Considerations in the Design of Bolted Joints for Weathering Steel

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In this discussion, some of the fundamentals that govern the behavior and design of bolted joints, in general, are first reviewed. Then, considerations that are particularly important in the design of joints in bare weathering steel are presented. The discussion is primarily concerned with high-strength bolts, since such bolts are the ones primarily used in construction today.

High-strength bolts were introduced about 25 years ago, and have gained wide acceptance for the construction of bridges, buildings, and structures of all types. Most are furnished to the requirements of ASTM specification A325, which includes three types distinguished by chemical composition: Type 1, a medium-carbon steel; Type 2, a low-carbon martensite steel; and Type 3, a weathering steel. All have a minimum tensile strength of 120 ksi for bolt diameters from ½- to 1 in., and 105 ksi for bolt diameters from 1⅛ to 1½ in. Also used for some higher load applications are ASTM A490 alloy bolts, which have a minimum tensile strength of 150 ksi for bolt diameters from ½- to 1 in. Both A325 and A490 bolts are quenched and tempered. Minimum tensile-strength requirements are summarized in Table 1.

Table 1. Bolt Specifications

ASTM A325 Bolts
Types:
1—Medium Carbon Steel
2—Low-Carbon Martensite Steel
3—Weathering Steel
Tensile Strengths:
120 ksi for ½ to 1 in. dia.
105 ksi for 1⅛ to 1½ in. dia.
ASTM A490 Bolts
Type:
Alloy Steel
Tensile Strength:
150 ksi for ½ to 1½ in. dia.

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High-strength bolts must be installed so that, when all fasteners are tightened, each fastener has a minimum tension equal to 70% of the specified bolt tensile strength. This is frequently done by the "turn-of-the-nut" method. The joint is first brought to a "snug tight" condition, and then each nut is rotated through ⅓ to ⅔ turn, depending upon the bolt length. As illustrated in Fig. 1, there is ample reserve tensile strength and ductility after installation.[1]

The high installation force introduces localized high contact pressures which tend to locally seat the joint. Studies have shown that the contact stresses between faying surfaces are mainly concentrated in a region equal to about two bolt diameters. (See Fig. 2.)

High-strength bolted joints are designed to resist shearing forces either as friction- or bearing-type connections, Fig. 3. In friction-type connections, the entire load is considered to be transferred by shearing stresses on the faying surfaces. In bearing-type connections, the entire load is considered to be transferred by bearing stresses between the fastener and the bolt hole, and then by shearing stresses on the cross section of the bolt. The contact surfaces of bearing-type connections are usually treated the same as

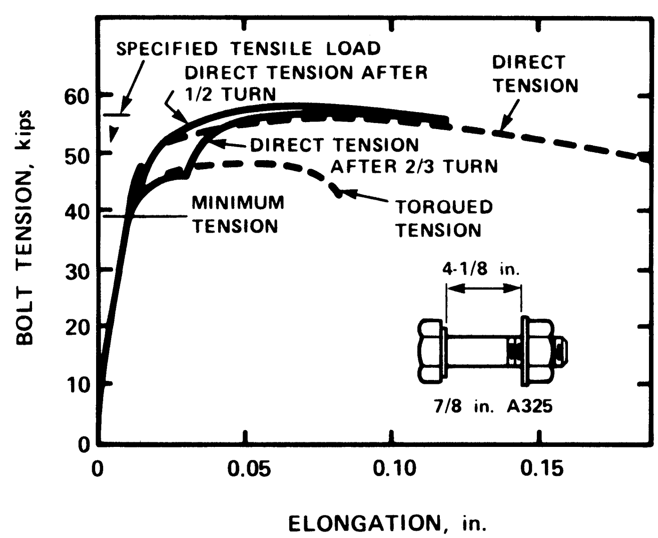


Fig. 1. Load-Elongation curves

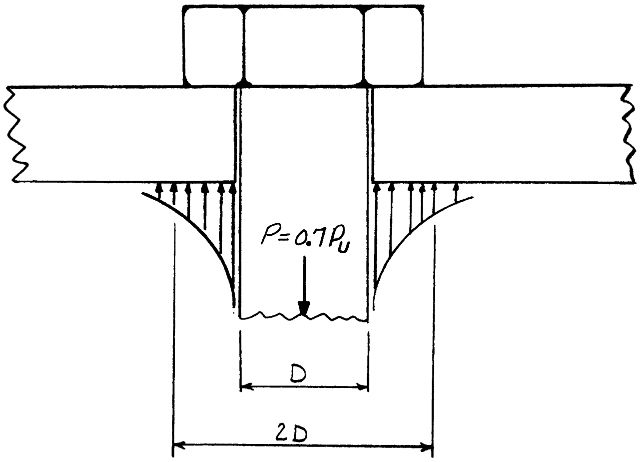


Fig. 2. High local contact pressure

the remainder of the structural member. The contact surfaces of friction-type joints may be left bare, or they may be protected by hot-dip galvanizing, inorganic zinc-rich paints, or metallized zinc or aluminum, in accordance with specified procedures.

