Failure Analysis of the World Trade Center 5 Building

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ABSTRACT: This article describes an analysis of the structural collapse that occurred in World Trade Center (WTC) building 5 due to fire exposure on 11 September 2001. It is hypothesized that the steel column-tree assembly failed during the heating phase of the fire. A failure analysis is performed to determine the response of the portion of the building frame that collapsed during the fire ignited by falling debris from the WTC towers. Results from a finite element, thermal-stress model confirm the column-tree failure hypothesis. Based on this model, the authors conclude that the catastrophic, progressive structural collapse occurred \[C24\] 2 hours into the fire exposure.

KEY WORDS: World Trade Center (WTC), structural fire protection, Finite Element Modeling (FEM), fire failure analysis, steel construction, heat transfer analysis, thermal-stress.

INTRODUCTION

WORLD TRADE CENTER building 5 (WTC 5) was a nine-story office and retail building at the World Trade Center (WTC) complex in New York, NY USA (Figure 1). WTC 5 was equipped with an automatic sprinkler system. The columns and floor assembly had spray-applied mineral fiber (SFRM) providing 3-hour and 2-hour fire resistance ratings, respectively [1].

On 11 September 2001, flaming debris from the collapse of the WTC towers penetrated the roof of WTC 5, causing a fire. The falling debris from the collapses of the WTC towers compromised the water supply system, causing the sprinkler system in WTC 5 to fail, and the priorities of the fire department forced it to abandon attempts to fight the fires. Hence, the fires in WTC 5 burned unchecked until the fuel from the building contents was consumed.
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 did not expect to see an interior structural collapse. This interior collapse encompassed an area of \(\sim 1000 \text{ m}^2\), and was due entirely to the influence of the fire (Figure 2). 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 3). The columns in this area remained straight and freestanding. The 9th and roof levels of the building were exposed to the fire, but did not collapse (Figure 4).

The major fire-induced collapse that occurred in WTC 5 involved the portion of the building that had Gerber framing (1.2 m-long girder stubs welded to columns, and simply supported central girder spans with shear connections to the ends of the stubs (Figure 5)). In other areas of the building (e.g., the 9th floor level) the 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.

Failure of framing during the heating phase would represent a clear risk to fire fighters 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

**Figure 1.** World Trade Center building 5 after the events of 11 September 2001 [1]. (The color version of this figure is available online.)
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 [2].

Figure 2. Aerial view of World Trade Center building 5 damage [1]. (The color version of this figure is available online.)

Figure 3. Internal collapse area in World Trade Center building 5 [1]. (The color version of this figure is available online.)
EVIDENCE AND PLANS

Forensic evidence is a key component to understanding the interior structural collapse that occurred in WTC 5 due to fire alone. Following the
events of 11 September 2001, the American Society of Civil Engineers and the Federal Emergency Management Agency formed a Building Performance Study (BPS) team consisting of many specialists. The BPS team surveyed the WTC 5 site and collected photographic evidence, as well as specimen samples to assist in a preliminary performance assessment of the building, and to supplement future study. The forensic evidence analyzed served as a baseline for this study and helped to reinforce specific findings derived from numerical methods.

The structural plans and details of WTC 5 were obtained from the Port Authority of New York and New Jersey using the Freedom of Information Act. These construction documents were referenced to derive the size of the structural elements and the detailed dimensions of the connections. This information was used in the development of the finite element model of the structural assembly.

**FIRE EVENT RECONSTRUCTION**

Based on the 2005 National Institute of Standards and Technology (NIST) report on WTC buildings 1 and 2, the effective heat of combustion and the peak heat release rate (peak HRR) per unit floor area of the fire that occurred within WTC 5 were estimated. More precisely, a series of calorimeter tests involving office cubicles (without jet fuel) and a full-scale Fire Dynamics Simulator (FDS) analysis were referenced to derive appropriate values for the effective heat of combustion and the peak HRR per unit floor area, respectively [3]. Lastly, a 1995 NIST report involving a survey of office buildings was used in the present study to derive an appropriate fuel load for WTC 5 [4].

Results from the 2005 NIST reports were used in the present study because WTC 5 had a similar office occupancy layout as did WTC 1 and 2, and the referenced 2005 NIST calorimeter tests did not include the effects of jet fuel (a component of the fuel in WTC 1 and 2, but not WTC 5).

