STEEL DESIGN – ADVANCED TOPICS

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Contents

Summary

The design of structural steel connections, including bolted connections, welded connections (loaded both concentrically and eccentrically) and column base connections is described. The use of plastic design as an alternative to conventional design is presented. The issue of fracture in structural steel components is discussed from a theoretical and practical perspective.

1. Introduction

As discussed in the previous chapter "*Structural Steel Analysis and Design: Fundamentals*", steel is a highly attractive material for civil construction, owing to its high strength as well as ductility, or deformation capacity. Consequently, the use of steel is widespread in the built environment. The fundamental properties of steel, and the types of steel are outlined in "*Structural Steel Analysis and Design: Fundamentals*" The basic design philosophy of Load and Resistance Factored Design (LRFD) is outlined as well, as is the design of structural steel members subjected to tension, compression, bending and combinations of these loads. Building on this framework, the main objective of this chapter is to introduce topics in steel design that are important, albeit not considered fundamental. The chapter begins by presenting the analysis and design of three major types of connections in steel structures, including bolted connections, welded connections and column base connections. The philosophy of plastic design is then presented, wherein the residual overstrength of a structure (owing to plastic redistribution) may be leveraged if adequate deformation capacity is present. The final topic of this chapter addresses fracture in structures. Recent occurrences of fracture in structural components (e.g. as observed in the 1994 Northridge, USA and 1995 Kobe, Japan) have reinforced the importance of fracture as a limit state. However, detailed design guidelines to protect against fracture are still not widespread. The section in this chapter provides an overview of the problem and the issues involved.

2. Bolted Connections

In the early 20^{th} century, riveting was the preferred method for connecting steel members. However, rapid advances in bolting and welding technology have led to the obsolescence of riveting, such that today, members in steel structures are typically connected through bolting, welding, or a combination of the two techniques.

Since the middle of the 21^{st} century, the use of high-strength bolts has almost entirely replaced riveting, which was prevalent before then. Unlike riveting, bolting may be performed by unskilled workers. Moreover, bolting is less noisy and less dangerous as compared to riveting, where heated rivets need to be tossed to the point of installation. While bolted connections are relatively cheap, and do not require the use of skilled workers, the obvious disadvantage of these connections (when compared to welded connections) is the loss of net area in the members, due to the introduction of the bolt-hole, necessitating the use of larger members. This section discusses commonly used bolted connections, their analysis, and methods for their design.

2.1. Types of Bolted Connections

Bolted connections are typically categorized based on the manner in which they are loaded. The most common types of bolted connections are shown in Figure 1. Figure 1a shows a lapped tension splice between two members, in which the bolts are loaded in shear. These types of connections will be referred to as bolted shear connections. Figure 1b shows a hanger type connection that subjects the bolts to tension. Figure 1c shows a bracket type connection, which is loaded eccentrically. In this section, we will focus on bolted shear connections, whereas a brief discussion of hanger type connections, and bolts loaded in shear and tension will be presented.

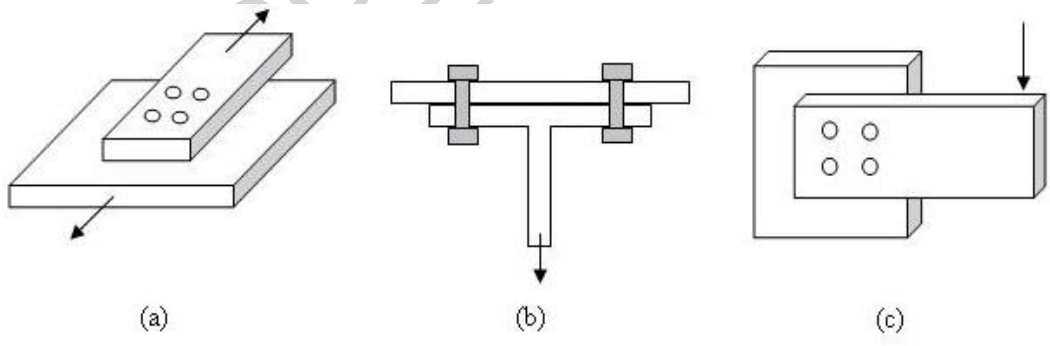

Figure 1. Types of bolted connections (a) Lapped tension splice (b) Hanger type connection (c) bracket type connection

