Structural Failure in Building Fires - Lessons Learned from 9/11

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ABSTRACT

One of the aftermaths of the collapse of the twin towers on 9/11 was a fire and subsequent collapse of major portions of World Trade Center 5 (WTC 5) and the complete collapse of World Trade Center 7 (WTC 7). These failures were due to connection detailing that did not account for the impact of fire.

In the case of WTC 5, a steel column-tree failed during the heating phase of the fire. Historically, it has been shown that a catastrophic failure during the heating phase of a fire endangers the lives of fire fighters and building occupants undergoing extended egress times. Typical facilities with extended egress times include high-rise buildings, or buildings relying upon defend-in-place strategies such as prisons, nursing homes and hospitals.

Computer software was used to reconstruct the WTC 5 fire event and predict the performance of the structural assembly when exposed to the fire. This analysis confirmed the failure was due to an unusual connection detail. In the case of WTC 7, the official study hypothesized that the failure was also due to the connection detail, one that is common in many unusual and non-symmetric framing schemes.

KEYWORDS

Fire; Structural failure.

INTRODUCTION

In 2001, one of the first major failures of a protected steel framed structure occurred due to an unwanted fire. This was repeated several times that day in the aftermath of the 9/11 terrorist attacks on the World Trade Center in New York City. This paper focuses on two of the failures, the first in World Trade Center building 5 (WTC 5) and the second in building 7 (WTC 7).

WTC 5 was a nine storey building constructed using a structural steel frame. There were two main framing systems; a moment resistant frame in the outer bays and the top storey (supporting the roof structure) and a simple frame using Gerber beams throughout much of the interior. WTC 5 was partially damaged the morning of 9/11 when World Trade Center 2 collapsed and flaming debris penetrated some of the building resulting in damage to WTC 5's building envelope and ignition of some of the building's contents. Major internal failures occurred as a result of the subsequent fire.

WTC 7 was a 47-storey structure with an unusual structural steel framing system due to site constraints. WTC 7 was damaged following the collapse of World Trade Center 1 (WTC 1) when the south façade was penetrated on multiple floors by flaming debris from WTC 1. The resultant fires in WTC 7 continued unabated throughout the day leading to a total collapse of the building.

This paper discusses the significant design elements that contributed to these structural failures and presents some thoughts on their significance.

BACKGROUND

The effect of fire on steel structures has been a topic of interest since the first steel buildings were designed and built. As steel structures replaced the inherently fire resistant masonry ones, engineers were faced with the challenge of providing a suitable means to prevent fire induced collapse in what would soon become a leading structural framing material in North America with use throughout the world. During the modern era this has resulted in the development and implementation of standard test protocols such as AS1530 Part 4 (which is similar to ISO834 and based on the developmental work for ASTM E119). By modern era we are referring to the late 1920s when researchers (led by Ingberg at the US National Bureau of Standards) conducted a series of large-scale tests to verify the appropriateness of this test method. The utility of the test protocol has resulted in its incorporation within most modern building codes. However, there are significant limitations (see for instance Barnett (1985, 1991); Buchanan, 2001; Parkinson, 2002; Underwriters Laboratory, 2004). These limitations include:

- The test is a large scale but not a full-scale test and "does not by itself incorporate all factors required for fire hazard or fire risk assessment of the materials, products or assemblies under fire conditions" (Underwriters Laboratory, 2004).
- The fire condition is representative of a fire test furnace, with the heat energy impact controlled by gas temperature rather than incident heat flux.
- There is no direct correlation between the fire test results, reported in units of time, and real building fire performance. For example, a test result of 120 minutes may be representative of a structural assembly that fails during a real fire in 240 minutes or in 15 minutes.
- In the case of structural framing systems, only individual beams (girders) and columns are tested. The effect of the connection details on the structural system's fire performance is ignored.

As an alternative to this prescriptive approach to structural fire safety, calculation techniques have been developed since the late 1970s to allow engineers to predict the heat energy exposure from real building fires, the impact of this heat energy exposure on structural elements and the resultant structural behaviour (Barnett, 1985).

