Failure of Welded Floor Truss Connections from the Exterior Wall during Collapse of the World Trade Center Towers

Failure of connections, as a result of overloading, occurred within the heat-affected zone of the base metals

BY S.W. BANOVIC AND T. A. SIEWERT

ABSTRACT. As part of the National Institute of Standards and Technology World Trade Center Investigation, failure modes of the connections attaching the composite floor system to the exterior wall of WTC 1 and WTC 2 were surveyed. Metallographic analyses of intact and failed welds of the main load-bearing truss seats complemented the survey to identify the location of metallurgical failure for these connections. Above the aircraft impact floors (94th to 99th in WTC 1 and 77th to 85th in WTC 2), the failure modes were randomly distributed. However, over 90% of floor truss connections at or below the impact floors of both buildings were either bent downward or completely sheared from the exterior wall suggesting progressive overloading of the floors below the impact zone following collapse initiation of the towers. Depending upon joint geometry, detachment of the main truss seats occurred either by fracture in the heat-affected zone of the base material, where the standoff plate detached from the spandrel, or through the weld metal, where the seat angle detached from the standoff plate. Failure in both cases was the result of a shear mechanism due to an overload condition. Exposure to fires prior to the collapse was not found to have an effect on the failure mode of the floor truss connections.

Introduction

A primary goal of the World Trade Center (WTC) investigation conducted by the National Institute of Standards and Technology (NIST) was to explore the building materials and construction and the technical conditions that contributed to the outcome of the disaster (Ref. 1). From an engineering standpoint, it was important to determine why and how WTC 1 (North Tower) and WTC 2 (South Tower) collapsed following the impacts of the aircraft, in order to apply the lessons learned to existing and future structures. The findings and conclusions of the WTC Investigation are intended to serve as a basis for 1) improvements in public safety through the way buildings are designed, constructed, maintained, and used, and 2) recommended revisions to current codes, standards, and practices regarding these issues. As part of the WTC Investigation, the Metallurgy Division and Materials Reliability Division of NIST analyzed the quality of the steel, weldments, and connections and assessed the damage and failure modes of the structural steel components. The overview report of the mechanical and metallurgical analysis of the steel (Ref. 2), as well as the complete technical reports covering all other aspects of the investigation, can be obtained on the NIST WTC Website (*http://wtc.nist.gov/*).

Gross and McAllister (Ref. 3) discussed the probable collapse sequence, based upon experimental work and finite element analyses, for the WTC towers that specified the main structural events leading to collapse initiation; the reader is referred to that document for details leading to the collapse. Immediately after collapse initiation, the potential energy of the structure (physical mass of the tower) above the impact floors (94th to 99th in WTC 1 and 77th to 85th in WTC 2) was released, developing substantial kinetic energy. The impact of this rapidly accelerating mass on the floors directly below led to overloading and subsequent failure of these floors. The additional mass of the failed floors joined that of the tower mass from above the impact area, adding to the kinetic energy impinging on the subsequent floors. The failure of successive floors was apparent in images and videos of the towers' collapse by the compressed air expelled outward as each floor failed and fell down onto the next. This mechanism appears to have continued until dust and debris obscured the view of the collapsing towers.

As the composite floor decking was most likely quite rigid due to the continuous concrete floor, the transverse bridging trusses, and the intermediate deck support angles, failure of the floor as a whole would be expected at the connections attaching the floor to the exterior wall and core. This paper characterizes the floor truss connections on recovered structural elements of the exterior wall. Damage is reported on only the exterior wall connections as the location of the exterior panels to which they were attached was known. The failure mode survey was supported with metallographic analyses of undamaged and failed welded joints to determine the location of metallurgical failure of the main load-bearing seats. The connections used in the core area are not discussed in this paper, as few were recovered and the as-built location of those that were could not be ascertained; information on these seats can be found in Ref. 4.

KEYWORDS

World Trade Center
Floor Truss Connections
Weld Failure
Collapse

Background

Tower Construction

The design of the WTC towers incorporated an innovative frame-tube concept for the structural system. Reference 1 describes the design in detail. There were four major structural subsystems — the exterior wall, the core, the floor system, and the hat truss. Figure 1 shows a typical floor plan and portions of the first three subsystems. As this paper presents data on

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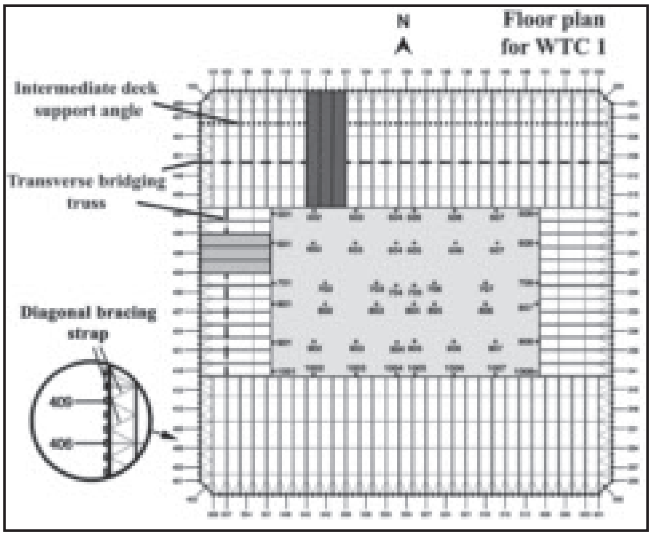


Fig. 1 — Typical floor plan showing three of the four major structural subsystems: numbered exterior columns, numbered core columns, and the floor system. Both 35 ft (light shaded) and 60 ft (dark shaded) composite floor assemblies are highlighted. Examples of traverse bridging trusses and intermediate deck support angles are also indicated.

Fig. 2 — Undated construction photograph of a composite floor assembly being hoisted into place.

the exterior wall truss connections only, the core and hat truss are not discussed further.

The majority of the buildings' exterior walls was composed of closely spaced, built-up box columns approximately 14 in. square by 36 ft long. Fifty-nine of these columns, each distinctly numbered (Fig. 1), were spaced at 40 in. on center along the face of the building. Adjacent columns were interconnected at each floor level by deep spandrels, typically 52 in. in depth. The exterior walls were designed to carry approximately 40% of the gravitational load and all of the lateral loads imposed by winds, potentially up to hurricane force.

Two lengths of composite floor trusses were used: 35 ft and 60 ft. Primary truss pairs were spaced at 6 ft 8 in. and typically constructed into assemblies spanning 20 ft in width — Figs. 1, 2. These assemblies were attached to the exterior walls at the interior intersection of the columns and spandrels — Fig. 3. Transverse bridging trusses and intermediate deck support angles directly supported the metal deck of the main trusses — Fig. 1. Once the entire floor was secured, a lightweight cast-in-place concrete slab, 4 in. thick, was poured over the metal deck.

