

The vertical shear capacity of the bolted double shear splice connecting the W18x50 section to the W24x61 cantilever can be estimated for the three-bolt connection from the bearing strength relationship given in Section B.4.2. This gives

$$V_u = 3 R_n = 3(3.0 d_b t_w F_u) \quad (B-15)$$

where  $d_b = 3/4$  inch is the bolt diameter,  $t_w = 3/8$  inch is the web thickness for a W18x50, and  $F_u = 60$  ksi is the approximate tensile strength of A36 steel.

Appendix A indicates the yield strength is  $0.9 F_y$  at  $200^\circ\text{C}$  ( $392^\circ\text{F}$ ) and the yield strength is  $0.5 F_y$  at  $550^\circ\text{C}$  ( $1,022^\circ\text{F}$ ). The large plastic deformation observed in the cantilever beam segments suggests that a significant loss of strength developed due to the fire. The fire temperatures reached in WTC 5 are not known, but if it is assumed for the purposes of this analysis that  $550^\circ\text{C}$  ( $1,022^\circ\text{F}$ ) was reached, this analysis estimates the cantilever plastic moment capacity as

$$\text{Room Temperature:} \quad M_p = 1.0 F_y Z_x = 1.0 (40) (152) = 6,080 \text{ in-k} \quad (B-16a)$$

$$550^\circ\text{C} (1,022^\circ\text{F}): \quad M_p = 0.5 F_y Z_x = 0.5 (40) (152) = 3,040 \text{ in-k} \quad (B-16b)$$

This corresponds to a shear force of about 126 kips at room temperature acting at the end of the 4-foot cantilever, which would be reduced to 63 kips at  $550^\circ\text{C}$  ( $1,022^\circ\text{F}$ ). The limiting shear yield strength of the cantilever is

$$\text{Room Temperature:} \quad V_p = 0.6 (1.0 \times 40) (23) (0.419) = 240 \text{ kips} \quad (B-17a)$$

$$550^\circ\text{C} (1,022^\circ\text{F}): \quad V_p = 0.6 (0.5 \times 40) (23) (0.419) = 120 \text{ kips} \quad (B-17b)$$

assuming the full web is effective. Hence, the three-bolt capacity is

$$\text{Room Temperature:} \quad V_u = 3[3(0.75)(0.375)(60)] \approx 152 \text{ kips} \quad (B-18a)$$

$$550^\circ\text{C} (1,022^\circ\text{F}): \quad V_u = 3[3(0.75)(0.375)(30)] \approx 76 \text{ kips} \quad (B-18b)$$

The double shear capacity of the three  $3/4$ -inch high-strength bolts is

$$\text{Room Temperature:} \quad R_u = 3[2(0.62)(1.0)(1.18 F_{ub})(A_b)] \approx 232 \text{ kips} \quad (B-19a)$$

$$550^\circ\text{C} (1,022^\circ\text{F}): \quad R_u = 3[2(0.62)(0.5)(1.18 F_{ub})(A_b)] \approx 116 \text{ kips} \quad (B-19b)$$

where  $F_{ub} = 120$  ksi is the bolt tensile strength at room temperature (see Section B.3.3) and  $A_b = 0.4418$  square inch is the bolt area. At  $550^\circ\text{C}$  ( $1,022^\circ\text{F}$ ), the bolt tensile strength is approximately  $0.51(1.18 F_{ub}) \approx 71$  ksi.

This verifies that the bolted shear connections have sufficient capacity to develop the reduced plastic moment capacity of the fire-weakened steel beam cantilever and sustain large vertical deformation.

The failures all appear to be a result of the large tensile force that developed in the structural system during the fire and/or as the structure cooled. As demonstrated in Section B.4.2, the tensile capacity of the bolted shear splice in the beam web can be estimated for a bolt as

$$R_u = L_e t F_u \leq 3 d t F_u \quad (B-20)$$

with  $L_e = 1.344$  inches is the edge distance,  $t = 0.375$  inch is the plate thickness in bearing, and  $F_u$  is the applicable tensile strength.

$$\text{Room Temperature:} \quad R_u = (1.344) (0.375) (60) = 30 \text{ kips/bolt} \quad (B-21a)$$

$$550^\circ\text{C} (1,022^\circ\text{F}): \quad R_u = (1.344) (0.375) (30) = 15 \text{ kips/bolt} \quad (B-21b)$$