John Fisher
Nestor Iwankiw

B Structural Steel and Steel Connections

B.1   Structural Steel

This appendix focuses mainly on the structural steel and connections in the WTC towers (WTC 1 and WTC 2), but column-tree connections in WTC 5 are also considered. Other WTC structures were fabricated from ASTM A36 and A572 grade steels, and their structural framing and connections are discussed in prior chapters.

The structural steel used in the exterior 14-inch by 14-inch columns that were spaced at 3 feet 4 inches on center around the entire periphery of each of the WTC towers was fabricated from various grades of high-strength steel with minimum specified yield stress between 36 kips per square inch (ksi) and 100 ksi (PATH-NYNJ 1976). Column plate thickness varied from 1/4 inch to 5/8 inch in the impact zone of WTC 1 for floors 89-101, and from 1/4 inch to 13/16 inch in the impact zone of WTC 2 for floors 77-87. Spandrel beams at each floor level were fabricated of matching steel and integrated into the columns as the columns and spandrel sections were prefabricated into trees. These trees were three columns wide and one to three stories high. The cross-sectional shape of the columns can be seen in Figure B-1. These varied in length from 12 feet 6 inches to 38 feet, depending on the plate thickness and location.

Figure B-1   Exterior column end plates.

The three columns in a panel were generally fabricated from the same grade of steel. The yield stress varied from 50 ksi to 100 ksi in increments of 5 ksi up to 90 ksi. Although most of the time the same grade of steel was used in all three columns, sometimes a column was fabricated from different grades. The difference was up to 15 ksi (i.e., 75 ksi, 85 ksi, and 90 ksi). The core columns were box sections fabricated

from A36 steel plate and were 36 inches x 14–16 inches with plate thickness from 3/4 inch to 4 inches. Above floor 84, rolled or welded built-up I-shaped sections were used.

The floor system was supported by 29-inch-deep open-web joist trusses with A36 steel chord angles and steel rod diagonals. Composite 1-1/2-inch, 22-gauge metal floor deck ran parallel to double trusses that were spaced at 6 feet 8 inches. The floor deck was also supported by alternate intermediate support angles and transverse bridging trusses that were spaced at 3 feet 4 inches. The bridging truss also framed into some periphery columns. Figure 2-2 (in Chapter 2) shows the layout of a typical floor. Because 13-foot-wide and 20-foot-wide modular floor units were prefabricated for construction, the outside two trusses shared a common top chord seat connection with adjacent panels. All double trusses were attached to every other periphery column by a seat angle connection and a gusset plate that was welded to the spandrel and top chord. Therefore, all truss supports had two trusses attached to the seat connection. A single bolt was used for each truss sharing a seat connection. The bottom chord of each pair of trusses was attached to the spandrel with visco-elastic dampers that had a slip capacity of 5 kips. At the core, the trusses were connected to girders that were attached to the box or H-shaped core columns by beam seats welded to the column faces.

B.2 Mechanical Properties

Nearly all of the steel plate was produced in Japan to ASTM standards or their equivalent. None of the mill test reports were available that describe the mechanical properties and chemical composition of the steel used in the WTC structures. Approximately 100 potentially helpful steel pieces were identified at the four salvage yards that had contracts to obtain and process the WTC steel debris. These pieces have been removed and transported to the National Institute of Standards and Technology (NIST) in Gaithersburg, Maryland, for storage and further study. No coupons were taken or tested to check material conformance with specification of any plate, rolled section, bolt, weld, reinforcing steel, or concrete. Visual examination of the debris did not identify any apparent deficiencies in the structural materials and connectors.

In lieu of actual WTC steel properties, typical stress-strain curves characteristic of 3 of the 12 steels used in the design and construction of the WTC complex are shown in Figure B-2 for three ASTM-designation steels with minimum specified yield strengths of 36 ksi (A36), 50 ksi (A441), and 100 ksi (A514). In general, as the yield strength of the steel increases, the yield-to-tensile-strength ratio (Y/T) also increases. For A36 steel, Y/T is approximately 0.6, whereas for A514 steel, Y/T is approximately 0.9. The yield plateau for five steels (yield points 36, 50, 65, 80, and 100 ksi) can be highly variable for structural steels, as is apparent from a comparison of the expanded initial portions of the five steels shown in Figure B-3. At the higher yield strength associated with quenched and tempered alloy steels, there may not be a distinct yield plateau; instead, the steels exhibit gradual yielding and nonlinear behavior with strain hardening.

High strain rates tend to increase the observed yield strength and tensile strength of steel, but may also reduce the ductility. There is a greater influence on the yield point than on the tensile strength. Figure B-4 compares the effect of a very high strain rate (100 in/in/sec) for a mild carbon steel with a more usual test speed of 850 micro in/in/sec. In this example, the yield point more than doubled, whereas the tensile strength was increased about 27 percent, and the Y/T ratio approached unity.

