



17
Steel Design Guide

High Strength Bolts

A Primer for Structural Engineers





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Geoffrey Kulak
Professor Emeritus
University of Alberta
Edmonton, Canada

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Chapter 1

INTRODUCTION

1.1. Purpose and Scope

There are two principal types of fasteners used in contemporary fabricated steel structures—bolts and welds. Both are widely used, and sometimes both fastening types are used in the same connection. For many connections, it is common to use welds in the shop portion of the fabrication process and to use bolts in the field. Welding requires a significant amount of equipment, uses skilled operators, and its inspection is a relatively sophisticated procedure. On the other hand, bolts are a manufactured item, they are installed using simple equipment, and installation and inspection can be done by persons with only a relatively small amount of training.

Engineers who have the responsibility for structural design must be conversant with the behavior of both bolts and welds and must know how to design connections using these fastening elements. Design and specification of welds and their inspection methods generally involves selecting standardized techniques and acceptance criteria or soliciting the expertise of a specialist. On the other hand, design and specification of a bolted joint requires the structural engineer to select the type of fasteners, understand how they are to be used, and to set out acceptable methods of installation and inspection. Relatively speaking, then, a structural engineer must know more about high-strength bolts than about welds.

The purpose of this Primer is to provide the structural engineer with the information necessary to select suitable high-strength bolts, specify the methods of their installation and inspection, and to design connections that use this type of fastener. Bolts can be either common bolts (sometimes called ordinary or machine bolts) or high-strength bolts. Although both types will be described, emphasis will be placed on high-strength bolts. Because many riveted structures are still in use and often their adequacy must be verified, a short description of rivets is also provided.

1.2. Historical Notes

Rivets were the principal fastener used in the early days of iron and steel structures [1, 2]. They were a satisfactory solution generally, but the clamping force produced as the heated rivet shrank against the gripped material was both variable and uncertain as to magnitude. Thus, use of rivets as the fastener in joints where slip was to be prevented was problematic. Rivets in connections loaded such that tension was produced in the fastener also posed certain problems. Perhaps most important,

however, the installation of rivets required more equipment and manpower than did the high-strength bolts that became available in a general way during the 1950's. This meant that it was more expensive to install a rivet than to install a high-strength bolt. Moreover, high-strength bolts offered certain advantages in strength and performance as compared with rivets.

Bolts made of mild steel had been used occasionally in the early days of steel and cast iron structures. The first suggestion that high-strength bolts could be used appears to have come from Batho and Bateman in a report made to the Steel Structures Committee of Scientific and Industrial Research of Great Britain [3] in 1934. Their finding was that bolts having a yield strength of at least 54 ksi could be pretensioned sufficiently to prevent slip of connected material. Other early research was done at the University of Illinois by Wilson and Thomas [4]. This study, directed toward the fatigue strength of riveted shear splices, showed that pretensioned high-strength bolted joints had a fatigue life at least as good as that of the riveted joints.

In 1947, the Research Council on Riveted and Bolted Structural Joints (RCRBSJ) was formed. This body was responsible for directing the research that ultimately led to the wide-spread acceptance of the high-strength bolt as the preferred mechanical fastener for fabricated structural steel. The Council continues today, and the organization is now known as the Research Council on Structural Connections (RCSC). The first specification for structural joints was issued by the RCRBSJ in 1951 [5].

At about the same time as this work was going on in North America, research studies and preparation of specifications started elsewhere, first in Germany and Britain, then in other European countries, in Japan, and elsewhere. Today, researchers in many countries of the world add to the knowledge base for structural joints made using high-strength bolts. Interested readers can find further information on these developments in References [6, 7, 8, 9].

1.3. Mechanical Fasteners

The mechanical fasteners most often used in structural steelwork are rivets and bolts. On occasion, other types of mechanical fasteners are used: generally, these are special forms of high-strength bolts. Rivets and bolts are used in drilled, punched, or flame-cut holes to fasten the parts to be connected. Pretension may be present in the fastener.

Whether pretension is required is a reflection of the type and purpose of the connection.

Rivets are made of bar stock and are supplied with a preformed head on one end. The manufacturing process can be done either by cold or hot forming. Usually, a button-type head is provided, although flattened or countersunk heads can be supplied when clearance is a problem. In order to install the rivet, it is heated to a high temperature, placed in the hole, and then the other head is formed using a pneumatic hammer. The preformed head must be held in place with a backing tool during this operation. In the usual application, the second head is also a button head.

As the heated rivet cools, it shrinks against the gripped material. The result of this tensile strain in the rivet is a corresponding tensile force, the *pretension*. Since the initial temperature of the rivet and the initial compactness of the gripped material are both variable items, the amount of pretension in the rivet is also variable. Destructive inspection after a rivet has been driven shows that usually the rivet does not completely fill the barrel of the hole.

The riveting operation requires a crew of three or four and a considerable amount of equipment—for heating the rivets and for forming the heads—and it is a noisy operation.

taken from the parent rivet or bolt.) Since the only reason for dealing with rivet strength today is in the evaluation of an existing structure, care must be taken to ascertain the grade of the rivets in the structure. Very old structures might have rivet steel of lesser strength than that reflected by ASTM A502. (This ASTM standard, A502, was discontinued in 1999.)

In fabricated structural steel applications, threaded elements are encountered as tension rods, anchor rods, and structural bolts. In light construction, tension members are often made of a single rod, threaded for a short distance at each end. A nut is used to effect the load transfer from the rod to the next component. The weakest part of the assembly is the threaded portion, and design is based on the so-called "stress area." The stress area is a defined area, somewhere between the cross-sectional area through the root of the threads and the cross-sectional area corresponding to the nominal bolt diameter. In the US Customary system of units, this stress area (A_{st}) is calculated as—

$$A_{st} = 0.7854 \left(D - \frac{0.9743}{n} \right)^2 \quad (1.1)$$

where D is the bolt diameter, inches, and n is the number of threads per inch.

