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Welded Tubular Connections—CHS Trusses

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29.1 Introduction

Truss connections in circular hollow sections (CHS) present unique design challenges. This chapter discusses the following elements of the subject: Architecture, Characteristics of Tubular Connections, Nomenclature, Failure Modes, Reserve Strength, Empirical Formulations, Design Charts, Application, and Summary and Conclusions.

29.2 Architecture

"Architecture" is defined as the art and science of designing and successfully executing structures in accordance with aesthetic considerations and the laws of physics, as well as practical and material considerations. Where tubular structures are exposed for dramatic effect, it is often disappointing to see grand concepts fail in execution due to problems in the structural connections of tubes. Such "failures" range from awkward ugly detailing, to learning curve problems during fabrication, to excessive deflections or even collapse. Such failures are unnecessary, as the art and science of welded tubular connections has been codified in the AWS Structural Welding Code [1].

A well-engineered structure requires that a number of factors be in reasonable balance. Factors to be considered in relation to economics and risk in the design of welded tubular structures and their connections include: (1) static strength, (2) fatigue resistance, (3) fracture control, and (4) weldability.

Static strength considerations are so important that they often dictate the very architecture and layout of the structure; certainly they dominate the design process and are the focus of this chapter. Many of the other factors also require early attention in design, and arise again in setting up QC/QA programs during construction; these are discussed further in sections of the Code dealing with materials, welding technique, qualification, and inspection.

29.3 Characteristics of Tubular Connections

Tubular members benefit from an efficient distribution of their material, particularly in regard to beam bending or column buckling about multiple axes. However, their resistance to concentrated radial loads are more problematic. For architecturally exposed applications, the clean lines of a closed section are esthetically pleasing and they minimize the amount of surface area for dirt, corrosion, or other fouling. Simple welded tubular joints can extend these clean lines to include the structural connections.

Although many different schemes for stiffening tubular connections have been devised [3], the most practical connection is made by simply welding the branch member to the outside surface of the main member (or chord). Where the main member is relatively compact (D/T less than 15 or 20), the branch member thickness is limited to 50 or 60% of the main member thickness, and a prequalified weld detail is used, the connection can develop the full static capacity of the members joined. Where the foregoing conditions are not met, e.g., with large diameter tubes, a short length of heavier material (or joint can) is inserted into the chord to locally reinforce the connection area. Here, the design problem reduces to one of selecting the right combination of thickness, yield strength, and notch toughness for the chord or joint can. The detailed considerations involved in this design process are the subject of this chapter.

29.4 Nomenclature

Non-dimensional parameters for describing the geometry of a tubular connection are given in the following list. Beta, eta, theta, and zeta describe the surface topology. Gamma and tau are two very important thickness parameters. Alpha (not shown) is an ovalizing parameter, depending on load pattern (it was formerly used for span length in beams loaded via tee connections).

β (beta)	d/D, branch diameter/main diameter
η (eta)	branch footprint length/main diameter
θ (theta)	angle between branch and main member axes
ζ (zeta)	g/D, gap/diameter (between balancing branches of a K-connection)
γ (gamma)	R/T, main member radius/thickness ratio
τ (tau)	t/T, branch thickness/main thickness

In AWS D1.1 [1], the term “T-, Y-, and K-connection” is used generically to describe simple structural connections or nodes, as opposed to co-axial butt and lap joints. A letter of the alphabet (T, Y, K, X) is used to evoke a picture of what the node subassemblage looks like.

29.5 Failure Modes

A number of unique failure modes are possible in tubular connections. In addition to the usual checks on weld stress, provided for in most design codes, the designer must check for the following failure modes, listed together with the relevant AWS D1.1-96 [1] code sections:

Local failure (punching shear)	2.40.1.1
General collapse	2.40.1.2
Unzipping (progressive weld failure)	2.40.1.3
Materials problems	
(fracture and delamination)	2.42, C4.12.4.4, and 2.1.3
Fatigue	2.36.6

29.5.1 Local Failure

AWS design criteria for this failure mode have traditionally been formulated in terms of punching shear. The main member acts as a cylindrical shell in resisting the concentrated radial line loads (kips/in.) delivered to it at the branch member footprint. Although the resulting localized shell stresses in the main member are quite complex, a simplified but still quite useful representation can be given in terms of punching shear stress, v_p:

$$\text{acting } v_p = f_n \tau \sin\theta \tag{29.1}$$

where f_n is the nominal stress at the end of the branch member, either axial or bending, which are treated separately. Punching shear is the notional stress on the potential failure surface, as illustrated in Figure 29.1. The overriding importance of chord thickness is reflected in tau, while $\sin\theta$ indicates that it is the radial component of load that causes all the mischief.

The allowable punching shear stress is given in the Code as:

$$\text{allowable } v_p = \frac{F_{yo}}{0.6\gamma} \quad \cdot \quad Q_q \cdot \quad Q_f \tag{29.2}$$

We see that the allowable punching shear stress is primarily a function of main member yield strength (F_{yo}) and gamma ratio (main member radius/thickness), with some trailing terms that tend towards unity. The term Q_q reflects the considerable influence of connection type, geometry, and load pattern, while interactions between branch and chord loads are covered by the reduction factor Q_f. Interactions between brace axial load and bending moments are treated analogous to those for a fully plastic section.

Since 1992, the AWS code also includes tubular connection design criteria in total load ultimate strength format, compatible with an LRFD design code formulation. This was derived from, and intended to be comparable to, the original punching shear criteria.