In fracture toughness tests where rapid load toughness is determined, the dynamic yield strength of certain steels can be estimated by the following equation taken from ASTM E1820 (ASTM 1999):

$$\sigma_{yd} = \sigma_{ys} + \frac{174{,}000}{(T + 460)\log_{10}(2 \times 10^7\ t)} - 27.2\ ksi \qquad \text{(B-1)}$$
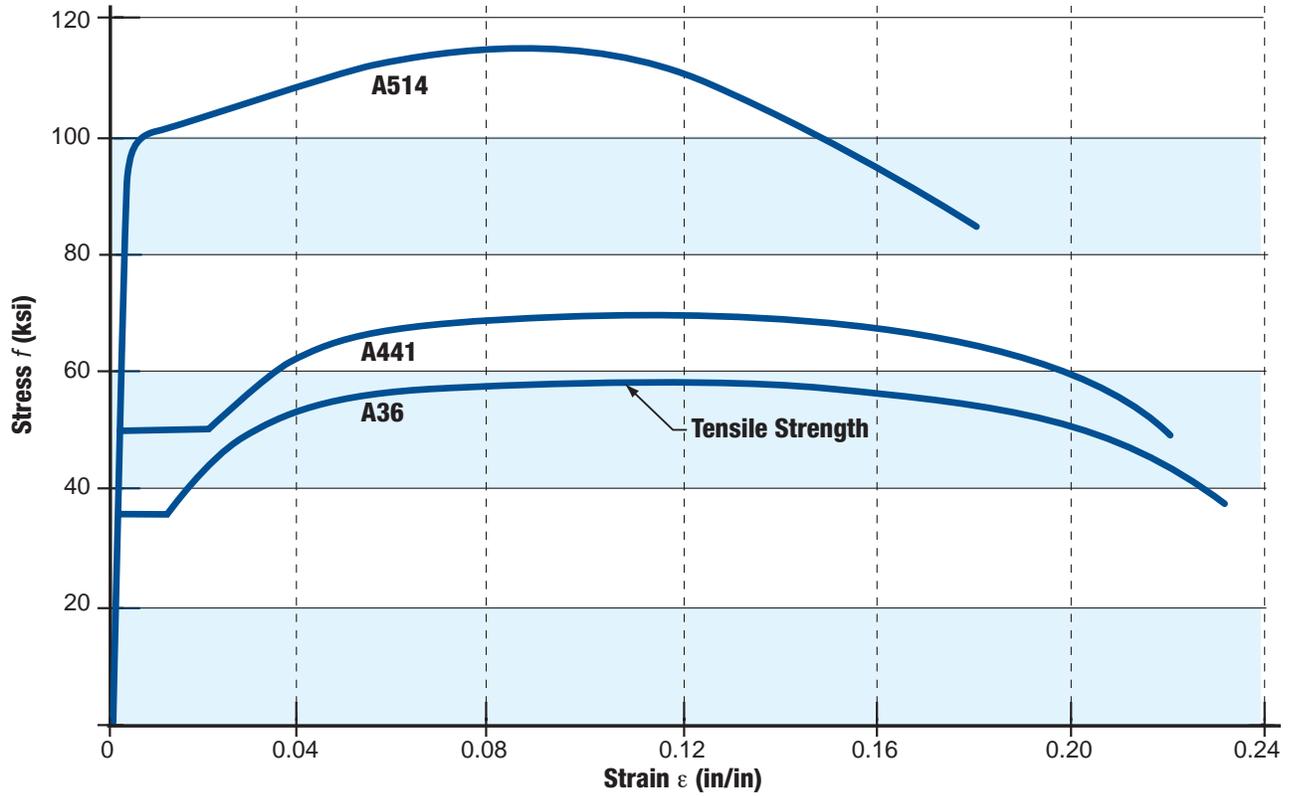
Figure B-2   Tensile stress-strain curves for three ASTM-designation steels (Brockenbrough and Johnston 1968, Tall 1974).
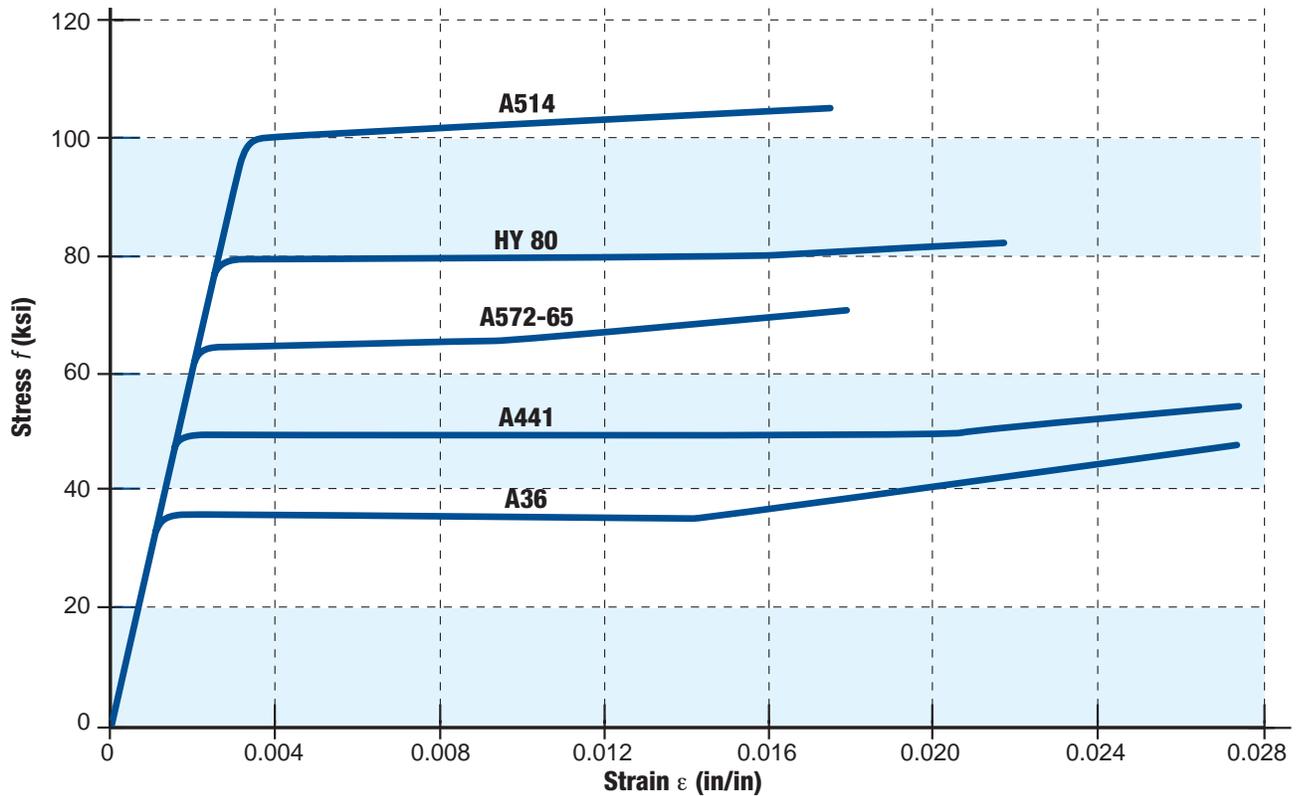

