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6 Bankers Trust Building

6.1 Introduction

The Bankers Trust building at 130 Liberty Street, also referred to as the Deutsche Bank building, withstood the impact of one or more pieces of column-tree debris raining down from the collapsing south tower (WTC 2). Although the debris sliced through the exterior façade, fracturing spandrel beam connections and exterior columns for a height of approximately 15 stories, the building sustained only localized damage in the immediate path of the debris from WTC 2 (hereafter referred to as the impact debris) (Figures 6-1 and 6-2). There were no fires in this building. The ability of this building to sustain significant structural damage yet arrest the progression of collapse is worthy of thorough study. Unlike WTC 1, 2, and 7, which collapsed completely, the Bankers Trust building provided an opportunity to analyze a structure that suffered a moderate level of damage, to explain the structural behavior, and to verify the analytical methods used. The following sections describe the building structure, the extent of damage, and the computational methods that were used to analyze the structure.

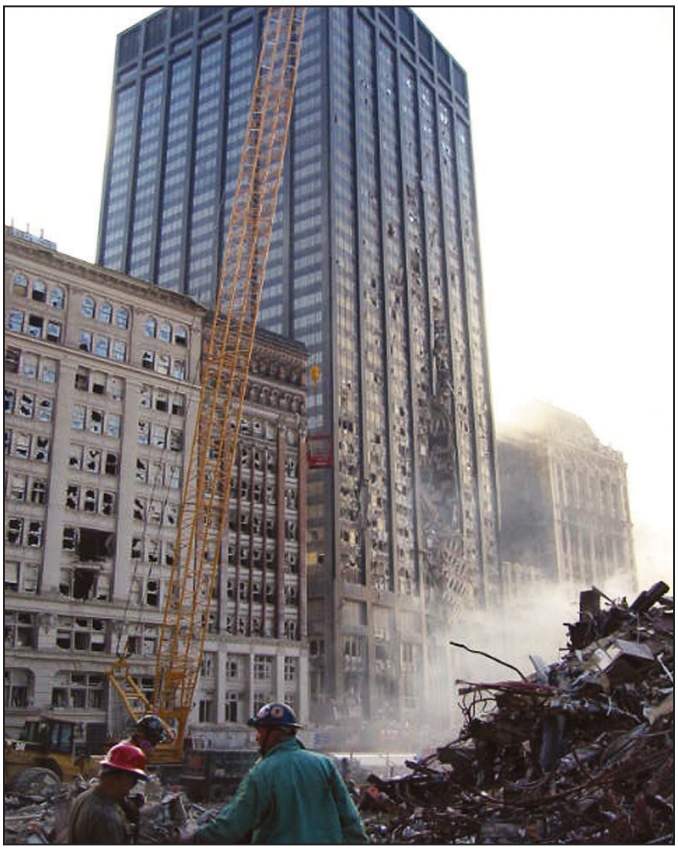


Figure 6-1
North face of Bankers Trust building with impact damage between floors 8 and 23.

6.2 Building Description

The Bankers Trust building is a steel-frame commercial office structure, designed and constructed circa 1971. Bankers Trust was designed by Shreve, Lamb & Harmon Associates P. C. Architects; Peterson and Brickbauer Associated Architects; the Office of James Rudderman Structural Engineers, and Jaros Baum and Bolles Mechanical and Electrical Engineers. The building measures 560 feet in height with 40 stories above grade and 2 below. It is located directly across Liberty Street from the former site of WTC 2, about 600 feet due south of the southeast corner of WTC 2. The floor numbering used in the building elevator system and referred to in this report omits the 13th floor and includes a mezzanine between the 5th and 6th floors.

Figure 6-2
Closeup of area of partial collapse. Note debris accumulated at the bottom of the damage area, resting on the 8th floor. Area of initial impact is not shown in this photo.

Above the second floor level, the building is essentially square in plan shape, with overall dimensions (centerline of exterior column lines) of 183 feet square. At the perimeter of the building, layout of columns in both the north-south and east-west directions consists of an exterior bay on each end that is 26 feet 3 inches wide and five interior bays that are 26 feet 0 inches wide. Interior column spaces vary slightly from this to accommodate the central elevator core (see Figure 6-3).

Girders and spandrel beams are typically deep wide-flange shapes, including W24, W27, W30, and W36 sections, but also including occasional built-up sections termed "wind girders." Girders and spandrel beams are moment-connected to columns at each intersection and at both axes of the columns. The girders' moment connections to column flanges were composed of top and bottom plates fillet-welded to the beam flanges and full-penetration welded to the column flange. These connections were designed for wind moment only, not the flexural capacity of the members, and are considered to be fully restrained, partial-strength connections. Lateral drift (stiffness) due to wind loads usually controls the design for moment frames. The beam web was connected to the column via a shear plate, fillet-welded to the column and bolted to the beam web. Girders that were connected to the column web utilized top plates fillet-welded to the beam top flange and full-penetration weld to the column web; there was no connection to the inside face of the column flanges. The bottom flange of the girder was connected to an extended stiffened seat that had its seat plate full-penetration-welded to the column web. The beam web had no connection to the column because the seated connection provided the necessary shear transfer. Girder-to-column shear connections utilize A325 high-strength bolts. Steel grade is ASTM A572 grade 50 except for the wind girders, which are A36. The columns are generally A36, except that some A588 steel is used at lower-level columns.

