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Letter from the Editor

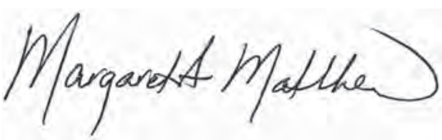
Dear Readers,

Hello and Happy New Year! As we head into the new year, I would like to take this opportunity to recognize all of the hard work of our reviewers, last year and every year. Their contributions are essential to the success of the *Journal* as we continue to strive to bring you the very best articles and information in the steel construction industry. A list of our 2019 reviewers is posted on the AISC website at www.aisc.org/ej.

Is there a steel design topic you would like to see in *EJ*? We are always looking for ideas for papers. Authors interested in submitting papers should visit our website at www.aisc.org/ej for author guidelines and submittal information.

Best wishes for a healthy and happy 2020!

Sincerely,



Margaret A. Matthew, P.E.
Editor

Dimensional Tolerances and Length Determination of High-Strength Bolts

JAMES A. SWANSON, GIAN ANDREA RASSATI, and CHAD M. LARSON

ABSTRACT

Structural engineers and detailers are often removed from the process of manufacturing bolts and, thus, the tolerances and variances that go along with common manufacturing processes. While this does not represent a problem in most cases, being familiar with the manufacturing processes and tolerances associated with high-strength bolts can help prevent some problems from occurring before the design process even begins, particularly when shorter bolt lengths are needed. This lack of familiarity, in some circumstances, might lead to mistaken assumptions regarding the location of the shear plane relative to the threads of the bolt, which may lead to incorrect designs. While an engineer might presume that bolt strength would not control in such short grips, this paper will discuss the cases in which this can become an issue. This paper summarizes the major variances between nominal and actual dimensions, evaluates some of the consequences that those variances can have on design, presents solutions to those issues, and culminates with a proposed design procedure for proper length determination of high-strength bolts with several illustrative examples.

Keywords: Structural bolt, high-strength bolt, fastener, A325, A490, F1852, F2280, F3125, F3148, threads excluded, ASME B18.2.6.

INTRODUCTION

Bolts, like any other manufactured product, are specified by nominal values but have acceptable variances or tolerances from those baseline values that are needed during manufacturing. Because the governing standards for high-strength bolts are maintained by ASTM International, which references several American Society of Mechanical Engineers (ASME) standards, the details of bolt manufacturing are often steps removed from the day-to-day attention of most structural engineers and connection detailers. The objective of this paper is to provide a summary of the dimensional tolerances that are associated with the manufacture of bolts, evaluate some of the consequences that those tolerances can have on structural steel design, and present solutions to those issues.

Carter, in a 1996 *Engineering Journal* paper, presented an analysis of tolerances associated with high-strength bolting with an emphasis on developing expedient methods of determining bolt length and when the threads of a bolt can be excluded from the shear plane of a bolted joint, thus increasing the design strength of the bolt. The paper focused on the length tolerance of the bolts and the thickness tolerances of the washers and nuts that are used to complete the bolting assembly, and culminated with a series of design tables for commonly used bolt sizes that are useful tools for structural engineers and detailers. Carter's paper identified that the tolerance on bolt length is more critical than the washer and nut thickness tolerances and that while additional tolerances on the shank length and thread transition region of bolts were considered, they were thought at the time to be small enough so as to be inconsequential.

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BOLT, NUT, AND WASHER GEOMETRY AND MANUFACTURING TOLERANCES

High-strength structural bolts are required by ASTM F3125/3125M-15a (2015) and F3148-17a (2017) to conform to the ASME Standard B18.2.6-19 (2019). Two types of dimensions, shown in Figures 1 and 2, are used in the latter standard: control dimensions and reference dimensions. Control dimensions are those dimensions that are used during manufacture to ensure quality control and conformance with standards. Reference dimensions, on the other hand, are dimensions that are typically provided for information only or for the purpose of calculating control dimensions, and not for quality control or demonstration of conformance to standards.