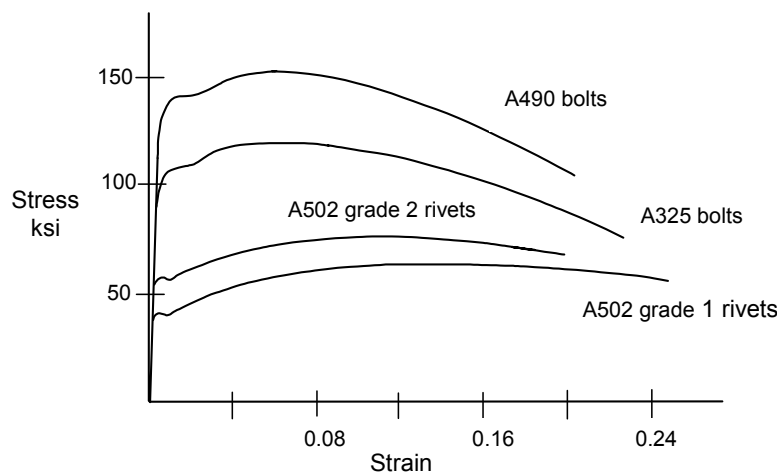


Fig. 1.1 Stress vs. Strain of Coupons taken from Bolts and Rivets

The ASTM specification for structural rivets, A502, provided three grades, 1, 2, and 3 [10]. Grade 1 is a carbon steel rivet for general structural purposes, Grade 2 is for use with higher strength steels, and Grade 3 is similar to Grade 2 but has atmospheric corrosion resistant properties. The only mechanical property specified for rivets is hardness. The stress vs. strain relationship for the two different strength levels is shown in Fig. 1.1, along with those of bolt grades to be discussed later. (The plot shown in Fig. 1.1 represents the response of a coupon

Threaded rods are not a factory-produced item, as is the case for bolts. As such, a threaded rod can be made of any available steel grade suitable for the job.

Anchor rods are used to connect a column or beam base plate to the foundation. Like tension members, they are manufactured for the specific task at hand. If hooked or headed, only one end is threaded since the main portion of the anchor rod will be bonded or secured mechanically into the concrete of the foundation. Alternatively, anchor rods can be threaded at both ends

and a nut used to develop the anchorage. Like threaded rods, anchor rods can be made of any grade of steel. One choice, however, is to use steel meeting ASTM A307, which is a steel used for bolts, studs, and other products of circular cross-section.¹ It is discussed below.

Structural bolts are loosely classified as either *common* or *high-strength*. Common bolts, also known as unfinished, ordinary, machine, or rough bolts, are covered by ASTM Specification A307 [11]. This specification includes the products known as *studs* and anchor bolts. (The term *stud* is intended to apply to a threaded product that will be used without a nut. It will be screwed directly into a component part.) Three grades are available in ASTM A307—A, B, and C. Grade B is designated for use in piping systems and will not be discussed here. Grade A has a minimum tensile strength of 60 ksi, and is intended for general applications. It is available in diameters from ¼ in. to 1½ in. Grade C is intended for structural anchorage purposes, i.e., non-headed anchor rods or studs. The diameter in this grade can be as large as 4 in. Structural bolts meeting ASTM A307 are sometimes used

Two strength grades of high-strength steel bolts are used in fabricated structural steel construction. These are ASTM A325 [12] and ASTM A490 [13]. Structural bolts manufactured according to ASTM A325 can be supplied as Type 1 or Type 3 and are available in diameters from ½ in. to 1½ in. (Type 2 bolts did exist at one time but have been withdrawn from the current specification.) Type 1 bolts use medium carbon, carbon boron, or medium carbon alloy steel. Type 3 bolts are made of weathering steel and their usual application is in structures that are also of weathering steel. A325 bolts are intended for use in structural connections that are assembled in accordance with the requirements of the Research Council on Structural Connections Specification (RCSC) [14]. This link between the product specification (ASTM A325) and the use specification (RCSC) is explicitly stated in the ASTM A325 Specification. The minimum tensile strength of A325 bolts is 120 ksi for diameters up to and including 1 in. and is 105 ksi for diameters beyond that value.²

The other high-strength fastener for use in fabricated

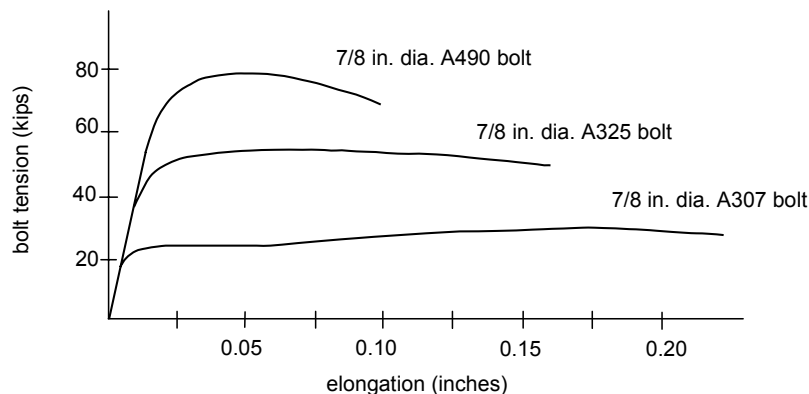


Fig. 1.2 Comparison of Bolt Types: Direct Tension

in structural applications when the forces to be transferred are not particularly large and when the loads are not vibratory, repetitive, or subject to load reversal. These bolts are relatively inexpensive and are easily installed. The response of an ASTM A307 bolt in direct tension is shown in Fig. 1.2, where it is compared with the two types of high-strength bolts used in structural practice. The main disadvantages of A307 bolts are its inferior strength properties as compared with high-strength bolts and the fact that the pretension (if needed for the type of joint) will be low and uncertain.

structural steel is that corresponding to ASTM A490. This fastener is a heat-treated steel bolt of 150 ksi minimum tensile strength (and maximum tensile strength of 170 ksi). As with the A325 bolt, it is intended that A490 bolts be used in structural joints that are made under the RCSC Specification. Two grades are available, Type 1 and Type 3. (As was the case with A325 bolts, Type 2 A490 bolts were available in the past, but they are no longer manufactured.) Type 1, available in diameters of ½ to 1½ in., is made of alloy steel. Type 3 bolts are atmospheric corrosion resistant bolts and are intended for

¹ ASTM F1554 –99 (Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength) is probably a more common choice today, however.