Relevant Construction Codes

The erection contract for the two towers was signed in early 1967, and thus, the fabrication contracts were based on the standards in place at the time of the design. This meant that ASTM A 36 could be specified for the lower-strength applications, while higher-strength grades were specified as modifications of other ASTM standards (like ASTM A 242 or ASTM A 441) or even fully proprietary grades (with acceptance criteria approved by the Project Engineer). Today, some of these steels would be classified as ASTM A 572 (Ref. 5). During construction, Skilling, Helle, Christiansen, & Robertson, structural engineers for the WTC towers, specified the steel to be used for each structural piece by the minimum specified yield strength (F_y). For example, a "50 ksi steel" is a steel with a minimum yield strength of 50,000 lb/in.^{2} This convention will also be used in this paper to identify various grades of steel used in fabrication of the towers.

Likewise, the welding consumable specifications were in a period of transition. This was before the American Welding Society (AWS) had taken the responsibility for these specifications, so the submerged arc electrodes were specified according to ASTM A 558 (today, AWS A 5.17 or A 5.23). The outer columns were welded according to the bridge code, AWS D 2.0, presumably because the D 1.0 code of 1966 was limited to strengths under 60 ksi.

Floor Truss Seat Details

The composite floor system was connected to the exterior wall and core columns by truss seats, which supported the floor dead and live loads, and by strap connections, which provided horizontal shear transfer between the floor slab and exterior wall as well as out-of-plane bracing for the exterior columns not directly connected to the floor trusses. The design drawings specified 84 types of connection details, which were designated by a four-digit detail number. The differences between connection details included the type of connections used (truss seat vs. strap connection), the location and dimensions of the seat angle, the placement of bolt holes on the seat angle, the dimensions of the standoff plates, the location of stiffener plates inside column, and the location of the damping unit. For simplicity, NIST categorized the connections into three groups based upon their general characteristics. A brief description of these groupings follow, with further details in Ref. 4.

Main (or primary) truss seat. This type of truss seat includes those that belong to the 1xxx, 2xxx, 3xxx, 4xxx, and 7xxx series. They were the main load-bearing connections and were located at every other column on all four faces of the exterior wall. These seats supported the main double trusses and continuous transverse bridging trusses. For the 2xxx, 3xxx, and 4xxx series seats, two standoff plates were welded directly to the spandrel plate in a vertical manner, with beads observed only on the outer side of the plates — Fig. 4. The seat angle was welded to both faces of each standoff plate. For the 7xxx series seats, the standoff plates were omitted, and the seat angle was welded directly to the column.

The seat angles had two slotted holes for attachment of the trusses with 0.875-in.- diameter ASTM A 325 bolts. Once the floor truss was lowered into place and fastened to the seat angle by two bolts, a gusset plate with a backing bar further secured the connection. The gusset plate was welded in the field to both the top chord of the truss and the interior face of the spandrel. Directly below the seat assembly another gusset plate was welded to the spandrel on all sides. This plate contained two bolt holes to connect the viscoelastic damper unit from the lower chord of the floor truss to the spandrel (Fig. 4B). The dampers were used to reduce occupant perception of wind-induced building motion.

Intermediate deck support angle seats. This type of seat was specific for details 5010 and 5510. These seats were used only on the 200 and 400 series faces of the exterior walls (Fig. 1) for columns x04, x08, x12, x48, x52, and x56, where "x" was either 2 or 4. These were the columns to which the intermediate support angles were connected. This seat consisted of a single, triangular stiffener plate (3.5- in. right angle, isosceles triangle with 0.375- in. thickness) welded vertically to the column on all sides — Fig. 5. A second plate (8 × 5 × 0.375 in.) placed on the top edge of the stiffener was subsequently welded on all sides to the spandrel and stiffener plate to complete the seat. Similar to the main truss seats, there were two slotted holes in the horizontal plate and a gusset plate with a backing bar that was welded to the top of the support angle. Viscoelastic damper units were not specified for this detail.

Strap connections. This type of connection included details in the 5xxx and 6xxx series. These connections were located on every other column on all four exterior faces of the buildings in between the columns with main truss seats. A single gusset plate was welded on one side — Fig. 6. The shape of the plate was found to vary from a full rectangular plate to two separate tabs of 4-in. lengths to a combination of the two. Regardless of the shape, two diagonal bracing straps with shear studs (Fig. 6B) were welded in the field to the plate and attached to the top chord of the trusses. On columns adjacent to those with intermediate deck support angle seats, the gusset plate was omitted and the two diagonal bracing straps were directly welded to the spandrel interior — Fig. 7. These connections were exclusively located on the 200 and 400 series faces of the buildings, within the first 14 columns from each corner — Fig. 1.

Experimental Procedure

Failure modes of the truss seats were identified from the survey of the recovered exterior wall panels. Documentation of field observations consisted of handwritten notes and photographic images of the floor truss connections. These data were tallied and analyzed according to various criteria.

Samples for metallographic evaluation, used to identify the location of metallurgical failure for these connections, were removed from the bulk and prepared using standard techniques. Microstructures were revealed using a combination of two solutions: 1) 4 g of picric acid and 96 mL of ethyl alcohol, and 2) 2 mL of nitric acid and 98 mL of ethyl alcohol.

Results

Damage to floor truss connections were documented as part of the NIST WTC Investigation (Ref. 4). Documentation included both a comprehensive survey of failure modes and metallographic analysis of failed welded joints associated with the main load-bearing truss seats. The information was used to gain an understanding of the types of failure modes and their spatial distribution within the towers.

Table 1 — Exterior Wall Connection Details Found on Recovered and Identified Panels