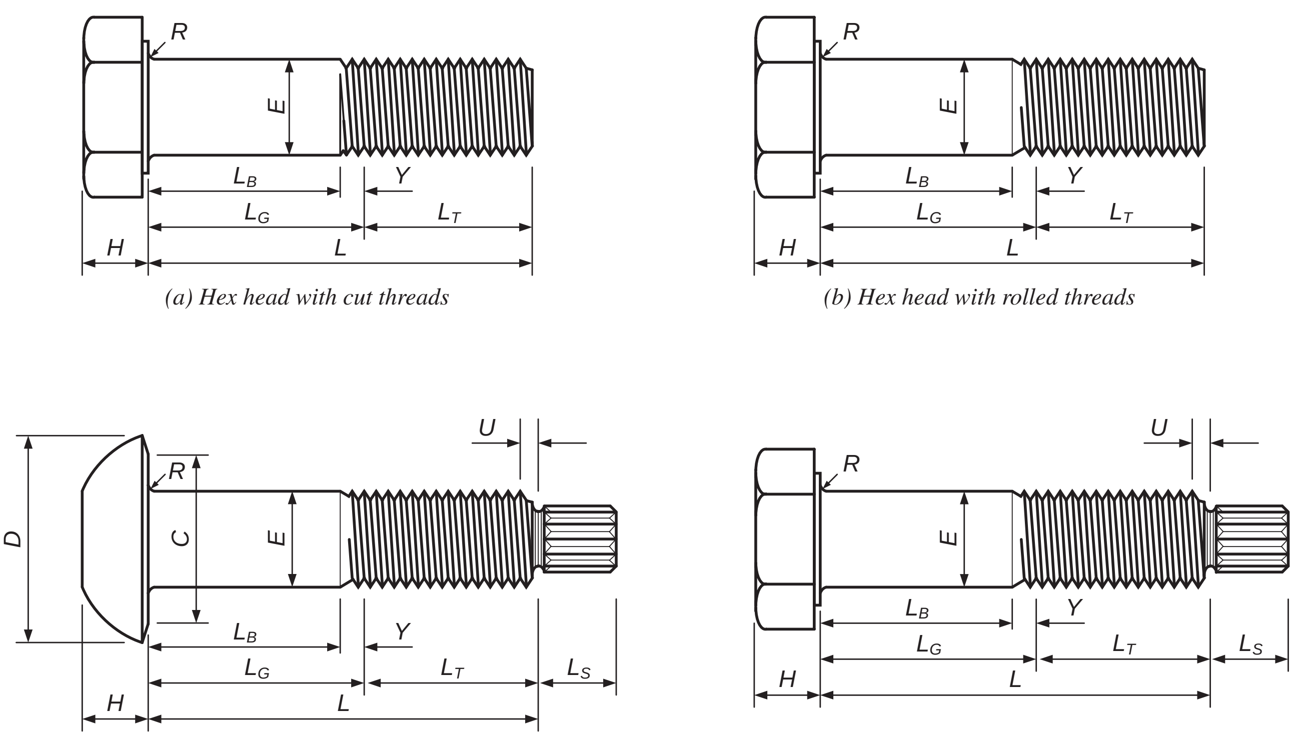
Bolt Diameter

ASTM F3125 high-strength structural fasteners are generally available in diameters ranging from ½ in. to 1½ in. in ⅛-in.-diameter increments, with the exception of 1⅜-in.

Table 1. Diameters and Tolerances for Standard Size High-Strength Bolts (ASME B18.2.6-19)				
Nominal Bolt Diameter, E or d_b (in.)	**Maximum Body Diameter (in.)**	**Minimum Body Diameter (in.)**	**Permitted Swell or Fin (in.)**	**Max Diameter plus Max Swell (in.)**
½	0.515	0.482	0.030	0.545
⅝	0.642	0.605	0.050	0.692
¾	0.768	0.729	0.050	0.818
⅞	0.895	0.852	0.060	0.955
1	1.022	0.976	0.060	1.082
1⅛	1.149	1.098	0.060	1.209
1¼	1.277	1.223	0.060	1.337
1⅜	1.404	1.345	0.090	1.494
1½	1.531	1.470	0.090	1.621