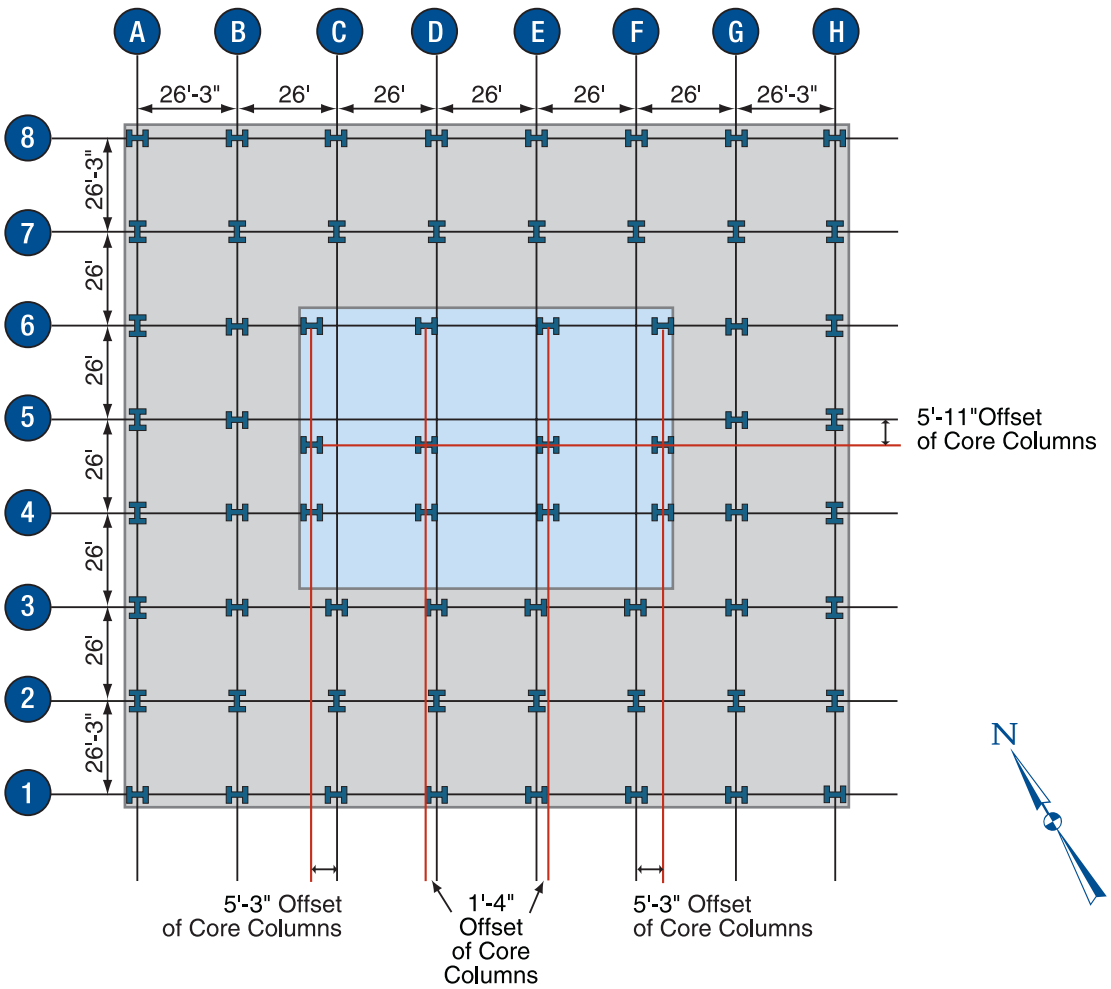


Figure 6-3 Floor plan above the 2nd level (ground floor extension not shown).

Floor beams are typically non-composite and consist of rolled W14, W16, and W18 shapes spanning north to south between girders, typically spaced at 8 feet 8 inches. The floor beams are fastened to the girders via shear connections and typically utilize a partial end plate welded to the floor beam and bolted to the girder with A307 bearing bolts. The floor itself typically consists of 1-1/2- inch metal deck supporting a 2-1/2-inch lightweight concrete slab. Floor beams are typically depressed 2 1/2 inches relative to the girders. Typical floor height (above the 5th floor) is 12 feet 2 inches. All beam-to-beam connection bolts appear to be A307. To accommodate this limited height, limited depth members with Grade 50 (50 ksi yield strength) steel were used.

A mechanical floor is present at the 5th level, and this story is taller, at 20 feet 8 inches. The stories below the 5th level are also taller than typical, with heights varying from 16 feet to 20 feet. To laterally stiffen the frame in these tall stories, a system of diagonal bracing is provided at the elevator core.

At the first story, the building extended outward to the north, with a large canopy structure. The canopy structure extended several column lines to the north of the main building line. A full basement underlies the entire structure.

6.3 Structural Damage Description

Debris from WTC 2 fell along the north side of the building. This debris completely crushed the single-story extension of the building, north of line 8, and collapsed it into the basement in this area. A column section from WTC 2 was embedded in the north edge of the floor slab of the 29th floor. It also appears that one section, or perhaps several sections, of exterior column trees from the south wall of WTC 2 plunged through the north wall of the building just above the 23rd floor. This impact area is illustrated in Figure 6-4. The zone of structural damage remained confined to one structural bay for several floors immediately below the point of impact before spreading to two and sometimes three bays in the floors below. However, although the pattern of damage was influenced by the structural response, most of the damage can be attributed to the path of the impact debris and not to progressive collapse.

