World Trade Center 5 Failure Analysis

By Kevin J. LaMalva, Jonathan R. Barnett, Ph.D. and Donald O. Dusenberry, P.E.

World Trade Center 5 (WTC 5) was a 9-story office and retail building at the World Trade Center complex in New York City, NY. On September 11, 2001, flaming debris from the collapse of the World Trade Center Towers penetrated the roof of WTC 5, causing a fire that burned unchecked until the fuel from building contents was consumed (FEMA, 2002, p. 4-4). While impact damage over a portion of the building and an intense fire throughout are not surprising given the assault this building received, engineers inspecting the building after the event were not expecting to see an interior collapse, due entirely to the influence of the fire. The floors collapsed between the 8th and the 4th levels in the eastern section of the building, where there was no initial impact damage (*Figure 1*).

The major fire-induced collapse that occurred in WTC 5 involved the portion of the building that had Gerber framing (girder stubs welded to columns, and simply supported central girder spans with shear connections to the ends of the stubs (*Figure 2*)), but not other areas of the building where girders spanned the full distance between columns. This fact, and observations at the site suggesting that the failure was early in the fire, raised the possibility that this structure had a vulnerability that led to premature failure, perhaps during the heating phase of the fire.

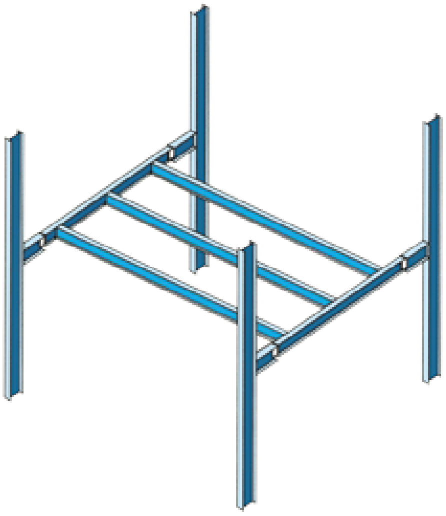

Figure 2: Typical Interior Bay Framing in WTC 5 (Floors 4, 5, 6, 7, and 8) (FEMA, 2002, p. 4-3).

Failure of framing during the heating phase would represent a clear risk to firefighters attempting to extinguish fires in buildings. Moreover, occupants in hospitals or multi-story buildings with vulnerable construction may be at risk when extended egress times or defend-in-place strategies keep them in the buildings during initial phases of fires. If the framing of WTC 5 failed during the heating phase, then it is possible that there is an unappreciated risk in a popular framing system in common use.

To resolve whether WTC 5 was unusually vulnerable to the effects of fire, the authors analyzed the response of the collapsed portion of the building frame to the fire that was ignited by falling debris.

Fire Event Reconstruction

The analysis of the circumstances leading to this failure required an understanding of the fire environment in the building. The 2005 National Institute of Standards and Technology (NIST) report (NIST, 2005) on WTC 1 and 2, provided a reference for the parameters describing the fire that occurred within WTC 5. Using this information, the Consolidated Fire and Smoke Transport Model (CFAST) software, developed by NIST, provided the fire temperature history models for structural analyses.

Finite Element Model Development

The analytical approach to evaluate the shear connection assembly for the failed girders included temperature-dependent material properties, fed into a geometrically non-linear, structural analysis model. The specific heat, conductivity, instantaneous coefficient of thermal expansion, and stress-strain curves for ASTM A36 steel, as derived from the literature, were converted into the input needed for ABAQUS, the structural finite element model (FEM) software capable of performing the required analyses.

The connections in WTC 5 failed by tear out of the web portion of the girder stubs. Chapter J3 of the *AISC Specification for Structural Steel Buildings* (AISC, S001) (LRFD) for single bolt tear out strength was a basis for evaluation of this behavior; plastic shear strain served as the failure criterion.