² The distinction of strength with respect to diameter arose from metallurgical considerations. These metallurgical restrictions no longer exist, but the distinction remains.

use in comparable atmospheric corrosion resistant steel components. They also can be supplied in diameters from $\frac{1}{2}$ to $1\frac{1}{2}$ in.

Both A325 and A490 bolts can be installed in such a way that a large pretension exists in the bolt. As will be seen, the presence of the pretension is a factor in some types of joints. This feature, and the concomitant requirements for installation and inspection, are discussed later.

There are a number of other structural fasteners covered by ASTM specifications, for example A193, A354, and A449. The first of these is a high-strength bolt for use at elevated temperatures. The A354 bolt has strength properties similar to that of the A490 bolt, especially in its Grade BD, but can be obtained in larger diameters (up to 4 in.) than the A490 bolt. The A449 bolt has strength properties similar to that of the A325 bolt, but it also can be furnished in larger diameters.³ It is often the specification used for high-strength anchor rods. Overall, however, A325, and A490 bolts are used in the great majority of cases for joining structural steel elements.

bolt and the nut has been satisfied, the main attribute of the nut is that it have a strength consistent with that of the bolt. Principally, this means that the nut must be strong enough and have a thread engagement deep enough so that it can develop the strength of the bolt before the nut threads strip.⁴ For the structural engineer, the selection of a suitable nut for the intended bolt can be made with the assistance of ASTM A563, Standard Specification for Carbon and Alloy Steel Nuts [15]. A table showing nuts suitable for various grades of fasteners is provided in that Specification. Washers are described in ASTM F436 [16]. The RCSC Specification [14] provides summary information for both nut and washer selection.

1.4. Types of Connections

It is convenient to classify mechanically fastened joints according to the types of forces that are produced in the fasteners. These conditions are tension, shear, and combined tension and shear. In each case, the force can be induced in several different ways.

Figure 1.3 shows a number of different types of joints that will produce shear in the fasteners. Part (a)

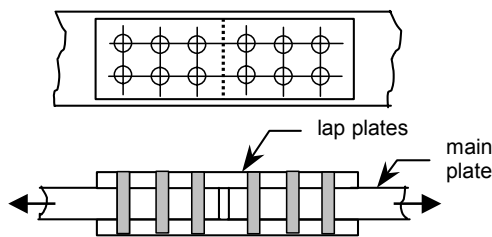


Fig.1.3(a) Lap Splice

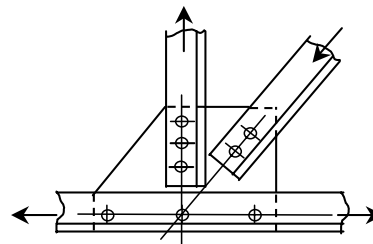


Fig. 1.3(b) Truss Joint

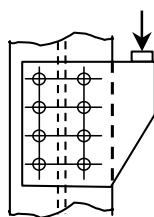


Fig. 1.3(c) Eccentric Joint

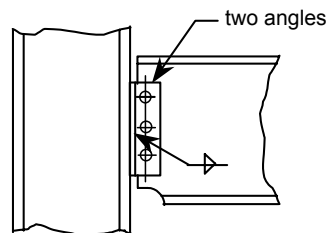


Fig. 1.3(d) Standard Beam Connection

Fig. 1.3 Bolted Joint Configurations

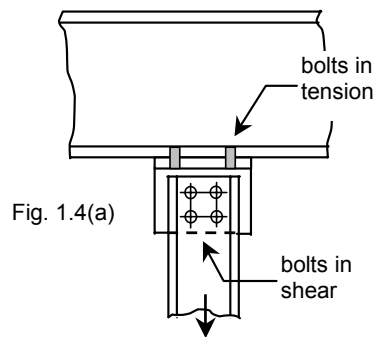
The nuts that accompany the bolts (and washers, if required) are an integral part of the bolt assembly. Assuming that the appropriate mechanical fit between the

³ Although the A354 and the A449 bolts have strength properties similar to the A325 and A490 bolts respectively, the thread length, quality assurance requirements, and packaging differ.

shows a *double lap* splice. The force in one main component, say the left-hand plate, must be transferred

⁴ Strictly speaking, this is not always required. If the only function of the bolt is to transfer shear, then the nut only needs to keep the bolt physically in place. However, for simplicity, the nut requirement described is applied to all bolting applications.

into the other main component, the right-hand plate. In the joint illustrated, this is done first by transferring the force in the left-hand main plate into the six bolts shown on the left-hand side of the splice. These bolts act in shear. Next, these six bolts transfer the load into the two splice plates. This transfer is accomplished by the bearing of the bolts against the sides of the holes in the plates.⁵ Now the load is in the splice plates, where it is resisted by a tensile force in the plate. Next, the load is transferred out of the splice plates by means of the six bolts shown on the right-hand side of the splice and into the main plate on the right-hand side. In any connection, understanding the flow of forces is essential for proper design of the



A joint in which tension will be induced in some of the fasteners is shown in Fig. 1.4 (a). This is the connection of a hanger to the lower flange of a beam. Figure 1.4 (b) shows a beam-to-column connection in which it is desired that both shear and moment be transmitted from the beam to the column. A satisfactory assumption for design is that all the shear force in the beam is in the web and all the beam moment is in the flanges. Accordingly, the fasteners in the pair of clip angles used to transfer the beam shear force are themselves loaded in shear. The beam moment (represented by a force couple located at the level of the flanges) is transmitted by the short tee sections that are