It appears that the direction of motion of the falling debris from WTC 2 was steeply angled down and to the east-southeast. As the falling debris smashed through the 23rd floor spandrel between column lines C and D, it and the debris it created at each floor continued to dive deeper into the building, causing more structural damage. Between the 19th and 22nd floors, the floor areas between column lines 7 and 8, and C and D were damaged or destroyed (Figure 6-5). The damage zone increased to include portions of the area between column lines D and E from the 18th floor down to the 9th floor (Figure 6-5).

Column splices are typically located at every second floor and are composed of thin splice plates bolted to the column flanges. Large axial tension loads were probably not a design condition for the column splices, and the splice plates appear to be minimal in thickness, offering little resistance to separation. The D-8 column splices at the 18th and 16th floors appear to have been overloaded, leaving a section of column suspended from the spandrels at floors 16 through 18. The column below this level and down to the 8th

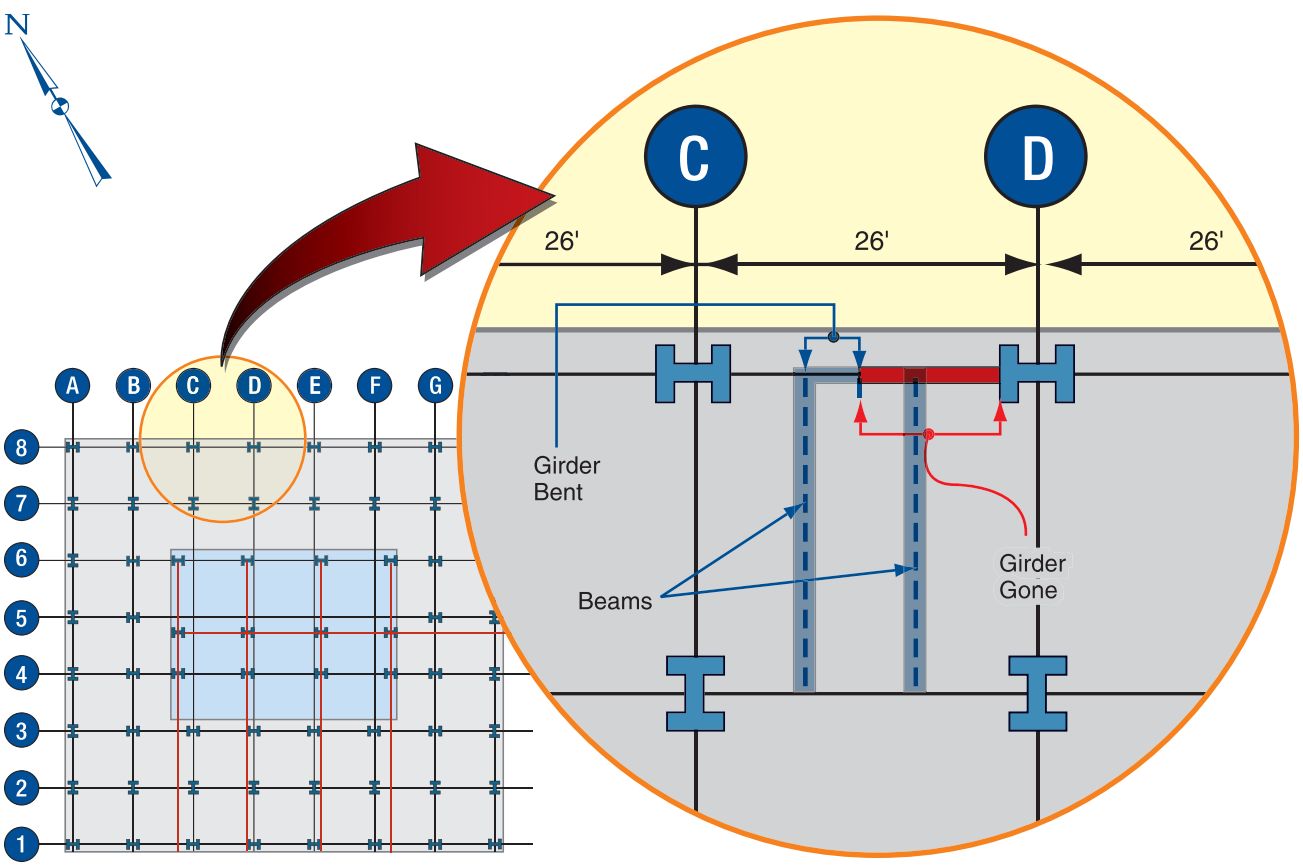


Figure 6-4 Area of initial impact of debris at the 23rd floor.

