Sensitivity Analyses of Structural Steel Connection Fire Performance

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Abstract: *In standard fire resistance furnace tests, structural specimens are unable to dissipate heat, as they are likely to do in building construction, by interconnection to cooler adjacent structural elements. Also, spray-applied fire resistive material (SFRM) coverage often is imperfect in buildings because it can be removed easily from steel, and poor application, natural deterioration, or other effects may cause localized deficiencies. Hence, standard furnace tests often do not capture the influences of important in situ features.*

This paper presents results of finite element, thermal-stress analyses that investigate the effects of connection proximity with respect to adjacent columns and the extent of SFRM coverage. More specifically, the performance of a series of model configurations that vary in connection proximity and SFRM coverage were studied under exposure from a typical office fire. These parametric studies demonstrate that minor changes to a connection assembly may have a significant effect on its fire performance. In some cases, relatively simple connection detailing changes can enhance the overall structure's fire resistance, even when SFRM coverage is not perfect.

Keywords: *Collapse, Coupled Analysis, Failure, Fire, Heat Transfer, Structural, Thermal-Stress*

1. Background

Fire resistance design as practiced in the U.S. usually follows prescriptive procedures, in which passive protection is specified in accordance with results of standardized tests on specific protection systems. The most widely-used U.S. building code for fire protection of structural assemblies is the International Building Code (IBC), which ranks structural assemblies based on the ASTM E 119 standard furnace test (ASTM E 119-07a, 2007). While this test, which has been largely unchanged for decades, can demonstrate the relative resistance that protection systems provide to structural components, it can neither test overall structural systems nor does it provide definitive information about the actual performance of structural components in fires.

In the ASTM E 119 standard furnace test, individual structural elements are tested in isolation, and connections between members are never included. Therefore, structural specimens are unable to dissipate heat, as they are likely to do in building construction by interconnection to cooler adjacent structural elements. For example, a protected steel column may act as a heat sink by conducting heat away from an adjacent steel connection. Also, spray-applied fire resistive material (SFRM) coverage often is imperfect in buildings because it can be removed easily from steel, and poor application, natural deterioration, or other effects may cause localize deficiencies. Hence, standard furnace tests often do not capture the influences of important in situ features.

2. Parametric Studies of Steel Connection Fire Performance

A 30 ft by 30 ft Gerber-type structural steel bay was the subject of parametric studies conducted to study the influence of connection proximity and SFRM coverage on fire performance. The steel bay consists of 4-ft long W24x61 girder stubs welded to the flanges of W14x103 columns and W18x50 central span girders connected between the girder stubs with shear connections. The shear connections are comprised of two 6.5 in. by 9.0 in. plates that are connected using six 15/16 in. high-strength bolts. Figure 1 shows an isometric view of the Gerber-type structural steel bay. The floor members support a 4-in. thick lightweight concrete slab. The columns and floor assembly have mineral fiber SFRM with thicknesses that are characteristic of 3-hour and 2-hour fire resistance ratings, respectively.

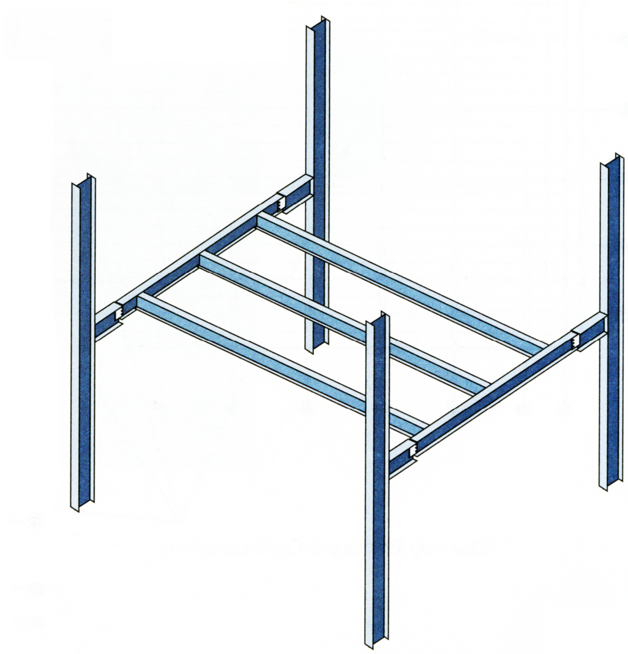

Figure 1. 30 ft by 30 ft Gerber-Type Framing System (FEMA, 2002)