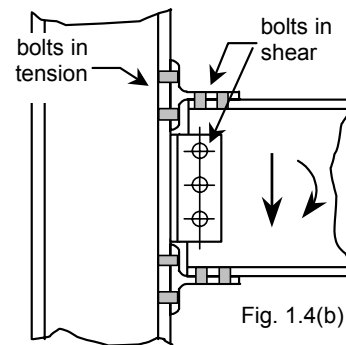


Fig. 1.4 Examples of Bolts in Tension

components, both the connected material and the fasteners. In the illustration, this visualization of the force flow (or, use of free-body diagrams!) allows the designer to see, among other things, that six fasteners must carry the total force at any given time, not twelve. More complicated arrangements of splice plates and use of different main components, say, rolled shapes instead of plates, are used in many practical applications. The problem for the designer remains the same, however—to understand the flow of forces through the joint.

Part (b) of Fig. 1.3 shows a panel point connection in a light truss. The forces pass out of (or into) the members and into (or out of) the gusset plate by means of the fasteners. These fasteners will be loaded in shear. Fig. 1.3 (c) shows a crane rail bracket. The fasteners again will be subjected to shear, this time by a force that is eccentric relative to the center of gravity of the fastener group. The standard beam connection (Fig. 1.3 (d)) provides another illustration of fasteners that will be loaded in shear. There are numerous other joint configurations that will result in shear in the fasteners.

fastened to the beam flanges. The connection of the tee section to the beam flanges puts those fasteners into shear, but the connection of the top beam flange tee to the

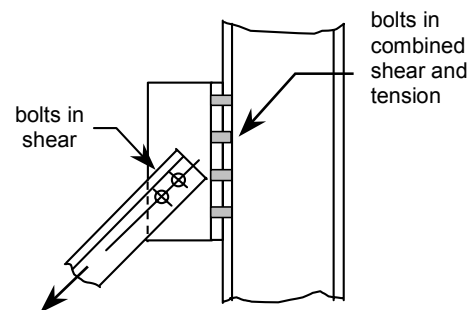


Fig. 1.5 Bolts in Combined Shear and Tension

column flange puts those fasteners into tension.

Finally, one illustration is presented where both shear and tension will be present in the fasteners. The inclined bracing member depicted in Fig. 1.5, shown as a pair of angles, is a two-force member. Considering the tension case, resolution of the inclined tensile force into its horizontal and vertical components identifies that the fasteners that connect the tee to the column must resist the applied forces in both shear and in tension.

⁵ Load transfer can also be by friction. This is discussed in Section 5.2.

The example of load transfer that was demonstrated by Fig. 1.3 (a) can be taken one step further, as is necessary to establish the forces and corresponding stresses in the connected material. Figure 1.6 shows the same joint that was illustrated in Fig. 1.3 (a), except that it has been simplified to one bolt and two plates. Part (a) shows the joint. A free-body diagram obtained when the bolt is cut at the interface between the two plates is shown in Fig. 1.6 (b). (A short extension of the bolt is shown for convenience.) For equilibrium, the force in the plate, P , must be balanced by a force in the bolt, as shown. This is the shear force in the bolt. If necessary, it can be expressed in terms of the average shear stress, τ , in the bolt by dividing by the cross-sectional area of the bolt. Going one step further, the bolt segment is isolated in Fig. 1.6 (c). This free-body diagram shows that, in order to equilibrate the shear force in the bolt, an equal and opposite force is required. The only place this can exist is on the right-hand face of the bolt. This force is delivered to the bolt as the top plate pulls up against the bolt, i.e., the bolt and the plate bear against one another. Finally, the short portion of the top plate to the right of the bolt, Fig. 1.6 (a), is shown in Fig. 1.6 (d). The force identified as a "bearing force" in Fig. 1.6 (c) must be present as an equal and opposite force on the plate in part (d) of the figure. This bearing force in the plate can be expressed as

a stress, as shown, if that is more convenient. Finally, since the plate segment must be in equilibrium, the pair of forces, $P/2$, must be present in the plate.

These are simple illustrations of how some connections act and the forces that can be present in the bolts and in the adjacent connected material. There are some other cases in which the load transfer mechanism needs to be further explained, for example, when pretensioned high-strength bolts are used. This will be done in later chapters.

1.5. Design Philosophy

For fabricated steel structures, two design philosophies coexist at the present time in the United States—limit states design and allowable stress design. In limit states design, commonly designated in the United States as Load and Resistance Factor Design, it is required that the "limit states" of performance be identified and compared with the effect of the loads applied to the structure. The limit states are considered to be *strength* and *serviceability*.

In the United States, the most commonly used specifications for the design of steel buildings are those of the American Institute of Steel Construction. In limit states design format, the AISC Load and Resistance Factor Design Specification (LRFD) is used [17]. If