29.5.2 General Collapse

In addition to local failure of the main member in the vicinity of the branch member, a more widespread mode of collapse may occur, e.g., general ovalizing plastic failure in the cylindrical shell of the main member. To a large extent, this is now covered by strength criteria that are specialized by connection type and load pattern, as reflected in the Q_q factor.

For balanced K-connections, the inward radial loads from one branch member is compensated by outward loads on the other, ovalizing is minimized, and capacity approaches the local punching shear limit. For T and Y connections, the radial load from the single branch member is reacted by beam shear in the main member or chord, and the resulting ovalizing leads to lower capacity. For cross or X connections, the load from one branch is reacted by the opposite branch, and the resulting double dose of ovalizing in the main member leads to still further reductions in capacity. The Q_q term also reflects reduced ovalizing and increased capacity, as the branch member diameter approaches that of the main member.

Thus, for design purposes, tubular connections are classified according to their configuration (T, Y, K, X, etc.). For these “alphabet” connections, different design strength formulae are often applied

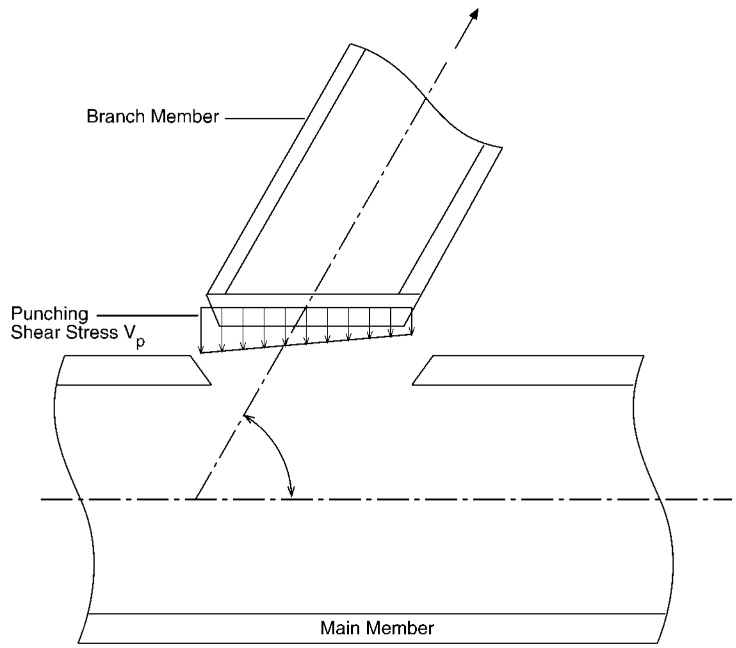

FIGURE 29.1: Local failure mode and punching shear V_p. (From Marshall, P., *Design of Welded Tubular Connections* (1992), Dev. Civ. Eng., Vol. 37. With kind permission from Elsevier Science, Amsterdam, The Netherlands.)

to each different type. Until recently, the research, testing, and analysis leading to these criteria dealt only with connections having their members in a single plane, as in a roof truss or girder.

Many tubular space frames have bracing in multiple planes. For some loading conditions, these different planes interact. When they do, criteria for the "alphabet" joints are no longer satisfactory. In AWS, an "ovalizing parameter" (alpha, Appendix L) may be used to estimate the beneficial or deleterious effect of various branch member loading combinations on main member ovalizing. This reproduces the trend of increasingly severe ovalizing in going from K to T/Y to X-connections, and has been shown to provide useful guidance in a number of more adverse planar (e.g., all-tension double-K [9]) and multi-planar (e.g., hub [11]) situations. However, for similarly loaded members in adjacent planes, e.g., paired KK connections in delta trusses, Japanese data indicate that no increase in capacity over the corresponding uniplanar connections should be taken [2].

The effect of a short joint can (less than 2.5 diameters) in reducing the ovalizing or crushing capacity of cross connections is addressed in AWS section 2.40.1.2(2) [1]. Since ovalizing is less severe in K-connections, the rule of thumb is that the joint can need only extend 0.25 to 0.4 diameters beyond the branch member footprints to avoid a short-can penalty. Intermediate behavior would apply to T/Y connections.

A more exhaustive discussion would also consider the following modes of general collapse in addition to ovalizing: beam bending of the chord (in T-connection tests), beam shear (in the gap of K-connections), transverse crippling of the main member sidewall, and local buckling due to uneven load transfer (either brace or chord). These are illustrated in Figure 29.2.