The Gerber-type structural steel framing method is commonly used for large buildings with regular repetitive layouts (Hemstad, 1999), such as the late World Trade Center Building 5 (WTC 5). WTC 5 was a 9-story office and retail building at the WTC complex in New York, NY. On September 11, 2001, flaming debris from the collapse of the WTC Towers penetrated the roof of WTC 5, causing an uncontrolled fire and a resulting interior structural collapse. The 8th through 5th floors within WTC 5 that collapsed solely due to this fire exposure utilized the Gerber-type framing configuration described above (Figure 1) (FEMA, 2002). Recent research of this interior collapse demonstrates that the framing failed during the heating phase of the fire (LaMalva, 2009). WTC 5 was sensitive to early failure because the Gerber beam design, with simple connections located away from columns, isolated the shear connections from their heat sinks to the rest of the "cooler" structure which reduced the tear-out strength of the shear connections.

The 9th floor of WTC 5 experienced a similar fire exposure as the 8th through 5th floors, but did not collapse. The only difference between the structural framing typical of the 8th through 5th floors and that on the 9th floor was the location of the shear connections; on the 9th floor the connections were made at the columns (FEMA, 2002). This observation raises interest about the influence of connection proximity on the fire performance of this steel framing assembly. Furthermore, the influence of localized insulation loss is worth investigating as a possible inhibitor to effective heat dissipation from the steel connections.

A series of configurations were studied to examine variations in the Gerber-type framing system described above in terms of connection proximity and SFRM coverage. These configurations are described below and were modeled using Abaqus/Standard. All of these configurations were analyzed using a typical office fire exposure to compare their relative fire performance characteristics. The office fire exposure reaches approximately 700 °C for approximately 90 minutes before burning out over the course of approximately 120 minutes.

2.1 Shear Connection Close to Column (the "Base Case")

Research of the interior collapse of WTC 5 demonstrated that the location of the shear connections, relatively far from the adjacent columns that functioned as heat sinks, hindered heat dissipation from the vulnerable connections to non-fire regions of the structure. As a result, the temperature of the steel in the vicinity of the shear connections increased at a similar rate as the span of the floor girder. This increase in temperature reduced the strength of the steel at the shear connections, which contributed to the early tear out failure (LaMalva, 2009). One possible method to prevent this early failure mechanism may be to locate the shear connection closer to the column interface, thereby allowing for more efficient heat dissipation from the connection.

Figure 2 shows a modification to the SFRM-protected Gerber-type framing assembly described above. The length of the girder stub is decreased from approximately 4 ft to 1 ft and the floor girder span is maintained from the original configuration. While this configuration may not represent an efficient structural design, it is studied here to explore the influence of connection proximity on fire performance of the assembly.

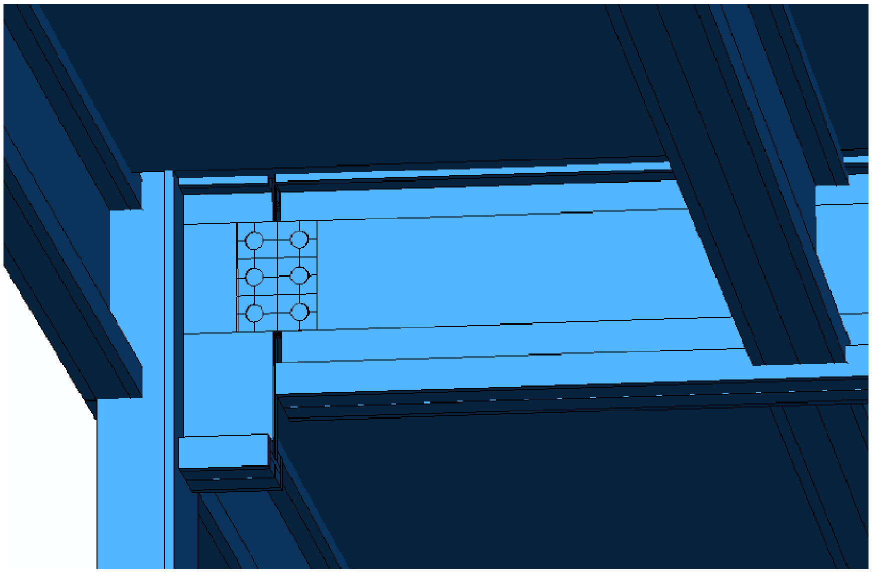

Figure 2. Close Connection Proximity Model (*Steel Insulation Is Shown*)