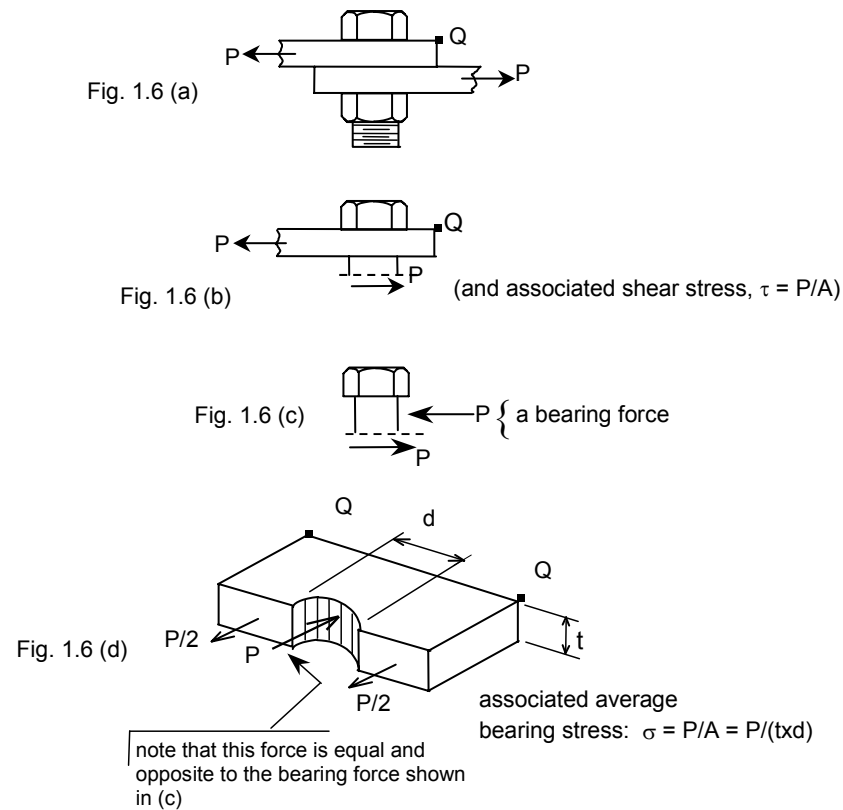


Fig.1.6 Bolt Forces and Bearing in Plate

allowable stress design (ASD) is used, then the AISC Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design, is available [18].

An example of a strength limit state is the compression buckling strength of an axially loaded column. The design strength is calculated according to the best available information, usually as expressed by a Specification statement of the nominal strength, which is then reduced by a *resistance factor*. The resistance factor, ϕ , is intended to account for uncertainties in the calculation of the strength, understrength of material, level of workmanship, and so on. In LRFD terminology, the product of the calculated ultimate capacity and the resistance factor is known as the *design strength*.

The loads that act on the structure are likewise subject to adjustment: few, if any, loads are deterministic. Therefore, the expected loads on a structure are also multiplied by a factor, the *load factor*. (More generally, load factors are applied in defined combinations to different components of the loading.) For most applications, the load factor is greater than unity. Finally, the factored resistance is compared with the effect of the factored loads that act on the structure.

In allowable stress design, the structure is analyzed for the loads expected to be acting (*nominal loads*) and then stresses calculated for each component. The calculated stress is then compared with some permissible stress. For example, a fraction of the yield stress of the material is used in the case of a tension member.

It is interesting to note that, for a long time, the design of mechanical fasteners has been carried out using a limit states approach. Even under allowable stress design, the permissible stress was simply a fraction of the tensile strength of the fastener, not a fraction of the yield strength. Indeed, it will be seen that there is no well-defined yield strength of a mechanical fastener: the only logical basis upon which to design a bolt is its ultimate strength.

The other limit state that must be examined is serviceability. For buildings, this means that such things as deflections, drift, floor vibrations, and connection slip may have to be examined. In contrast to the situation when the ultimate limit state is under scrutiny, these features are to be checked under the nominal loads, not the factored loads.

One of the most important features of bridge design (and other structures subjected to moving or repetitive loads) is fatigue. Some specifications put this topic in the category of ultimate limit state, whereas others call it a serviceability limit state. The principal design specification for fatigue in highway bridges in the United States, the rules of the American Association of State Highway and Transportation Officials (AASHTO), creates a separate limit state for fatigue [19]. This is done primarily because the so-called *fatigue truck*, used to

calculate stresses for the fatigue case, does not correspond to either the nominal load or to the usual factored load.

A full discussion of allowable stress design and limit states design can be found in most books on the design of fabricated steel structures. See, for example, Reference [20].

1.6. Approach Taken in this Primer

In this document, the usual approach is to describe the phenomenon under discussion in general terms, provide enough background information by way of research or, in some cases, theoretical findings, to enable a description of the phenomenon to be made, and then to provide a design rule. This is then linked to the corresponding rule in the principal specification, that of AISC [17], and only the LRFD rules will be discussed. In a few cases, the reference specification will be that of AASHTO [19].

Chapter 2

STATIC STRENGTH of RIVETS

2.1 Introduction

As discussed in Chapter 1, rivets have not been used in the fabrication and erection of structural steel for many years. However, there are still reasons why a structural engineer may need to know about the behavior of rivets. Because they can be present in existing buildings and bridges, it follows that one objective is the necessity of evaluating the strength of these elements when a structure is considered for such things as renovation or the determination of safety under increased load levels. In this Chapter, the static design strength of rivets is examined. The fatigue strength of a riveted connection, the other major area of interest, is more logically treated in Chapter 7, Fatigue of Bolted and Riveted Joints.

2.2 Rivets Subject to Tension

The tensile stress vs. strain response for ASTM A502 rivet steel (i.e., undriven rivets) was shown in Fig. 1.1. The tensile strength is about 60 ksi for Grade 1 and about 80 ksi for Grade 2 or 3. After the rivet has been driven, the tensile strength can be significantly increased [21]. At the same time, however, the ductility of the driven rivet is considerably less than that of the material from which it was driven. Most tension tests of driven rivets also show a decrease in strength with increasing rivet length (grip). The residual clamping force that is present in a driven rivet does not affect the ultimate strength of the rivet. In principle then, the design tensile strength of a rivet is simply the product of the minimum tensile strength of the rivet material multiplied by a resistance factor.