Section 8 of the "*Structural Steel Analysis and Design: Fundamentals*" describes various types of structural fasteners and their properties. This section emphasizes the design of connections using these fasteners. Modern construction practice typically requires the use of high-strength bolts. The yield strength of these bolts is typically in the range of 550MPa to 650MPa. These bolts have hexagon heads and are used with semifinished hexagon nuts. The bolts typically feature only a small threaded portion. Figure 2 shows a high-strength bolt, with a nut-washer assembly. High strength bolts are typically tightened such that predictable tensile force develops in them, thereby resulting in a predictable clamping force in the joint. In fact, the joints are often designed to develop sufficient friction (i.e. slip-resistance) at service loads. These types of joints are referred to as slip-critical joints. If this degree of slip-resistance is not required, then the joints are referred to as bearing-type joints. To facilitate discussion of these various types of connections, the strength of the fasteners under various types of loading is first addressed.

Figure 2. High strength bolt and nut assembly

2.2. Nominal Strength of Bolts

This section addresses the nominal strength of bolt fasteners with respect to common loading modes, i.e. tension, shear, bearing and a combination of tension and shear. Once these basic relationships are established, they may be used to determine the strength of bolted connections wherein the bolts may be loaded in any of these modes.

2.2.1. Tensile Strength of Bolts

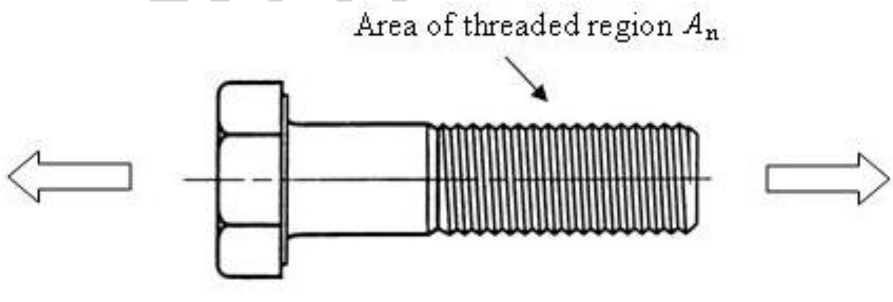

Figure 3. Bolt loaded in pure tension

If a bolt is loaded in pure tension, as indicated in Figure 3, the strength may be determined as

$$R_n = F_u^b A_n, \qquad (1)$$

where R_n is the nominal strength in tension, F_u^b is the ultimate strength of the bolt material, and A_n is the net cross sectional area through the threaded portion of the bolt.

Typically, this is taken conservatively as 75% of the gross (unthreaded) area of the bolt (A_b). Thus, Eq.(1) may be simplified to

$$R_n = F_u^b (0.75 A_b)\,. \tag{2}$$

2.2.2. Shear Strength of Bolts

Consider the lapped joint with a single bolt, subjected to a force P as shown in Figure 4a. The bolt is subjected predominantly to shear forces, such that shear failure will occur in the bolt, wherein the shear stress f_v in the bolt may be calculated as

$$f_v = R_n / A_b = P / (\pi \times d_b{}^2 / 4)\,, \tag{3}$$

where d_b is the cross-sectional diameter of the bolt, and A_b is the bolt cross-sectional area. The bolt is subjected to only a small degree of bending, and consequently, this eccentricity is neglected, assuming that the bolt fails in pure shear. If the failure shear stress of the bolt material is known, then the strength of the connection may be determined from Eq. (3) above as

$$R_n = A_b \times f_v\,. \tag{4}$$

If the connection features two splice plates, as shown in Figure 4b, then the bolt is subjected to two (rather than one) shear planes, such that the stress on each of the planes is only half of the stress that the bolt in Figure 4a is subjected to. In this case, the strength of the connection may be determined as