Using information from the NIST reports, a HRR versus time curve that peaks at 16 MW and encompasses a floor area of 84 m² was estimated for the fire that occurred in WTC 5. The HRR curve assumes a time-squared growth phase and a decay period that consumes one-third of the fuel load. Using this curve as input, the Consolidated Fire and Smoke Transport Model (CFAST) software developed by NIST was used to estimate the upper layer gas temperature history of the fire (Figure 6) [5]. CFAST was chosen for this analysis because it provides accurate results for relatively homogeneous fire environments in the upper gas layer. This was the case with the WTC 5 post-flashover fire; therefore, more sophisticated software such as FDS was not required for this analysis.
In typical building compartments, post-flashover fires would be ventilation controlled. In the case of WTC 5, the large holes in the roof of the western portion of the building allowed smoke venting from the eastern portion (Figure 2). Therefore, a fuel-controlled fire was simulated using CFAST by providing ample ventilation from the compartment. The high ventilation conditions provide the fire with an abundance of oxygen, but it also serves to vent heat from the upper gas layer of the compartment. Figure 6 shows the resulting upper layer gas temperature history.

**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 nonlinear, structural analysis model. Temperature-dependent stress–strain curves for A36 steel were derived from experimental data (Figure 7) [6]. These curves for nominal stress and strain were converted to true stress and true plastic strain for input into ABAQUS, the structural finite element modeling (FEM) software capable of performing the required analyses [7]. The following temperature-dependent properties of steel were also derived from the literature: specific heat, thermal conductivity, and the instantaneous coefficient of thermal expansion [3,8].
The connections in WTC 5 failed by tear out of the web portion of the girder stubs. This type of failure occurs when the bolts bear against the weak side (i.e., acting toward the free end of the member) of the bolt holes. Failure criteria were developed using ABAQUS 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* (LRFD). The shear strain states in the ABAQUS model compared to capacities in accordance with Chapter J3 for single bolt tear out strength served as the failure criterion [9].

ABAQUS was used to create a structural model that encompasses the stress behavior of the four structural bays of interest on the 8th floor (hypothesized as the initial region of failure). This model serves as the foundation for the final model: a sequentially coupled, thermal stress analysis of the four structural bays of interest. This model utilizes symmetry boundary conditions to capture the behavior of several structural bays. Eight-node linear brick elements (C3D8R)\(^1\) were used to mesh the structural model and frictional contact between assembly parts was defined (Figure 8). Under service loading, the maximum deflection of the central girder span was reasonable considering general structural engineering tolerances for beams.

ABAQUS was also used to create a thermal model that encompasses the heat transfer behavior of the four structural bays of interest on the 8th floor of WTC 5 when exposed to the reconstructed fire (Figure 9 shows the

\(^1\)C3D8R is a three-dimensional element used in ABAQUS.
symmetric section). Eight-node convection/diffusion brick elements (DCC3D8) were used to mesh the thermal model. Temperature-dependent data for the steel conductivity and specific heat properties were included in the model, as well as similar property definitions for the insulation and concrete slab [8]. Ideal thermal contact between the insulation and the steel surfaces was assumed for this model (i.e., small air voids at the interface were neglected). Turbulent natural convection and radiation from the upper gas layer, as derived from CFAST, heated the structure during the model simulation. The temperature distribution of the steel assembly in 3D space

Figure 8. Model of the shear connection. (The color version of this figure is available online.)

Figure 9. Thermal model assembly (steel insulation shown). (The color version of this figure is available online.)
and time is essential for the understanding of the thermal-stress behavior, since steel strength is highly temperature-dependent.

The thermal model was combined with the structural model to create a sequentially coupled, thermal stress model. Thermal creep strain was programmed into the model as a function of stress, temperature, and time. Moreover, the thermal expansion of steel was also included in the model.

In the first step of the thermal-stress simulation, the gravity load was applied under ambient conditions. The second step of the simulation applied the temperature distribution history of the steel assembly as a function of 3D space and time, as derived from the thermal model results. Therefore, the thermal-stress model simulated the response of the structural assembly to the estimated fire exposure.

**FINITE ELEMENT MODELING RESULTS**

Modeling the effects of insulation on the framing and heat sinks to nonfire regions, the analyses demonstrated a significant temperature gradient across the length of the girder stub after 2 hours of fire exposure (Figure 10). Whereas the steel in the vicinity of the shear connection reached \( \sim 650^\circ\text{C} \), the steel at the column interface reached only \( \sim 400^\circ\text{C} \).