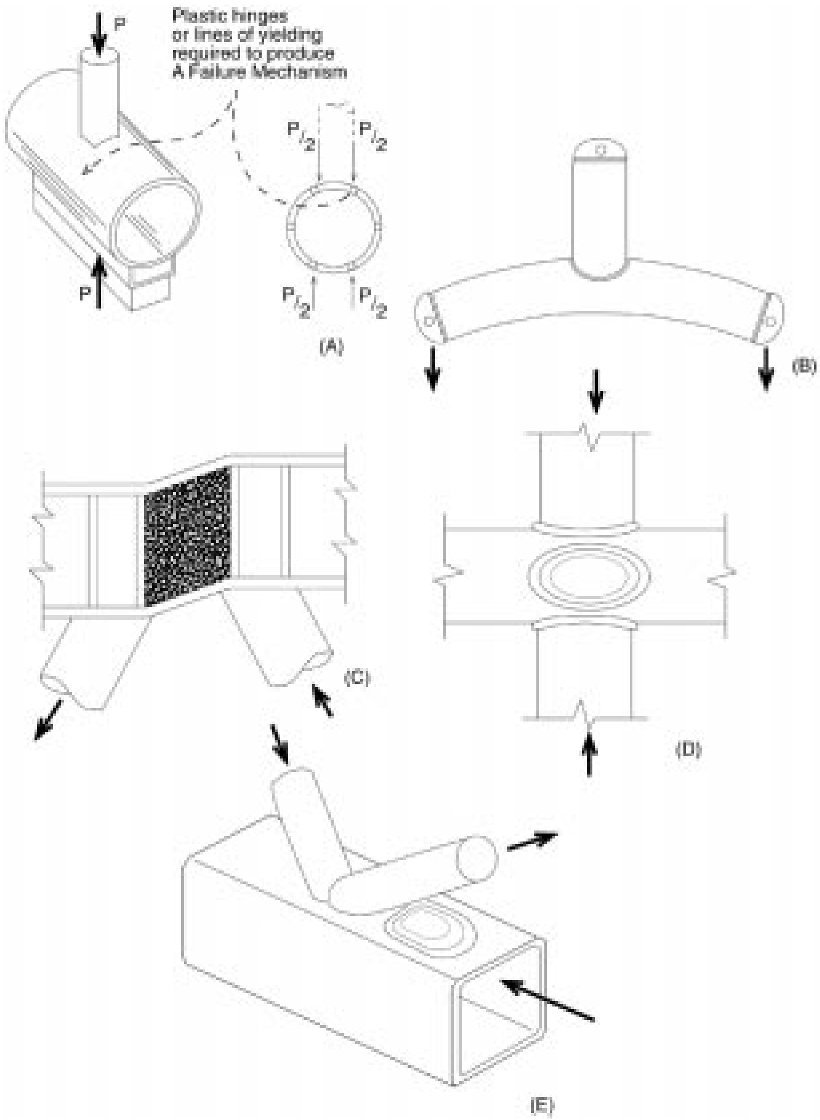

FIGURE 29.2: Failure modes — general colapse. (a) Ovalizing, (b) beam bending, (c) beam shear in the gap, (d) sidewall (web) crippling, and (e) local buckling due to uneven distribution of axial load. (From Marshall, P., *Design of Welded Tubular Connections* (1992), Dev. Civ. Eng., Vol. 37. With kind permission from Elsevier Science, Amsterdam, The Netherlands.)

29.5.3 Unzipping or Progressive Failure

The initial elastic distribution of load transfer across the weld in a tubular connection is highly non-uniform, as illustrated in Figure 29.3, with the peak line load often being a factor of two higher than that indicated on the basis of nominal sections, geometry, and statics. Some local yielding is required for tubular connections to redistribute this and reach their design capacity. If the weld is a weak link in the system, it may "unzip" before this redistribution can happen. Criteria given in the AWS code are intended to prevent this unzipping, taking advantage of the higher reserve strength in weld allowable stresses than is the norm elsewhere. For mild steel tubes and overmatched E70 weld metal, weld effective throats as small as 70% of the branch member thickness are permitted.

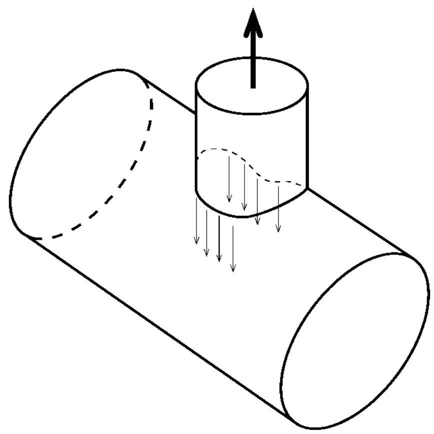

FIGURE 29.3: Uneven distribution of load across the weld. (From Marshall, P., *Design of Welded Tubular Connections* (1992), Dev. Civ. Eng., Vol. 37. With kind permission from Elsevier Science, Amsterdam, The Netherlands.)

29.5.4 Materials Problems

Most fracture control problems in tubular structures occur in the welded tubular connections, or nodes. These require plastic deformation in order to reach their design capacity. Fatigue and fracture problems for many different node geometries are brought into a common focus by use of the "hot spot" stress, as would be measured by a strain gauge, adjacent to and perpendicular to the toe of the weld joining branch to main member, in the worst region of localized plastic deformation (usually in the chord). Hot spot stress has the advantage of placing many different connection geometries on a common basis with regard to fatigue and fracture.

Charpy impact testing is a method for qualitative assessment of material toughness. The method has been, and continues to be, a reasonable measure of fracture safety when employed with a definitive program of nondestructive testing to eliminate weld area flaws. The AWS recommendations for material selection (C2.42.2.2) and weld metal impact testing (C4.12.4.4) are based on practices that have provided satisfactory fracture experience in offshore structures located in moderate temperature environments, i.e., 40°F (+5°C) water and 14°F (−10°C) air exposure. For environments that are either more or less hostile, impact testing temperatures should be reconsidered based on LAST (lowest anticipated service temperature).

In addition to weld metal toughness, consideration should be given to controlling the properties of the heat affected zone (HAZ). Although the heat cycle of welding sometimes improves hot rolled base metals of low toughness, this region will more often have degraded toughness properties. A number of early failures in welded tubular connections involved fractures that either initiated in or propagated through the HAZ, often obscuring the identification of other design deficiencies, e.g., inadequate static strength.