Figure B-3   Expanded yield portion of the tensile stress-strain curves (Tall 1974).
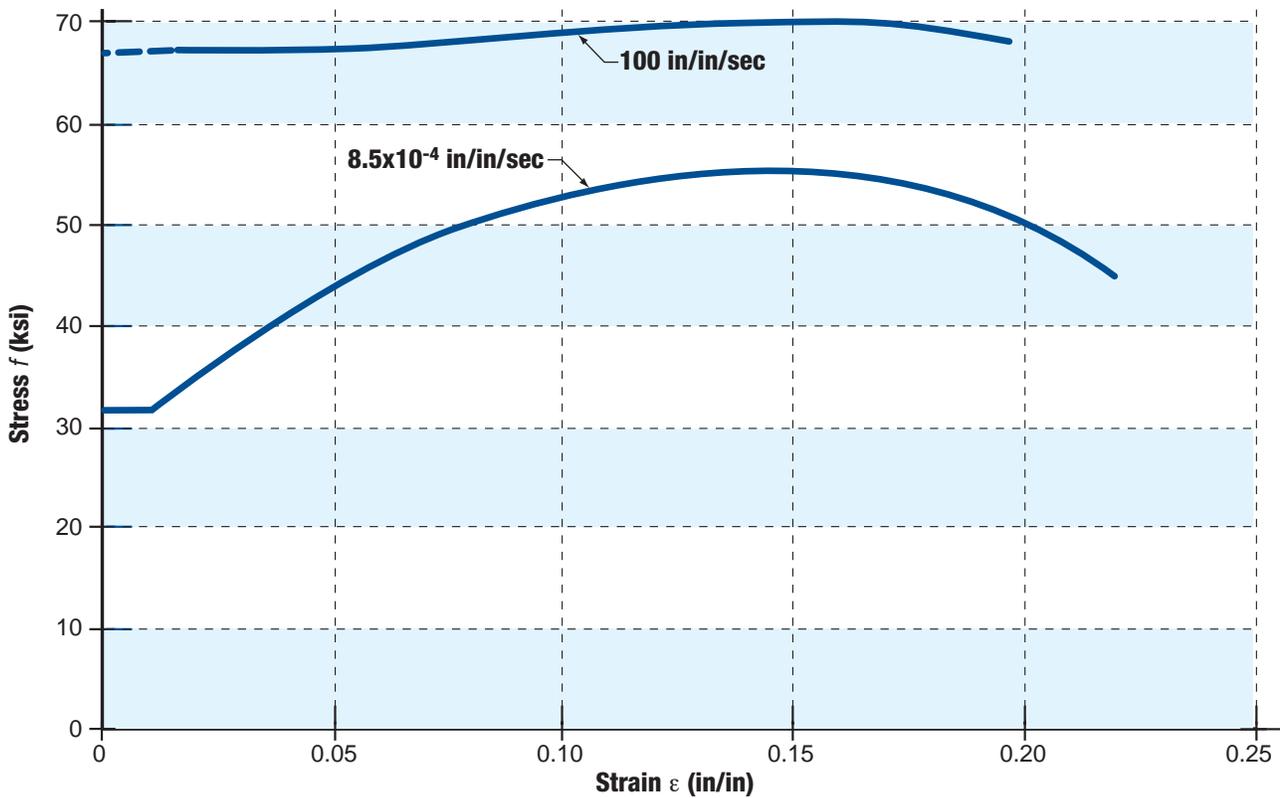
Figure B-4   Effect of high strain rate on shape of stress-strain diagram (Tall 1974).