$$R_n = 2 \times A_b \times f_v\,. \tag{5}$$

The situation represented by Figure 4a is typically termed single shear, whereas the situation represented by Figure 4b implies that the bolt is subjected to double shear. Moreover, the shear strength is found experimentally to be approximately 62% of the ultimate tensile strength. Thus, Eq. (5) above may be generalized to

$$R_n = m A_b (0.62 F_u^b)\,, \tag{6}$$

where m, the number of shear planes passing through the bolt is generally 1 or 2. The above equation may be used to calculate the strength of a single fastener where the shear planes do not pass through the threaded region of the bolt. However, if shear planes do pass through the threaded region, the gross cross sectional area of the bolt must be replaced by the threaded area $A_n = 0.75 A_b$. The equations described above are valid for a single bolt, however, when connections with multiple bolts are constructed, the strength of the connection is less than that determined by adding the strength of the individual fasteners due to non-uniform distribution of forces in the bolts. Thus, for these, the strength per bolt is calculated as 80% of the strength determined as per Eq.

(6). For threads not in the plane of shear, the strength of the connection may be determined based on a per-fastener strength

$$R_\mathrm{n} = 0.8 \times mA_\mathrm{b}(0.62F_\mathrm{u}^\mathrm{b}) = mA_\mathrm{b}(0.5F_\mathrm{u}^\mathrm{b}). \qquad (7)$$

Similarly for threads in the plane of shear, the strength may be determined as

$$R_\mathrm{n} = 0.8 \times mA_\mathrm{b}(0.62 \times 0.75 \times F_\mathrm{u}^\mathrm{b}) = mA_\mathrm{b}(0.37F_\mathrm{u}^\mathrm{b}). \qquad (8)$$

The design strength is determined by multiplying the above strength values by a resistance factor φ. In both cases, the φ factor of 0.75 must be used.

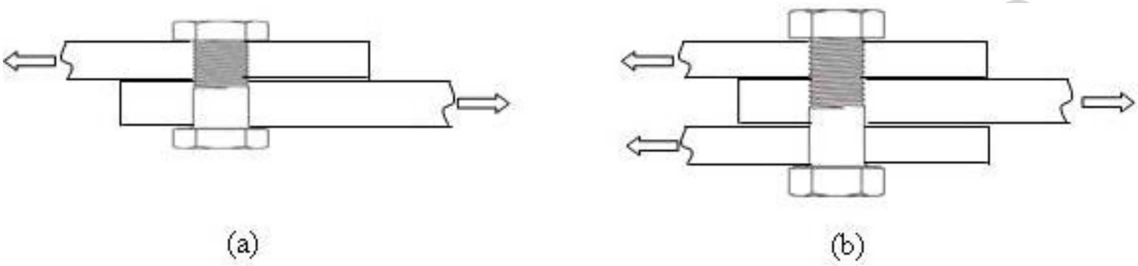

Figure 4. Lapped bolted joint (a) single shear and (b) double shear

2.2.3. Bearing Strength

The bearing limit states that are possible around a bolt hole are illustrated schematically in Figure 5.

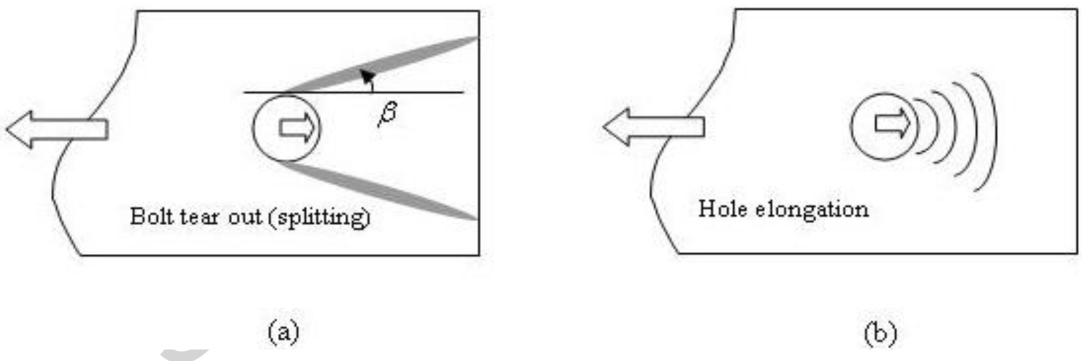

Figure 5. Bearing limit states at bolt hole

Referring to Figure 5, bolt bearing may produce either shear tear out (splitting) of the plate (Figure 5a), or excessive deformations of the bolt hole in the bearing region (Figure 5b). The bearing resistance will, in general, depend on the end distance between the edge of the hole, and the edge of the member, as illustrated in Figure 6.