A more rigorous approach to fatigue and fracture problems in welded tubular connections has been taken by using fracture mechanics [5]. The CTOD (crack tip opening displacement) test is used to characterize materials that are tough enough to undergo some plasticity before fracture.

Underneath the branch member footprint, the main member is subjected to stresses in the thru-thickness or short transverse direction. Where these stresses are tensile, due to weld shrinkage or applied loading, delamination may occur — either by opening up pre-existing laminations or by laminar tearing in which microscopic inclusions link up to give a fracture having a woody appearance, usually in or near the HAZ. These problems are addressed in API joint can steel specifications 2H, 2W, and 2Y. Pre-existing laminations are detected with ultrasonic testing. Microscopic inclusions are prevented by restricting sulfur to very low levels ($<$ 60 ppm) and by inclusion shape control metallurgy in the steel-making ladle. As a practical matter, weldments that survive the weld shrinkage phase usually perform satisfactorily in ordinary service.

Joint can steel specifications also seek to enhance weldability with limitations on carbon and other alloying elements, as expressed by carbon equivalent or P_{cm} formulae. Such controls are increasingly important as residual elements accumulate in steel made from scrap. In AWS Appendix XI [1], the preheat required to avoid HAZ cracking is related to carbon equivalent, base metal thickness, hydrogen level (from welding consumables), and degree of restraint.

29.5.5 Fatigue

This failure mode has been observed in tubular joints in offshore platforms, dragline booms, drilling derricks, radio masts, crane runways, and bridges. The nominal stress, or detail classification approach, used for non-tubular structures fails to recognize the wide range of connection efficiencies and stress concentration factors that can occur in tubular structures. Thus, fatigue design criteria based on either punching shear or hot spot stress appear in the AWS Code. The subject is also summarized in recent papers on tubular offshore structures [7, 8].

29.6 Reserve Strength

While the elastic behavior of tubular joints is well predicted by shell theory and finite element analysis, there is considerable reserve strength beyond theoretical yielding due to triaxiality, plasticity, large deflection effects, and load redistribution. Practical design criteria make use of this reserve strength, placing considerable demands on the notch toughness of joint-can materials. Through joint classification (API) or an ovalizing parameter (AWS), they incorporate elements of general collapse as well as local failure. The resulting criteria may be compared against the supporting data base of test results to ferret out bias and uncertainty as measures of structural reliability. Data for K, T/Y, and X joints in compression show a bias on the safe side of 1.35, beyond the nominal safety factor of 1.8, as shown in Figure 29.4. Tension joints appear to show a larger bias of 2.85; however, this reduces to 2.05 for joints over 0.12 in. thick, and 1.22 over 0.5 in., suggesting a thickness effect for tests that end in fracture.

For overload analysis of tubular structures (e.g., earthquake), we need not only ultimate strength, but also the load-deflection behavior. Early tests showed ultimate deflections of 0.03 to 0.07 chord diameters, giving a typical ductility of 0.10 diameters for a brace with weak joints at both ends. As more different types of joints were tested, a wider variety of load-deflection behaviors emerged, making such generalizations tenuous.

Cyclic overload raises additional considerations. One issue is whether the joint will experience a ratcheting or progressive collapse failure, or will achieve stable behavior with plasticity contained at local hotshots, a process called "shakedown" (as in shakedown cruise). While tubular connections have withstood 60 to several hundred repetitions of load in excess of their nominal capacity, a conservative analytical treatment is to consider that the cumulative plastic deformation or energy absorption to failure remains constant.

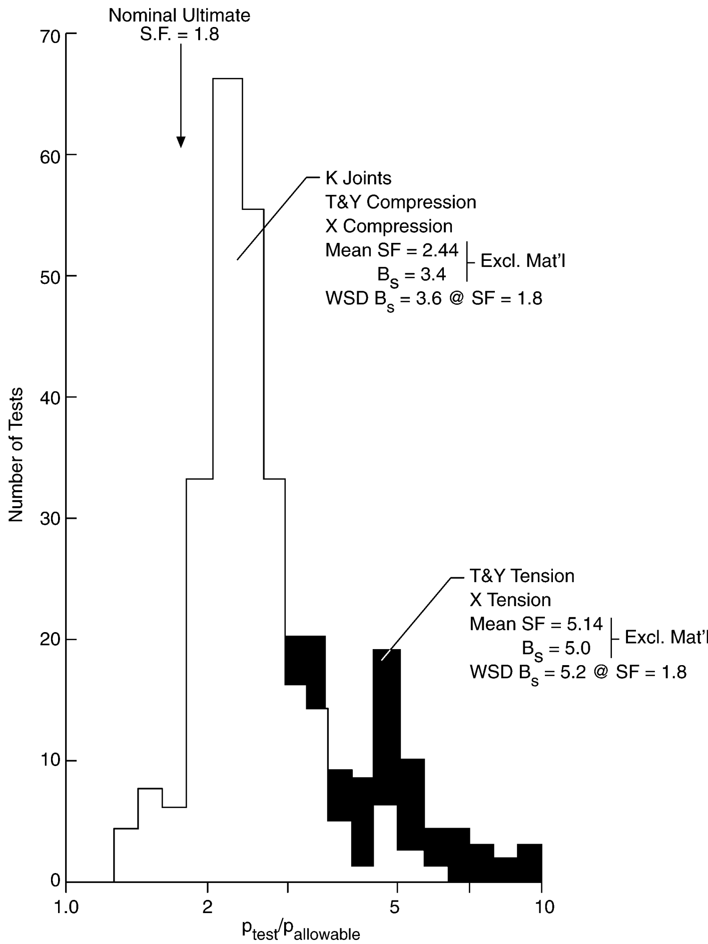

FIGURE 29.4: Comparison of AWS design criteria with the WRC database. (From Marshall, P., *Design of Welded Tubular Connections* (1992), Dev. Civ. Eng., Vol. 37. With kind permission from Elsevier Science, Amsterdam, The Netherlands.)