The FEM analyzed the stress behavior of the four structural bays of interest on the 8th floor (hypothesized as the initial region of failure). This model served as the foundation for the final model:

Figure 1: Internal Collapse Area in WTC 5 (FEMA, 2002, p. 4-18).

a sequentially-coupled, thermal stress analysis of the four structural bays of interest, employing symmetry boundary conditions to capture the behavior of several structural bays.

Finite Element Modeling Results

Modeling the effects of insulation on the framing and heat sinks to non-fire regions, the analyses show that the temperature at the shear connection to the center span could have been as much as 400 Celsius (752 degrees Fahrenheit) hotter than in the girder stub at the column face after two hours of fire exposure (*Figure 3*).

The results of the thermal-stress model (a combination of the thermal and structural models) show that the steel girder assembly expanded as it heated, tending to close the gap between the simple span segment and the girder stub. This expansion caused relatively harmless compressive stress concentrations around the bolts, as the bolts were forced into the webs.

At the same time, as the temperature of the steel assembly increased, its rigidity decreased and the floor girder began to deflect significantly. This deflection caused the end of the center segment of the girder to rotate, and the lower flange of the center segment to contact and deform the girder stub web. This caused a fulcrum point that changed the response of the connection as temperatures continued to rise.

After 2 hours, the loss of rigidity in the steel "outpaced" its thermal expansion. As the girder end continued to rotate in response to mid-span deflection, the direction of action of the top bolt of the shear connection reversed, with the bolt beginning to pull toward the end of the web in the direction tending to cause tear out (*Figure 4*).

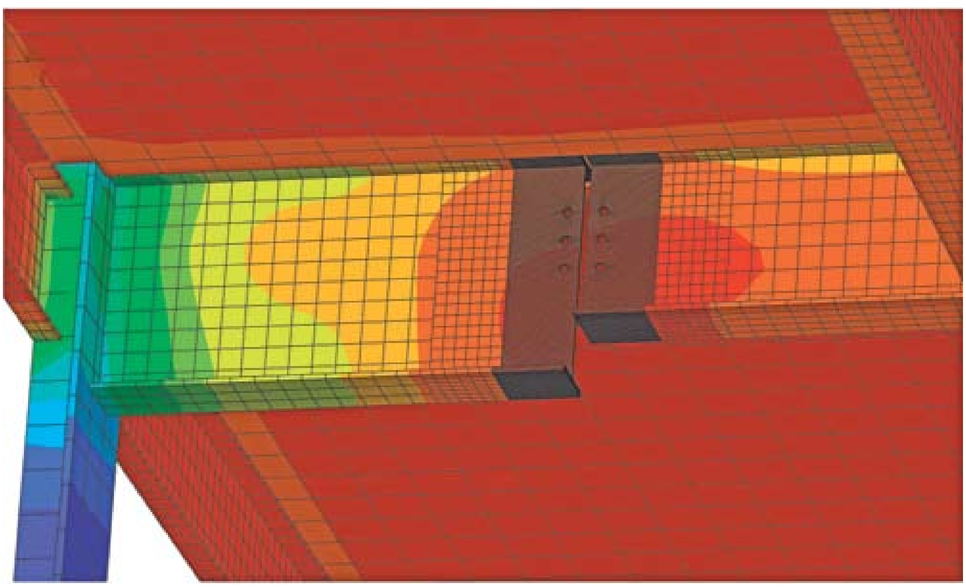

Figure 3: Steel Temperature Distribution (2 Hours of Fire Exposure) (Steel Insulation Not Shown).