WORLD TRADE CENTER 5 (WTC 5)

The fires that resulted from the impact of burning debris subsequent ignitions inside WTC 5 during the 9/11 events, resulted in an internal disproportionate structural frame failure and collapse in the eastern portion of WTC 5 from the forth to the eighth floor. This was the first recorded massive failure of a protected structural steel framed building.

As reported by ASCE/FEMA (2002) the nine-storey (see Figure 1) steel framed building's floors consisted of 0.1 m (four-inch) thick lightweight concrete fill on metal decking. Floors five through eight used a Gerber Beam design consisting of a 1.22 m (four-foot) long beam stem shop-welded to a column, forming a column tree, with a simply-connected floor girder bolted to the beam stem using sheer tabs. A typical section is illustrated in Figure 2 (LaMalva, 2007). The Gerber beam design was developed by Gerber in the 19th century and locates structural hinges at inflection points to allow settlement without an increase in the maximum bending stress (Schierle, 2011). The main advantage of the Gerber design is that the high cost of field erected moment splices is avoided and the reduced depth of the girder allows for easy routing of building services without the need to penetrate structural webs. This design approach was popular in the US through the 1970s. There appears to have been a worldwide resurgence of the Gerber beam design in recent years, due to the changing cost of steel construction. However, there is no indication that Gerber beams have been used in Australia.

Figure 1. WTC 5**.**

The areas of WTC 5, which did not collapse, used more conventional systems consisting of either a moment frame or girders resting on column seats bolted directly to a column. The steel framing was protected against a fire's heat energy impact by a sprayed vermiculite coating (Grace Monokote MK-5). The coating thickness was designed to provide the structural adequacy component of the fire resistance level (FRL)[1] of 3-hours for columns, and 2-hours for girders, beam stems, shear tabs and the concrete slab.

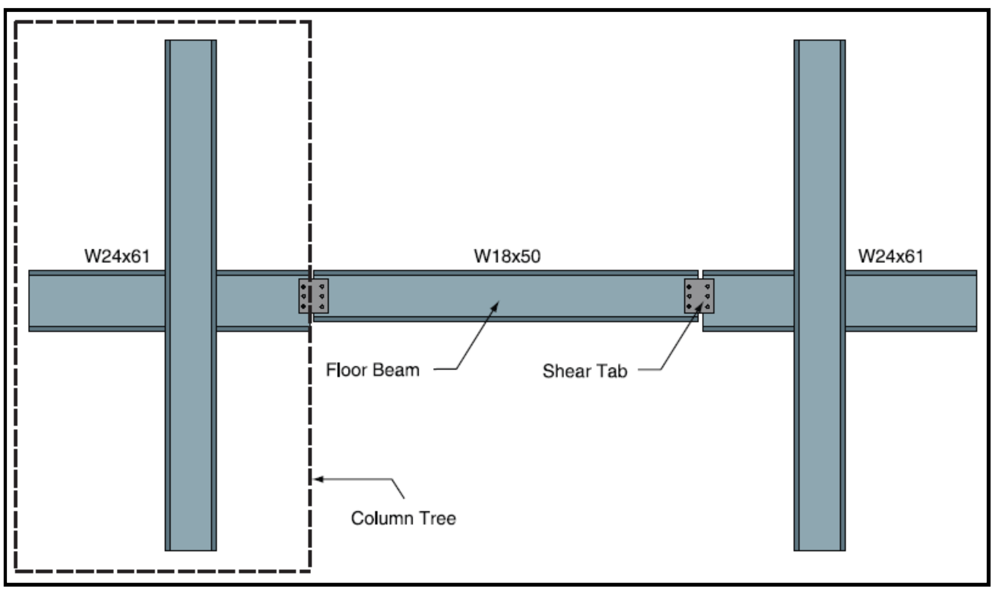

Figure 2. Typical Gerber beam configuration (not to scale).