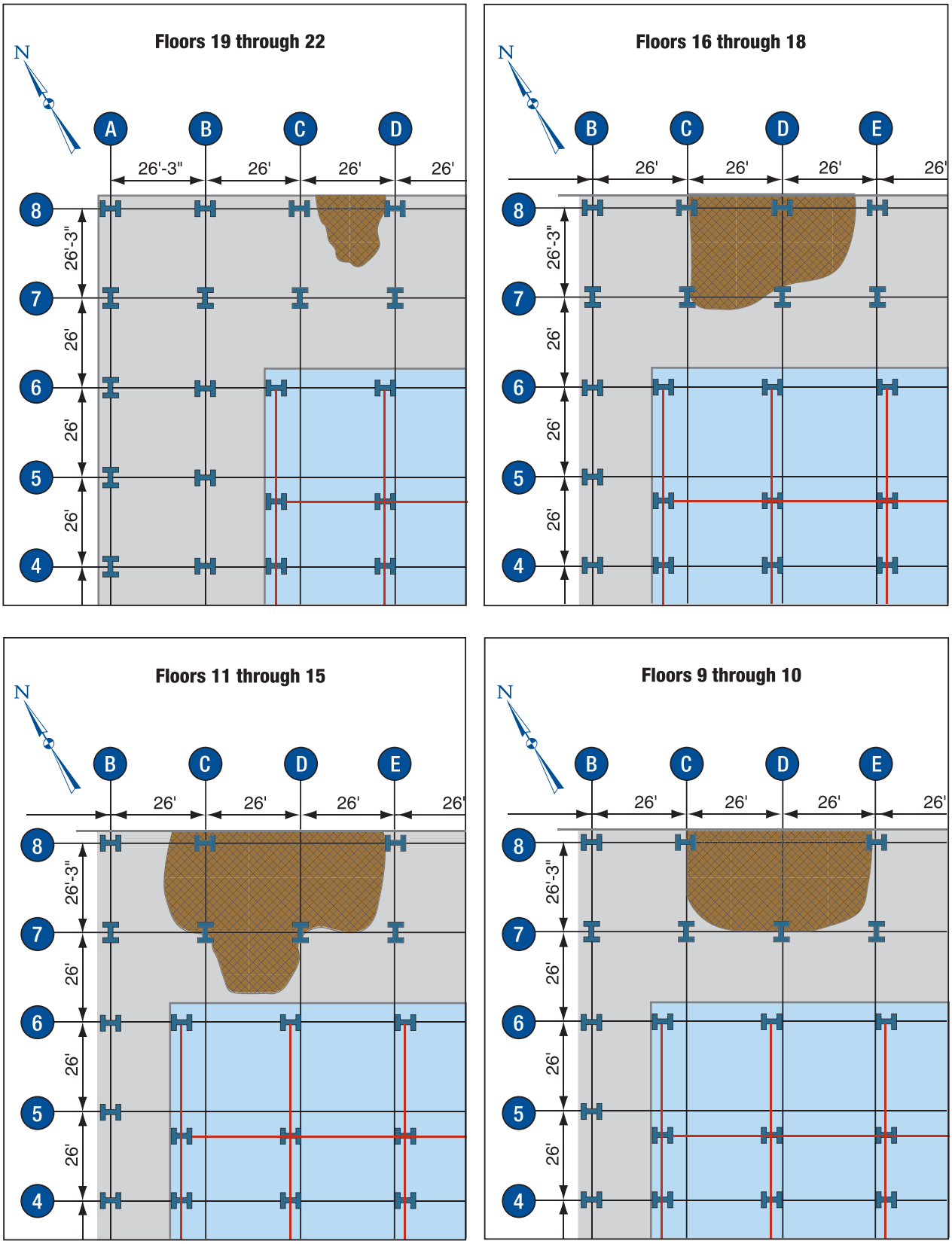


Figure 6-5 Approximate zones of damage – 19th through 22nd floors, 16th through 18th floors, 11th through 15th floors, and 9th through 10th floors.

floor was either ejected from the building or folded into the general debris during the partial collapse. In the area between the 8th and 16th floors, the damage area generally increased as the partial collapse progressed downward to include, and sometimes exceed, portions of the two bays bounded by columns C-8 to E-8 and C-7 to E-7, with column D-8 missing in the area of the 12th floor.

At the point of impact and at the floors immediately below, it appears the impact debris sliced through the spandrels. However, at lower elevations, the spandrel beams were not fractured and separation occurred at the spandrel-to-column connections. In the typical failure mode of the girder-to-column-flange connection, the weld and heat-affected zone of the top flange plate pulled out of the column and left a crater in the flange. The bottom flange plate was overloaded in bending at the column face without creating a crater in the column flange. The beam shear connection was typically left in place. The shear failure occurred mostly in the beam web and, in some cases, through the bolt line in the shear connection plate.

Girders that were connected to the column web had their top flange plate overloaded in tension at the weld zone interface. Normally, a crater was created where the top flange plate was welded to the column web. The bottom flange sat on a stiffened seat. Typically, the seated connection was left in place; however, the beam pulled off of the seat, separating at the fillet welds and leaving behind the fillet welds that attached the beam flange to the seat top plate. Figure 6-6 shows the stiffened seat at the weak axis side of the column. Figure 6-7 shows that the shear connection remains on the flange side, but only a piece of beam remains hanging on the web side.

Floor-beam-to-girder connections typically used a partial-depth end plate welded to the floor beam web and bolted to the girder with A307 bearing bolts. Typically, failure occurred in one of two modes, either the bolts were overloaded in tension and the connection pulled off, or the partial end plate sheared at or near the weld line. At most of the connections, it appears there was some amount of bolt failure. Figure 6-8 shows a shear connection with half of the end plate remaining in place.