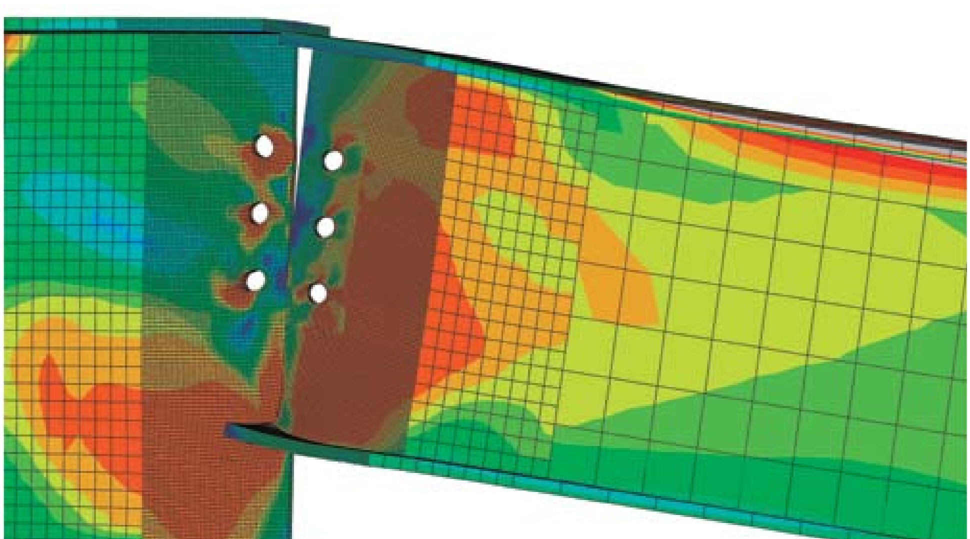

Figure 4: Stress Distribution (2 Hours of Fire Exposure).

The calculations predicted that the plastic shear strain in the girder web quickly – over the course of only minutes after the fulcrum formed – reached values that were triple and quadruple the failure limit. At this point the top bolt would tear out, followed almost instantaneously by the failure of the remaining two bolts, unzipping the connection.

The failure predicted by the finite element model can be seen in a connection specimen that was preserved from WTC 5. The angles at which the bolts pried against the bolt holes are similar in the model and the specimen (*Figure 5*, showing the model at initiation of prying, and the damaged web after the failure). Moreover, photographs of the interior collapse area show that the failed girder stubs are deformed at the fulcrum points.

The sequentially-coupled, thermal-stress model estimated that the catastrophic structural collapse within WTC 5 occurred approximately 2 hours after the initiation of the fire. This is during the heating phase of the fire, when firefighters normally would be in the building.

Conclusions

It is not the precise time of failure which is paramount, but the fact that the structure failed uncharacteristically during the fire's heating phase, rather than during the cooling phase when most fire-induced collapses occur. This building 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 via the columns.

The collapse involved four floors, and might have progressed all of the way down to the ground level, if it had not been for the moment-type connections utilized for the 4th floor.

The fire that destroyed WTC 5 was a severe complete burn-out fire. As such, it is not unreasonable that the structure would experience substantial damage. However, the failure of the building to achieve the preferred performance, with the framing system surviving at least into the cooling phase of the fire, follows from the absence of analyses and design for fire exposure.

The present approach to fire protection engineering in much of the United States is primarily prescriptive, often employing propriety products to insulate structural elements and active fire suppression systems to control fire growth. Such approaches would not lead to an appreciation for vulnerabilities such as apparently existed in some of the detailing in WTC 5. Analytical, performance-based approaches, more akin