When tubular joints and members are incorporated into a space frame, the question arises as to whether computed bending moments are primary (i.e., necessary for structural stability, as in a sidesway portal situation, and must be designed for) or secondary (i.e., an unwanted side effect of deflection which may be safely ignored or reduced). When proportional loading is imposed, with both axial load and bending moment being maintained regardless of deflection, the joint simply fails when it reaches its failure envelope. However, when moments are due to imposed lateral deflection, and then axial load is imposed, the load path skirts along the failure envelope, shedding the moment and sustaining further increases in axial load.

Another area of interaction between joint behavior and frame action is the influence of brace bending/rotation on the strength of gap K-connections. If rotation is prevented, bending moments develop which permit the gap region to transfer additional load. If the loads remain strictly axial, brace end rotation occurs in the absence of restraining moments, and a lower joint capacity is found. These problems arise for circular tubes as well as box connections, and a recent trend has been to conduct joint-in-frame tests to achieve a realistic balance between the two limiting conditions. Loads that maintain their original direction (as in an inelastic finite element analysis) or, worse yet, follow the deflection (as in testing arrangements with a two-hinge jack), result in a plastic instability of the compression brace stub which grossly understates the actual joint strength. Existing data bases may need to be screened for this problem.

29.7 Empirical Formulations

Because of the foregoing reserve strength issues, AWS design criteria have been derived from a database of ultimate strength tubular joint tests. Comparison with the database (Figure 29.4) indicates a safety index of 3.6 against known static loads for the AWS punching shear criteria. Safety index is the safety margin, including hidden bias, expressed in standard deviations of total uncertainty. Since these criteria are used to select the main member chord or joint can, the choice of safety index is similar to that used for sizing other structural members, rather than the higher safety margins used for workmanship-sensitive connection items such as welds or bolts.

When the ultimate axial load is used in the context of AISC-LRFD, with a resistance factor of 0.8, AWS ultimate strength is nominally equivalent to punching shear allowable stress design (ASD) for structures having 40% dead load and 60% live load. LRFD falls on the safe side of ASD for structures having a lower proportion of dead load. AISC criteria for tension and compression members appear to have made the equivalency trade-off at 25% dead load; thus, the LRFD criteria given by AWS would appear to be conservative for a larger part of the population of structures.

In Canada, using these resistance factors with slightly different load factors, a 4.2% difference in overall safety factor results — within calibration accuracy [10].

29.8 Design Charts

Research, testing, and applications have progressed to the point where tubular connections are about as reliable as the other structural elements with which designers deal. One of the principal barriers to more widespread use seems to be unfamiliarity. To alleviate this problem, design charts have been presented in "Designing Tubular Connections with AWS D1.1", by P. W. Marshall [4].

The capacity of simple, direct, welded, tubular connections is given in terms of punching shear efficiency, E_v, where

$$E_v = \frac{\text{allowable punching shear stress}}{\text{main member allowable tension stress}} \tag{29.3}$$

Charts for punching shear efficiency for axial load, in-plane bending, and out-of-plane bending appear as Figures 29.5 through 29.9. Note that for axial load, separate charts are given for K-connections, T/Y connections, and X connections, reflecting their different load patterns and different values of the ovalizing parameter (alpha). Within each connection or load type, punching shear efficiency is a function of the geometry parameters, diameter ratio (beta) and chord radius/thickness (gamma), as defined earlier. For K-connections, the gap, g, between braces (of diameter d) is also significant, with the behavior reverting to that of T/Y connections for very large gap. Punching shear efficiency cannot exceed a value of 0.67, the material limit for shear.

29.8.1 Joint Efficiency

The importance of branch/chord thickness ratio tau (t/T) and of angle $(\sin\theta)$ becomes apparent in the expression for joint efficiency, E_j, given by:

$$E_j = \frac{E_v \quad \cdot \quad Q_f}{(t/T)\sin\theta} \quad \cdot \quad \frac{F_{yo}(\text{chord})}{F_y(\text{branch})} \tag{29.4}$$

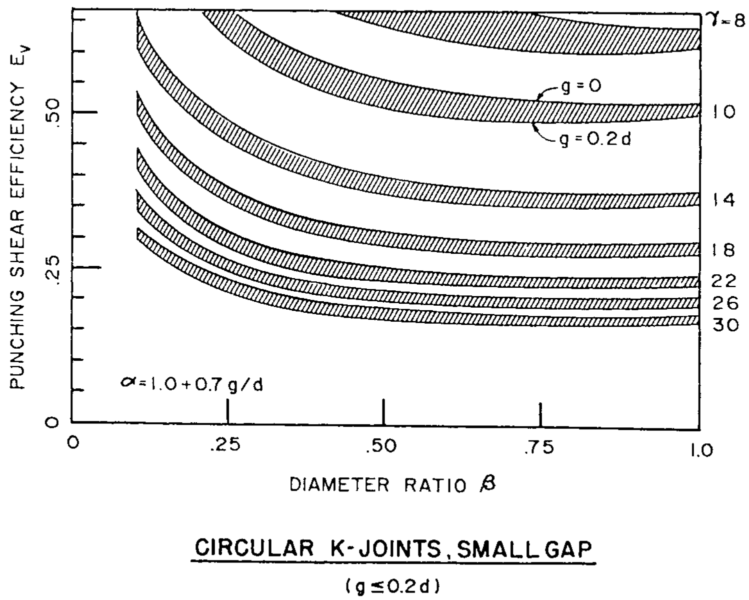

FIGURE 29.5: Values of Q_q for axial load in K-connections. (From Marshall, P.W., Designing Tubular Connections with AWS D1.1, *Welding J.*, March, 1989. With permission from the American Welding Society.)