[1] In the US the lowest value of the FRL is reported as the "Fire Resistance Rating". For structural components the only value that is relevant is the first of the three numbers reported in the FRL, or the "Structural Adequacy".

The Collapse

As illustrated in Figure 3 (ASCE/FEMA, 2002) the column tree failure was extensive. As observed by the author during a visit to the site, the failure was due simply to the fire as there was no evidence of impact damage in the vicinity of the collapse. This failure raised two major questions,

1. Was the collapse due to a defect in construction or design (for example was there insufficient fire protection?
2. Did the collapse happen during the heating phase or the cooling phase of the fire?

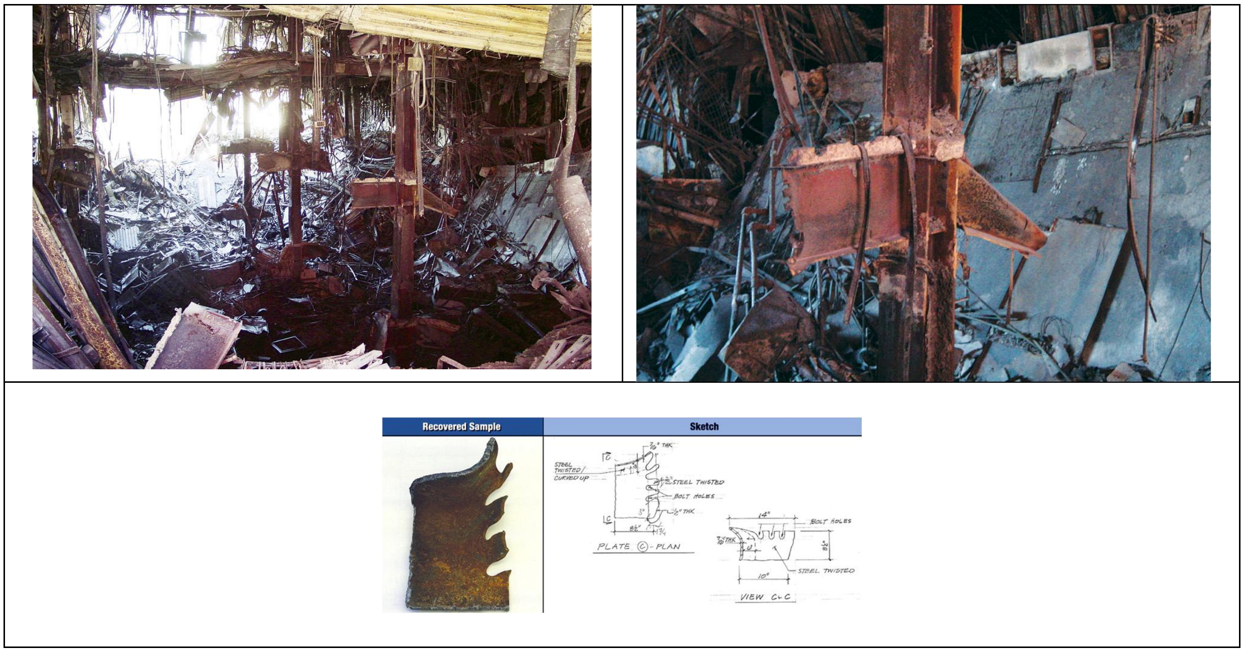

Figure 3. Column Tree Failures

The forensic evidence suggested that the building was compliant with the fire protection requirements of the local building code at the time of construction; areas where the spray on vermiculite protection was in place were protected with appropriate amounts and types of insulation. Likewise, because the columns remained plumb after the collapse, there is a strong likelihood that the collapse occurred during the heating phase of the fire. If it had occurred during the cooling phase, during uneven temperature and loading conditions, it is more likely than not that the columns would not have remained plumb. The author implemented a research program to determine if this qualitative assessment could be verified.