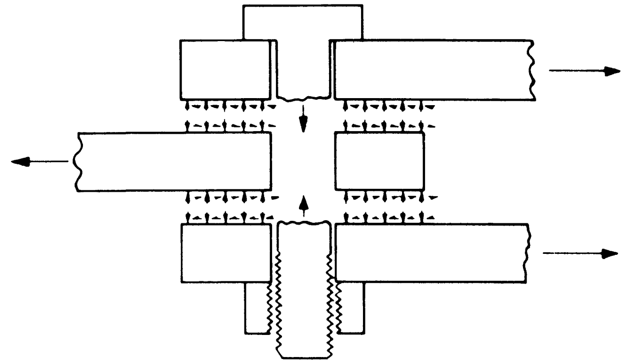
For convenience, both types of joints are designed on the basis of the shear stress on the nominal area of the bolt. The allowable shear stresses range up to 30 ksi for A325 bolts and up to 40 ksi for A490 bolts.[2] Lower allowable stress on friction-type joints are specified to prevent slippage. Friction-type connections are specified for shear connections subjected to stress reversal or severe stress fluctuation, or where slippage would be undesirable. The ultimate strength of either type of joint under static loading is about the same. Thus, the bearing-type joint is used predominantly because higher allowable stresses are permitted. Allowable shear stresses are summarized in Table 2.

Next, consider the design of joints in bare weathering steel, such as USS COR-TEN steel. Many years have passed since atmospheric-corrosion-resistant high-strength low-alloy steels were first used in the unpainted (bare) condition in the construction industry. These steels have been specified for such diverse applications as buildings, railroad hopper cars, bridges, light standards, transmission towers, plant structures, conveyor belt systems, hoppers, etc., because they are relatively inexpensive and require little maintenance. Under alternate wetting and drying conditions, a protective oxide coating forms that is resistant to further corrosion. However, if such atmospheric-corrosion-resistant steels remain wet for prolonged periods, their corrosion resistance will not be any better than that of carbon steel. Thus, the design of the structure must minimize ledges, crevices, and other areas that can hold water or collect debris.

Table 2. Allowable Shear Stresses (ksi)*

Connection Type	A325 Bolts	A490 Bolts
Friction	17.5	22.0
Bearing—Threads in Shear Plane	21.0	28.0
Bearing—Threads not in Shear Plane	30.0	40.0

* For standard holes. Allowable stresses differ for oversize and slotted holes, and for various surface conditions in friction-type connections. See *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Research Council on Riveted and Bolted Structural Joints, 1976.



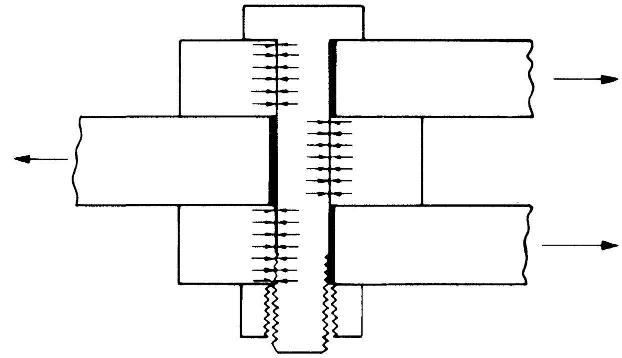
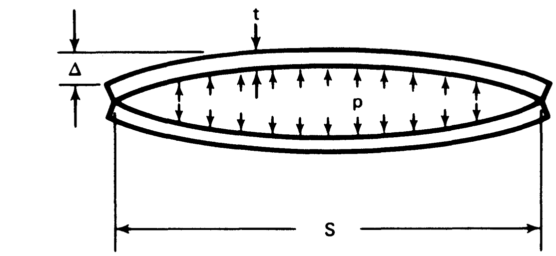


Fig. 3. Types of connections

Experience with bolted joints in exposed frameworks of bare weathering steel has indicated that, if the stiffness of the joint is adequate and the joint is tight, the space between two faying surfaces of weathering-type steel seals itself with the formation of corrosion products around the periphery of the joint. However, if the joint design does not provide sufficient stiffness, continuing formation of corrosion products within the joint leads to expansive forces which can (1) deform the connected elements such as cover plates, and (2) cause large tensile loads on the bolts.