Where σ_{ys} is room temperature static yield strength in ksi, t = loading time in milliseconds, and T is the test temperature in °F.

In *Making, Shaping, and Treating of Steel* (USX 1998), it is noted that a tenfold increase in rate of loading increased a 0.12 percent carbon steel yield strength by 7 ksi, but the influence on tensile strength was negligible.

High impacts that create notches can also lead to brittle fracture at stresses that are less than the dynamic yield strength. This is also true if triaxial stress conditions exist from constraint.

The high-temperature characteristics of structural steel are discussed in Appendix A.

B.3 WTC 1 and WTC 2 Connection Capacity

B.3.1 Background

Connections are typically designed to transfer the joint forces to which they are subjected. Generally, simple equilibrium models are used to proportion the mechanical or welded connectors and the plate or beam elements used in the connection for the required design loads (Fisher, et al. 1978; Kulak, et al. 1987; Lesik and Kennedy 1990; Salmon and Johnson 1996).

According to available information, steel connections in the WTC structures were designed in accordance with the AISC specifications that were applicable at the time to resist the required design loads. This section focuses on the ultimate limit strengths of the connectors and the various connections that were used to construct the WTC towers. Standard practice is that the design of connectors and connections provide a factor of safety of at least two against the various design strength limit states. Significant deformations can be expected when these limit states are reached.

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

$$F_n = C_t F_u \qquad (B-2)$$

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

$$F_v = C_s F_u \qquad (B-3)$$

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.

Figure B-5
Column tree showing bolt bearing shear failures of spandrel connection.

The nominal strength of a linear weld group loaded in-plane through the center of gravity is (Fisher, et al. 1978; Lesik and Kennedy, 1990).

$$\text{F}_w = 0.6\ F_{EXX}\,(1.0 + 0.5\ \sin^{1.5}\theta)\tag{B-4}$$

where F_{EXX} is the electrode classification number (minimum specified tensile strength) and θ is the angle of loading measured from the longitudinal axis in degrees. Hence, when the load is parallel to the weld, the capacity is $0.6\ F_{EXX}$, and when it is perpendicular to the longitudinal weld axis, it increases to $0.9\ F_{EXX}$ or more. This increased strength of fillet welds transverse to the axis of loading was not recognized in the AISC Specification when the WTC was designed and built.

Figure B-6 shows the shear failure of the fillet welds that connected a built-up wide-flange column to the top end of a box core column. The basic limit states for bolts and welds described by Equations B-2, B-3, and B-4 are used in Chapter J of the AISC Load and Resistance Factor Design Specification along with resistance factors to design structural steel building connections (Fisher, et al. 1978). Additional connection strength design provisions are covered in Chapter K of the same specification for flanges and webs subjected to concentrated forces (Fisher, et al. 1978). Those relationships can also be used to assess the ultimate capacity, or strength, of structural members and connections subjected to concentrated tension and compression forces.

Discontinuities such as porosity and slag seldom cause a significant loss of static strength. Common imperfections are permitted within limits and accommodated by the provisions of the design standards. On the other hand, lack of fusion or cracks can have a major impact on strength and can result in joint

Figure B-6
Shear fracture failure of fillet welds connecting a W-shape column to a box core column.

failure at loads below the design load. This will depend on the size of the discontinuity or defect and its orientation to the applied loads.

B.4 Examples of WTC 1 and WTC 2 Connection Capacity

B.4.1 Bolted Column End Plates

Collapse of the WTC towers resulted in failure of many of the bolts in bolted end plate connections as the columns were subjected to large and unanticipated out-of-plane bending. In the majority of cases, the A325 high-strength bolts reached their tensile capacity and failed in the threaded stress area. The example shown in Figure B-7 examines the flexural capacity of the bolted end plate in a column in the impact area where the column plate thickness was 1/4 inch.