			Seat Angle				Standoff Plate		Gusset Plate			
Exterior Wall Seat Detail Number	Number of Observations	NIST Type	Horizontal length (in.)	Thickness (in.)	Distance between centerline of holes (in.)	Distance between floor level and seat (in.)	Vertical length (in.)	Thickness (in.)	Dimesions (in.)	Thickness (in.)	Distance between floor level and plate (in.)	Optional stiffener (in.)
1111	14	Main truss seat	16	½	9	8½	8	5/16	na	na	na	na
1113	1	Main truss seta	16	⅜	9	10	8	⅜	na	na	na	na
1212	11	Main truss seat	16	⅜	3¼	8¾	8	⅜	na	na	na	na
1311	8	Main truss seat	16	½	10½	8½	8	5/16	na	na	na	na
1313	2	Main truss seat	16	½	3¼	8½	8	5/16	na	na	na	na
1411	45	Main truss seat	16	⅝	10½	8½	9	⅜	na	na	na	na
1511	3	Main truss seat	16	⅝	10½	8½	10	⅜	na	na	na	na
1611	2	Main truss seat	16	¾	10½	8½	11	⅜	na	na	na	na
2110	1	Main truss seat	16	⅝	8½	14⅝	9	⅜	na	na	na	na
2310	1	Main truss seat	16	⅝	10½	14⅝	9	⅜	na	na	na	na
2410	2	Main truss seat	16	⅝	10½	14⅝	10	⅜	na	na	na	na
2610	1	Main truss seat	16	¾	10¼	14⅝	12	⅜	na	na	na	na
4120	1	Main truss seat	16	½	5½	18½	9	⅜	na	na	na	na
4424	2	Main truss seat	19	⅞	2¾	22¾	17	⅜	na	na	na	na
7010	1	Main truss seat	9	⅝	2¼	13 3/16	na	na	na	na	na	na
7494	1	Main truss seat	16	⅞	3½	23¾	na	na	na	na	na	na
5010	5	Intermediate support angle seat	na	na	na	na	na	na	8 × 5	⅜	8⅞	5/16
5510	4	Intermediate support angle seat	na	na	na	na	na	na	8 × 5	⅜	8⅞	5/16
5010	2	Strap connection (with gusset plate)	na	na	na	na	na	na	8 × 5	⅜	8-⅞	5/16
5110	30	Strap connection (with gusset plate)	na	na	na	na	na	na	14 × 4	⅜	4	5/16
5210	42	Strap connection (with gusset plate)	na	na	na	na	na	na	14 × 4	⅜	4	5/16
6220	1	Strap connection (with gusset plate)	na	na	na	na	na	na	14 × 4	⅜	13 3/16	5/16
5110	7	Strap connection (with gusset plate)	na	na	na	na	na	na	na	na	na	na
5210	6	Strap connection (with gusset plate)	na	na	na	na	na	na	na	na	na	na

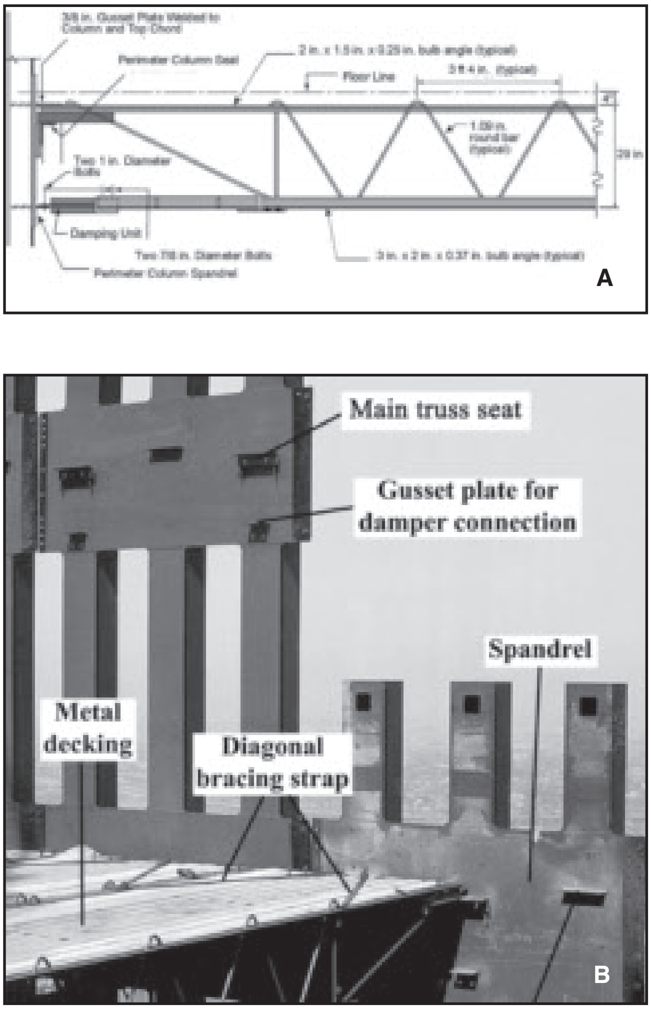


Fig. 3 —A — Schematic showing attachment of the floor truss to the exterior wall; B — undated construction photograph showing the various features of the structural subsystems and their connections.

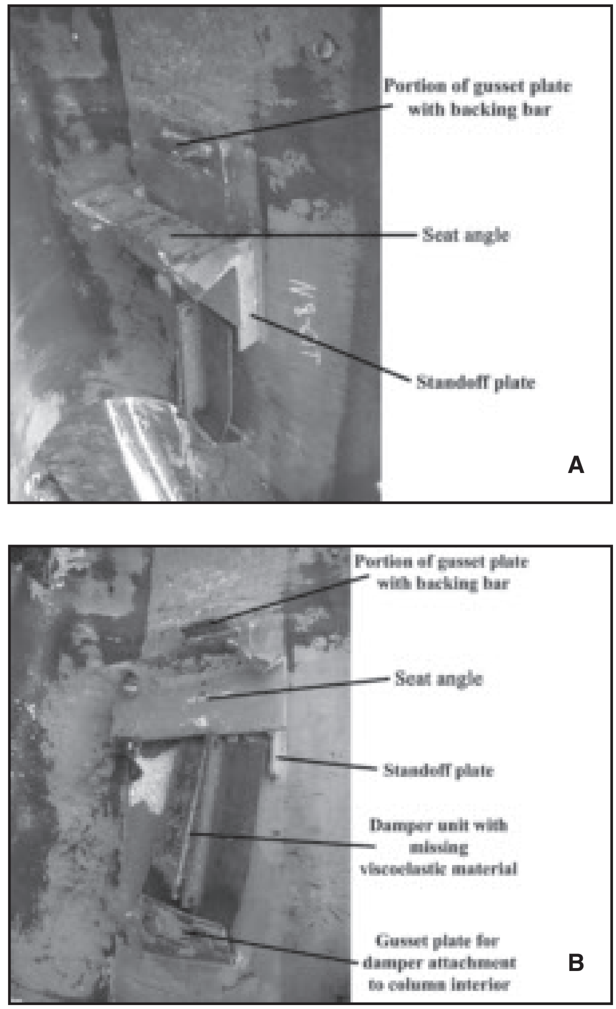


Fig. 4 — Example of a main truss seat from a recovered exterior panel (A142: 97–100). Connection shown is on column 143 at the 100th floor.

Table 2 — Statistical Data of Damage and Failure Modes for Recovered Main Truss Seats from Exterior Wall Panels

Exterior Wall Panel Description	Panels Considered	Number of Observations of Main Truss Seats	Description of Seat Angle: Intact relatively undeformed	Intact bent upwards	Intact bent downward	Intact bent inward	Missing both standoff plates remain	Missing one/both standoff plates missing
WTC 1 panels by impact region	WTC 1 panels in impact region	15	12	0	27	27	7	27
	WTC 1 panels outside of impact region	51	25	12	27	2	12	22
WTC 1 panels exposed to fire	Connectors exposed to fire	18	17	5	28	11	17	22
	Connectors not exposed to fire	48	25	10	27	6	9	23
WTC 1 panels separated by floor	Panels above 95th floor	38	36	16	21	11	5	11
	Panels at and below 95th floor	28	3	0	36	4	18	39
WTC 2 panels separated by floor	Panels above 78th floor	20	35	5	30	5	10	15
	Panels at and below 78th floor	10	0	10	20	0	40	30

Unless otherwise noted, values are in percentages of observations.