The AISC LRFD Specification provides rules for the design tension strength (ϕR_n) of ASTM A502 rivets. In accordance with Article J3.6 of the Specification, this is to be calculated as:

$$\phi R_n = \phi F_t A_b \quad (2.1)$$

where ϕR_n = design tension strength in tension, kips

ϕ = resistance factor, taken as 0.75

F_t = nominal tensile strength, taken as 45 ksi for ASTM A502 Grade 1 hot-driven rivets or as 60 ksi for Grade 2 hot-driven rivets

A_b = cross-sectional area of the rivet according to its nominal diameter, in.²

The product $F_t A_b$ obviously is the ultimate tensile strength (nominal strength) of the rivet shank. The value of the resistance factor ϕ recommended in the AISC Specification, 0.75, is relatively low, as it is for most connection elements. There is no research available that identifies the appropriate value of the resistance factor, ϕ , for rivets in tension. However, the case of high-strength bolts in tension can be used as a basis of comparison. In Reference [22], it was established that $\phi = 0.85$ is a satisfactory choice for high-strength bolts in tension. This is also the value recommended in the *Guide* [6]. Thus, selection of the value 0.75 is a conservative choice for rivets, but it results in values that are consistent with those used historically in allowable stress design.

It is not uncommon for mechanical fasteners acting in tension to be loaded to a level that is greater than that corresponding to the total applied load divided by the number of fasteners. This is the result of prying action produced by deformation of the connected parts. It is advisable to follow the same rules for prying action in the case of rivets in tension as are recommended for bolts in tension. Prying action is discussed in Chapter 6.

The most common need for the strength calculation of a rivet or rivet group in tension will be to determine the strength of an existing connection. The integrity of the rivet heads should be closely examined. If the head is not capable of resisting the force identified in Eq. 2.1, then the calculation simply is not valid. Rivet heads in such structures as railroad bridges can be severely corroded as a result of the environmental conditions to which they have been subjected over the years.

2.3 Rivets in Shear

Numerous tests have been carried out to determine the shear strength of rivets—see, for example, References [21, 23, 24]. These tests all show that the relationship between the shearing force that acts on a rivet and its corresponding shearing displacement has little, if any, region that can be described as linear. Thus, the best description of the strength of a rivet in shear is its ultimate shear capacity. In order to be able to compare rivets of different basic strengths, it is usual to relate the shear strength to the tensile strength of the steel from which the rivet is made. The results [21, 23] indicate that the value of this ratio (shear strength / tensile strength) is about 0.75, and that the ratio is not significantly affected by the grade of rivet or whether the shear test was done

on driven or undriven rivets. However, there is a relatively wide spread in the value of the ratio, from about 0.67 to 0.83, according to the work in References [21 and 23].

Typical shear load vs. shear deformation tests are shown in Fig. 2.1 [25]. These tests are for 7/8 in. dia. A502 Grade 1 rivets with two different grip lengths, 3 in. and 4½ in. Because of greater bending in the longer rivets (and un-symmetrical loading in the case of these tests), there was greater deformation in these rivets in the early stages of the test. However, the ultimate shear strength was unaffected by grip length. Since driving of the rivet increases its tensile strength, the corresponding shear strength is likewise expected to increase. Thus, the shear strength of Grade 1 A502 rivets can be expected to be at least $0.75 \times 60 \text{ ksi} = 45 \text{ ksi}$ and that for Grade 2 or Grade 3 rivets will be about $0.75 \times 80 \text{ ksi} = 60 \text{ ksi}$. (The multiplier 0.75 is not a resistance factor. It is the value of the ratio shear strength / tensile strength mentioned above.)

As was the case for rivets in tension, there have not been any studies that have explored the resistance factor for rivets in shear. The value recommended in the *Guide* [6] for bolts in shear is 0.80. In Reference [22], the resistance factor recommended is 0.83 for ASTM A325 bolts and 0.78 for ASTM A490 bolts.

In the AISC LRFD Specification, Section J3.6 requires that the design shear strength (ϕR_n) of a rivet is to be taken as—

$$\phi R_n = \phi F_v A_b \quad (2.2)$$

where ϕR_n = design shear strength, kips

ϕ = resistance factor, taken as 0.75

F_v = nominal shear strength, taken as 25 ksi for ASTM A502 Grade 1 rivets or as 33 ksi for Grade 2 and Grade 3 hot-driven rivets

A_b = cross-sectional area of the rivet, in.² The calculation of A_b should reflect the number of shear planes present.

Comparing the nominal shear strength values given in the Specification for the two rivet grades (25 ksi or 33 ksi) with the corresponding experimentally determined values (45 ksi or 60 ksi), it can be seen that the permissible values under the AISC LRFD rules are significantly conservative. When evaluating the shear strength of rivets in an existing structure, these conservative elements of the design rule can be kept in mind.

The effect of joint length upon shear strength applied to bolted shear splices (Section 5.1.) should also be

applied for long riveted connections. See also Section J3.6 of the AISC LRFD Specification.

2.4 Rivets in Combined Shear and Tension

It was explained in Section 1.4 (and with reference to Fig. 1.5) that fasteners must sometimes act under a combination of tension and shear. Tests done by Munse and Cox [23] form the basis for the design rule for this case. The tests were done on ASTM A141 rivets (which are comparable to A502 Grade 1 rivets), but the results are considered to be reasonable for application to all grades of rivets. The test variables included variation in grip length, rivet diameter, driving procedure, and manufacturing process [23]. The only one of these

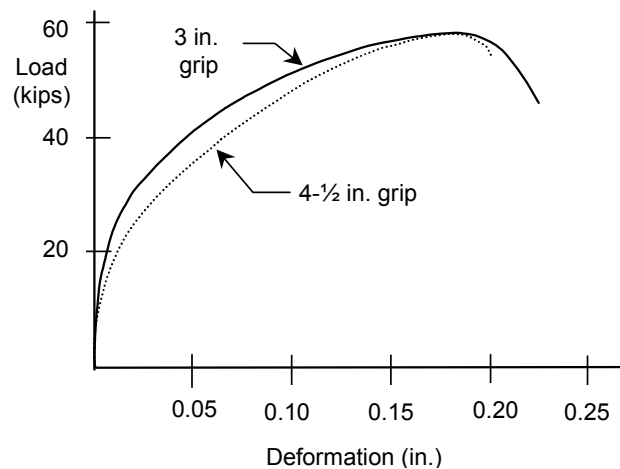


Fig. 2.1 Shear vs. Deformation Response of A502 Grade 1 Rivets

variables that had an influence on the behavior was grip length: long grip rivets tended to show a decrease in strength with length. This is consistent with tests done on rivets loaded in shear only. As the loading condition changed from tension-only to shear-only, deformation capacity decreased. This also is consistent with observations for rivets in tension and rivets in shear.

