

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.