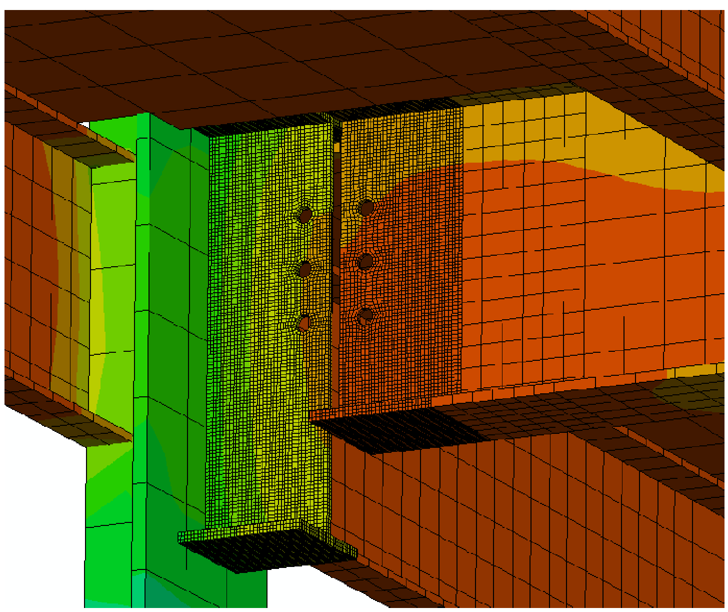

Figure 3. Temperature Distribution After 2 Hours (Close Proximity Model)
(*Steel Insulation Not Shown*)

Failure criteria were developed using Abaqus/Standard by creating an analytical model for a single pre-tensioned bolt connecting two plates, and comparing the results to relationships in Chapter J3 of the AISC Specification for Structural Steel Buildings (AISC, 2001). The shear strain states in the Abaqus/Standard model compared to ultimate capacities in accordance with Chapter J3 for single bolt tear out strength served as the failure criterion.

Figure 3 shows the steel temperature distribution at the shear connection after 2 hours of simulated fire exposure. Theoretical failure of the connection occurred after approximately 4 hours of fire exposure by bolt tear-out (Figure 4). Significantly, the shear connection failed after the fire burned out completely and the structural assembly was cooling. Whereas the model developed to study the interior collapse of WTC 5 failed during the heating phase because of the floor girder's large deflection coupled with inefficient heat dissipation from the connection (LaMalva, 2009), this modified configuration failed due to thermal contraction of the floor girder after the fire burnout during the cooling phase.

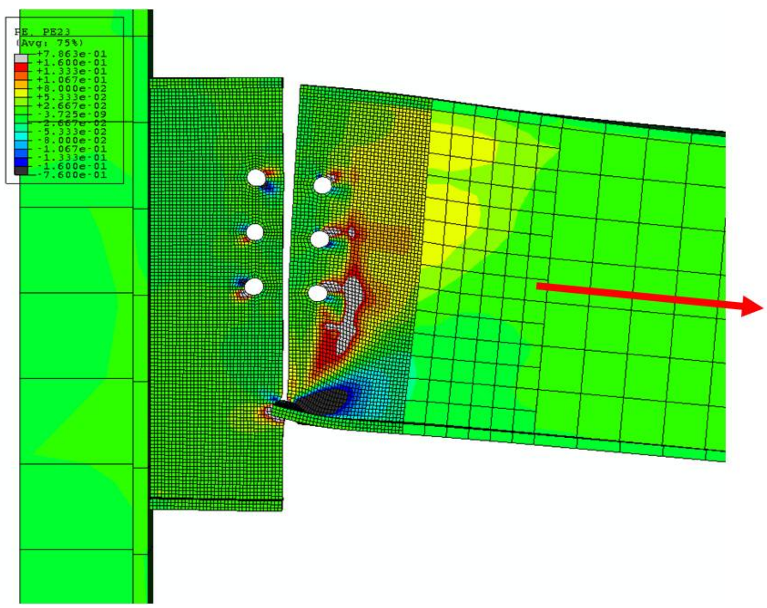

Figure 4. Connection Failure Due to Thermal Contraction (Close Proximity Model) (*Steel Insulation Not Shown*) (Plastic Shear Strain at Top Bolt Hole = 16% and Rapidly Increasing)

There have been documented cases in which steel connections fail during the cooling phase of a fire exposure. For example, following the fire within the One New York Plaza building in 1970, shear connections between beams and girders failed due to the thermal contraction of the members during cooling (Powers, 1970). Moreover, partial connection failures attributed to member thermal contraction were observed during the Cardington fire tests in 1997 (British Steel, 1998).