Analysis of WTC 5

As part of his post-graduate studies, Kevin LaMalva (2007) used computer analysis techniques to predict the failures that were observed by the author. LaMalva modelled the fires using the zone fire model CFAST Version 6 (Peacock 2008). This provided the thermal loading input he used as input to an ABAQUS (ABAQUS, 2004) Version 6.6 model. ABAQUS was used to model the heat transfer to the steel, the subsequent heating of the steel frame, and the structural response.

Fire Prediction. Using information gathered by ASCE/FEMA and NIST, and computer fire modelling using CFAST, LaMalva predicted the average time history of the fire in WTC 5. This is depicted in Figure 4 where it is compared to the standard furnace time-temperature curve used in AS 1530 Part 4. Of interest is that the predicted peak temperature of the actual fire occurs at 12 minutes exposure in the AS 1530 furnace fire.

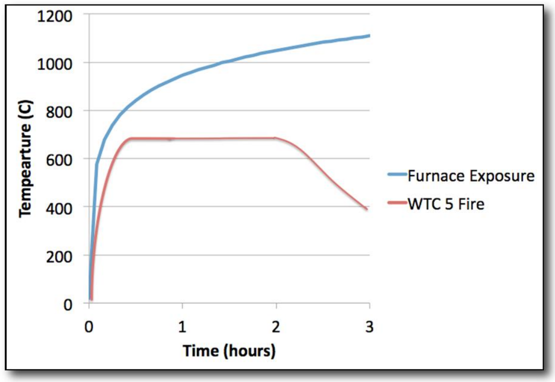

Figure 4. Time-Temperature Exposure

Structural Response. Using the finite element mesh depicted in Figure 5, LaMalva used ABAQUS to predict the time to failure of the shear tab connection between the girder and the beam stem using a sequentially-coupled, thermal stress model. After modelling the transient behaviour of the shear tab exposed to the WTC 5's temperature the ABAQUS results are presented in Figure 6. The maximum predicted average steel temperature was 450 °C. The predicted temperature at the boltholes was 660 °C. The stress levels at the boltholes indicated imminent failure. At this point in time the maximum plastic shear strain at the top bolthole was predicted to be 0.47. Therefore, the thermal-stress model predicted a "runaway" bolt rupture failure at approximately two hours of fire exposure, during the heating phase of the fire. In addition, a qualitative comparison of the predicted deformations matched the deformations observed in the samples collected at the site.

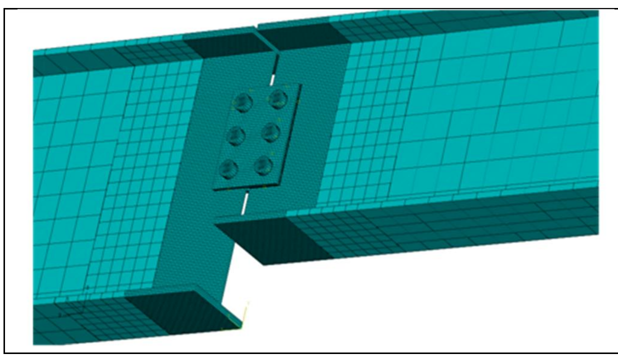

Figure 5. ABAQAS Finite Element Mesh

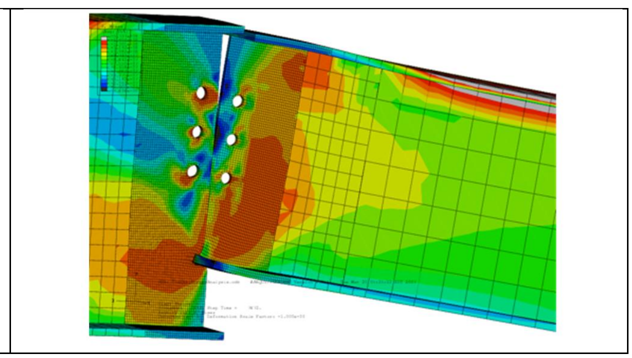

Figure 6. Stress Distribution at two hours

Analysis. The structural failure in WTC 5 occurred at much lower fire temperatures than would be expected for a protected steel system. The failure most likely occurred at a fire temperature of 700 °C whereas for the case of a structure with an FRL of 2-hours, the fire exposure temperature would be 1049 °C. This is primarily because the failure was due to a failure at the connection of the girder to the beam stem, not a failure of the girder itself.