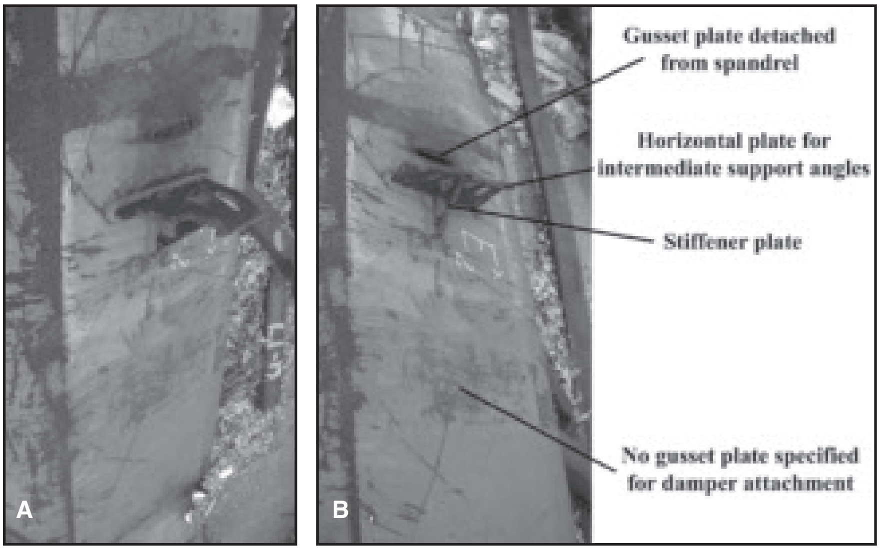


Fig. 5 — Example of an intermediate deck support angle seat from a recovered exterior panel (A451: 85–88). Connection shown is on column 452 at the 88th floor.

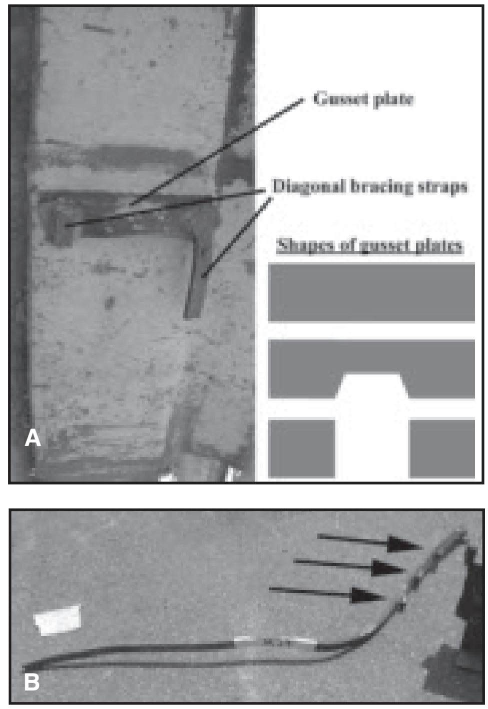


Fig. 6 — Example of a strap connection from a recovered exterior panel (A115 : 89–92). A — Recovered gusset plate, from column 116 at the 90th floor, and observed variations; B — portion of a diagonal bracing strap with shear studs (arrows).

Failure Modes of the Floor Truss Connections

Failure modes for the floor truss connections found on recovered exterior panels were surveyed. This paper focuses on panels with known locations within the towers. The truss connections were not visible in pre-collapse photographs, and thus, the analysis relied solely on the observations of the recovered components. A standard, exterior wall panel has nine connections. However, there were many cases where only partial panels were recovered, reducing the number of observations per panel. From the 42 recovered panels with identified locations, there were 21 types of connection details (Table 1). As the number of intermediate deck support angle seats (9 total) and strap connections welded directly to the spandrel (13 total) were too limited to draw meaningful conclusions, only the main truss seats and strap connections with gusset plates will be discussed further. Data for these connections can be found in Tables 2 and 3, respectively. Failure statistics regarding the intermediate deck support angle seats and strap connections directly welded to the spandrel can be found in Ref. 4.

Field observations characterized the condition of the floor truss connections. Descriptions for the main truss seats included the following:

- seat angles intact and relatively undeformed — Fig. 8A,
- seat angles intact and bent up — Fig. 8B,
- seat angles intact and bent down — Fig. 8C,
- seat angles intact and bent toward the spandrel — Fig. 8D,
- seat angles missing with standoff plates remaining — Fig. 8E, or
- seat angles missing with standoff plates missing — Fig. 8F.

Similarly, the strap connections with gusset plates were characterized as follows:

Table 3 — Statistical Data of Damage and Failure Modes for Recovered Strap Connections from Exterior Wall Panels

Exterior Wall Panel Description	Panels Considered	Number of Observations of Strap Connectors	Intact bent upward	Intact bent downward	Partial remain fracture in plate	Fracture in plate along leg of weld
WTC 1 panels by impact region	Panels in impact region	18	28	17	0	55
	Panels outside of impact region	39	13	18	13	56
WTC 1 panels exposed to fire	Connectors exposed to fire	16	13	13	18	56
	Connectors not exposed to fire	41	20	20	5	55
WTC 1 panels separated by floor	Panels above 9th floor	34	26	12	0	62
	Panels at and below 9th floor	23	4	26	22	48
WTC 2 panels separated by floor	Panels above 78th floor	12	25	17	0	58
	Panels at and below 78th floor	6	0	100	0	0

Unless otherwise noted, values are in percentages of observations

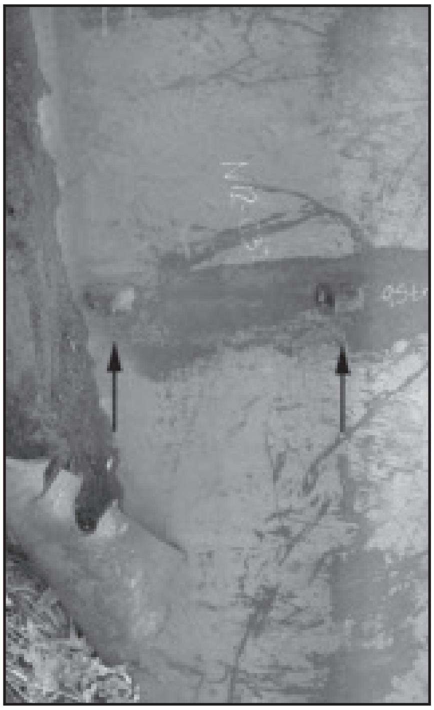


Fig. 7 — Example of strap connections without a gusset plate from a recovered exterior panel (A206: 92–95). Connection shown is on column 205 at the 95th floor. Portions of diagonal bracing strap that were directly welded to the spandrel are indicated with arrows