An elliptical interaction curve was fitted to the test results [23]. The mathematical description of the curve is:

$$\frac{x^2}{(0.75)^2} + y^2 = 1.0 \quad (2.3)$$

where x = ratio of calculated shear stress (τ) to tensile strength of the rivet (σ_u) (i.e., $x = \tau / \sigma_u$)

y = ratio of calculated tensile stress (σ) to tensile strength of the rivet (σ_u) (i.e., $y = \sigma / \sigma_u$)

An alternative representation of the test results was also suggested by the researchers [26]. This form, which

approximates the elliptical interaction equation with three straight lines, is the model used in the AISC LRFD Specification. In the AISC Specification (Table J3.5), A502 rivets of Grade 1 are permitted a nominal tension stress (ksi) under conditions of combined tension and shear of

$$F_t = 59 - 2.4 f_v \leq 45 \quad (2.4)$$

and for A502 Grade 2 and 3 rivets, the expression is:

$$F_t = 78 - 2.4 f_v \leq 60 \quad (2.5)$$

Equations 2.4 and 2.5 use the AISC LRFD notation for stresses. The resistance factor $\phi = 0.75$ must be applied to the result obtained by Equation 2.4 or 2.5, and then the design tension strength of the rivet (now reduced by the presence of shear) can be determined using Equation 2.1.

In applying these rules, it is apparent that the nominal tensile stress is limited to the nominal tensile strength of the rivet, which is 45 ksi for Grade 1 and 60 ksi for Grade 2 and 3. It should be remembered, as well, that there is also a limit on the calculated shear stress, f_v (computed under the factored loads). It must be equal to or less than the nominal shear strength multiplied by the resistance factor. The nominal shear stress is 25 ksi for A502 Grade 1 rivets and 33 ksi for Grade 2 and 3 rivets.

An advantage of the straight-line representation is that it identifies the range of shear stress values for which a reduction in tensile strength needs to be made. For example, a reduction in tensile strength for Grade 1 rivets is required when the calculated shear stress under the factored loads is between 5.8 ksi and the maximum permitted value of 18.8 ksi (i.e., $25 \text{ ksi} \times \phi = 0.75$). At the former, the nominal tensile stress is, of course, 45 ksi, and at the latter it has been reduced to 21.5 ksi.

The elliptical representation and the straight-line representation fit the test data about equally well when the forms presented in Reference [26] are applied. In the formulation used by AISC (Equations 2.4 and 2.5 above), the result will be conservative. It has already been pointed out in this Chapter that the rules given in the AISC LRFD Specification for the tension-only and the shear-only cases are themselves conservative.

Chapter 3

INSTALLATION OF BOLTS AND THEIR INSPECTION

3.1 Introduction

The installation of bolts, both high-strength bolts and common bolts, is presented in this chapter. This is accompanied by information on the inspection process that is necessary to ensure that the expectations of the installation have been met. Further information on the physical characteristics and mechanical properties of bolts is also included.

High-strength bolts can be installed in a way such that an initial pretension (or, preload) is present. The installation of ordinary bolts (ASTM A307) does not result in any significant pretension. For some applications, the presence of a pretension affects how the joint performs, and the inspection of installation of high-strength bolts should reflect whether or not bolt pretension is required. Whether bolts should be pretensioned is important in both the installation and inspection processes. Because of this importance, advice is given as to when pretensioned bolts should be required.

3.2 Installation of High-Strength Bolts

A bolt is a headed externally threaded fastener, and it is intended to be used with a nut. High-strength bolts were introduced in Section 1.3, and for structural applications two types of bolts are used—ASTM A325 and ASTM A490. Washers may or may not be required (see Chapter 8), depending on the application. Both the bolt head and the nut are hexagonal. The shank is only partially threaded, and the threaded length depends on the bolt diameter. Complete information on these details can be obtained in the relevant specifications [12, 13].

Not all structural bolts used in practice precisely meet the definition just given. Two other bolt configurations are in common use. These are bolts that meet or replicate the ASTM A325 or A490 requirements, but which have special features that relate to their installation. One is the "twist-off" bolt, which is covered by ASTM Specification F1852. It is described in Section 3.2.4. The other case is different from the conventional bolt–nut set only by the addition of a special washer that acts as an indicator of the pretension in the bolt. Its installation and other characteristics are described in Section 3.2.5.

Bolts meeting the requirements of ASTM Standards A325 and A490 were first described in Section 1.3. It was noted there that the ultimate tensile strength level for A325 bolts is 120 ksi or 105 ksi. The former applies to bolts of diameter up to and including 1 in. and the latter for bolts greater than 1 in. diameter. There is no maximum ultimate tensile strength specified for A325 bolts. The other kind of high-strength bolt used in

structural practice, ASTM A490, has a specified ultimate tensile strength of 150 ksi (and a maximum tensile strength of 170 ksi) for all diameters. In each case, the mechanical requirements of the specifications also make reference to a so-called proof load. This is the level up to which the bolt can be loaded and then unloaded without permanent residual deformation. In mild structural steels, this is termed the yield strength. However, in the case of the high-strength bolts there is no well-defined yield strength and all the design strength statements for high-strength bolts use the ultimate tensile strength as the basic parameter. Hence, the designer need not be concerned about the proof load.

It is required that the nuts for high-strength bolts used in normal structural applications are heavy hex nuts that conform to the requirements of ASTM Standard A563 [15]. (If the bolts are to be used in high-temperature or high-pressure applications, then another ASTM Standard is used for identifying the appropriate nuts.) When zinc-coated A325 bolts are to be used, then the nuts must also be galvanized and tapped oversize. In this case, requirements for lubrication of the nuts and a rotation capacity test for the bolt–nut assembly are specified in ASTM Standard A325. (This is discussed in Section 8.5.)

