STRUCTURAL RESPONSE OF WTC 7 FLOOR SYSTEMS TO FIRE

Therese McAllister[1], Ph.D., P.E., M. ASCE
John L. Gross[2], Ph.D., P.E., F. ASCE

[1]National Institute of Standards and Technology, Building and Fire Research Laboratory, Gaithersburg, Maryland 20899; Ph: (301) 975-6078; email: therese.mcallister@nist.gov

[2]National Institute of Standards and Technology, Building and Fire Research Laboratory, Gaithersburg, Maryland 20899; Ph: (301) 975-6068; email: john.gross@nist.gov

ABSTRACT

The WTC 7 investigation by the National Institute of Standards and Technology (NIST) identified several factors that alone, or in combination, led to fire-induced failures of the floors, and subsequently, total collapse of the 47-story WTC 7 building. At the present time, a sensitivity study is being conducted to determine the relative contribution of the identified factors, which includes the presence or absence of shear studs on girders, connection types, asymmetric framing, and bay span lengths. The technical basis for the identified structural factors is presented in this paper.

INTRODUCTION

The investigation into the collapse of the World Trade Center (WTC) 7 by the National Institute of Standards and Technology (NIST) recommends that buildings be explicitly evaluated to ensure the adequate performance of the structural system under maximum credible (infrequent) design fires with any active fire protection system rendered ineffective. Of particular concern are the effects of thermal expansion in buildings with one or more of the following features:

- long-span floor systems which experience significant thermal expansion and sagging effects,
- connection designs (especially shear connections) that cannot accommodate thermal effects,
- floor framing that induces asymmetric thermally-induced (i.e., net lateral) forces on girders,
- shear studs that could fail due to differential thermal expansion in composite floor systems, and
- lack of shear studs on girders.

A detailed ANSYS[1] (2007) analysis model was used to determine how and where the building failure began in response to the uncontrolled fires that burned for hours. The model accounted for nonlinear geometric effects, temperature dependent behavior of the structural members and connections, such as thermal expansion, stiffness and strength degradation, the sequential failure of structural framing and connections under fire conditions, and removal of failed elements (with user intervention). Since the spatial and temporal changes in the temperature of structural members were slow relative to the dynamic characteristics (i.e., natural frequencies) of the building, a non-linear static procedure with an implicit solution algorithm that guaranteed equilibrium at each time step was used.

An important feature of the WTC 7 ANSYS model was the user-defined elements, also referred to as break elements, developed to model the behavior and failure of floor framing connections at elevated temperatures (see Erbay *et al.* 2010). The structural floor connections were analyzed for all possible failure modes. Break elements were used to develop models of connections between structural components that simulated possible failure modes and the associated failure loads. Use of break elements in these models allowed the analysis to continue through subsequent failures of connections. Such modeling of connections using break elements were used on the east side of Floors 7 to 14, where the fires were dominant prior to collapse.

Contact elements were used between the floor slab and girder to allow the slab to transfer gravity loads, but not shear forces, to the girder. Contact elements also allowed the girders to sag independently of the floor slab, and prevented slab penetration through the girder in the analysis. To model composite action between floor beams and slabs, break elements were used to represent shear studs. These break elements represented the temperature-dependent shear stud capacity and captured shear failures between the slab and the shear stud.

Temperature data were input at 30 min intervals for the observed fires. The temperature data were based on a Fire Dynamics Simulator (FDS) analysis of the fires. Further details of the model development are given in McAllister *et al.* (2008). The technical basis for each of the identified structural features is described in the following sections.

---

[1] Certain commercial software or materials are identified to describe a procedure or concept adequately. Such identification is not intended to imply recommendation, endorsement, or implication by NIST that the software or materials are necessarily the best available for the purpose.

CONNECTION DESIGNS AND THERMAL EFFECTS

To determine the capacity of structural connections in the composite floor system, possible failure modes were evaluated for each floor connection type. The floor connection types included single-plate (fin) shear tabs, double-angle shear connections, and seated connections. Failure modes included weld failure, bolt failure (both shear and tension), plate tear-out, and block shear failure. Connection element capacities (or ultimate strength), were based on the AISC LRFD design provisions (AISC 2005). To determine failure capacities instead of design capacities, resistance factors were set to 1.0.

The shear connections in WTC 7 were designed to support gravity loads, but they were not designed for horizontal loads from the floor beams due to thermal expansion or other fire effects.