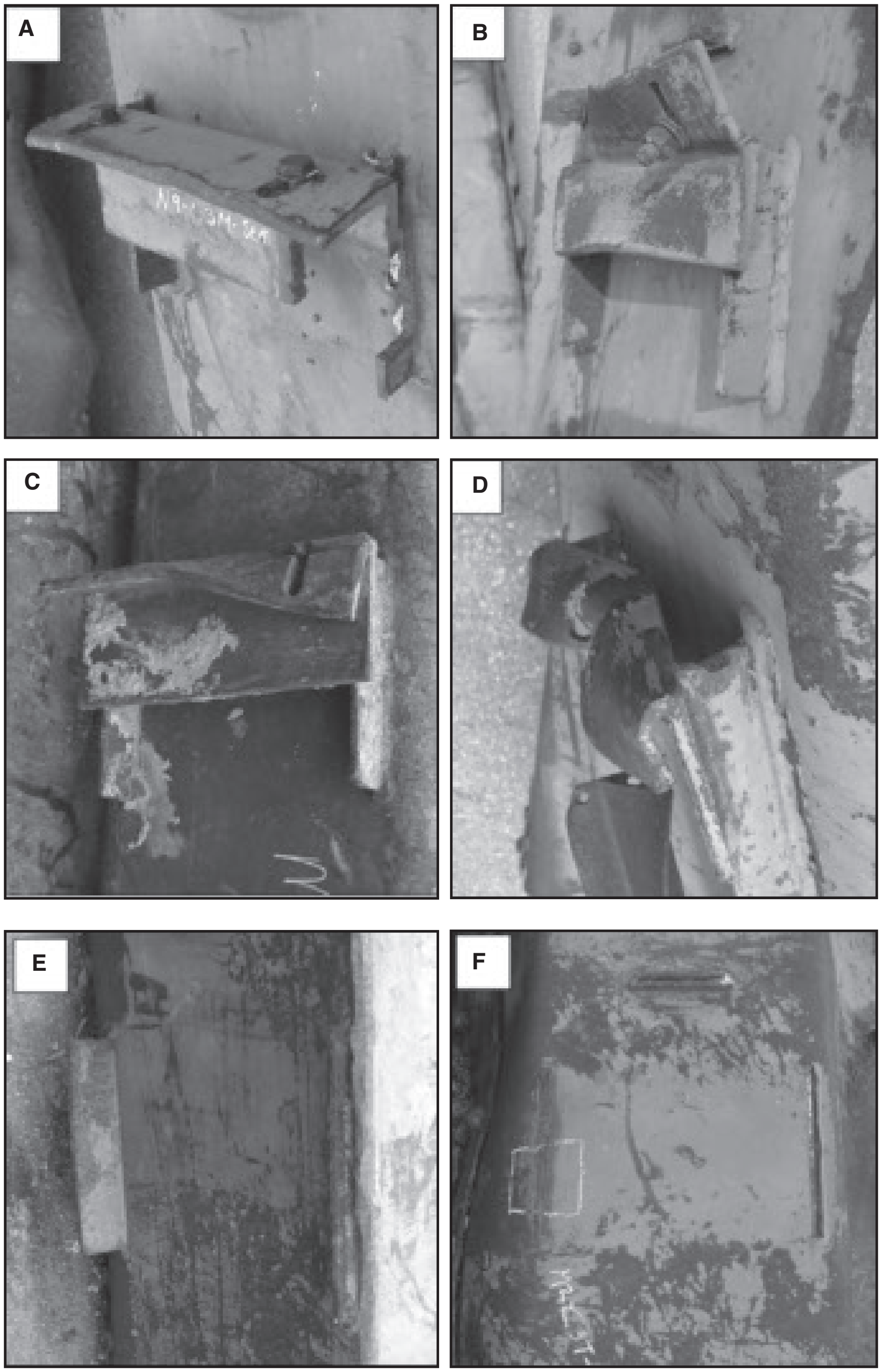


Fig. 8 — Description of main truss seat conditions from field observations. A — Intact and relatively undeformed from panel A154: 101–104 (connection is on column 153 at the 103rd floor); B — intact and bent up from panel A154: 101–104 (connection is on column 155 at the 102nd floor); C — intact and bent down from panel A130: 90–93 (connection is on column 131 at the 92nd floor); D — intact and bent toward the spandrel from panel B206: 83–86 (connection is on column 206 at the 84th floor). E — seat angle missing with standoff plates remaining from panel A124: 70–73 (connection is on column 123 at the 71st floor); F — seat angle missing with standoff plates missing from panel A130: 96–99 (connection is on column 129 at the 99th floor).

- gusset plates intact and bent upward,
- gusset plates intact and bent downward,
- fracture in gusset plate, or
- fracture along the leg of the weld.

Discussions of these damage states and failure modes follow:

Floor Truss Connections by Location Relative to the Aircraft Impact Region

Failure modes for the connections from WTC 1 were analyzed to determine whether they were different inside vs. outside the impact region. Panels with similar impact damage were not recovered for WTC 2. As the aircraft impact region of WTC 1 was located on the 100 series face (north face) of the building, only main truss seat and strap connections were located in the impact region. There were 33 observations from the impact region and 90 outside of this area. Damage statistics for main truss seats (66 total) are found in Table 2 while those for strap connections (57 total) are in Table 3. The middle portions of the seat angle were slightly more likely to be bent toward the spandrel in the impact region. This failure mode was likely a result of the aircraft impact as the columns were pushed towards the core before the top truss chords gave way or broke loose from the seat angle. The most common failure mode of the strap connections, regardless of location, was shear fracture along the leg of the weld at the gusset plate.

