

Many of the bearing seat brackets and the damper angle connections on the column/spandrel beam plate were completely sheared off. Only the weld segments remained on face of the column/spandrel beam plate (Figure B-11). This mode of failure appears to be due to excessive vertical overloads on the floor system. This is in contrast with the failure mode exhibited in Figure B-9 where the bearing seat bracket has pulled away from the column/spandrel plate, after fracture of the top chord gusset plate.

B.4.4 WTC 5 Column-tree Shear Connections

Chapter 4, Section 4.3.2, noted that limited structural collapse had occurred in WTC 5 as a result of failure at the shear connections between the infill beams and the column tree beam stub cantilevers. It is visually apparent from Figures 4-18, 4-19, 4-20, and 4-21 that the fire-weakened structural members formed diagonal tension field failure mechanism in the cantilever beam webs and plastic hinge moments in the cantilever beam near the column face. The following analysis examines the capacity of the cantilever beams and the shear connections between the cantilever and the infill beams.

The magnitude of the shear force acting at the end of the cantilever section can be estimated from the plastic moment capacity and the plastic shear yield capacity (AISC 2001). The plastic moment capacity of a W24x61 steel section is

$$M_p = F_y Z_x \quad (\text{B-13})$$

and the shear yield capacity is

$$V_p = 0.6 F_y d t_w \quad (\text{B-14})$$

where $F_y = 40$ ksi is the approximate yield strength of the A36 steel sections at room temperature, Z_x is the plastic section modulus, d is the beam depth, and t_w is the web thickness.



*Figure B-11
Shear failure of floor truss
connections from column/
spandrel plate.*