Heat transfer to the shear connection from the fire exposure must be conducted across the length of the girder stub before it reaches the column, which acts as a heat sink to the rest of the 'cooler' structure in nonfire regions. Therefore, the temperature of the steel in the vicinity of the shear connection remained comparable to the center segment of

![Figure 10. Steel temperature distribution (2 hours of fire exposure) (steel insulation not shown). (The color version of this figure is available online.)](image-url)
the girder span. This effect was exacerbated by the heating of the girder stub length during the fire exposure, reducing the thermal flow from the shear connection.

Results from the thermal model predicted that the insulation delays the transmission of heat to the steel during the fire exposure as expected. Moreover, the temperature of the steel members varied according to elevation due to the heat sink effect of the concrete slab. Two-dimensional quasi-steady heat transfer hand calculations that do not account for heat sink effects were performed. These hand calculations predict a steel temperature of \( \sim 650^\circ C \) after 2 hours of the specified temperature exposure, which agrees well with the modeling results as it pertains to temperatures experienced away from the column.

As in almost all structural fire reconstructions the greatest uncertainty is the prediction of the fire exposure. In the situation where there is a large thermal capacitance of the exposed structural system, such as present when a beam is protected by SFRM, small variations in the fire exposure are thermally damped by the SFRM. To test this concept, the thermal modeling was repeated for a temperature history, which quickly grows to 1000\(^\circ\)C for \( \sim 30 \) minutes before dropping to 720\(^\circ\)C until the decay phase of the exposure. After 2 hours of exposure the average steel temperature is \( \sim 700^\circ C \), which does not mark a dramatic change from the original scenario.

The results of the thermal-stress model (a combination of the thermal and structural models) predicted 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 deflected 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 of fire exposure, 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 11).

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 approximately 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 (see Figure 12, showing the model result at initiation of prying and the

![Stress distribution (2 hours of fire exposure).](image11)

**Figure 11.** Stress distribution (2 hours of fire exposure). (The color version of this figure is available online.)

![Equivalent strain comparison.](image12)

**Figure 12.** Equivalent strain after 2 hours of fire exposure (finite element model) compared to a recovered sample [1]. (The color version of this figure is available online.)
damaged web after the failure). Moreover, photographs of the interior collapse area show that the failed girder stubs are deformed at the fulcrum points (Figure 13).

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

**CONCLUSIONS**

The fire that occurred within WTC 5 is estimated using relevant data and computer modeling. The original structural drawings of WTC 5 are referenced to determine the dimensions of the steel assembly accurately. Using this information, finite element models are created. It is predicted that WTC 5 experienced a catastrophic, progressive structural collapse during the heating phase of its fire exposure. More precisely, the thermal-stress model predicts a ‘runaway’ bolt rupture failure at ~2 hours of real fire exposure.

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 structural
failures occur (e.g., One New York Plaza). NIST concluded that WTC 7 failed during the heating phase of its fire exposure, but the failure mechanism was different from WTC 5 [10]. WTC 5 was sensitive to early failure because the Gerber beam design, which has simple connections located away from columns, isolated the shear connections from their heat sinks, i.e., the rest of the ‘cooler’ structure.

The collapse involved four floors, and might have progressed all the way down to the ground level, if it had not been for the moment-type connections utilized for the 4th floor. The 9th floor of the building experienced a similar fire exposure as the 8th floor, but it did not collapse. In fact, forensic evidence demonstrates that the beams on the 9th floor reached the catenary phase and remained stable (Figure 4). The only difference between the structural assembly on the 8th floor and that on the 9th floor is the location of the shear connections; on the 9th floor, the connections were made at the columns.

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 for fire exposure.

The present approach to structural fire protection in much of the USA is primarily prescriptive, often employing propriety products to insulate structural elements and active fire suppression systems to control fire growth. Most commonly, structural fire protection systems are validated using fire tests in accordance with the ASTM E-119 standard [11]. Structural connections are never included in these tests; therefore, such approaches would not lead to an appreciation for vulnerabilities such as apparently existed in some of the structural detailing in WTC 5. Analytical, performance-based approaches, more akin 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 structural detailing changes could have enhanced the structure’s fire resistance. Greater clear distance for the bolt holes from the edge of the girder stub web would have enhanced the tear out strength of the structural assembly. Alternatively, slotted holes in the girder webs would have allowed more girder rotation without developing.

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2One New York Plaza was a 50-story office building which experienced a two-floor fire in 1970. There is forensic evidence that suggests that certain steel connections failed during the cooling phase of the fire, but these failures did not precipitate a disproportionate collapse.
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.

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