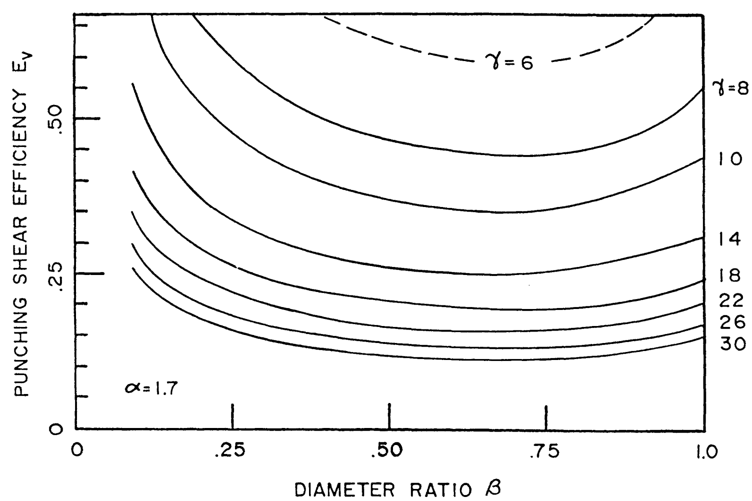

FIGURE 29.6: Values of Q_q for axial load in T- and Y-connections. (From Marshall, P.W., Designing Tubular Connections with AWS D1.1, *Welding J.*, March, 1989. With permission from the American Welding Society.)

where Q_f is the derating factor to account for chord utilization (described in the next section), and the ratio of specified minimum yield strengths F_{yo}/F_y drops out if chord and branch are of the same material. In LRFD, joint efficiency is the characteristic ultimate capacity of the tubular connection, as a fraction of the branch member yield capacity. In ASD, joint efficiency is the branch member nominal stress (as a fraction of tension allowable) at which the tubular connection reaches

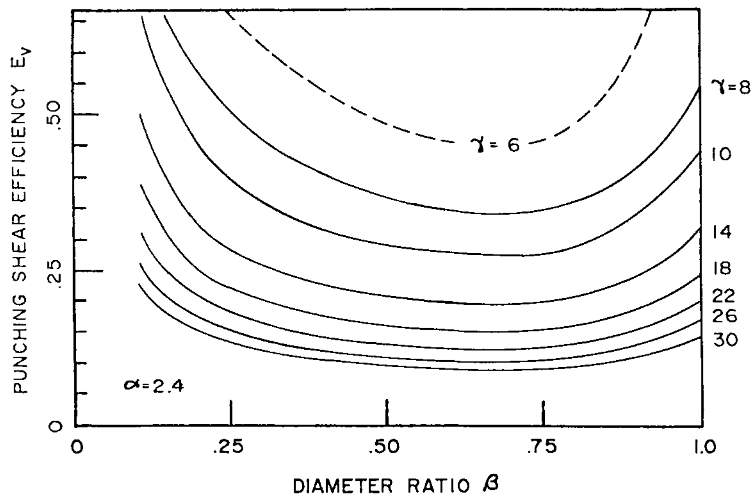

FIGURE 29.7: Values of Q_q for axial load in X-connections and other configurations subject to crushing. (From Marshall, P.W., Designing Tubular Connections with AWS D1.1, *Welding J.,* March, 1989. With permission from the American Welding Society.)

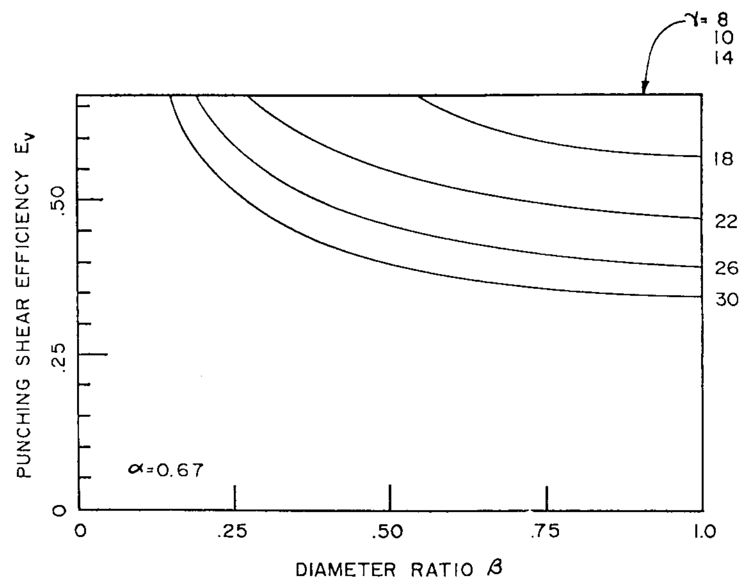

FIGURE 29.8: Values of Q_q for in-plane bending. (From Marshall, P.W., Designing Tubular Connections with AWS D1.1, *Welding J.,* March, 1989. With permission from the American Welding Society.)

its allowable punching shear. Connections with 100% joint efficiency develop the full yield capacity of the attached branch member, in either design format.