WORLD TRADE CENTER 7 (WTC 7)

WTC 7 was damaged when debris from WTC 1 entered the building on multiple floors (this occurred around 10:30 a.m. on 11 September 2001). The debris damaged the south facing façade but there was no indication that the structural system was damaged. The debris was the ignition source for a series of fires, which spread around the building. The fires continued until 5:20 p.m. when the building collapsed. The ASCE/FEMA (2002) teams conducted the initial investigation of the collapsed followed by the National Institute of Standards and Technology (NIST), which lead a major investigation (Gann, 2008). Other studies were conducted as part of litigation surrounding the event. Nonetheless, as limited forensic work was conducted at the scene, much of the following assessment is based on the theoretical studies reported by NIST (Gann, 2008).

Figure 7 - WTC 7 in the dust cloud from the collapsing WTC 1 tower

WTC 7 was a 47-storey office (see Figure 7 (ASCE/FEMA, 2002)) building completed in 1987. It was roughly trapezoidal in shape with a length of 100 m on the north, 75 m on the south and 45 m wide. It had an unusual design as the original plan was for a much smaller floor area. Caissons were installed for this original design around and through a large subsurface electrical substation. The final design required a unique structural framing grid to transfer the load to the original caissons. The superposition of the original foundation plan with the final one as well as the resultant unsymmetrical typical floor-framing plan is shown in Figure 8 (ASCE/FEMA, 2002).