2.2 Incomplete SFRM Coverage

SFRM is applied to structural steel members to delay the transmission of heat from a fire to the steel. Since steel properties degrade as its temperature increases, the integrity of the insulation system to the overall structural fire performance is very important. Standard furnace tests demonstrate that steel members which are not insulated do not perform nearly as well in fire as do their insulated counterparts. When fully insulated, the short girder stub model (i.e., the "base case") survived well into the cooling phase of the fire exposure. In order to examine the influence of incomplete SFRM coverage, the base case model was modified for various patterns of missing insulation and subjected to the same fire exposure.

2.2.1 Case A: Small Section of Missing SFRM on the Floor Girder

A small section of SFRM, measuring 4-1/2 in. by 1-1/2 in., on a single face of the floor girder was removed from the base case model (Figure 5). This condition might represent an installation deficiency or local removal of SFRM by maintenance activities over the life of a building. When the fire exposure is applied to this modified assembly, the steel temperature in the vicinity of the missing SFRM rose more quickly than the base case during the initial growth phase of the fire. However, after approximately 2 hours of fire exposure, the temperature distribution of the steel was very similar to the base case. Failure occurred approximately 4 hours after initiation of the simulated fire exposure, essentially unchanged from the base case.

2.2.2 Case B: Small Section of Missing SFRM on the Girder Stub

A small section of SFRM, measuring 3 in. by 1-1/2 in., was removed from the base case model (Figure 6). This scenario is nearly identical to Case A except the small section of missing SFRM is located on a single face of the girder stub, not the floor girder. As observed in Case A, the temperature distribution of the steel was very similar to the base case after significant heating. Failure occurred approximately 4 hours after initiation of the simulated fire exposure, which is essentially unchanged from the base case.

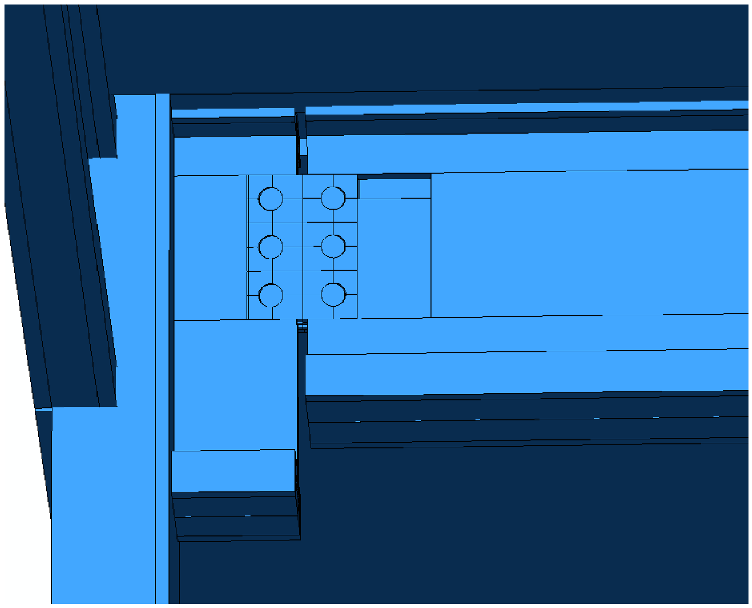

Figure 5. Incomplete SFRM Case A (*Steel Insulation Is Shown*)