As the spandrel connections failed and the floor slabs collapsed, a portion of the rubble accumulated into a two-story pile while the remainder fell out of the building onto the low-rise roof on the north side of the building. The impact debris finally came to rest on the two-story-deep pile of debris that was on the 8th floor. Figures 6-9 and 6-10 illustrate the damage. A major component of WTC 2, approximately 40 feet tall by 30 feet wide, remained lodged in the debris pile and was clearly visible as it hung from the face of the building. Yet despite this weight, the floor supporting the debris deflected a maximum of 6 inches. Figure 6-11 shows the extent of debris in the lobby of the building.

6.4 Architectural Damage Description

In addition to the destruction of the canopy structure north of column line 8, and the collapse of the floor areas between the 8th and 23rd floors, there was general damage to the entire north façade of the building. Nearly every window was broken on the western half of the north face, between column lines B and E below the 23rd floor. This window breakage would appear to be attributable to the following causes:

- Localized damage in the areas impacted by column trees falling from WTC 2.
- Smaller debris blown from WTC 1 and WTC 2. In particular, several small chunks of lightweight concrete, which appeared to be from WTC 2 floor slabs, were thrown through the north windows of the building. These debris items ranged in size from small fragments that caused bullet-size holes in the windows to large chunks with a maximum dimension of approximately 12 inches. Many of these chunks landed as far as 15 feet from the exterior building line and appeared to be traveling almost horizontally when they penetrated the building façade.

Figure 6-6 Moment-connected beams to columns.

Figure 6-7 Column with the remains of two moment connections.

Figure 6-8
Failed shear connection of beam web to column web.

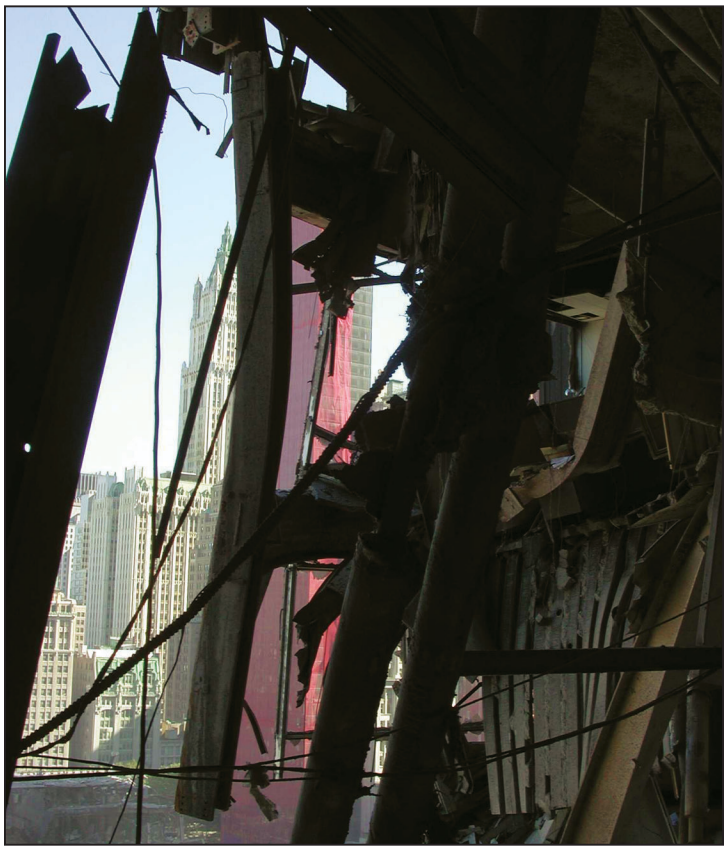


Figure 6-9
Suspended column D-8 at the 15th floor. Note separation at column splices.

Figure 6-10 Area of collapsed floor slab in bays between C-8, E-8, C-7, and E-7, from the 15th floor.

Figure 6-11 Bankers Trust lobby (note debris has been swept into piles).

After the exterior glazing was penetrated by the debris, the dust cloud resulting from the collapse of the towers deposited a layer of dust an inch or more thick throughout the northern part of the building. The 2nd floor lobby area had extensive broken glass, general debris, and dust (Figure 6-11). Figure 6-12 shows a typical office near the collapsed area at the 8th floor.

Although fire sprinkler piping was damaged in the collapse area, causing water to flow on the floor, in general, sprinkler piping throughout other portions of the building remained intact and the building was basically dry. Water pressure in the domestic system was available in upper floors. Ceiling systems generally remained intact, except at the collapsed areas of the building.

6.5 Fireproofing

The structural steel sections were fireproofed with a spray-applied non-asbestos fireproofing material. The thickness on the beam flanges was observed to be on the order of 1/2 inch thick. Many of the rolled steel shapes appeared to be almost completely bare of fireproofing where directly impacted by debris; the remainder of the fireproofing appeared intact even in the damaged areas. Because fires were not ignited in combination with this structural damage, the damaged fireproofing did not affect the performance of the building.

6.6 Overall Assessment

Except for the canopy structure on the north side of the building, which was crushed by falling debris, the building withstood the debris impact well. Excluding the framing and supported floors in the immediate zone of impact and several floors immediately below this area, the structure remained in good condition and serviceable. Repair of the structure should be feasible.

Figure 6-12
Office at the north side of the 8th floor.

6.7 Analysis

A 3-D model was developed to better understand the performance of the building in response to impact from debris and to identify specific design features that contributed to this performance. The model was developed based on information obtained from the following sources: structural drawings (Rudderman 1971–1975), *Draft Structural Engineering Evaluation for the New York Department of Design and Construction (*Nordenson et al. 2001*)*, and personal accounts (Smilow 2001).