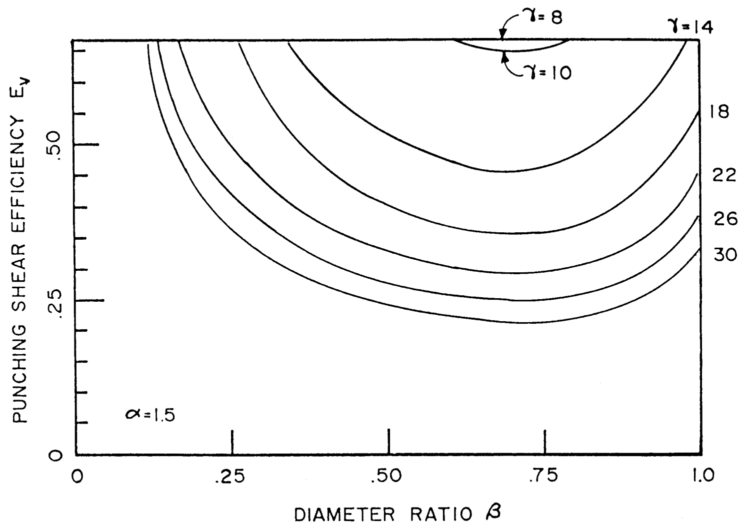

FIGURE 29.9: Values of Q_q for out-of-plane bending. (From Marshall, P.W., Designing Tubular Connections with AWS D1.1, *Welding J.*, March, 1989. With permission from the American Welding Society.)

29.8.2 Derating Factor

In most structures, the main member (chord) at tubular connections must do double duty, carrying loads of its own (axial stress f_a and bending f_b) in addition to the localized loadings (punching shear) imposed by the branch members. Interaction between these two causes a reduction in the punching shear capacity, as reflected in the Q_f derating factor, shown in Figure 29.10.

In-plane bending experiences the most severe interaction, as localized shell bending stresses at the tubular intersection are in the same direction and directly additive to the chord's own nominal stresses over a large part of the cross-section. For chords with very high R/T (gamma) and high nominal compressive stresses, buckling tendencies further reduce the capacity for localized shell stresses. Out-of-plane bending is less vulnerable to both these sources of interaction, as high shell stresses only occupy a localized part of the cross-section, and are transverse to P-Δ effects. Axially loaded connections of the types tested thus far exhibit intermediate behavior (although the gap region in K-connections might be expected to behave more like in-plane bending).

29.9 Application

What follows is a step-by-step design procedure for simple tubular trusses, applying the charts presented in the foregoing section.

Step 1. Lay out the truss and calculate member forces using statically determinate pin-end assumptions. Flexibility of the connections results in secondary bending moments being lower than given by typical rigid-joint computer frame analyses.

Step 2. Select members to carry these axial loads, using the appropriate governing design code, e.g., AISC. While doing this, consider the architecture of the connections along the following guidelines:

1. Keep compact members, especially low D/T for the main member (chord).
2. Keep branch/main thickness ratio (tau) less than unity, preferably about 0.5.

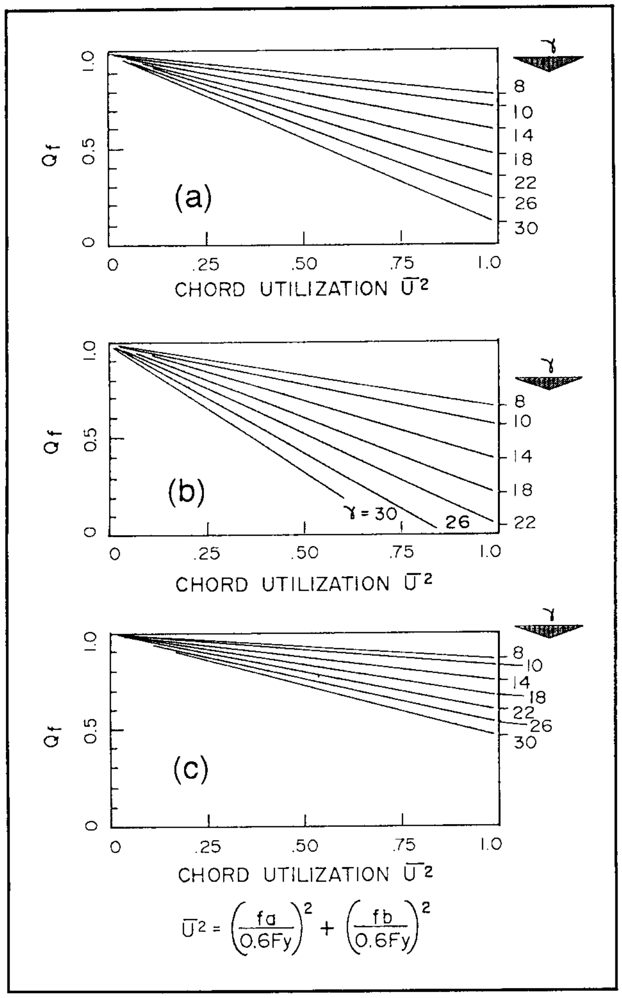

FIGURE 29.10: Derating factor Q_f for (a) axial loads in branch, (b) in-plane bending, and (c) out-of-plane bending. (From Marshall, P.W., Designing Tubular Connections with AWS D1.1, *Welding J.*, March, 1989. With permission from the American Welding Society.)