The shear connections were evaluated for their vertical (gravity) load capacity for room temperature material properties. The average vertical failure load-to-design load ratio was approximately 3.2 for the floor beam and girder end connections in the core area and 4.4 for the floor beams and girders in the tenant area; the values ranged from a minimum of 1.9 to a maximum of 7.2. A vertical failure load to design load ratio of approximately 2.0 to 4.0 is within the typical range expected for these types of connections.

The failure mode for a connection subject to horizontal loading depends on the direction of load. For instance, if a bolted shear connection between a floor beam and supporting girder is subject to a compressive load from the beam as it undergoes thermal elongation, the failure mode would likely be bolt shear, but if it is subject to a tensile load, the failure mode could be tearout or weld fracture. Since the shear connections were not designed for horizontal loads, no comparison of horizontal load capacity to design load could be made.

Detailed connection models were developed for each connection type using beam elements, gap and contact elements, and user-defined break elements that captured the applicable temperature-dependent failure modes for vertical and horizontal loads. Failure in connections was determined by checking the exceedance of either a limiting force or a deformation limit (for example, “walking off” the seat). The inclusion of contact elements in the connection models allowed for slip and construction clearances (gaps) to be taken into account. The connection models included temperature-dependent properties and allowed for a different response in horizontal tension and compression.

The temperature of connection and shear stud components (e.g., bolts, plates, angles, and welds) were taken as the average of the temperature between two steel components (e.g., a girder and a column) for connection components and between the slab and the beam for shear studs. Connection and shear stud capacity at elevated temperature were modeled using room temperature capacity and the tensile strength reduction factor for steel at elevated temperatures.

After approximately 4.0 h of heating by fires that moved across the tenant floor areas of WTC 7[2], thermal expansion of steel beams and girders within the structural system resulted in (1) bolt shear, (2) walk-off of seated connections after bolts had sheared, and (3) failure of connection welds to beam webs. These failures were due to thermally induced axial forces in the beams and girders as their thermal expansion was restrained by the adjacent structural system.

Shear failure of all the bolts or failure of the weld in the connections resulted in a loss of horizontal and vertical support to beams or girders. In seated connections, the shear failure of bolts at the bearing seat and top clip or plate, caused loss of horizontal support but not vertical support. Loss of vertical support occurred when a beam or girder "walked off" the bearing seat or when a bearing seat weld failed.

LONG-SPAN FLOOR SYSTEMS

Shear connections for floor framing typically have a nominal 12.7 mm (0.5 in.) gap between the end of the floor beam or girder and the girder or column to which it is connected. The thermal expansion of the floor beam or girder is first resisted by the end connections. For floor beams or girder with shear stud connectors to a concrete floor slab the thermal expansion of the floor beam or girder exerts shear forces on the shear stud connectors as well as the end connections. The effect of thermal expansion on shear studs is discussed in the next section.

If the shear studs and end connections (such as a seated connection) fail under horizontal forces induced by thermal expansion of the floor beam or girder, the beam or girder will continue to thermally elongate until it encounters restraint by contact with the supporting girder or column. Thus, the gap limits the length of expansion that can occur before the beam or girder is restrained by the member to which it is connected.

As a steel beam or girder is heated by a fire, it undergoes thermal expansion and elongates in proportion to its length and the temperature increase. With no restraint to thermal expansion, the change in length due to uniform heating, δ_T, is computed as (McAllister et al. 2008)

$$\delta_T = \alpha \, \Delta T \, L \qquad (1)$$

where

α is the coefficient of thermal expansion (ºC)$^{-1}$

---

[2] Fires were observed to burn on Floors 7 to 9 and Floors 11 to 13 starting in the early afternoon until the building collapsed at 5:20:33 p.m. EDT.

ΔT is the increase in temperature, ºC

L is the length if the member (in.)

For example, if a 13.7 m (45 ft or 540 in.) long floor beam or girder is uniformly heated so as to increase its temperature by 600 ºC, and the coefficient of thermal expansion is taken to be 1.4×10^{-5} / ºC, the elongation would be,

$$\delta_T = (1.4 \times 10^{-5} / ºC) \times (600\ ºC) \times 13.7\ m\ (540\ in.) = 114\ mm\ (4.5\ in.)$$

Table 1 shows examples of the temperature increase required for steel members of varying lengths to elongate by 25.4 mm (1.0 in.).