bolts, which are not widely produced. ASTM F3148 high-strength bolts are currently available in diameters ranging from ⅝ in. to 1⅛ in. in ⅛-in. increments. The diameter of a bolt, specified as E in Figures 1 and 2 and in ASME B18.2.6, but more commonly referred to as d_b in structural engineering contexts, is a control dimension and has an over/under tolerance ranging from approximately 0.015 in. for small bolts to approximately 0.030 in. for large bolts. Actual values of diameters permitted are shown in Table 1. In addition to the diameter tolerance, an allowance is included for a swell or fin under the head of the fastener that may occur during manufacturing.[1] As can be deduced from the table, a bolt produced at the maximum body diameter with the maximum permitted swell may have a final measured diameter

1 Swells and fins result primarily during hot forging of fastener heads. Hot forging is used in the manufacture of relatively few common bolt sizes, however—mostly for larger diameter or longer bolts. Bolts up to approximately 6 in. long are typically cold formed.



(a) Hex head with cut threads

(b) Hex head with rolled threads

(c) Twist-off spline drive with button head

(d) Twist-off spline drive with hex head

Fig. 1. Geometry of ASTM F3125 bolts (ASME B18.2.6-19).

Table 2. Tolerances on Bolt Length		
Nominal Bolt Diameter, *E* or d_b (in.)	Nominal Bolt Length, $L \leq 6$ (in.)	Nominal Bolt Length, $L > 6$ (in.)
½	+0.00, –0.12	+0.00, –0.19
⅝	+0.00, –0.12	+0.00, –0.25
¾ to 1	+0.00, –0.19	+0.00, –0.25
1⅛ to 1½	+0.00, –0.25	+0.00, –0.25

that is more than 1⁄16 in. larger than its nominal diameter and, in the case of a 1½-in. bolt, is nearly ⅛ in. larger than the nominal diameter. As Shaw points out (2015), this, in part, led to the increase in the size of standard holes for 1-in.-diameter and larger bolts in the 2016 AISC *Specification* (AISC, 2016) and in the 8th edition of the AASHTO *LRFD Specification* (AASHTO, 2017) and has been approved for inclusion in the next edition of the RCSC *Specification*, expected to be published in 2020.

Bolt Length, Shank Length, and Thread Length

The length of the bolt, specified as *L* in Figure 1, is a control dimension and for bolts without splines (ASTM F3125 Grades A325 and A490) is measured parallel to the axis of the bolt from the underside of the head of the bolt—the bearing surface of the head—to the end of the bolt. For bolts with twist-off splines (ASTM F3125 Grades F1852 and F2280), the length, *L*, is measured from the bolt bearing surface to the center point of the groove between the threaded portion of the bolt and the spline drive (ASME B18.2.6-19). For F3148 spline drive bolts (ASTM F3148-17a), the length, *L*, is measured from the bolt bearing surface to the first indication of thread near the spline, as shown in Figure 2. The tolerances on the overall length of a bolt, shown in Table 2, range from +0 in. to approximately –⅛ in. or –¼ in. depending on the diameter and nominal length of the bolt. A looser tolerance for bolts longer than 6 in. reflects once-common manufacturing methods but is likely no longer required due to improved production practices, while methods vary from one manufacturer to another and even one machine to another.

ASME B18.2.6 provides specifications for bolts ranging in length from 1½ in. to 10 in. in ¼-in.-long increments. However, the AISC *Steel Construction Manual* (2017) states that high-strength bolts are generally furnished in length increments of ¼-in. only up to a length of 5 in. and then in length increments of ½-in. for longer bolts. The RCSC *Specification* (2015), however, notes that the transition from ¼-in. to ½-in. increments occurs at a length of 6 in. In practice, availability of bolts of specific length is a function of several variables, including manufacturing methods, tooling, and market factors. A good rule of thumb is that bolt lengths up to four diameters are generally stocked for all diameters up to 1½ in. and bolt lengths up to eight diameters are commonly stocked for the more commonly used diameters of ¾ in. to 1⅛ in. Longer bolts or bolts in ¼-in. length increments are available given sufficient lead time and appropriate coordination with a supplier.