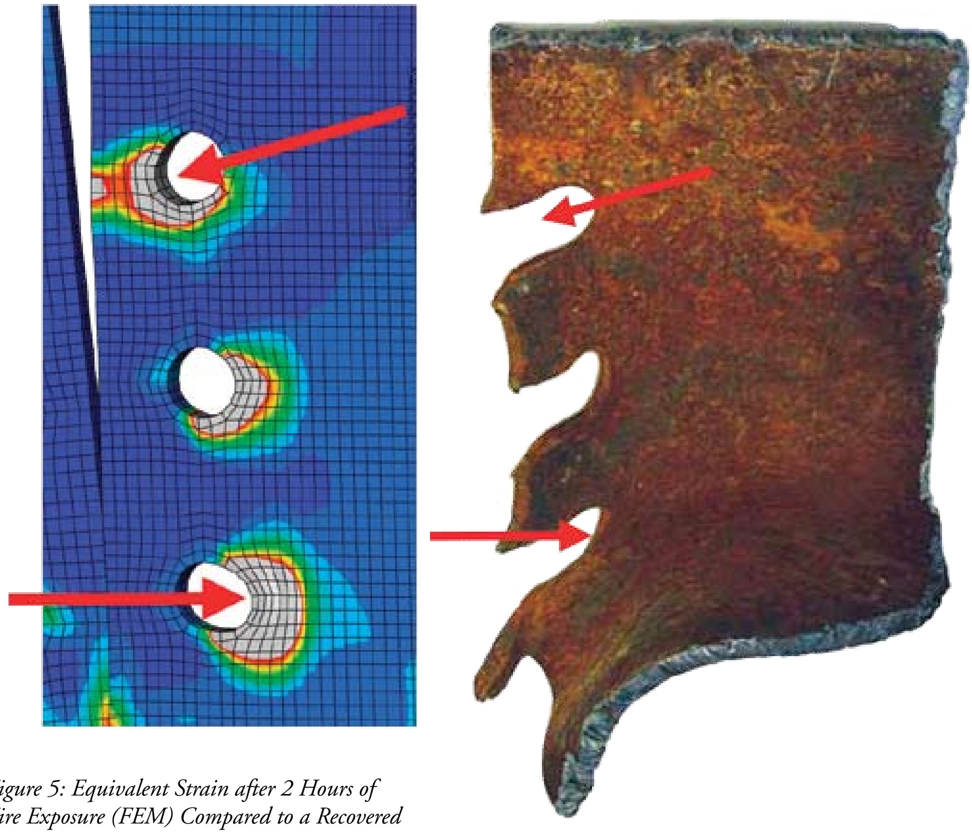

Figure 5: Equivalent Strain after 2 Hours of Fire Exposure (FEM) Compared to a Recovered Sample (FEMA, 2002, p. 4-19).

to common design for wind, seismic, and other environmental loads, are more likely to 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.

In the case of WTC 5, relatively simple detailing changes would have enhanced the structure's fire resistance. Slotted holes in the girder webs, or increased spacing between the end of the girder stubs and the beginning of the simply supported center spans, would have allowed more girder rotation without developing the prying action that tore out the girder webs. Keeping the shear connection near the face of the column would have reduced the temperature of this critical connection, thereby maintaining higher temperature-induced tear-out strengths during the fire.

In the more general case, we must acknowledge that many buildings in current use have unappreciated vulnerabilities. While analyzing and retrofitting for these vulnerabilities in existing buildings could be undertaken if justified for certain framing systems (e.g., perhaps for the Gerber system, if risk analyses and system testing verified heightened risk for the building system generally), finding the critical shortcomings in the present building stock would be a prohibitive exercise. However, modest expenditures of engineering effort during the design phase for new buildings can reveal fire performance weaknesses that can be avoided, often at minimal cost to construction.▪

Kevin J. LaMalva is an Engineer at Simpson Gumpertz & Heger, Inc. Mr. LaMalva was the recipient of the 2007 Society of Fire Protection Engineers (SFPE) Student Scholar Award. Mr. LaMalva can be reached via email at ***kjlamalva@sgh.com****.*

Donald O. Dusenberry, P.E. is a Senior Principal with Simpson Gumpertz & Heger Inc. Mr. Dusenberry is an investigator of structural failure, damage, and deterioration, both for remedial design and forensic testimony. He has written several articles based on his investigations of fire effects on buildings. Mr. Dusenberry can be reached via email at ***dodusenberry@sgh.com****.*

Jonathan R. Barnett, Ph.D. is a Staff Consultant at Simpson Gumpertz & Heger, Inc. Dr. Barnett is a fire safety expert with a specialty in the analytical modeling of fire dynamics, smoke movement, the behavior of structures exposed to fires and occupant egress. As a Fellow in the SFPE he is currently a member of the standards-setting committee on Calculating Fire Exposures to Structures. Dr. Barnett can be reached via email at ***jbarnett@sgh.com****.*

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