Table 1. Examples of Thermal Elongation of Steel Members

Member Length	Temperature Increase Required for Member to Elongate 25.4 mm (1.0 in.)
13.7 m (45 ft)	132 ºC
9 m (30 ft)	198 ºC
6 m (20 ft)	297 ºC

In the WTC 7 analyses (where beam lengths were up to approximately 16 m (50 ft)), and for the shorter beam lengths in Table 1, significant elongations occur below temperatures of associated with a decrease in steel strength and stiffness (i.e., approximately 400 ºC). Additionally, forces associated with restraint of thermal elongation can be well in excess of forces that cause yielding or fracture.

SHEAR STUD CONNECTORS IN COMPOSITE FLOOR SYSTEMS[3]

A shear stud connector in a composite floor system is shown in Figure 1. Two sources were found for predicting shear stud strength in a composite floor system with a metal deck. For shear studs in a 76 mm (3 in.) metal deck, where the applied load is perpendicular to the metal deck ribs, the shear stud strength, Q_{sc}, is given by (Rambo-Rodenberry, 2002)

$$Q_{sc} = R_p\ R_n\ R_d\ A_s\ F_u = 19.5\ \text{kip} \qquad (2)$$

where

[3] The policy of NIST is to use metric units in all its published materials. The equations in this section were developed for use by the U.S. construction industry in inch-pound units, so the metric conversion is not presented in this section.

R_p = 0.68 for e ≥ 2.2 in. (e is the distance from center of stud to mid-height of deck web on the loaded side) for strong position studs

R_n = 1.0 for one stud per rib or staggered position studs

R_d = 1.0 for all strong position studs

A_s = 0.44 in.^{2} for 0.75 in. shear stud cross sectional area

F_u = 65 ksi (this ultimate strength value has been shown to accurately predict published experimental results for shear stud failure in a composite floor system)

Values for the shear stud strength, Q_n, as given by AISC (2005) Table 3-21 for metal deck ribs that are perpendicular to the floor beam are

Q_n = 21.5 kip Strong direction of stud per rib

and

Q_n = 17.2 kip Weak direction of stud per rib

The average of the strong and weak direction capacities give a shear stud strength of

Q_n = 19.4 kip Average of strong and weak directions

The estimated shear stud strength by equation (2) and the average for the weak and strong direction strengths for AISC Table 3-21 were essentially the same. Since the direction of loading for a shear stud was not predictable, either because its actual position relative to the metal deck rib or the direction of thermally-induced loads was not known, the shear stud capacity was modeled as Q_{sc} = 19.5 kip. This value was used for forces that were both perpendicular and parallel to the metal deck ribs.

When a floor beam is exposed to fire, it thermally elongates. Thermal elongation is resisted by shear studs (when present) and the member end connections. The force on a single stud due to thermal elongation, Q_s, is

$$Q_s = \frac{AE(T)\alpha\Delta T}{n} \qquad (3)$$

where

A is the steel member cross-sectional area (in^2)

E(T) is the temperature dependent modulus of elasticity (ksi)

α is the coefficient of thermal expansion (ºC)$^{-1}$

ΔT is the increase in temperature, ºC

n is the number of shear studs along the member

Assume that the steel heated to 200 ºC. Substituting A=16.2 in.^{2} (W24x55), E(T)=50,000 ksi, α=1.4x10^{-5}/ ºC, ΔT=100 ºC, and n=22, one obtains,

$$Q_s = \frac{(16.2)x(50000)x(1.4x10^{-5})(100)}{22} \cong 52\ kips \tag{4}$$

Thus, for a nominal shear capacity of approximately 20 kip at room temperature for a ¾ in stud, relatively low temperatures increases in the steel members can develop forces that exceed the stud shear capacity when conditions where the concrete floor slab is restrained from thermal expansion and the steel members are able to thermally elongate relative to the slab. For example, for asymmetrical framing of floor beams into girders (see next section), and the absence of shear studs on the girders leads to a floor system which the concrete slab remains restrained by the surrounding floor slab and the steel framing has little restraint to thermal expansion. Once the shear stud connectors have failed, a floor beam will have little resistance to lateral-torsional buckling and the thermal elongation will be resisted only by the end connections.