The thread length, L_T, is the distance from the last complete thread near the shank to the extreme end of the bolt for Grades A325 and A490 bolts, to the center point of the groove for Grades F1852 and F2280 bolts, or to the first indication of thread for F3148 bolts, as is shown in Figures 1 and 2. The thread length of structural bolts is generally shorter than that of similar nonstructural bolts so as to more easily allow the threads of the bolt to be excluded from the shear plane, thus increasing the strength of the bolt when it is subjected to shear. Although many resources, including the AISC *Manual* and the RCSC *Specification*, include

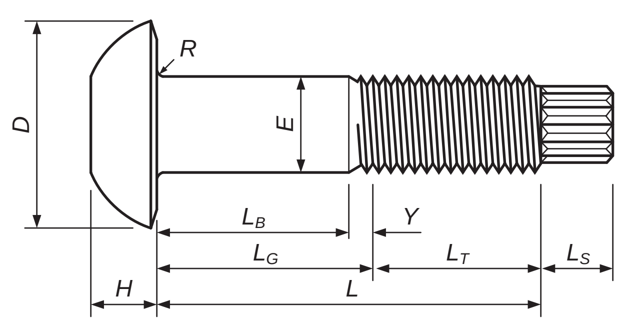


Fig. 2. Geometry of ASTM F3148 bolts.

Table 3. Bolt Lengths, Thread Lengths, and Shortest Lengths Routinely Produced			
Nominal Bolt Diameter, E or d_b (in.)	Thread Length, L_T (in.)	Transition Length, Y (in.)	Shortest Length Produced (in.)*
½	1.00	0.19	1¼
⅝	1.25	0.22	1½
¾	1.38	0.25	1¾
⅞	1.50	0.28	2
1	1.75	0.31	2¼
1⅛	2.00	0.34	2¾
1¼	2.00	0.38	2¾
1⅜	2.25	0.44	Special order
1½	2.25	0.44	See text

* Bolts are occasionally produced in shorter lengths for special orders.

tables detailing the length of threads, L_T, also shown here in Table 3, the thread length is a reference dimension in ASME B18.2.6, intended for calculation purposes only, and may actually vary from published nominal values.

Instead of controlling the thread length of a bolt, the geometry of the bolt is controlled by the overall length of the bolt, L, the grip gaging length, L_G, and the body length, L_B. The grip gaging length, L_G, is a control dimension measured from the bearing surface of the head to the face of a thread ring gage (Figure 3) that is threaded onto the bolt by hand until it stops at the thread runout. The bolt body length, L_B, which is also a control dimension, is basically the length of the shank or body of the bolt. The body length is more precisely defined as the distance measured from the bearing surface of the head to the last scratch of thread for bolts with cut threads, as shown in Figure 1(a), or to the top of the extrusion angle for bolts with rolled threads, as shown in Figures 1(b)–1(d). The transition length, Y, is a reference dimension that represents the length of the transition region between the threads and body.

The grip gage length and body length are used as control dimensions by specifying a maximum grip gage length,

Fig. 3. A thread ring gage.

$L_{G,max}$, and a minimum body length, $L_{B,min}$, which are calculated as shown in Equations 1 and 2 using the nominal overall length and the reference dimensions L_T and Y. Values of $L_{G,max}$ and $L_{B,min}$ are tabulated in ASME B18.2.6 for each diameter and length of fastener and are also shown herein (in part) as Table A1. Because L_T and Y are reference dimensions, no tolerances are provided for these dimensions and actual measured dimensions on finished product may vary from these published values.