A rational model of the mechanism can be developed.[3] Assume that the connected elements act as beams spanning between fasteners and are subjected to pressures from the development of corrosion products. If the beam is sufficiently stiff to limit the deflection to a low level, the corrosion products seal the edge before significant internal corrosion occurs. A simple structural analysis, Fig. 4, shows the deflection can be limited to a constant value if the



$$\Delta = \frac{pS^4}{384\ EI} = \frac{pS^4}{32\ Ebt^3}$$

$$S^4 = \frac{\Delta}{p}\ 32\ Ebt^3$$

$$S = Ct^{3/4}$$

WHERE:
Δ = DEFLECTION
p = PRESSURE FROM CORROSION
E = MODULUS OF ELASTICITY
I = MOMENT OF INERTIA
b = WIDTH
t = THICKNESS
S = BOLT SPACING

Fig. 4. Model of joint bowing

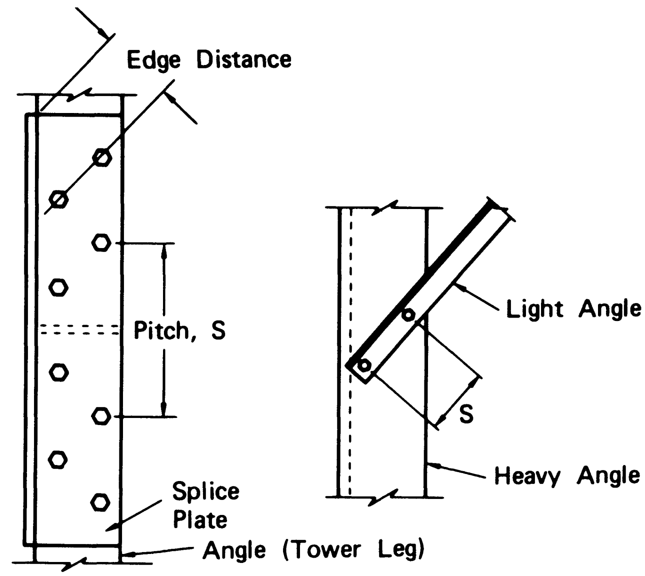


Fig. 6. Tower joints

spacing (or pitch, S) expressed in terms of the thickness (t) is

$$S = Ct^{3/4}$$

where C is a constant that can be determined by service experience. This equation would plot as a family of curves (Fig. 5); calibration against service experience would determine the appropriate curve. Such calibration has been done by inspections of numerous bare COR-TEN steel transmission towers (Fig. 6). The tower joints were connected by bolts furnished to another specification, but had a measured tensile strength approaching that specified for A325 bolts. However, the bolts were only installed to something equivalent to a snug tight condition. It was found from these inspections (Fig. 7) that if the pitch is limited to $14t$ or 7-in. maximum, significant bowing of the joint does not occur. Similarly, if the edge distance is limited to $8t$ or 5-in. maximum (Fig. 8), significant lift-up at the joint edges does not occur. The limit on edge distance is the same as that specified for many years by AASHTO for all types of structures, but is more conservative than that specified by AISC, (AISC does not specify a maximum pitch; AASHTO values for maximum pitch are somewhat different.)