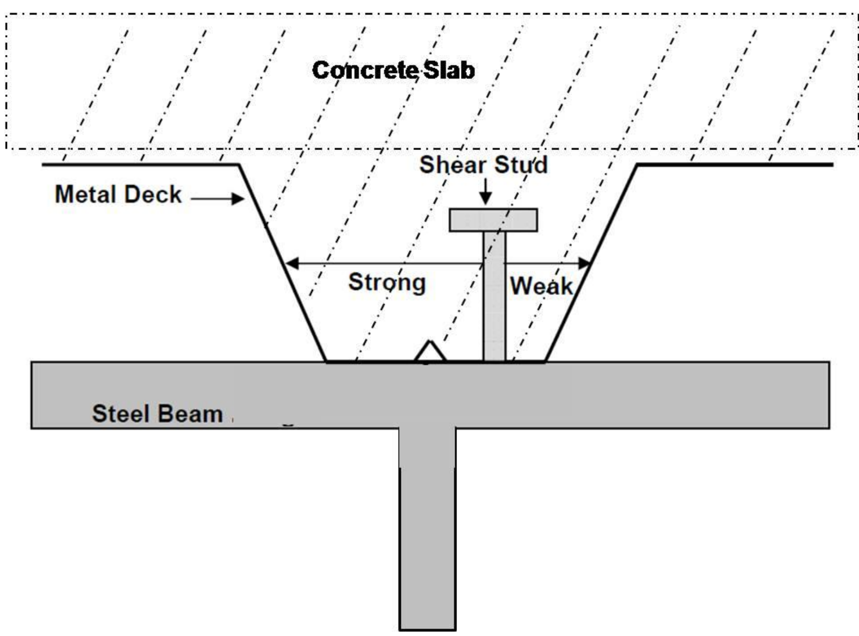

Figure 1. Shear stud connector in a composite floor system (McAllister *et al.* 2008).

ASYMMETRIC FLOOR FRAMING AND LACK OF SHEAR STUDS

Asymmetric framing refers to floor beams framing into a girder from one side only, as shown in Figure 2. The floor beams provide lateral bracing to the girder under gravity load conditions. In WTC 7, floor beams framed into only one side of a girder in several locations. Additionally, only the floor beams and exterior girders (spandrel beams) had shear stud connectors to the concrete floor slab; the girders did not have shear stud connectors.

When the floor beams are heated in an asymmetric floor framing arrangement, they thermally expand and laterally push near the top of the girder, as shown in Figure 3. The thermal elongation of the floor beams is initially restrained by the shear studs, but once the shear studs fail at relatively low temperatures, there is little restraint to the thermal elongation of the floor beams from the weak axis of most girders.

In WTC 7, the lateral stiffness of the girder about its weak axis was about three orders of magnitude smaller than the axial stiffness of the floor beams. The lateral displacement of the girder was only resisted by puddle welds between the metal deck for the concrete slab and the top flange of the girder and any friction that developed between these two surfaces after the puddle welds failed. The girder lost vertical support when the girder end "walked-off" the bearing seat. This occurred when beams that framed into the girders from one side thermally expanded and the resulting compressive forces in the beams pushed laterally on the girder from one side, sheared the girder bolts at the seated connection, and then continued to laterally push the girder until it walked off the bearing seat.

PLANNED SENSITIVITY STUDIES

To address the structural factors identified in the WTC 7 Investigation that alone, or in combination, led to fire-induced failures of the floors, a sensitivity study is being conducted. The study will consider steel-framed structures with composite floors to determine the relative significance of the presence or absence of shear studs on girders, connection types, asymmetric framing, and bay span lengths. Planned analyses include the following parameter variations:

- Skew framing (compare skew and rectangular)
- Studs on beams (spacing and/or capacity)
- Studs on girder (add studs)
- Opposing beam framing
- Girder seat connections (replace with alternative design, e.g., double angles)
- Beam connections (replace with alternative design, e.g., double angles)
- Slab boundary conditions
- Span lengths

While the listed structural features will be varied separately, it is likely that they are not independent parameters. The structural response of a composite floor system depends on the combined effect of these features, the degree of which depends on the particular geometry and construction of a floor system and heating scenario.