3. Select branch members to aim for large beta (branch/main diameter ratio), subject to avoidance of large eccentricity moments.
4. In K-connections, use a minimum gap of 2 in. between the braces for welding access. For small tubes, this may be reduced to 20% of the branch member diameter. Connection eccentricities up to 25% of the chord diameter may be used to accomplish this. Reconsider truss layout if this gets awkward.

Step 3. Calculate and distribute eccentricity moments and moments due to loads applied in-between panel points. These are not secondary moments, and must be provided for. They may be allocated entirely to the chord, for connection eccentricities less than 25% of the chord diameter, but should be distributed to both chord and braces for larger eccentricities, portal frames, or Vierendeel type trusses. Recheck members for these moments and resize as necessary.

Step 4. For each branch member, calculate A_y, utilization against member-end yield at the joint. For allowable stress design,

$$A_y = \frac{|f_a|}{0.6F_y} \text{ or } \frac{|f_b|}{0.6F_y} \tag{29.5}$$

where

f_a = nominal axial stress
f_b = bending in the branch

Where used, the 1/3 increase is applicable to the denominator.

Step 5. Also calculate chord utilization, using the formula in Figure 29.10 with chord nominal stresses and specified minimum yield strength. Use the appropriate chart in the figure to determine the derating factor Q_f. At heavily sheared gap K-connections and at eccentric bearing shoes, it may (rarely) also be necessary to check beam shear in the main member, and its interaction with other chord stresses, e.g., using AISC criteria. For circular sections, the effective area for beam shear is half the gross area.

Step 6. For each end of each branch member, calculate the joint efficiency E_j using Equation 29.4 and the appropriate charts for punching shear efficiency E_v. Joint efficiencies less than 0.5 are sometimes considered poor practice, rendering the structure vulnerable to incidental loads which the members could resist, but not the weaker joints.

Step 7. For axial loading alone, or bending alone, the connection is satisfactory if member-end utilization is less than joint efficiency, i.e., $A_y/E_j \leq 1.0$. For the general case, with combinations of axial load and bending, the connection must satisfy the following interaction formula:

$$(A_y/E_j)^{1.75}_{\text{axial}} + (A_y/E_j)_{\text{bending}} \leq 1.0 \tag{29.6}$$

Step 8. To redesign unsatisfactory connections, go back to Step 2 and

1. increase the chord thickness, or
2. increase the branch diameter, or
3. both of the above.

Consider overlapped connections (AWS section 2.40.1.6) or stiffened connections only as a last resort. Overlapped connections increase the complexity of fabrication, but can result in substantial reductions in the required chord wall thickness.

Step 9. When the designer thinks he is done, he should talk to potential fabricators and erectors. Their feedback could be valuable for avoiding unnecessary, difficult, and expensive construction headaches. Also make sure they are familiar with, and prepared to follow, AWS Code requirements for special welder qualifications, and that they are capable of coping the brace ends with sufficient accuracy to apply AWS prequalified procedures. Considerable savings can be realized by specifying partial joint penetration welds for tubular T-, Y-, and K-connections with no root access, where these are appropriate to service requirements. Fabrication and inspection practices for welded tubular connections have been addressed by Post [12].

29.10 Summary and Conclusions

This chapter has served as a brief introduction to the subject of designing welded tubular connections for circular hollow sections. More detail on the background and use of AWS D1.1 in this area can be found in [6].

References

[1] AWS D1.1-96. 1996. *Structural Welding Code — Steel*, American Welding Society, Miami, FL.

[2] Kurobane, Y. 1995. Comparison of AWS vs. International Criteria, ASCE Structures Congress, Atlanta, GA.

[3] Marshall, P.W. 1986. Design of Internally Stiffened Tubular Joints, *Proc. IIW/AIJ Intl. Conf. on Safety Criteria in the Design of Tubular Structures*, Tokyo.

[4] Marshall, P.W. 1989. Designing Tubular Connections, *Welding J.*

[5] Marshall, P.W. 1990. Advanced Fracture Control Procedures for Deepwater Offshore Towers, *Welding J.*

[6] Marshall, P.W. 1992. *Design of Welded Tubular Connections: Basis and Use of AWS D.1.1*, Elsevier Science Publishers, Amsterdam.

[7] Marshall, P.W. 1993. API Provisions for SCF, S-N and Size-Profile Effects, *Proc Offshore Tech. Conf.*, OTC 7155, Houston, TX.

[8] Marshall, P.W. 1996. Offshore Tubular Structures, *Proc. AWS Intl. Conf. on Tubular Structures*, Vancouver.

[9] Marshall, P.W. and Luyties, W.H. 1982. Allowable Stresses for Fatigue Design, *Proc. Intl. Conf. on Behavior of Off-Shore Structures*, BOSS-82 at MIT, McGraw-Hill, New York.

[10] Packer, J.A. et al. 1984. Canadian Implementation of CIDECT Monograph 6, IIW Doc. XV-E-84-072.

[11] Paul, J.C. 1992. The Ultimate Behavior of Multiplanar TT- and KK-Joints Made of Circular Hollow Sections, Ph.D. thesis, Kumamoto University, Japan.

[12] Post, J.W. 1996. Fabrication and Inspection Practices for Welded Tubular Connections, *Proc. AWS Intl. Conf. on Tubular Structures*, Vancouver.

[13] Wardenier, J. 1987. Design and Calculation of Predominantly Statically Loaded Joints Between Round (Circular) Hollow Sections, van Leeuwen Technical Information No. 7.