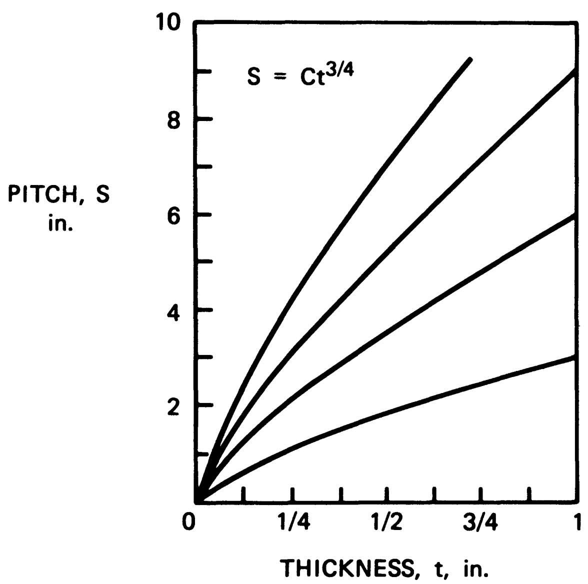


Fig. 5. Lines of constant deformation

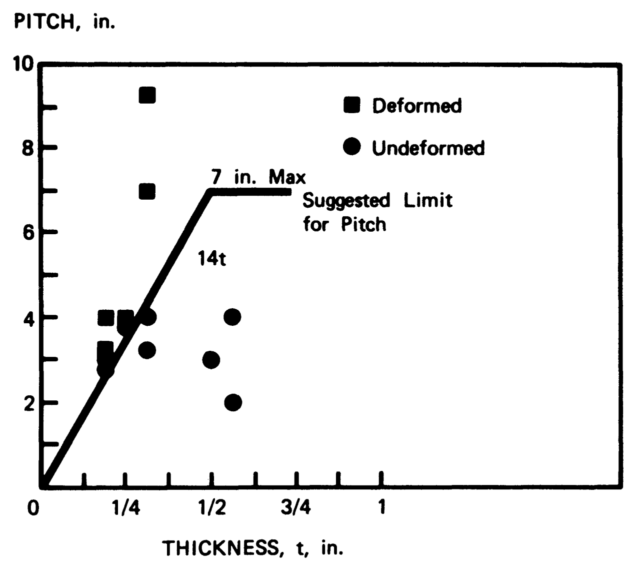


Fig. 7. Pitch limits

These guidelines were recently confirmed by observations made on 46 bolted-joint specimens exposed for about 7 years at Kure Beach, North Carolina and Monroeville, Pennsylvania.[4] Joint designs that met the suggested maximum pitch developed very little bowing between bolts during exposure at either site. Also, edge distortion was greatly reduced on joints that met the suggested maximum edge distance.

To obtain some quantitative evaluations of this phenomenon, both finite-element stress analyses and ultimate-load tests were conducted on tower joints removed from service. The joint design, Fig. 9, had a wide spacing that significantly exceeded the preceding guideline. The finite-element model (Fig. 10) included shell-type elements possessing membrane and bending stiffness arranged as shown. Both the main angle and the splice angle were included in the model. A uniform pressure was incrementally applied over the faying surface, so that deflections and bolt forces could be obtained as a function of the applied pressure. A curve of pressure versus corner deflection was plotted. Superposition of measured values of corner deflection showed that the pressure developed by corrosion products was equivalent to a uniform value of about 1200 psi (Fig. 11).