Floor Truss Connections Exposed to Pre-Collapse Fire

Failure modes for floor truss connec-

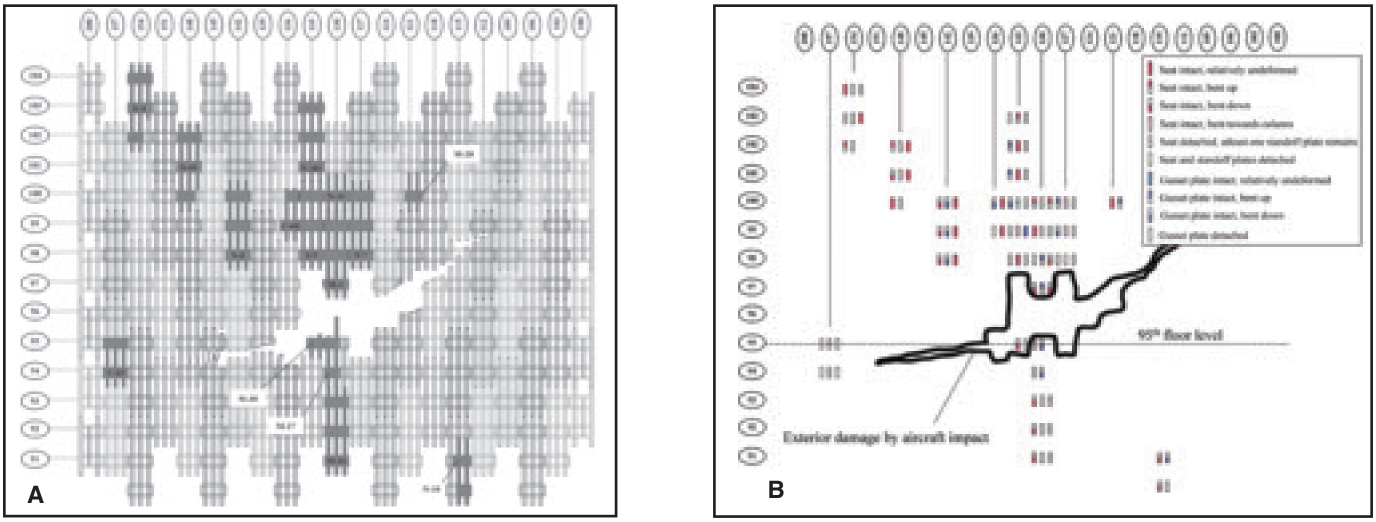


Fig. 9 — A — Damage diagram of WTC 1 north face showing the location of recovered exterior panels; B — observed condition of recovered floor truss connection; those at or below the 95th floor level are either missing or bent downward.

tions from WTC 1 were evaluated to ascertain whether exposure to pre-collapse fires produced different failure modes; similar fire-exposed panels were not recovered from WTC 2. Photographic images, video images, and other relevant information were used to develop detailed time lines for the spread and growth of fires at the peripheries of WTC 1 and WTC 2 (Ref. 6). This information was correlated with recovered exterior panels to determine the pre-collapse fire exposure for a given connection (Ref. 4). Based upon this work, 34 connections experienced direct exposure to pre-collapse fires, while results were inconclusive for 89 other connections. Tables 2 and 3, show that exposure to pre-collapse fires did not significantly change the distribution of failure modes compared to those connections that were not observed to have direct exposure to fire.

Floor Truss Connections Separated by Floor Elevation

WTC 1

Failure of floor truss connections from WTC 1 were analyzed to determine whether connections above and below the impact region failed by different mechanisms. There were 51 observations at or below the 95th floor and 72 above the 95th floor. The spatial display of a fraction of this information for WTC 1 near the impact region of the north face is seen in Fig. 9. Both pictorially and statistically (Tables 2 and 3), an overwhelming majority of main truss seats and strap connections were either bent down or completely missing below the impact zone. Below the 95th floor level, 93% of the main truss seats failed by these modes compared with only 37% of the seats above the 95th floor. For the strap connections, 96% were either bent down or torn off below the 95th floor and 74% were similarly deformed above this level.

WTC 2

The 78th floor was the lowest floor of the impact region on the south face of WTC 2. There were 32 observations above the 78th floor and 16 at or below this floor. Similar to WTC 1, the floor truss connections had a higher tendency to be bent downward or be missing below the 78th floor than above. Table 2 shows that 90% were damaged as such at or below the 78th floor vs. 55% above this level. For the strap connections (Table 3), all at or below the 78th floor were bent downward or sheared off and 75% were similarly damaged above this floor. These data cannot be easily shown spatially, similar to the image in Fig. 9, because the recovered panels from WTC 2 were more uniformly distributed on the four faces of the tower.

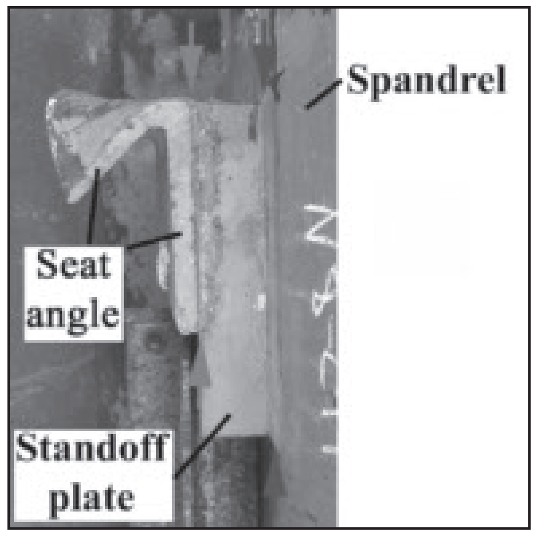


Fig. 10 — Location of the two welds analyzed from the main truss seats (A142: 97–100; column 143, 100th floor).

Characterization of Welds from Main Truss Seats

Two welded joints from the main truss seat were metallurgically examined to determine the location of metallurgical failure. These truss seats were chosen as they were the primary load-carrying connections. As seen in Fig. 10, the two welds were located between 1) the spandrel plate and standoff plates, and 2) the standoff plates and the seat angle. Both intact and damaged welds were investigated and the behaviors of the failed joints were characterized with respect to the base metal, heat-affected zone (HAZ), and weld metal.

Intact Weld between Spandrel Plate and Standoff Plates

Figure 11 shows, in cross section, one of the undamaged welds between a standoff plate and a spandrel plate. Both plates are ferrite-pearlite steels with minimum yield strengths of 42 and 65 ksi, respectively (1 ksi equals 1000 lb/in.^{2}). As stated previously, the two standoff plates were fillet welded directly to the spandrel plate. Multipass beads were observed only on the outboard face of the plates. The specific arc welding process was not found

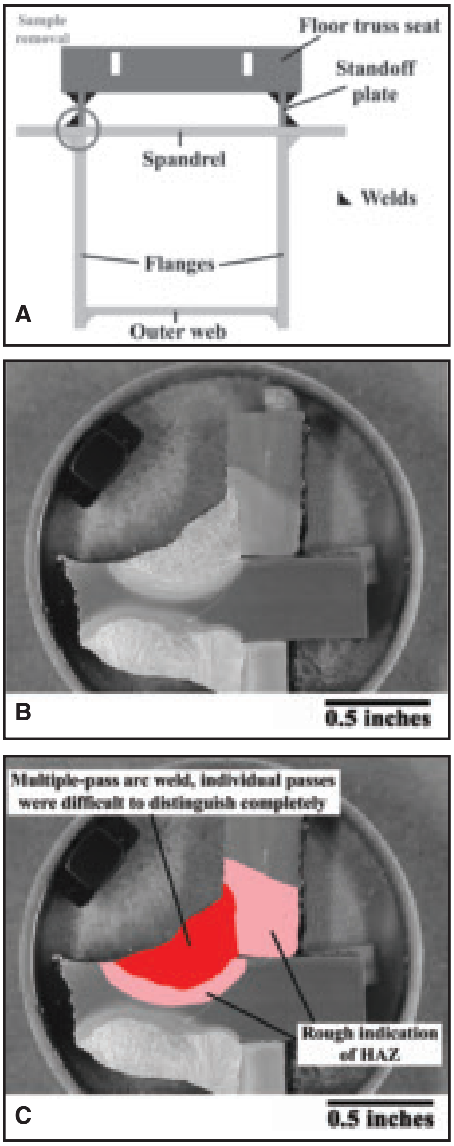


Fig. 11 — Cross section of fillet weld between spandrel and standoff plate from an undamaged main truss seat (A157: 93–96; column 157, 94th floor). A — Schematic showing the location of sample removal; B — metallographically prepared specimen; C — HAZ and weld were determined via optical means. Two percent nital and four percent picral etch.