All major structural entities were simulated within the ANSYS model: all major framing, intermediate framing in regions of damage, steel decking/slab, columns, and vertical bracing. Given the level of modeling detail required to simulate the behavior of the structure in the regions of damage, three levels of refinement were incorporated within the model to use analytical resources more efficiently:

Level 1: No visible structural damage

Level 2: Local structural damage

Level 3: Partial or imminent collapse

Because there are no eyewitness accounts available that can clarify the order of collapse of portions of the structure, two types of analyses were considered.

Nominally, the final state of the structure could be analyzed to determine the current state of stress redistribution within existing structural members. This analysis clarifies why the damage was arrested.

A more complex analysis could be performed, namely an analysis tracking the partial progression of damage. This type of analysis consists of parametric studies performed to determine the velocity of the impact debris that was required to:

(a) collapse the region of slab between column lines C and D at the 16th through 21st floors;

(b) remove the exterior column D-8 below the 16th floor; and

(c) extend the region of damaged slabs to the two bays bounded by column lines C and E.

These calculations are based on engineering principles and methods. Although they attempt to quantify the extent of the damaged regions, they provide little intrinsic value to the understanding of the building performance. It is highly unlikely the calculations would indicate a unique sequence of events resulting in the observed collapse patterns. Analyses of the surviving structure are much more informative and indicate the redundancy of the steel moment-frame system. These calculations indicate why the collapse was arrested at the 8th floor and why the region of slab loss did not extend beyond the two exterior bays on either side of column line D. Calculations demonstrating the redistribution of load could be performed statically, allowing for inelastic deformations.

Two different types of girder-column moment connections failed as a result of the debris impact, and the performance of these connections had a major effect on the extent of collapse. These two connections were analyzed to better understand their capacity to resist extreme loading. Although the design drawings show some connection information and the observations of the damaged building (Smilow 2001) identified some critical dimensions, there was not enough information to develop detailed finite element models. Therefore, parametric evaluations based on engineering judgment were required to identify the trends in the reserve capacities of these critical connections.

At the time this report was published, a parametric study had been completed to determine the behavior of typical moment connection details in preparation for a subsequent suite of full 3-D frame parametric studies. This suite of calculations was not completed in time for inclusion in this report.

6.7.1 Key Assumptions

Key assumptions were made in the modeling of the building. These assumptions are based on engineering judgment and basic principles of physics:

1. Boundary Conditions:

    a. The structural model was fixed at the base of Concourse Level A.

    b. Vertical rollers were placed at frame 4; given the localization of damage, approximations can be made to model half of the building and restrain the structure a single bay beyond the region of spot damage.

    c. The two-story canopy, north of frame line 8, was excluded from the model; because the structure (frame lines 8 through 10) suffered heavy damage, the collapsed region would not influence the response of the structure in the floors above.

2. Static Nonlinear Analysis:

    a. The nonlinear analysis was performed accounting for large-deflection geometric nonlinearities.

    b. The inelastic response of connections was simulated via nonlinear springs and localized inelastic material properties.

3. Multi-phase Loading:

    a. Application of the gravitational loading.

    b. Removal of missing structure; damaged and missing members were removed in a final-state analysis, and selected members were removed sequentially in an analysis tracking the partial progression of damage.

6.7.2 Model Refinement

Three levels of refinement were incorporated into the model to make more efficient use of the analytical resource for the regions of structural damage. These levels of refinement were developed based on the classification of damage patterns depicted in the draft evaluation prepared by Nordenson (2001*)* and supplemented by observations of structural damage (Smilow 2001).

In refinement level 1, all major framing was modeled explicitly. All moment and shear connections are assumed to be elastic within regions modeled with this refinement. The steel deck/slab system in conjunction with intermediate framing was modeled using orthotropic plates.

In refinement level 2, all major framing (spandrel beams) was modeled explicitly. Intermediate framing (beams) was not incorporated in the orthotropic plate definition; it was modeled explicitly. All moment connections are modeled explicitly, simulating all spandrel/column connections. All shear connections are assumed to be elastic within regions modeled with this refinement. The steel deck/slab system is not represented explicitly.

In refinement level 3, all major framing (spandrel beams) and intermediate framing (beams) were modeled explicitly. All moment connections and shear connections were modeled explicitly, simulating all spandrel/column connections and beam/spandrel connections, respectively. The steel deck/slab system was not represented explicitly.

Plate elements and beam elements were refined systematically to obtain key output data in regions of heavy damage with correspondingly less refinement farther away from heavy damage areas. In regions

modeled using refinement level 3, a minimum of four beam elements were used between columns. The plate elements resting on the beam elements were meshed to match the resolution of the beam elements within this region of refinement. In regions modeled using refinement level 2, a minimum of two beam elements were used between columns.

The plate elements resting on the beam elements were also meshed to match the resolution of the beam elements within this region of refinement. In regions modeled using refinement level 1, single elements were used between columns. This region was modeled to simulate the behavior of regions that were not subjected to structural damage and, because no damage patterns were anticipated, the plate elements in this region of refinement were meshed with minimum discretization.

6.7.3 Simulation of Nonlinear Behavior

The ANSYS model simulates the inelastic response of the structure system by means of nonlinear springs. All moment connections and shear connections in regions of heavy damage (refinement level 3) were modeled using nonlinear rotational and translational springs.