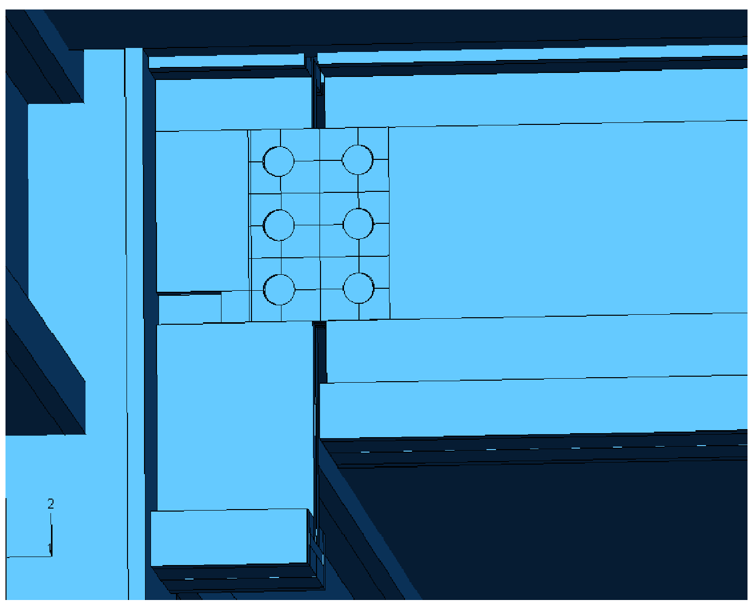

Figure 6. Incomplete SFRM Case B (*Steel Insulation Is Shown*)

2.2.3 Case C: Large Section of Missing SFRM on Girder Stub

A relatively large section of SFRM, measuring 4 in. by 9 in., on a single face of the girder stub was removed from the base case model (Figure 7). This might represent a dislodged section of the SFRM that was not properly adhered to the steel surface at application. This representation of missing SFRM might also be characteristic of localized damage in areas of high vehicle or pedestrian traffic during the construction process.

After approximately 2 hours of fire exposure, the temperature distribution in the vicinity of the shear connection changed significantly from the base case (Figure 8). Failure of the shear connection theoretically occurred approximately 2 hours and 20 minutes into the fire exposure. The failure mode is similar to that observed in the model developed to study the interior collapse of WTC 5, for the tear out strength of the girder stub web steel was not sufficient to resist the forces that developed from the girder's large deflection (Figure 9). Since the failure for this case was during the heating phase of the fire, the performance of this assembly would be deemed as insufficient.

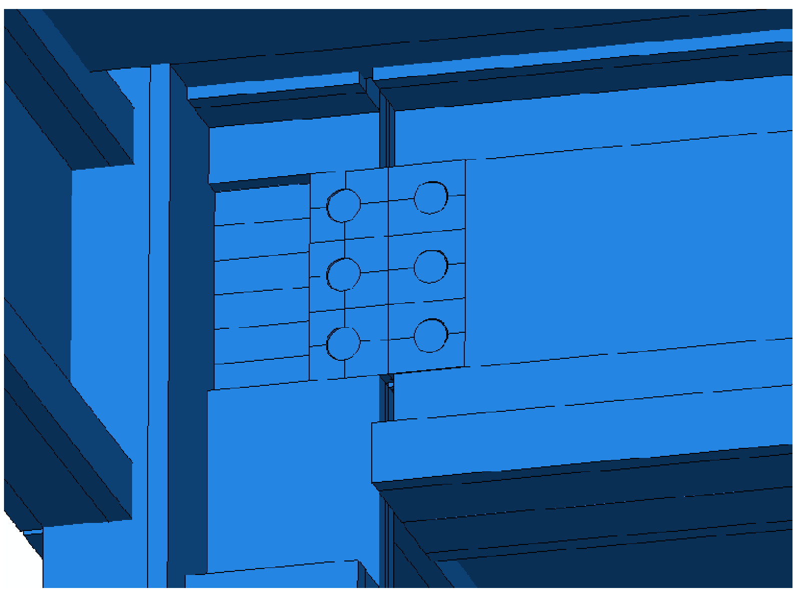

Figure 7. Incomplete SFRM Case C (*Steel Insulation Is Shown*)

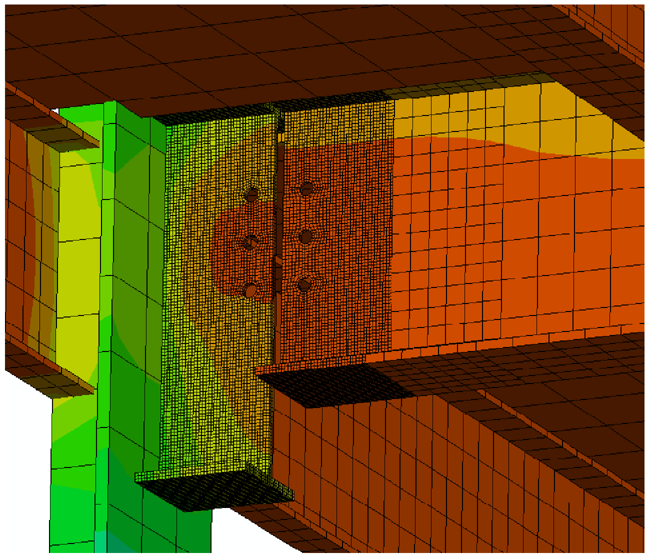

Figure 8. Temperature Distribution After 2 Hours (SFRM Case C)
(*Steel Insulation Not Shown*)