The simple moment capacity of the bolt group is 20 to 30 percent of the plastic moment capacity of a column fabricated from steels with a 50 to 100 ksi yield point, assuming no axial load in the columns. The end plates at the columns splice have a 11-3/4-inch x 14-inch cross-section. The columns are subjected to axial load from the dead load acting on the structure. For the as-built structure, the moments acting on the bolted splice are small, because the splices were located at the column inflection points and the resultant of the applied axial load and moment is within the middle third of the 12-inch-deep bearing connection. Assuming an axial stress of 20 ksi in the column, the corresponding axial force acting on the base plate is 280 kips. As the columns lose lateral support and deform out-of-plane from overloading eccentricities and from the thermal effects, the bending moment acting on the column splice does not introduce significant forces into the bolted end plate connection until the eccentricity exceeds 2 inches. As the eccentricity increases, the applied bending moment will exceed the bolt preload stress when the eccentricity reaches approximately 4 inches. Continued deformation will exceed the ultimate moment capacity of the connection and result in instability as the eccentricity approaches 4.5 inches.

It also should be noted that the column splices were staggered midheight at each floor, as was illustrated in Chapter 2. As a result, two-thirds of the perimeter columns were continuous at each floor's
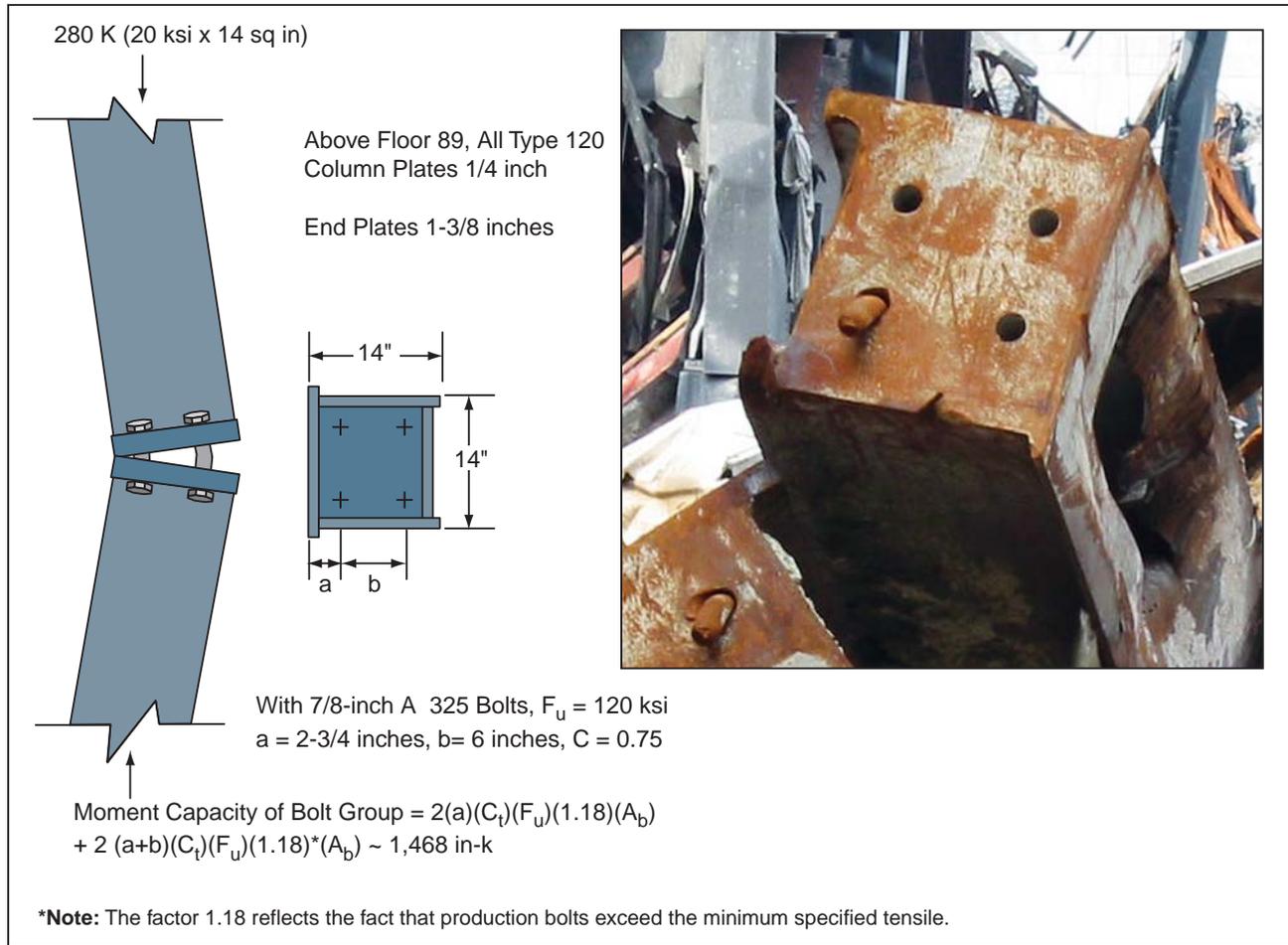
280 K (20 ksi x 14 sq in)