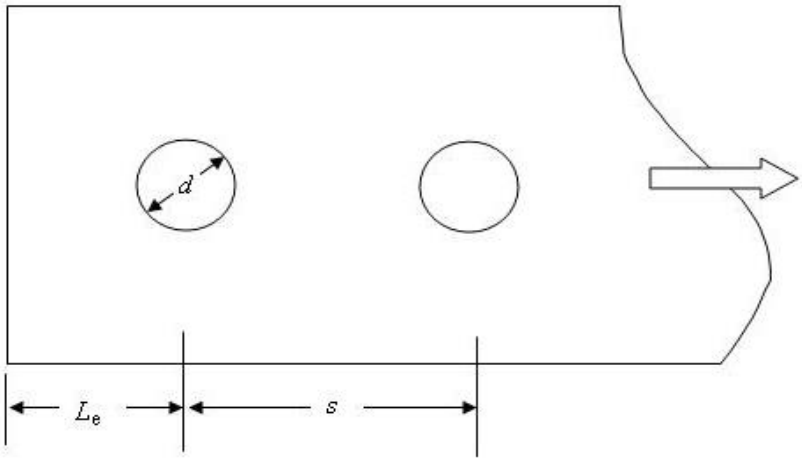

Figure 6. Bearing stress calculation for multiple bolt holes

While the actual tearing or splitting will occur along the angled lines as shown in Figure 5a, conservatively, the angle β indicated in the Figure 5a may be taken as zero. Thus, the strength associated with splitting of the plate may be calculated as the total area over which shear is active, times the shear strength. This may be determined as (for Hole 1, which is nearest to the edge)

$$R_\mathrm{n} = 2\times\left(L - d/2\right)\times t\times\tau\,, \tag{9}$$

where d is the diameter of the bolt hole, and t is the thickness of the connected plate or member. The shear strength of the plate material denoted as τ may be assumed as $\tau = 0.70F_\mathrm{u}$. Thus, Eq. (9) above reduces to

$$R_\mathrm{n} = 1.4\times F_\mathrm{u}\left(L_\mathrm{e} - d/2\right)\times t\,. \tag{10}$$

If multiple bolts are present in a connection, then the capacity for each bolt hole may be calculated based on the clear distance between edges of the adjacent holes. Thus, for Hole 2, which is farther from the edge, the capacity may be calculated as

$$R_\mathrm{n} = 1.4\times F_\mathrm{u}\left(s - d\right)\times t\,. \tag{11}$$

It is recommended that center-to-center spacing s between the holes should be at least 2.67 times the hole diameter. Even if the clear distance between the holes (or between the hole and the edge) is large enough such that splitting is avoided, the holes may suffer excessive elongation. To prevent this, the capacity associated with each hole, must be determined as the minimum of the ones predicted by Eqs. (10) and (11) above, and Eq. (12) below

$$R_\mathrm{n} = 2.4F_\mathrm{u}dt\,. \tag{12}$$

In Eq. (12), the capacity is dependent only on the hole diameter and plate thickness and is independent of the edge distances. Thus, for each hole, the strength may be determined as

$$(R_{\mathrm{n}} = 1.4\times F_{\mathrm{u}}\left(L_{\mathrm{e}} - d/2\right)\times t \quad \text{OR} \quad R_n = 1.4\times F_u\left(s-d\right)\times t) \le R_n = 2.4 F_u dt\,. \qquad (13)$$

Once the capacity for each hole is determined in this way, the capacity for all the holes may be added to determine the total strength of the connection associated with bearing. While the nominal strength is R_{n}, the design (or available) strength may be determined as $\varphi\times R_{\mathrm{n}}$, where $\varphi = 0.75$.