Bolts are installed by first placing them in their holes and then running the nut down on the bolt thread until it contacts the connected plies. This can be done either manually, by using a spud wrench,¹ or using a power tool, which is usually a pneumatic impact wrench. The expectation is that the connected parts will be in close contact, although in large joints involving thick material it cannot be expected that contact is necessarily attained completely throughout the joint. The installation process should start at the stiffest part of the joint and then progress systematically. Some repetition may be required. The condition of the bolts at this time is referred to as *snug-tight*, and it is attained by the full effort of the ironworker using a spud wrench or by running the nut down until the air-operated wrench first starts to impact. The bolt will undergo some elongation during this process, and there will be a resultant tensile force developed in the bolt. In order to maintain equilibrium, an equal and opposite compressive force is developed in the connected material. The amount of the bolt tension at the

¹ A spud wrench is the tool used by an ironworker to install a bolt. It has an open hexagonal head and a tapered handle that allows the worker to insert it into holes for purposes of initial alignment of parts.

snug-tightened condition is generally large enough to hold the parts compactly together and to prevent the nut from backing off under static loads. As an example, in laboratory tests snug-tight bolt pretensions range from about 5 to 10 kips for 7/8 in. diameter A325 bolts. In practice, the range is probably even larger.

For many applications, the condition of snug-tight is all that is required. Because use of snug-tightened bolts is an economical solution, they should be specified whenever possible. If the function of the joint requires that the bolts be pretensioned, then bolt installation must be carried out in one of the ways described following. Whether or not the bolts need to be pretensioned is described in Section 3.3.

3.2.1 Turn-of-Nut Installation

If the nut continues to be turned past the location described as snug-tight, then the bolt tension will continue to increase. In this section, the installation process described is that in which a prescribed amount of turn of the nut is applied. This is an elongation method of controlling bolt tension. Alternatively, a prescribed and calibrated amount of torque can be applied, as described in Section 3.2.2.

As the nut is turned, conditions throughout the bolt are initially elastic, but local yielding in the threaded portion soon begins. Most of the yielding takes place in the region between the underside of the nut and the thread run-out. As the bolt continues to elongate under the action of turning the nut, the bolt load (pretension) vs. elongation response flattens out, that is, the bolt pretension force levels off.

Figure 3.1 shows the bolt pretension obtained by turning the nut on a certain lot of A325 bolts [27]. These were 7/8 in. diameter bolts that used a grip length of 4–1/8 in. (In this laboratory study, the snug-tight condition was uniquely established for all bolts in the lot by setting

the snug-tight load at 8 kips.) It can be seen that the average response is linear up to a load level slightly exceeding the specified proof load, then yielding starts to occur in the threads and the response curve flattens out. Also shown in the figure is the range of elongations that were observed at 1/2 turn past snug, which is the RCSC Specification requirement [14] for bolts of the length used in this study. The specified minimum bolt pretension is 39 kips for A325 bolts of this diameter, and it can be observed that the measured pretension at 1/2 turn is well above this value. (The minimum bolt pretension required is 70% of the minimum specified ultimate tensile strength of the bolt [14].)

Figure 3.1 also shows that the specified minimum tensile strength of the bolt (i.e., direct tension) is well above the maximum bolt tension reached in the test (i.e., torqued tension). This reflects the fact that during installation the bolts are acting under a condition of combined stresses, tension and torsion.

The results of the bolt installation shown in Fig. 3.1, which is typical of turn-of-nut installations, raise the following questions:

- How do such bolts act in joints, rather than individually as depicted in Fig. 3.1?
- If the bolts subsequently must act in tension, can they attain the specified minimum tensile strength?
- Does the yielding that takes place in the bolt threads (mainly) affect the subsequent strength of the bolt in shear, tension, or combined tension and shear?
- What is the margin against twist-off of the bolts in the event that more than 1/2 turn is applied inadvertently?
- How sensitive is the final condition (e.g., bolt pretension at 1/2 turn) to the level of the initial pretension at snug-tight?

The first three items in the list apply to bolts installed by any procedure: the others are specific to turn-of-nut installations.

Several of these questions can be addressed by looking at the behavior of bolts that were taken from the same lot as used to obtain Fig. 3.1 when they were installed in a large joint [6]. Figure 3.2 shows the bolt elongations and subsequent installed pretensions for 28 of these bolts installed to 1/2 turn of nut beyond snug-tight.

The individual bolt pretensions can be estimated by projecting upward from the bolt elongation histogram at the bottom of the figure to the plot of bolt pretensions obtained by the turn-of-nut installation. Even though there is a large variation in bolt elongation for these 28 bolts (from about 0.03 in. to nearly 0.05 in.), the resultant pretension hardly varies at all. This is because the bolts have entered the inelastic range of their response. Thus, the turn-of-nut installation results in a reliable level of

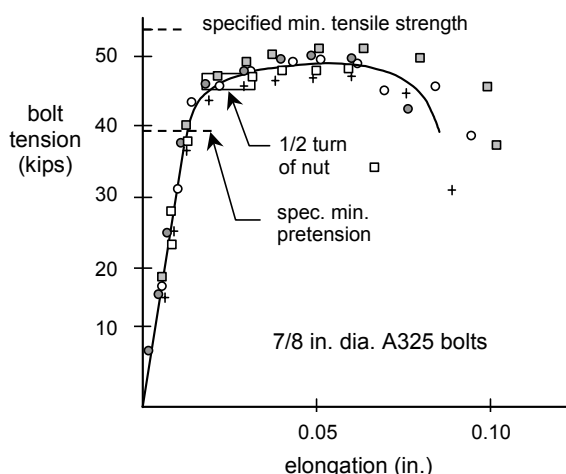


Fig. 3.1 Load vs. Elongation Relationship, Torqued Tension