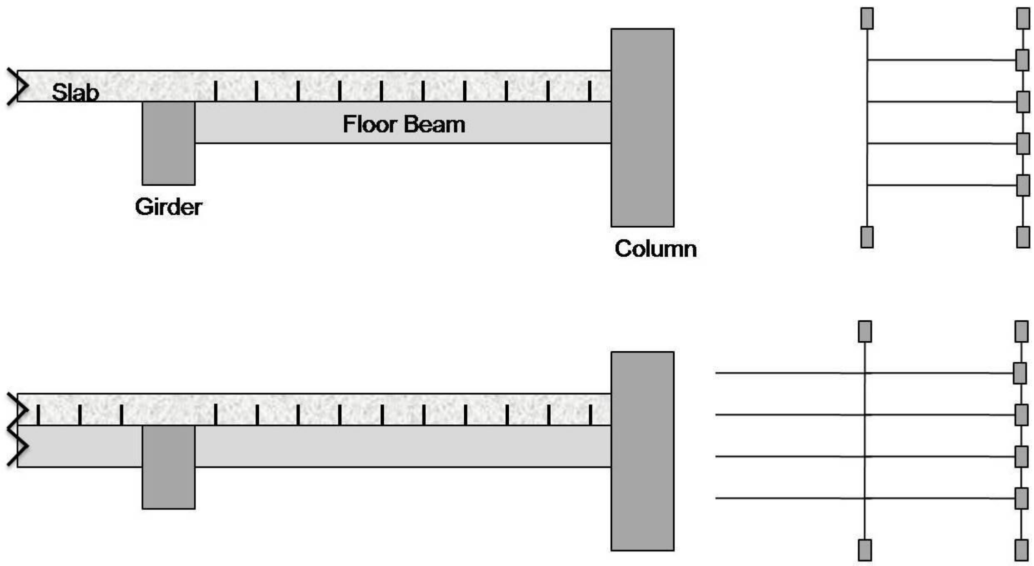

Figure 2. Schematic of asymmetric floor framing (top) and symmetric floor framing (bottom) relative to the girder.

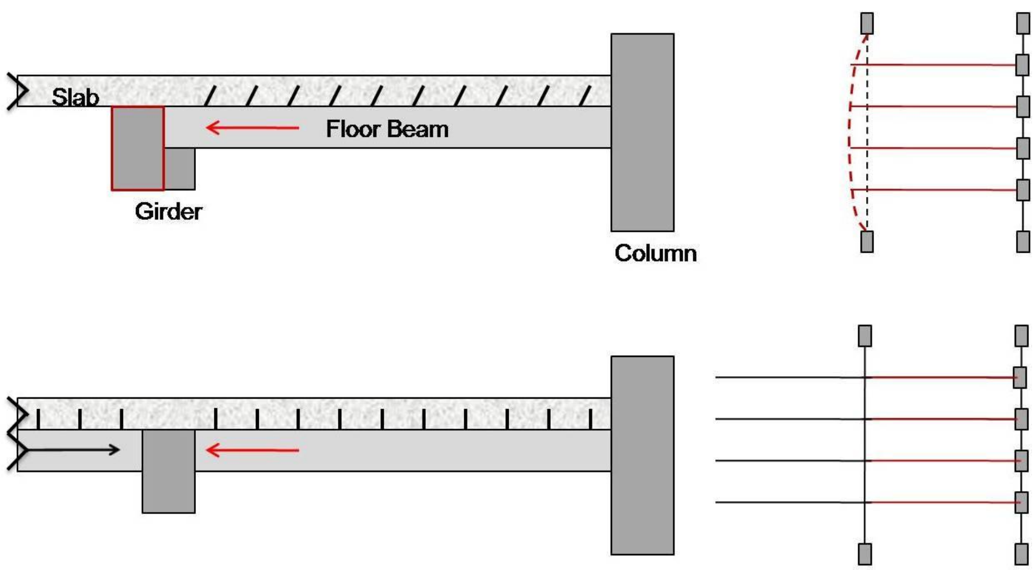

Figure 3. Schematic of effects of heating from fire on asymmetric floor framing (top) and symmetric floor framing (bottom).

REFERENCES

AISC (2005), American Institute of Steel Construction Inc., *Manual of Steel Construction*, Thirteenth Edition, Chicago, IL.

ANSYS (2007), ANSYS Mechanical Release 11.0, ANSYS Inc., Southpointe, 275 Technology Drive, Canonsburg, PA 15317

Erbay, O.O., A.T. Sarawit, and M.S. Zarghamee (2010). *Modeling behavior and failure of steel connections subjected to elevated temperatures*, Proceedings of the ASCE Structures Congress 2010, Orlando, FL.

McAllister, T. P., R. G. Gann, J. D. Averill, J. L. Gross, W. L. Grosshandler, J. R. Lawson, K. B. McGrattan, H. E. Nelson, W. M. Pitts, K. R. Prasad, F. H. Sadek. (2008). *Federal Building and Fire Safety Investigation of the World Trade Center Disaster: Structural Fire Response and Probable Collapse Sequence of World Trade Center Building 7.* NIST NCSTAR 1-9. National Institute of Standards and Technology. Gaithersburg, MD, November.

M. D. Rambo-Roddenberry (2002), *Behavior and Strength of Welded Stud Shear Connectors,* Dissertation submitted to the faculty of the Virginia Polytechnic Institute andState University, April 8, 2002