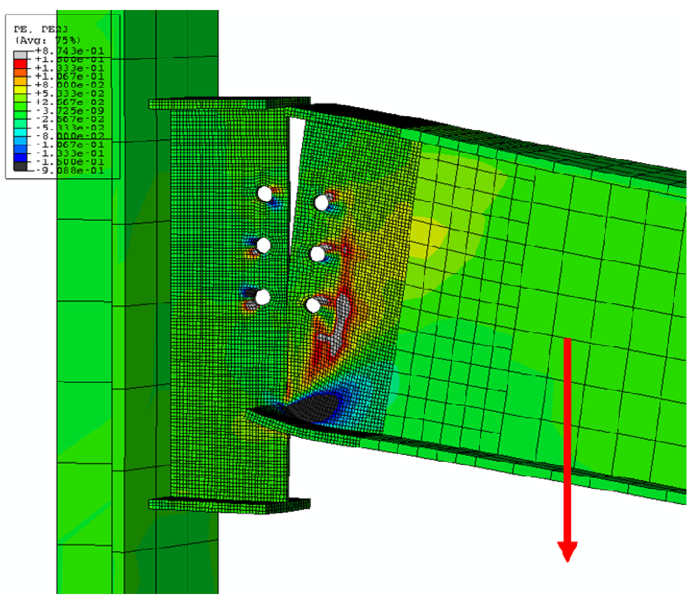

Figure 9. Connection Failure Due to Fulcrum Mechanism (SFRM Case C)
(*Steel Insulation Not Shown)* (Plastic Shear Strain at Top Bolt Hole = 16% and Rapidly Increasing)

2.2.4 Case D: Large Section of Missing SFRM on Floor Girder

The SFRM was removed from the lower flange face of the floor girder across its entire span (Figure 10). This might represent damage from an explosion or severe damage due to maintenance operations. While the temperature of the bottom flange increased rapidly, after approximately 1 hour of fire exposure the theoretical temperature distribution in the vicinity of the shear connection was not changed significantly from the base case. Failure of the shear connection occurred approximately 1 hour and 15 minutes into the fire exposure.

The failure mode for this case is similar to that observed in the model used to study the interior collapse of WTC 5, but the cause is different. The theoretical strength of the steel in the vicinity of the shear connection was essentially unchanged from the base case. However, the increased steel temperature across the span of the floor girder increased its mid-span deflection from 15 in. for the base case to 21 in. for this scenario. This increase in deflection amplified the rotation at the shear connection, ultimately causing tear out failure to occur early in the fire exposure. This case demonstrated the poorest fire performance of all the studied configurations.