Above Floor 89, All Type 120 Column Plates 1/4 inch

End Plates 1-3/8 inches

14"

14"

a   b

With 7/8-inch A 325 Bolts, F_u = 120 ksi
a = 2-3/4 inches, b= 6 inches, C = 0.75

Moment Capacity of Bolt Group = $2(a)(C_t)(F_u)(1.18)(A_b)$
$+ 2 (a+b)(C_t)(F_u)(1.18)*(A_b) \sim 1{,}468$ in-k

Note: The factor 1.18 reflects the fact that production bolts exceed the minimum specified tensile.

Figure B-7 Bent and fractured bolts at an exterior column four-bolt connection.

midheight elevation. This resulted in staggered failure patterns, as the bolted end plate connections and spandrel beam connections failed during the resulting instability and collapse. The exception to this staggered pattern was the splices at mechanical floors, which were not staggered, and the bolts were supplemented with welds.

B.4.2 Bolted Spandrel Connections

Collapse of the WTC towers resulted in failure of the bolted shear splices that connected the spandrel beams together at each of the prefabricated column trees. Several modes of failure were observed in these connections. Figure B-5 showed an example of bearing strength failure of the spandrel plate. The loading appeared to be a combination of unanticipated moment and tensile loading. The following example examines the shear rupture capacity of one of the bolts. The spandrel plate thickness was assumed to be 3/8 inch, which was observed at the columns with 1/4-inch plate used in Figure B-7.

The ultimate bearing strength capacity is given by (AISC 2001; Fisher, et al. 1978)

$$R_u = L_c\, t\, F_u \le 3.0\; dt\, F_u \le F_v\, A_s \tag{B-5}$$

where d is the nominal bolt diameter (7/8 inch), L_c is the clear distance, in the direction of force, between the edge of the hole and the edge of the spandrel plate (1-5/16 inches), F_u is the tensile strength of the spandrel plate, and t is the thickness of the spandrel plate (0.375 inch).

This results in the following bearing capacity of a single bolt

$$R_u = L_c\, t\, F_u = 1.3125 \times 0.375 \times 90 = 44.3\ kips \tag{B-6}$$

This is well below the single shear capacity of the bolt, which is

$$F_v A_s = 0.62 \times 120 \times 1.18 \times 0.6013 = 52.8\ kips \tag{B-7}$$

Hence, the failure mode observed in Figure B-5 is consistent with the predicted capacity.

B.4.3   Floor Truss Seated End Connection at Spandrel Beam and Core

The floor system supported by 29-inch-deep prefabricated steel trusses consisted of 4 inches of lightweight concrete fill on a 1-1/2-inch corrugated deck that ran parallel with the truss (PATH-NYNJ 1976). As noted in the introduction, alternate truss supports had two joists attached to the seat connection.

Figure B-8 shows the end of the top chords that were connected to every other exterior column/ spandrel beam and the core support channel beams. The top chords were supported on bearing seats at each end of the two trusses. At the exterior column/spandrel beam, a gusset plate was groove-welded to the spandrel face and fillet-welded to the top chord angles. At the bearing seat, two 5/8-inch A325 bolts in 3/4-inch x 1-1/4-inch slotted holes connected the trusses' top chords to the bearing seat with a single bolt in the exterior angle of each truss. The lower chord was attached to the exterior column/spandrel beam with a visco-elastic damping unit connected to a small seat with two 1-inch A490 bolts that provided a slip-resistant connection. The damping unit had a capacity of about 5 kips.

At the core, the top chords were supported by bearing seats with two vertical stiffeners. Two 5/8-inch A325 bolts were installed in 3/4-inch x 1-3/4-inch slotted holes in the seat plate and standard holes in the top chord outside angles.

Figure B-9 shows several of the failure modes of the truss connections to the chord bearing seat and spandrel beam. The gusset plate welded to the spandrel beam and the top chord failed by tensile fracture of the plate. The gusset plate connection was primarily resisting the floor diaphragm support to the column. After fracture, the slotted holes in the seat would allow rigid body motion of the trusses until the 5/8-inch bolts came into bearing. That resulted in partial fracture in the seat of the fillet welds attaching the fill plate to the spandrel beam. The seat angle welded connection to the fill plate remained intact as this separation occurred and final block shear failure developed in the outstanding angle leg at the two slotted bolt holes.