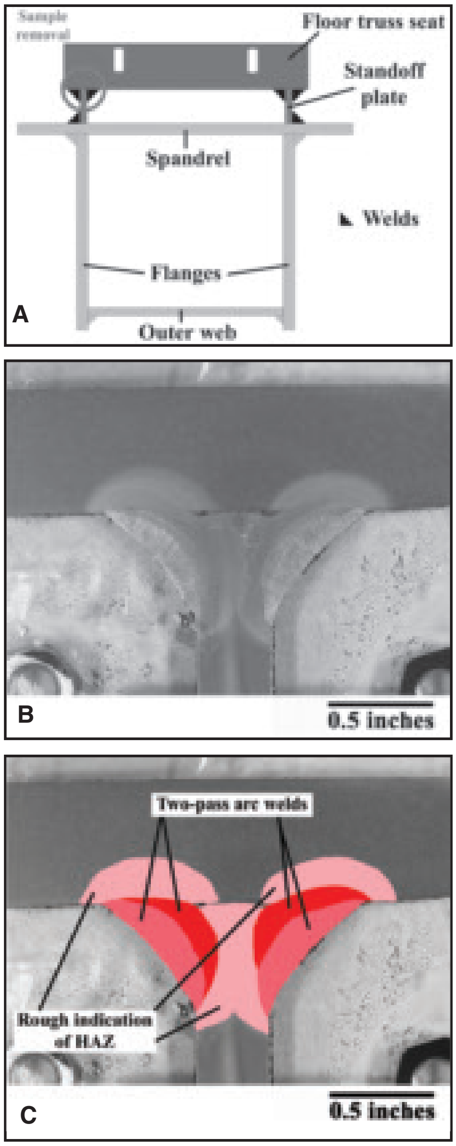


Fig. 12 — Cross section of fillet weld between standoff plate and seat angle from an undamaged main truss seat (A154: 101–104; column 153, 103rd floor). A — Schematic showing the location of sample removal; B — metallographically prepared specimen; C — HAZ and weld were determined via optical means. Two percent nital and four percent picral etch.

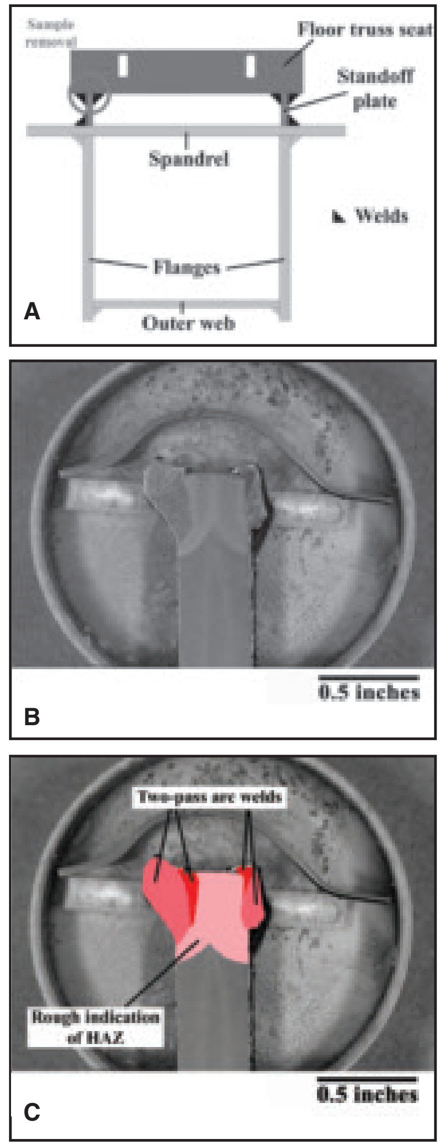


Fig. 13 — Cross section of fillet weld between standoff plate and seat angle from a damaged main truss seat (A157: 93–96; column 157, 94th floor). A — Schematic showing the location of sample removal, B — metallographically prepared specimen, C — HAZ and weld were determined via optical means. Two percent nital and four percent picral etch.

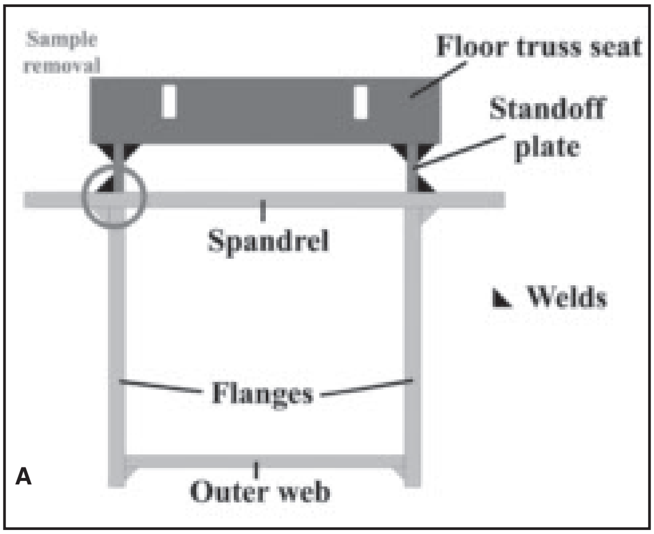
within the design drawings, but due to the short welding length, shielded metal arc welding (SMAW) or flux cored arc welding (FCAW) was most likely used. Typically, these joints were overwelded, meaning that a $\frac{3}{8}$- or $\frac{1}{2}$-in. weld may have been deposited where only a $\frac{5}{16}$-in. weld was specified. The HAZ in the standoff plate extends through the entire thickness of the plate; the HAZ in the spandrel plate is shallower. This difference is most likely related to welding procedures and angle of the electrode with respect to the two plates. Of the five welds randomly selected for examination, all were of good quality and contained no visible surface flaws (e.g., cracks, undercuts, or overlaps) or subsurface defects (e.g., cracks in weld metal, porosity, incomplete fusion, incomplete penetration).