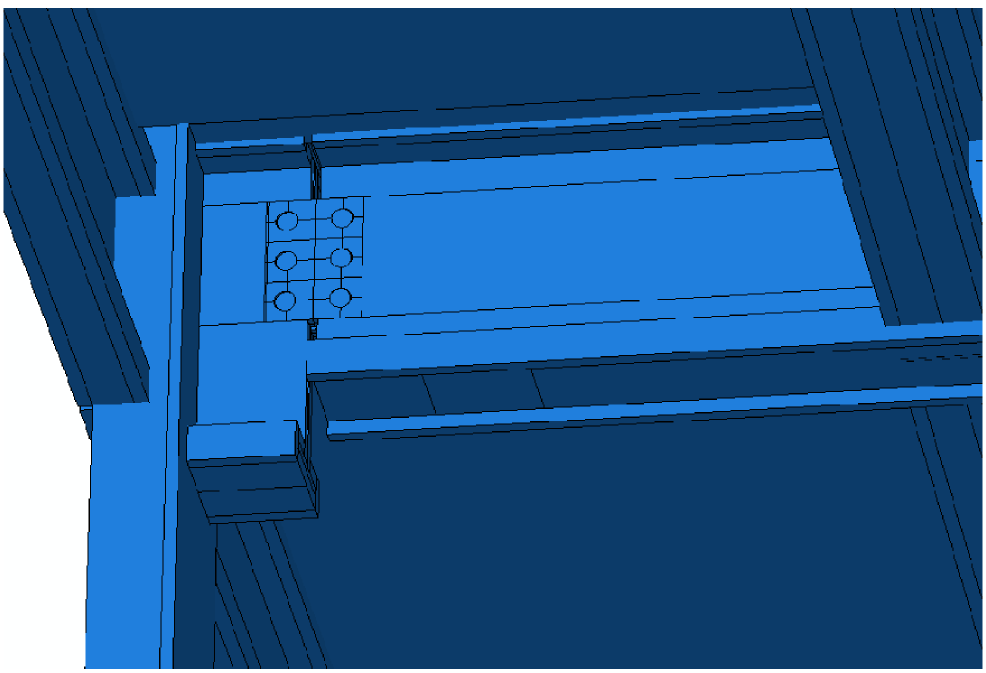

Figure 10. Incomplete SFRM Case D (*Steel Insulation Is Shown*)

2.2.5 Case E: Small Section of Missing SFRM Adjacent to Column

A 1-1/2 in. strip of SFRM was removed from the entire periphery of the girder stub section at the connection to the column (Figure 11). This might represent damage to non-ductile, SFRM in an earthquake. After approximately 2 hours of fire exposure, the temperature distribution in the vicinity of the shear connection changed significantly from the base case (Figure 12).

Theoretical failure of the shear connection occurred approximately 2 hours and 10 minutes into the fire exposure for this scenario. Similar to Case C, the removed SFRM allowed the flow of heat to a region of the steel assembly which effectively inhibited heat dissipation from the shear connection to the cooler periphery structure. Introducing a significant heat concentration between the shear connection and the column diminished their thermal interaction via conduction heat transfer. The failure for this case is during the heating phase which represents insufficient fire performance.

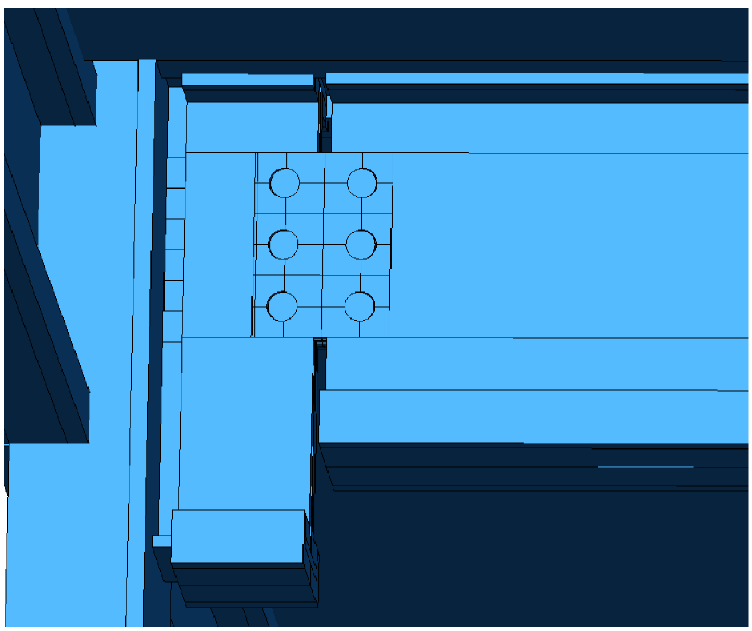

Figure 11. Incomplete SFRM Case E (*Steel Insulation Is Shown*)

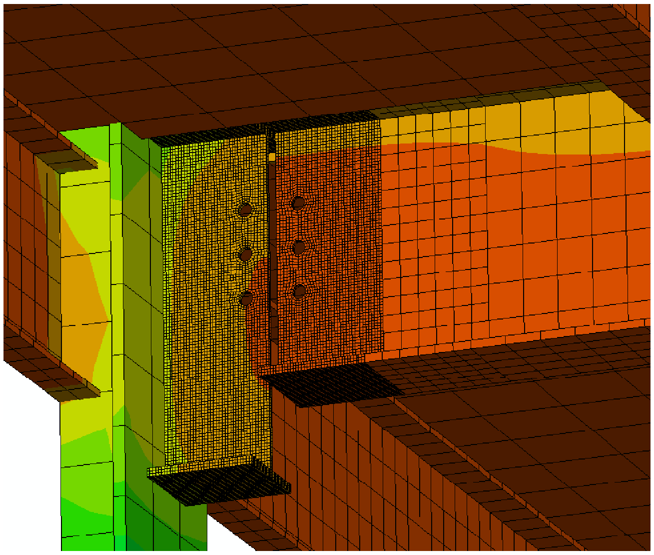

Figure 12. Temperature Distribution After 2 Hours (SFRM Case E)
(*Steel Insulation Not Shown*)