2.2.4. Bolts Subjected to Combined Shear and Axial Stress

In several situations, the bolts are loaded in a combination of axial tension and shear. In these situations, the shear and tension forces interact, such that if part of the strength in shear has been used by the load, then the full strength in tension is not available. Based on experimental data, this interaction equation may be represented by the following elliptical relationship, and represented graphically in Figure 7

$$\left(\frac{R_{\mathrm{ut}}}{\varphi_{\mathrm{t}} R_{\mathrm{nt}}}\right)^2 + \left(\frac{R_{\mathrm{uv}}}{\varphi_{\mathrm{v}} R_{\mathrm{nv}}}\right)^2 \le 1\,, \qquad (14)$$

where the terms in the numerators of the above equation are the factored tension (R_{ut}) and shear (R_{uv}) loads on the bolt, whereas the corresponding denominators are the design strengths in tension and shear respectively, such that both the φ-factors are 0.75. The terms in the denominator may be calculated as per Eq. (2) for tension, and Eqs. (6) and (7) for the shear strength of bolts, where the threads may or may not be in the plane of shear. Equation (14) may be simplified into a linear equation as indicated on Figure 7.

These situations may arise, as discussed earlier in connections where eccentric loading is present, although the load is not in the plane of the bolts. If high strength bolts are used, typically the tension in the high strength bolts prevents separation of the two attached components. In these cases, the tension and shear in the bolts may be determined in a straightforward manner though elastic analysis. The tension and shear thus calculated may be used in conjunction with the formula of Eq. (14) to evaluate safety of the connection.

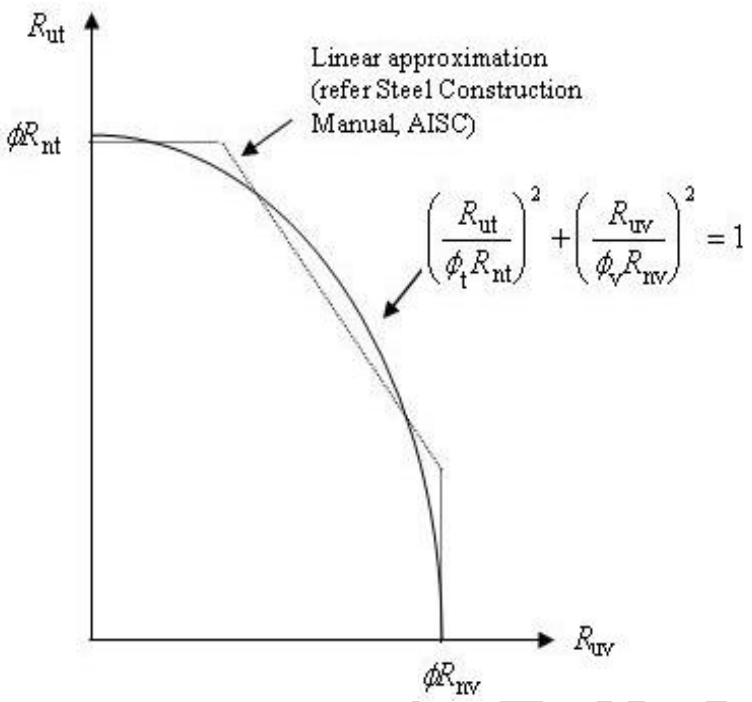

Figure 7. Interaction diagram between bolt shear and tension

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Biographical Sketch

Amit Kanvinde received his Bachelor of Technology in 1999 from the Indian Institute of Technology, Mumbai, India. Subsequently, he earned his MS (2000) and PhD (2004) degrees in Structural Engineering at Stanford University in California. His doctoral dissertation focused on fracture in steel structures. After completing his PhD, he joined the University of California at Davis, where he conducts research on various aspects of the response of steel structures. His recent research has addressed welded connections, column base connections, as well as earthquake-resistant braced frames. He remains at UC Davis as an Associate Professor, where he teaches classes on steel design at both the undergraduate and the post-graduate level.