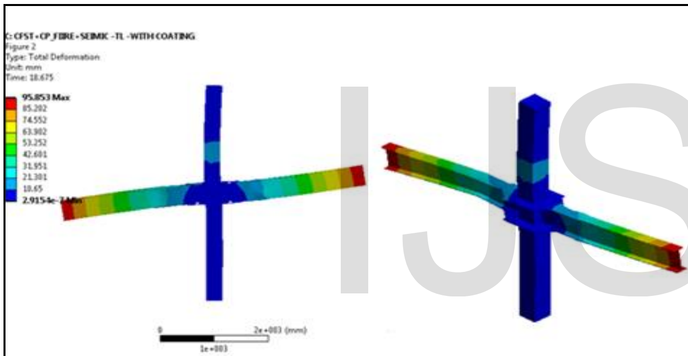

Figure 18: Total Deformation of Joint under with fire exposure

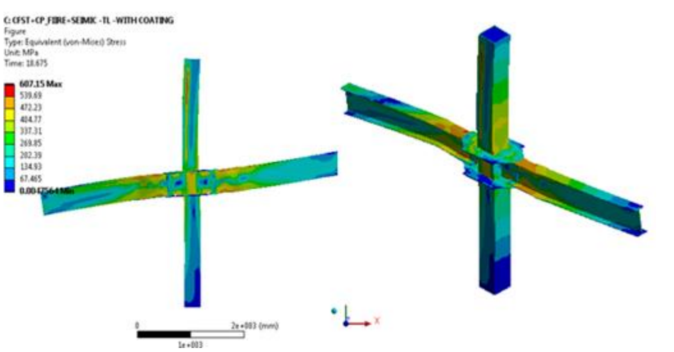

Figure 19: Equivalent Stress under without fire exposure

3.4 Result from study

TABLE 5
STUDY FROM VARIOUS CASES

Case No	CASE	RIGHT BEAM					LEFT BEAM		
			TIME	DISPLACEMENT	LOAD		TIME	DISPLACEMENT	LOAD
I	Without Fire Case	+	18.7	98	2.58E+05		20.55	92.4	2.65E+05
		-	20.55	-92.4	-261280		18.7	-98	-261760
		AVG	19.625	95.2	259.53				
II	Fire at bottom Left		18.7	98	2.67E+05		20.4	67.2	2.68E+05
			16.75	-105	-2.59E+05		18.4	-56	-2.64E+05
						AVG	19.4	61.6	265.825
III	Fire at bottom right	+	18.498	69.738	2.68E+05		16.75	105	2.59E+05
		-	20.54	-90.72	-2.67E+05		18.675	-94.544	-2.69E+05
		AVG	19.519	80.229	267.71	AVG			
IV	Fire at top left		18.7	98	2.68E+05		20.4	67.2	2.68E+05
			20.6	-100.8	-2.73E+05		18.6	-84	-2.72E+05
						AVG	19.5	75.6	270.085
V	Fire at top right		18.7	98	2.76E+05		20.4	67.2	2.68E+05
			20.4	-67.2	-269430		18.55	-77	-2.70E+05
		AVG	19.55	82.6	272.76				

3.5 Inference

The Force-Displacement indicates the connection load carring capacity degradation that start only after 13.235% with loading cycles.The lateral strength degradation for case II 15.72 %, is while for case III isb35.29%.for case IV is 20.588%, and for case V is 13.235% .It indicates that the Top right bay when exposed to fire has more load carrying capacity.This indicates that this bay can resist more load compared to others of the bay and can perform well during a seismic performance. The cycles starts degradation only after 4% drift\It can be also found that in case V the structure behave like a soft storey mechanisim. Resulting in complete failure of the structure during seismic analysis.

4 COMPARISON OF RESULTS

When comparing with results based on loading carrying capacity, it can be clearly seen that when coating is applied on to the beam and column there is a much difference in strength degradation. In unprotected cases strength degradates from 2.94% while in protected case strength degrades only from 13.235% with number of cycles. For fire case II,only the left bay is subjected to a temperature of about 1000C and consequently the material strength degrades while in protected case, the beam is subjected to a temperature of 200C or below. Therefore the strength degration is less or nil during fire case.

When considering cases II and III, plastic hinge is formed at base of column indicating weak floor.It can also be noted that hinge is made to form inthe beam by providing external diaphragmn.

It can also be found that the drift is more than 4% which is acceptable in protected condition for seismic zone areas.

5 CONCLUSION

The seismic vulnerability of a fire exposed frame was studied considering various fire scenario. Without Fire case, Plastic Hinge formed at 4% drift When peak temperature is reached the strength of steel degrades and adverse effectof sory drift has been found.Reduction in strength at 4.06% indicates that the structure performance better during seismic activities. The decrease in displacement indicates the decrease in drift angle.In protected case the drift angle shows a drift of about 4% which is good. Moment capacity of top left is found to be better when exposed to fire during unprotected case whereas top right when exposed to fire will behave as a soft storey mechanism.The increase in number of cycles indicates the increase in ductility.Fire exposure on unprotected bay can result in the formation of weak beam to column connection that fails drastically during an earthquake.Hence, protected condition can improve the time of failure,increase load carrying capacity and also increase ductility.

REFERENCES

[1] Quayyum, S., and T. Hassan. 2017. "Initial residual stresses in hot-rolled wide-flange shapes: A computational technique and influence on structural." J. Struct. Eng. 143 (5): 04017013.

[2] 2. Cristian Vulcu; Aurel Stratan; Adrian Ciutina; and Dan Dubina,2017 "Beam-to-CFT High-Strength Joints with External Diaphragm. II: Numerical Simulation of Joint Behavior" DOI: 10.1061/(ASCE)ST.1943-541X.0001693.

[3] Adan, S. M. 2011. "Design parameters for steel special moment frame connections.In Proc., 2011 SEAOC Convention. Las Vegas: Structural Engineers Association of California.

[4] Shahriar Quayyum, A.M.ASCE1; and Tasnim Hassan,"Seismic Performance Of Fire Exposed moment Frame" DOI: 10.1061/(ASCE)ST.1943-541X.0002201.

[5] Naveed Alam, Ali Nadjai , Faris Ali ,Walid Nadjai ," Structural response of unprotected and protected slim floors in fire "Journal of Construc- tional Steel Research 142 (2018) 4454, https://doi.org/10.1016/j.jcsr.2017.12.009

[6] Raul Zaharia and Dan Pintea ," Fire after Earthquake Analysis of Steel Moment Resisting Frames"Department of Steel Structures and Struc- tural Mechanics.

[7] Quiel, S. E., and M. E. Garlock. 2008. "Modeling high-rise steel framed buildings under fire." In Proc., 2008 ASCE/SEI Structures Congress. Reston, VA: ASCE.

[8] Adan, S. M. 2011. "Design parameters for steel special moment frame connections." In Proc., 2011 SEAOC Convention. Las Vegas: Structural Engineers Association of California.

[9] AISC. 2016. Specification for structural steel buildings. ANSI/AISC 360. Chicago: AISC.

[10] Morrison, M. L. 2015. "Innovative seismic performance enhancement techniques for steel building moment resisting connections." Ph.D.dissertation, Dept. of Civil, Construction and Environmental Engineering, North Carolina State Univ.

[11] Morrison, M. L., and T. Hassan. 2016. "Resilient welded steel moment connections by enhancing beam buckling resistance." J. Constr. Steel Res. 127:77–91.https://doi.org/10.1016/j.jcsr.2016.07.