

B.3.2 Observations

1. The exterior tree columns were spliced using bolted end plate connections.
2. All column end plate bolted connections appeared to fail from the unanticipated out-of-plane bending of the column tree sections due to either the aircraft impacts or the deformation and buckling of the unbraced columns as the floor system diaphragms were destroyed by the impacts and fires. The bolts were observed to exhibit classical tensile fracture in the threaded area. Most bolts were also bent in the shank. Figure B-1 shows the column end plates and holes with some fractured and bent bolts. No evidence of plastic deformation was observed in the end plates.
3. Column splice requirements in the AISC Specifications (1963) indicated in Section 1.15.8 that “Where compression members bear on bearing plates and where tier-building columns are finished to bear, there shall be sufficient rivets, bolts, or welding to hold all parts securely in place.”

B.3.3 Connectors

The connectors generally used for steel structures are either high-strength bolts or welds. The project specifications indicated that bolts were to meet the ASTM A325 or A490 standards.

Bolts are designed based on their nominal shank area A_b for tension, shear, or some combination. For tension, the nominal strength (per unit of area) of a single bolt is provided by

$$F_n = C_t F_u \quad (\text{B-2})$$

where F_u is the minimum specified tensile strength and $C_t = 0.75$, which is the ratio of the stress area to the nominal shank area. An analysis of A325 bolts produced in the 1960s and 1970s indicated that, on average, the bolts exceeded the minimum specified tensile strength by 18 percent (Kulak, et al. 1987).

For shear, the nominal strength of a single bolt is provided by

$$F_v = C_s F_u \quad (\text{B-3})$$

The average shear coefficient $C_s = 0.62$ for a single bolt. This coefficient is reduced to $C_s = 0.5$ to account for connection lengths up to 50 inches parallel to the line of force. When threads are not excluded from the shear plane, the coefficient C_s is further reduced to $C_s = 0.4$.

Other failure modes are possible as bolts transfer forces from one component into another by bearing and shear of the fastener. This can result in bearing deformations and net section fracture of the connected elements. Other failure modes are shear rupture or bearing strength as bolts shear connected material between the bolt and a plate edge, or block shear, which combines the tension and shear resistance of the connected elements. An example of one of these failure modes can be seen in Figure B-5, which shows a spandrel beam bolted shear connection that has failed in end zone shear as the connection was subjected to moments and/or tensile loading. This indicates that all elements in this example were at their ultimate load capacity when the spandrel connection failed.

The commonly used weld connectors are either fillet welds or groove welds. Complete joint penetration groove welds are designed for the same basic capacity as the connected base metal and match its capacity. Fillet welds and partial joint penetration groove welds are designed to resist a calculated or specified load by sizing for the weld throat area, which is the effective cross-sectional area of the weld.