The capacity of the 3/8-inch x 4-inch A36 steel gusset plate can be estimated as:

$$R_u = A_g F_u = (1.5\ inches^2)\,(60\ ksi) = 90\ kips \tag{B-8}$$

The bearing capacity of the two 5/8-inch bolts connecting the top chord angles to the seat angle is

$$R_u = 2L_c\, t\, F_u = 2(1\ inch)(0.375\ inch)(60\ ksi) = 45\ kips \tag{B-9}$$

The shear capacity of the two 5/8-inch A325 bolts is

$$2\, F_v A_x = 2 \times 0.62 \times 120 \times 1.18 \times 0.307 = 53.9\ kips \tag{B-10}$$

The block shear rupture strength provided in Chapter J of the AISC LRFD specification (Fisher, et al. 1978) can be used to assess the tensile force that separated the floor joist from the bearing seat

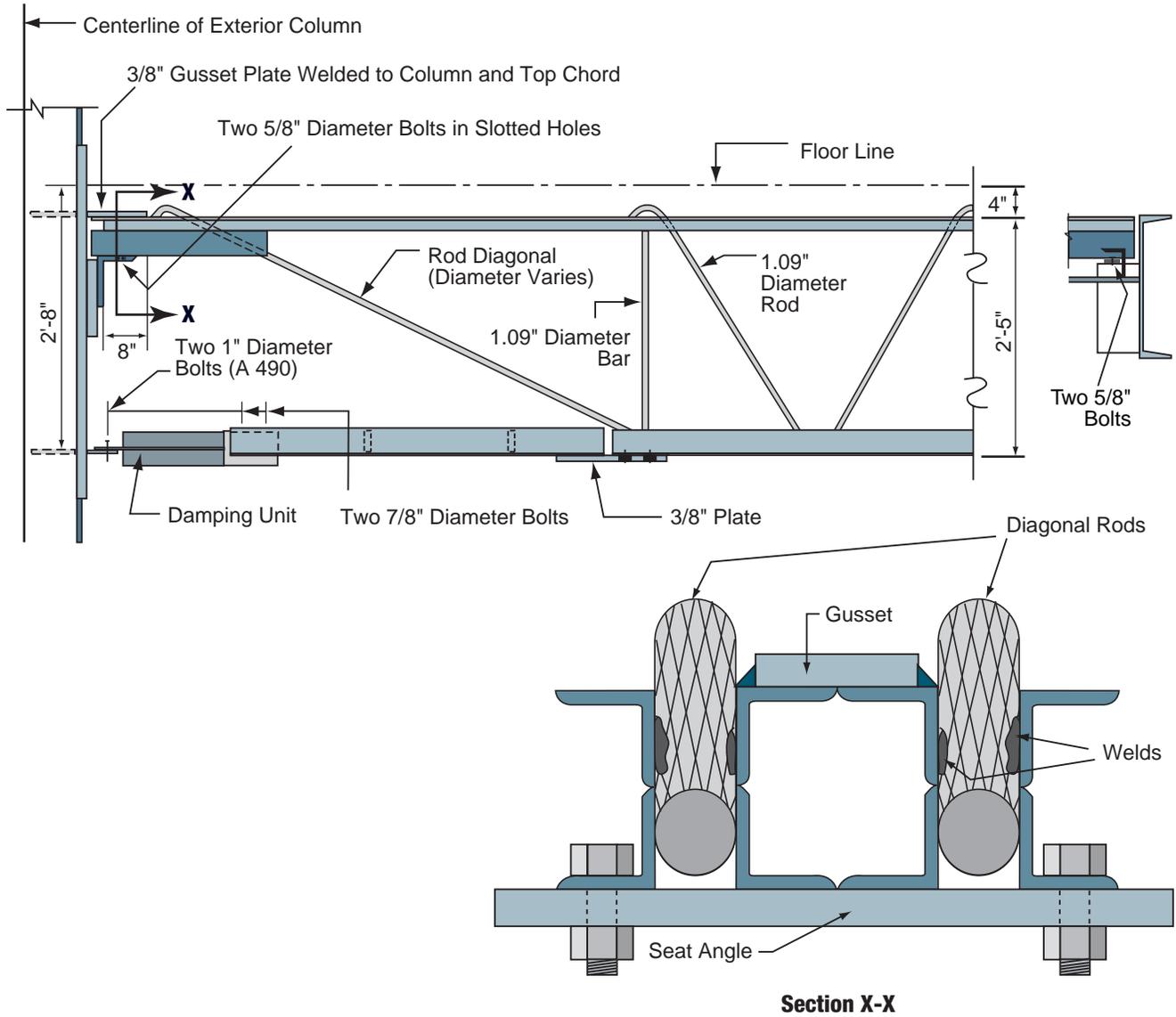$$R_u = 0.6\, F_u A_{nv} + F_u A_{nt} \tag{B-11}$$

Section X-X

Figure B-8   Typical truss top chord connections to column/spandrel beam and to the core beam.

where A_{nv} is the net shear area = 0.375 inch x 1.5 inches = 0.5632 inch^2.