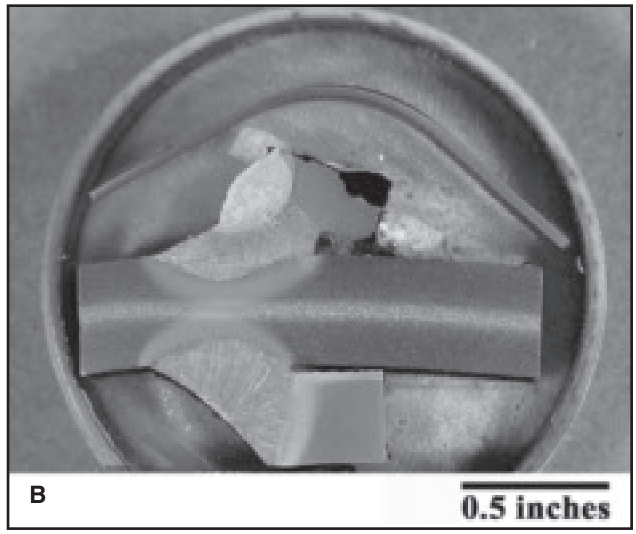
Intact Weld between Standoff Plates and Seat Angle

A two-pass fillet weld, on both faces of the standoff plate, joined the seat angles to the standoff plates — Fig. 12. This clearly indicates that the standoff plate/seat angle assemblies were prefabricated and then welded to the spandrel. Again, SMAW or FCAW was probably used for joining. The combined HAZ from both fillet welds spans the entire standoff plate. Again, the HAZ in the seat angle is shallower. All five welds randomly chosen for examination were of good quality with no visible flaws or defects.

Failed Weld between Standoff Plates and Seat Angle

Table 1 shows that nearly 25% of the main truss seats at or below the impact regions had the seat angles detached with at least one of the standoff plates remaining. Again, the seat angles probably detached via a shear mechanism as the buildings collapsed. Figure 13 shows the failed cross section of one of these joints. Failure occurred within the weld metal at the weld-seat angle interface. Similar failure features were observed on other samples





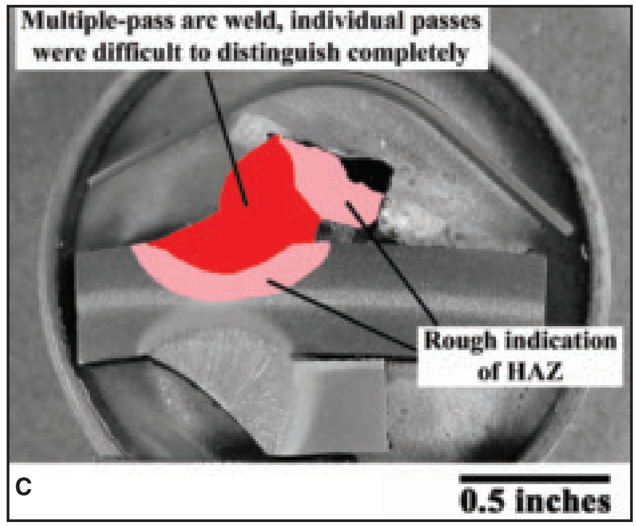


Fig. 14 — Cross section of fillet weld between spandrel and standoff plate from a damaged main truss seat (A130: 96–99; column 129, 99th floor). A — Schematic showing the location of sample removal, B — metallographically prepared specimen, C — HAZ and weld were determined via optical means. Two percent nital and four percent picral etch.

examined that had a failed joint between the standoff plates and the seat angle. Attempts were made to microscopically view the fracture surfaces; however, a corrosion product was found on the surfaces due to environmental degradation. Even after removal of the corrosion product, the positive identification of the microscopic fracture mechanism (cleavage facets or dimpling) was not possible.

Failed Weld between Spandrel Plate and Standoff Plates

Table 1 also shows that nearly 40% of the main truss seats at or below the impact regions from both towers had at least one of the standoff plates detached from the spandrel in the process of removing the seat assembly from the column. The plates probably detached via a shear mechanism as the buildings collapsed due to the overloading of the floors. Figure 14 shows one of these failed joints in cross section. Failure occurred almost exclusively within the HAZ of the standoff plate. Similar failure features were observed on all samples metallographically analyzed that had a failed joint between the spandrel plate and standoff plates.

Discussion of Floor Truss Connection Failures

Observations of deformation and failure modes for the floor truss connections of the recovered exterior panels revealed a consistent pattern when the panels were separated by floor elevation: those at or below the impact region vs. those above the impact region. Of the 51 floor truss connections (both main truss seats and strap connections) at or below the impact floors for WTC 1, 94% were either bent downward or completely detached. Only 48% of the 72 floor truss connections above the impact floors were deformed in a similar manner. Analogous results were found for WTC 2, where 94% of the 16 floor truss connections below the impact floors were bent down or missing, while only 63% of the 32 floor truss connections above this region experienced similar damage characteristics. This distribution of truss connection damage was most likely a result of overloading the floors below the impact region during the collapse of the building.

Detached main truss seats failed near one of two welded joints associated with the standoff plates. Inspection of the weld failures showed that fracture typically occurred in the location with the lowest cross-sectional area. The joints between the standoff plate and spandrel failed primarily in the HAZ of the standoff plate because this path is shorter than the one that traverses the stronger weld material. Additionally, the standoff plate had the lowest cross-sectional area for load transfer at this joint. Conversely, when the seat angles were detached from the standoff plates, failure occurred in the weld metal at the seat angle/standoff plate interface. Figure 12 shows that the fillet weld between the standoff plate and the seat angle did not fully penetrate the portion of the standoff plate in contact with the seat. Thus, the joint failed there because that location had the lowest cross-sectional area for load transfer.

It was noted that separating the floor truss connections into other categories (inside vs. outside of the impact zone; exposure to pre-collapse fires) revealed no significant change in the distribution of failure modes. This fact was quite interesting, particularly for those connections exposed to pre-collapse fires. Some of the main truss seats experienced considerable fires raging on the floor just below their location for upward of 30 min (Ref. 4). However, the spray-applied fire-resistive material (SFRM) applied to the seats appeared to perform as required, as metallurgical analysis of many of these fire exposed seats showed no alteration in their microstructure (Ref. 4).

Summary

Analysis of the connections supporting the composite floor system of the WTC towers showed that at and below the impact floors, the greater majority (above 90%) of the floor truss connections were either bent downward or completely removed from the exterior column. This was probably related to the overloading of the floors below the impact region after collapse initiation. Depending upon weld joint geometry, detachment of the main load-bearing seats was a result of either fracture in the heat affected zone of the base material (standoff plate detached from spandrel) or through the weld metal (seat angle detached from standoff plate). Failure in both cases was assumed to be a result of a shear mechanism as a result of overloading from floors above impacting those below. There did not appear to be a significant change in distribution of failure modes of the floor truss connections when comparing those connections inside vs. outside of the impact region or those exposed to pre-collapse fires and those that were not.

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The authors wish to thank Dr. W. Pitts of the Building and Fire Research Laboratory at NIST for collecting, cataloging, and giving access to the WTC photographic archive used to determine exposure of exterior wall panels to pre-collapse fire.

Disclaimer

The policy of NIST is to use the International System of Units (metric units) in all publications. In this document, however, units are presented in metric units or the inch-pound system, whichever is prevalent in the discipline.

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