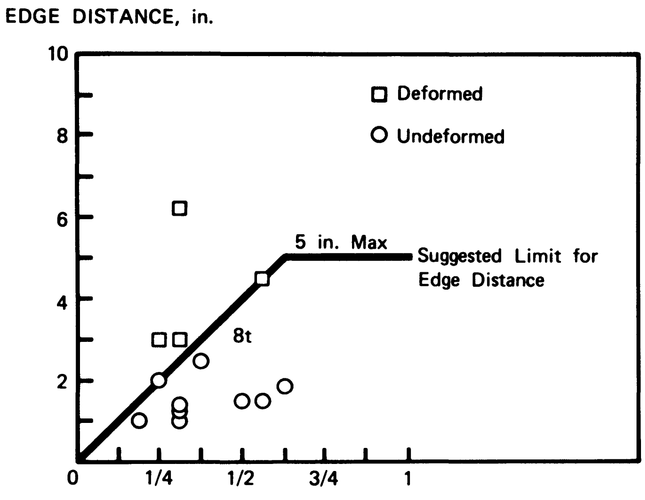


Fig. 8. Edge distance limits

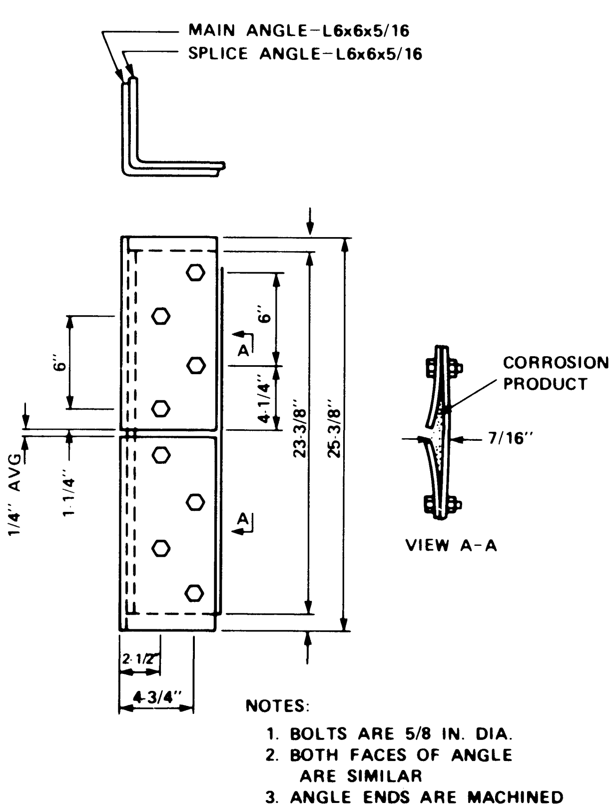


Fig. 9. Tower splice test specimen

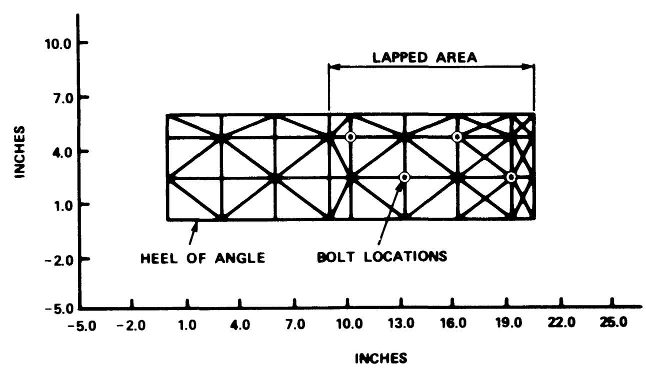


Fig. 10. Finite element model for specimen

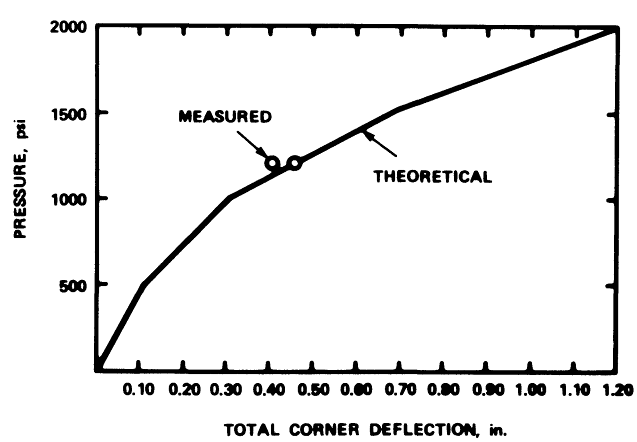


Fig. 11. Relationship between corner deflection and pressure

Plots of theoretical bolt stress versus applied pressure resulted in straight-line relationships (Fig. 12). The average bolt stress at 1200 psi was about 70% of the measured bolt tensile strength, equal to the minimum for a properly installed high-strength-bolt joint. Measurements of bolt stress showed reasonable correlation with the theoretical analysis. Apparently, if corrosion continued, bolt failures could be expected.