The net tension area $A_{nt} = A_{nv}$. Hence, assuming an average tensile strength for A36 steel of Fu = 60 ksi results in a maximum resisting force

$$R_u = 2[0.6\ (60)(0.563) + 60\ (0.563)] = 108\ \textit{kips} \tag{B-12}$$

It is probable that, once the 3/8-inch gusset plate fractures, the next lower bound resistance is provided by the bearing capacity of the two 5/8-inch bolts on the beam seat angle. This failure also tore off the ends of the angle even though the tensile capacity of those segments was predicted to be higher.

Figure B-9   (A) Visco-elastic damper angles bolted to angle welded to spandrel plate and (B) failed bearing seat connection.

It should also be noted that each truss top chord provided a horizontal diagonal plate brace (1-1/2 inches x 1/2 inch) to the two adjacent columns. These members were welded to welded bracket plates on each adjacent column/spandrel member, as illustrated in Figure B-10. In this case, it would appear that the diagonal plate braces fractured on their gross section or tore the bracket plate. The component of ultimate strength of the two diagonal plate braces normal to the column/spandrel member is about 85 percent of the tensile capacity of braces, which would be 76 kips.

Figure B-10   (A) Bracket plate welded to the column/spandrel plate and (B) horizontal plate brace with shear connectors welded to the failed bracket.

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

$$M_p = F_y Z_x \tag{B-13}$$

and the shear yield capacity is

$$V_p = 0.6 F_y d t_w \tag{B-14}$$

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.

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

$$V_u = 3 R_n = 3(3.0 d_b t_w F_u) \tag{B-15}$$

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 °C (392 °F) and the yield strength is 0.5 F_y at 550 °C (1,022 °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 °C (1,022 °F) was reached, this analysis estimates the cantilever plastic moment capacity as

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

550 °C (1,022 °F): $\qquad M_p = 0.5 F_y Z_x = 0.5 (40) (152) = 3,040$ in-k $\qquad$ (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 °C (1,022 °F). The limiting shear yield strength of the cantilever is

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

550 °C (1,022 °F): $\qquad V_p = 0.6 (0.5 \times 40) (23) (0.419) = 120$ kips $\qquad$ (B-17b)

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

Room Temperature: $\qquad V_u = 3[3(0.75)(0.375)(60)] \cong 152$ kips $\qquad$ (B-18a)

550 °C (1,022 °F): $\qquad V_u = 3[3(0.75)(0.375)(30)] \cong 76$ kips $\qquad$ (B-18b)

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

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

550 °C (1,022 °F): $\qquad R_u = 3[2(0.62)(0.5)(1.18 F_{ub})(A_b)] \cong 116$ kips $\qquad$ (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 °C (1,022 °F), the bolt tensile strength is approximately $0.51(1.18 F_{ub}) \cong 71$ 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

$$R_u = L_e t F_u \le 3 d t F_u \tag{B-20}$$

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.

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

550 °C (1,022 °F): $\qquad R_u = (1.344) (0.375) (30) = 15$ kips/bolt $\qquad$ (B-21b)

The photographs in Figure 4-22, in Chapter 4, indicate that the deformed structure subjected the bolted shear connection to a large tensile force. At 550 °C (1,022 °F), the ultimate resistance of the three bolts is about 45 kips. The capacity increases to about 90 kips at room temperature. Failure occurred between these bounds.

Tensile catenary action of this type of floor framing members and their connections has not been a design requirement or consideration for most buildings. For the analysis shown here, with assumed fire temperatures, increasing the end distance L_e to 2.25 inches would increase the tensile capacity of the three bolts to about 76 kips at 550 °C (1,022 °F) and 152 kips at room temperature, because the resistance would increase to the limit in Equation B-20.

B.5 References

AISC. 2001. *Manual of Steel Construction*, LRFD. 3rd Edition. Chicago.

ASTM. 1999. ASTM E1820 *Standard Test Method for Measurement of Fracture Toughness*.

Brockenbrough, R.L., and Johnston, B.G. 1968. *USS Steel Design Manual*. USS, Pittsburgh, PA.

Fisher, J. W., Galambos, T. V., Kulak, G. L., and Ravindra, M. K. 1978. "Load and Resistance Factor Design Criteria for Connectors," *Journal of Structural Division*. ASCE, Vol. 104, No. ST9, September.

Kulak, G. L., Fisher, J. W., and Struik, J. H. A. 1987. *Guide to Design Criteria for Bolted and Riveted Joints*. 2nd Edition. Wiley, NY.

Lesik, D. F. and Kennedy, D. J. L. 1990. "Ultimate Strength of Fillet Welded Connections Loaded In-Plane," *Canadian Journal of Civil Engineering*. Vol. 17, No. 1.

PATH-NYNJ. 1976. PATH-NYNJ Document 761101, *The World Trade Center: A Building Project Like No Other*. May.

Salmon, C. G., and Johnson, J. E. 1996. *Steel Structures, Design and Behavior*. 4th Edition. Harper Collins.

Structural Steel Design. 1974. L. Tall, Ed. 2nd Edition. Ronald Press.

USX. 1998. *Making, Shaping, and Treating of Steel*.