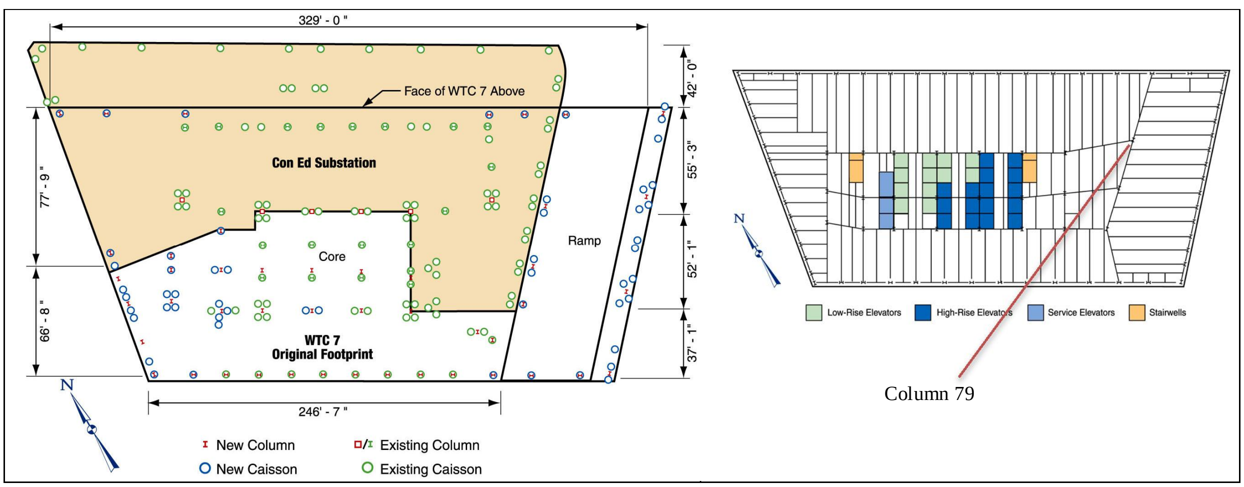

Figure 8. WTC 7 Foundation Layout and Typical Framing Plan

Failure Analysis of WTC 7

The initial fires in WTC 7 started near the south façade and with time progressed in a clockwise direction around the building. By about 4:00 p.m. the fire had reached the east end of the 13th floor. The structure in this portion of the building was protected with a sprayed vermiculite coating, which provided an FRL of 3-hours for the columns and 2-hours for the beams (ASCE/FEMA, 2002). This eastern end of the building was the location of column 79. As illustrated in Figure 9, the framing into column 79 consisted of two roughly north-south pointing girders and two east-west girders. If we consider the girder spanning between column 79 and column 80 we note that there are long beams framing into this girder supporting the floor slab towards the exterior façade to the east. This is similar to the situation between column 79 and column 44. As the fire progressed to this portion of the floor plate, these long beams expanded, pushing against the girders between columns 79 and column 80 and between column 79 and column 44. Unlike a normal Cartesian framing system, there were no beams framing into these girders to the west. Therefore, the only mechanism to keeping the girders on their column seats at column 79 were simple connections consisting of a shear plate and a couple of bolts. This connection was not designed for the stress caused by the thermal thrust from the beams framing into the girders. Computer analysis using ANSYS predicted that with temperatures as low as 400 °C these simple connections would fail and the beams would push the girders off the column 79 supports. NIST's study showed that a failure of this type on the 13th floor would lead to a general cascading failure resulting in the disproportionate collapse of the building. This is illustrated in Figure 10 showing an LS-DYNA simulation of WTC 7 (Gann, 2008).

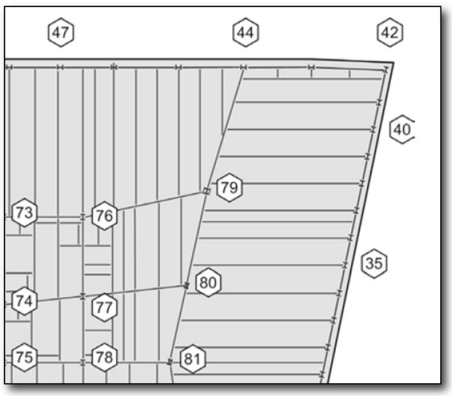

Figure 9. Framing Near Column 79

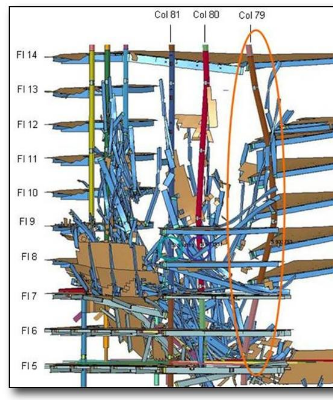

Figure 10. Predicted Collapse Around Column 79

Discussion

Failure in AS1530 Part 4 is normally considered to occur with an average beam temperature or a column temperature greater than 538 °C. In WTC 7 column 79 most likely buckled when its temperature was less than 300 °C. The failure, due to thermal expansion of girders and beams and the resultant lack of frame stability, is not predicted by the test standard.

CONCLUSIONS

The collapse of WTC 7 and the partial collapse of WTC 5 were the first major collapses of protected steel structures due to fire. The common feature in both was that collapse was due to a failure of a critical connection. In both cases collapse was not due to failure to comply with the building code. The Fire Resistance Rating of the frames met the building code requirements. If these buildings had been located in Australia they would have met the requirements of AS1540 Part 4. Nonetheless, under similar circumstances, collapse would have occurred.

The significance of this finding is that in situations where an unusual connection or framing scheme is used, simply meeting the Deemed to Satisfy provisions of the building code's FRL requirement may not result in a safe structure. Under conditions of unusual design, the design team may have to consider going beyond the minimum requirements specified in the code. This may be particularly important for facilities where a defend in place fire engineering strategy is employed, such as in health care facilities or prisons, or where an alternative solution is developed which includes a reduced FRL and where fire brigade response may conflict with the time to potential structural collapse.

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