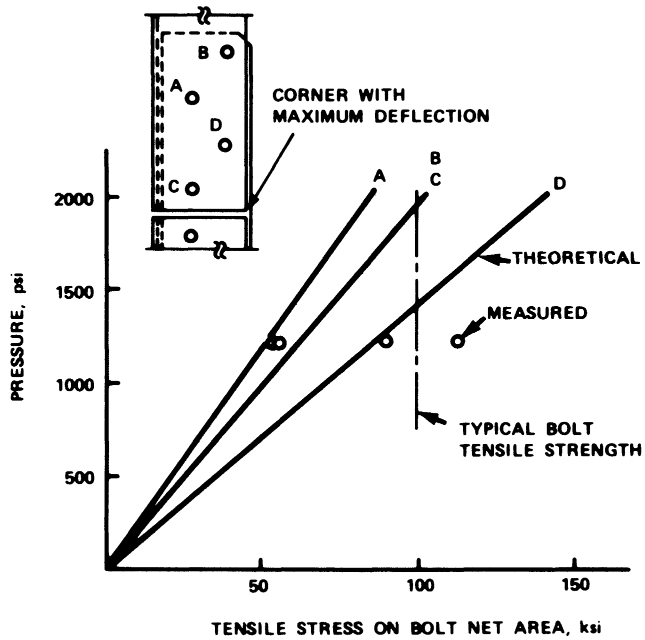


Fig. 12. Relationship between bolt stress and pressure

Typical load-deflection curves from stub-column compression tests of a joint showed high strength and ductility (Fig. 13). The ultimate strength of the joint was actually about 10% more than what would be calculated theoretically, neglecting any interaction between bolt shear and bolt tension. However, if the tensile stress should increase through continued prying from increased corrosion products, the interaction of shear and tension would reduce the ultimate load of the joint. Ultimate failure is expected to occur when the following relationship (Fig. 14) is satisfied:

$$\frac{f_v}{F_{us}} + \frac{f_t}{F_u} = 1$$

where f_v is the shear stress on the bolt, f_t is the tensile stress on the bolt, F_{us} is the ultimate shear strength of the bolt, and F_u is the ultimate tensile strength of the bolt.

Consequently, in the design of bolted joints in bare COR-TEN steel, it is important to adhere to the following guidelines:

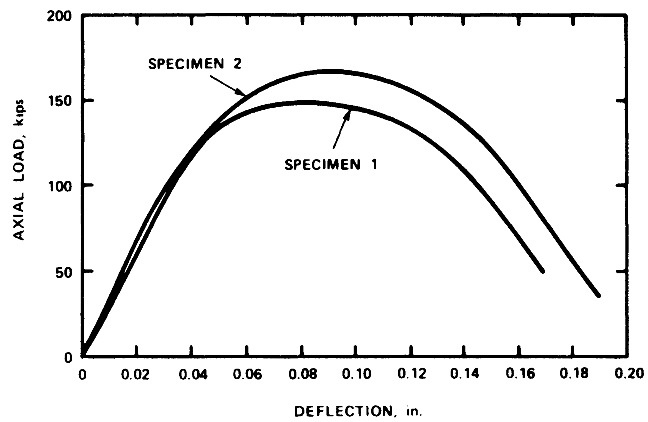


Fig. 13. Load-Deflection curve for joint specimen from tower

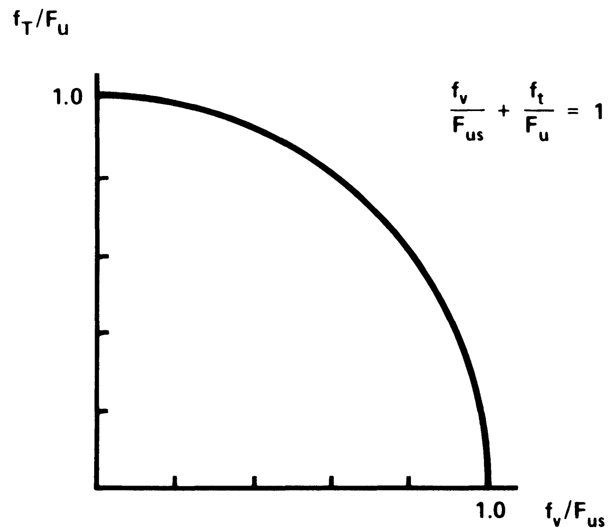


Fig. 14. Interaction of shear and tension

1. Limit pitch to 14 times the thickness of the thinnest part (7-in. maximum).
2. Limit edge distance to 8 times the thickness of the thinnest part (5-in maximum).
3. Use fasteners such as ASTM A325, Type 3, installed in accordance with specifications approved by the Research Council on Riveted and Bolted Structural Joints.

REFERENCES

1. *Fisher, J. W. and J. H. A. Struik* Guide to Design Criteria for Bolted and Riveted Joints *Wiley-Interscience, New York, 1973.*
2. Specification for Structural Joints Using ASTM A325 or A490 Bolts *Approved by the Research Council on Riveted and Bolted Structural Joints, 1976.*

3. *Brockenbrough, R. L. and R. J. Schmitt* Considerations in the Performance of Bare High-Strength Low-Alloy Steel Transmission Towers *Paper No. C75 041-9, IEEE, Jan. 1975.*
4. *Brockenbrough, R. L. and W. P. Gallagher* Effect of Clamping Pressure and Joint Geometry on Bowing and Distortion of Bare COR-TEN Steel Bolted Joints *Paper presented at 11th COR-TEN Conference, South Africa, June 1982.*

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