All explicit shear connections were modeled using nonlinear vertical springs. The remaining translational degrees of freedom were assumed to be rigidly constrained. All column splices were assumed to provide continuity based on observations of the structure (Smilow 2001). All explicit moment connections were modeled using nonlinear spring elements. The properties of these nonlinear elements were determined from three-dimensional quasi-static analyses of representative connections, whose response produces a moment rotation relationship. The inelastic response of the connection was simulated by using a elasto-plastic model for the girder, plating, and weld material.

Contrary to the structural drawings, inspection of the floor structure revealed that the steel deck/slab system was not explicitly attached to the supporting spandrel beam elements; it was resting on the underlying spandrel and beam system. This structural assembly was simulated by rigidly constraining the vertical degrees of freedom of the plate elements to the underlying spandrel/beam system. This finite element construct accounted for the transmittal of weight of the deck/slab system, along with additional amounts of reported debris, to the supporting beam elements.

6.7.4 Connection Details

The behavior of a typical fully rigid, partial strength wind-moment connection about the strong axis of the column was studied. The connection of the W18x50 girder to the W14x426 column between girder line 7-8 at frame line D on the 14th floor was modeled as a representative connection. The top and bottom moment plates (estimated as 5/8 inch x 6 inches x 24 inches and 3/8 inch x 10-1/2 inches x 24 inches, respectively) were welded to girder flanges with a 1/4-inch weld. The shear plate (estimated as 5/16 inch x 3 inches x 12 inches) was bolted to each girder web with four 7/8-inch-diameter bolts. Although the design wind moment was estimated to be 2,930 kip-in, the connection capacity was estimated to be 10,800 kip-in.

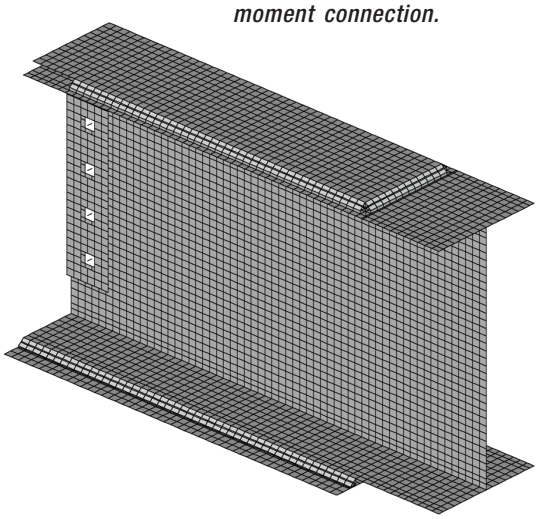
Similarly, the behavior of a typical fully rigid, partial-strength wind-moment connection about the weak axis of the column was studied. The connection of the W24x68 girder to the W14x426 column between girder line C-D at frame line 7 on the 15th floor was modeled as a representative connection. The top and bottom moment plates were estimated as 3/8 inch x 12 inches x 14 inches with a 1/4-inch weld, and the shear in the connection was resisted by a seat, estimated as 1/2 inch x 5 inches x 12 inches, stiffened with a 3/8-inch x 8-inch seat plate. Although the design wind moment was estimated to be 2,830 kip-in, the connection capacity was estimated to be 7,500 kip-in, thus confirming the frame design was governed by stiffness and not strength.

Both connections were modeled in three dimensions in ANSYS, using shell, beam, and continuum elements. All weld material was simulated with a bilinear kinematic hardening material with brittle fracture capabilities at a specified ultimate strain. Figures 6-13 and 6-14 illustrate the details of the finite element models of the two different connection details.

6.7.5 Connection Behavior

Both models were subjected to numerous load combinations to determine the overall behavior of the connection. The weld material was assumed to have a nominal yield strength of 50 ksi. Each model was then subjected to a monotonically increasing moment about the transverse axis of beam bending (M_Y) and the principal strains in the welds were evaluated at the end of each load increment. If the strains in any of the weld elements exceeded the specified ultimate strain, the weld element was considered to have fractured and the modulus of elasticity was reduced by several orders of magnitude. Because the ultimate strain in the weld corresponding to fracture is an unknown quantity, several values were assumed in order to determine the connection behavior. Values of 0.5 percent, 1.0 percent, 10 percent, and 20 percent strain were assumed, and the moment curvature relations for the connection were developed. Based on these calculations, it was observed that the welds fracture before plastic hinges occur when the ultimate strain in the welds is assumed to be less than 1 percent. Furthermore, the connection was observed to degrade very quickly with the onset of weld fracture. The first onset of yielding in the welds was observed at a M_Y value of 1,000 kip-in and 1,400 kip-in, for the shear plate and seat connections, respectively. In the absence of wind moments, the connections were found to be able to support a considerable increase in gravity loads over their dead and live load design values. However, the connection offered little resistance to torsional loads and a significant reduction in capacity of the connection with respect to out of plane bending.

Figure 6-13
3-D ANSYS model of flange and shear plate moment connection.