3. Observations from Parametric Studies

The sequentially-coupled, nonlinear thermal-stress modeling capabilities of Abaqus/Standard have been used to study the structural fire performance of a steel connection assembly under varying conditions. The proximity of connections in relation to their heat sinks has a significant effect on their fire performance for the structural steel assembly analyzed. The analyses demonstrate that locating the shear connection 3 ft. closer to the column, as compared to the WTC 5 configuration, resulted in a dramatic improvement in its fire performance. This modified framing assembly more closely resembles that used for the roof and 9th floors of WTC 5 in which the shear connections were made directly at the column interfaces. Those floors of WTC 5 did not suffer collapse even after the steel cooled following the fire.

Small regions of missing SFRM did not significantly affect the fire performance of the structural assembly analyzed. However, other types of frames may behave differently during fire exposure and, therefore, have different dependency on full insulation coverage. Relatively large regions of missing SFRM significantly affected the temperature distribution of the steel assembly analyzed. For cases C and E (insulation missing from the girder stub web and column connection, respectively), the large regions of missing insulation reduced the structure’s fire endurance, for heat dissipation from the shear connections to the cooler periphery structure was inhibited during the fire exposure. For Case D (insulation missing from the bottom face of floor girder flange), the large section of missing insulation resulted in increased deflection of the floor girder, which increased forces at the shear connection and caused early failure.

4. Performance Based Approach

In performance-based design of steel frames, connection performance potentially controls the survivability of the structural system. Since connections are not included in standard furnace tests, performance data of connections under fire exposure is limited. This scarcity of information confounds performance assessment when compounded by the large variety of possible structural steel connections that may be utilized for structural frames. In order for a structural steel frame to resist collapse during fire exposure, the connections must be able to withstand fire-induced forces and deflections, which may change dramatically during the course of a fire. Moreover, steel properties degrade at elevated temperatures, reducing the load–carrying capability of steel connections during fire.

The parametric studies reported herein demonstrate that minor changes to a connection assembly may have a significant effect on its fire performance. In some cases, relatively simple connection detailing changes can enhance the overall structure's fire resistance, even when fireproofing is not perfect. For example, engineers may choose to provide added connection flexibility and increase bolt edge distances in anticipation of large member deflection during a fire's heating phase and tensile forces due to member thermal contraction during the cooling phase.

Although fire endurance requirements usually are acknowledged early in the planning of a building, they are not a conscious part of structural design (Fitzgerald, 2004). Analytical approaches, more akin to common design for wind, seismic, and other environmental loads, could reveal critical aspects of building performance in fires, and provide engineers with the understanding they need to create designs that are robust, raise safety for occupants and firefighters, and are cost efficient (LaMalva, 2009). Performance-based structural fire protection engineering would offer a much broader and sounder technical basis for design which building stakeholders could use to make critical decisions during the planning of a building.

Analytical approaches have many merits, but the prescriptive codes for structural fire protection should not be dismissed. Modern prescriptive codes uphold a high level of quality in building construction, and there is no question that the good practices incorporated into these codes significantly raise the level of fire safety in buildings. Unfortunately, the level of risk associated with code compliance remains unknown to decision-makers as it relates to structural fire protection (Fitzgerald, 2004).

Performance-based approaches to structural fire protection would certainly be warranted for structures with the following conditions: high consequences of failure, hazardous fuel loads; elevated risk of terrorism or arson; or concerns about post-earthquake fires. In these cases, the designer may choose to exceed the minimum requirements of the building codes in order to raise the overall level of safety and provide redundancy. Alternatively, performance-based approaches may be used to justify cost efficient designs or application of nonconventional building features which may not be thoroughly addressed in the building codes.

Performance-based structural fire protection engineering requires knowledge of connection performance under fire conditions. Structural and fire protection engineers collectively possess the expertise required to evaluate this type of performance.

5. References

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6. Acknowledgement

The author would like to express his sincere appreciation to Simpson Gumpertz & Heger Inc. (SGH) for their professional support of this research. He would also like to extend special thanks to Donald Dusenberry, P.E. (Senior Principal at SGH) and Jonathan Barnett, Ph.D. (Staff Consultant / Fire Safety at SGH) for their ongoing guidance and support during this research effort. Research relating to the internal failure of WTC 5 was funded by the Society of Fire Protection Engineers (SFPE) Educational and Scientific Foundation.