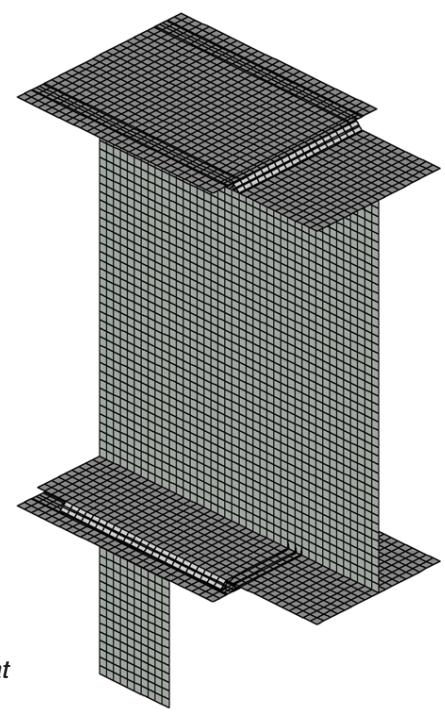


Figure 6-14
3-D ANSYS model of flange and seat moment connection.

The computed results show a sensitivity of the moment curvature relations to the ultimate strain of the weld material and the out-of-plane moments that may have been applied to the connections. The connection sensitivity to out-of-plane and twisting moments significantly influenced the capacity of the connection to resist abnormal loading. The significant reserve capacity of these connections to gravity loads, over an order of magnitude by some estimates, are quickly eroded when the connection is subjected to out-of-plane bending. Therefore, as members were twisted by the collapse of adjacent bays, the connections were less able to withstand the weight of the accumulated debris. This phenomenon may explain why many connections failed and may also explain the sequence of weld fracture. This in turn may have influenced the modes of failure in different connections.

6.8 Observations and Findings

An evaluation of the damage patterns revealed several interesting interpretations. The spandrels were sheared by the impactor, between column lines C and D, from the 23rd to the 19th floors. The D-8 column splices failed at the 18th floor and at the 16th floor, but there are no clues to indicate why column splice tension overload occurred at this location. However, unlike the spandrels above, the girder-column connections at column lines C and D failed. Although severed from the column above and below, column D-8 remained suspended from the girders spanning between column lines E and D. These girders developed large vertical and lateral deformations (twisting). The twisting and bending of these girders may have extended the zone of collapse to bays bounded by column lines C and E. If the column splices had not failed at the 16th and 18th floors, it is possible the extent of collapse may have been limited to the single bay in the path of the impactor. This enlarged zone of damage continued until the collapse was arrested on the 8th floor. It is unlikely that dynamic effects caused the damage to column D-8 below the 16th floor; otherwise, the collapse should have progressed all the way to the ground. It is possible that the column splice failures and the resulting large deformations (twisting) of the spandrels caused the remaining portion of column D-8 to lose lateral bracing, and the collapse was not arrested until the energy of the impactor and debris pile was sufficiently diminished to halt the collapse. If this actually accounted for the enlargement of the damage zone, the restraint of the twisting deformations may have prevented the failure of column D-8.

Although a considerable amount of debris fell from the upper floors onto the first-floor extension to the north, a two-story deep pile of debris accumulated on the 8th floor. By one estimate, although the debris distributed some of its weight by bridging action, the net effect would have been a 500-percent increase in dead load moment for the supporting beam. Based on the computed results, and in the absence of wind, it appears that the connections would have been able to support more than 500 percent of the estimated dead load moments before any hinging would occur. This may explain why multiple stories of debris came to rest at the 8th floor without incurring additional damage to the structure.

Because column D-8 failed below the 16th floor, the beam-to-column moment connection was the single most significant structural feature that helped limit the damage. The portion of the building above the collapsed floors was held in place by frame action of the perimeter. Static elastic analyses of the moment frame show very high stress levels; however, there was a negligible deformation directly above the damaged structure. Furthermore, connections that enable the beams to develop some membrane capacity improve a structure's ability to arrest collapse. The typical floor beam end connections with their A307 bolts were overloaded in direct tension. High-strength bolts would have provided significantly greater tensile ability and possibly held more beams in place through catenary action. Inelastic analyses demonstrate the role of the weaker connections in the response of the structure. Finally, stronger column splices may have made it more difficult for the damaged column to separate from the upper column. Heavier column splices could have allowed the damaged column to function as a hanger and limit the amount of collapsed area, or they could have tended to pull more of the frame down.

6.9 Recommendations

It is difficult to draw conclusions and more detailed study is required to understand how the collapse was halted. As better descriptions of the structural details become available, the observed patterns of damage may provide useful information in the calibration of numerical simulation tools. Some issues requiring further study are:

1. Whether the observed damage in the column flange, and not at the beam flange, of the moment frames top connection plates is due to high restraint in the welds.
2. Why the bottom flange welded connection has typically failed at the fillet weld to beam interface and not at the fillet weld to seat plate interface.
3. The impact response of various moment-connected details.
4. Whether composite construction would reduce local collapse zones. (There were no shear connectors to provide composite action between the floor beams and slab. Composite construction would have increased the capacity of the members and may have dissipated more of the impact energy; however, it may have also pulled a greater extent of the adjoining regions into the collapse zone.)
5. Whether perimeter rebar in the slabs could improve the structural response by providing catenary action and tensile force resistance in the slabs to reduce local collapse zones.
6. Whether the partial-strength connections permitted members to break away from the structure, thereby limiting the extent of damage. (If the moment connections had been designed for the capacity of the sections [as opposed to fully rigid partial strength based on design load and stiffness requirements], the building performance is likely to have been different.)
7. Whether the collapse zone would have been limited if the spandrels on the 16th, 17th, and 18th floors had not been so grossly distorted through twisting.

6.10 References

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