$$L_{G,max} = L_{nom} - L_T \quad (1)$$

$$L_{B,min} = L_{G,max} - Y \quad (2)$$

ASME B18.2.6 states that when the minimum body length, $L_{B,min}$, is short enough, that the bolt shall be threaded full length. Specifically, it says that when $L_{B,min} \le 2.5p$ for $d_b \le 1$ in. or when $L_{B,min} \le 3.5p$ for $d_b > 1$ in., the bolt shall be threaded full length, where p is the pitch of the threads on the bolt. However, ASME B18.2.6 also says that bolts that are threaded full length are permitted to have an unthreaded length under the head that is not longer than $2.5p$ for bolts 1 in. in diameter or smaller and $3.5p$ for bolts larger than 1 in. in diameter. One implication of this is that there are some lengths of bolts an engineer may expect to be fully threaded that may, in fact, have a short unthreaded length under the head. Another implication is that there are some lengths of bolts an engineer may expect to definitely have a shank but may, in fact, be fully (or mostly) threaded. The former case is generally of little concern in most cases, but the latter case may lead to a serious design issue.

With two exceptions, bolts that are fully threaded do not carry a special designation identifying them as such. The first exception is for bolts manufactured with nonstandard dimensions, which are designated with an S—A490S, for example. The second exception is that Grade A325 bolts up to a length of four times their diameter may be manufactured as fully threaded and are designated with a T—A325T, for example. Bolts with the "T" designation are for users who may want a longer bolt that is fully threaded so they do not need to order or inventory multiple shorter sizes. These bolts are common in markets such as the metal building industry and are used in multiple grip ranges and in the threads included condition. The fully threaded T bolts are also permitted to have an unthreaded length under the head that is not longer than 2.5p for bolts 1 in. in diameter or smaller and 3.5p for bolts larger than 1 in. in diameter.

Illustrative Cases

The next several cases consider ⅞-in.-diameter bolts of varying length. For all ⅞-in.-diameter structural bolts, there are 9 threads per in. (TPI); thus the pitch of these bolts is p = ⅑ in. and 2.5p = (2.5)/(9 in.) = 0.28 in. Further, all ⅞-in.-diameter structural bolts have a threaded length, L_T, a reference dimension, equal to 1½ in. Finally, the transition length, Y, also a reference dimension, has a value of 0.28 in. for all ⅞-in.-diameter structural bolts.

Case 1

First, consider a ⅞ in.-9×1½ in. bolt. The maximum grip gage length, $L_{G,max}$, is calculated using Equation 1 and the minimum body length, $L_{B,min}$, is calculated using Equation 2:

$$\begin{aligned} L_{G,max} &= L_{nom} - L_T \\ &= 1\tfrac{1}{2}\text{ in.} - 1\tfrac{1}{2}\text{ in.} \\ &= 0.00\text{ in.} \end{aligned} \tag{1}$$

$$\begin{aligned} L_{B,min} &= L_{G,max} - Y \\ &= 0.00\text{ in.} - 0.28\text{ in.} \\ &= -0.28\text{ in.} \end{aligned} \tag{2}$$

Therefore, use $L_{B,min}$ = 0.00 in.

Because $L_{B,min}$ is less than 2.5p, however, $L_{G,max}$ is taken as 2.5p = 0.28 in. Because the value of $L_{B,min}$ is less than 2.5p, the bolt is considered to be fully threaded. Because the nominal thread length for a ⅞-in.-diameter bolt is 1½ in., which is equal to the nominal overall bolt length, the bolt would simply be considered as fully threaded.

Based on the ASME B18.2.6 standard, the ⅞ in.-9×1½ in. diameter bolt may be produced without an unthreaded body, or the bolt may have a small unthreaded and transition length that could be as long as 2.5p. The diameter of this unthreaded and transition length may be as large as the full nominal diameter (over/under tolerances) but may be as small as the pitch diameter of the bolt. These two alternatives, both of which are in compliance with ASME B18.2.6, are shown as Figures 4(a) and 4(b), respectively. (The tolerance for overall bolt length is not illustrated in Figure 4.)

Case 2

Next, consider a ⅞ in.-9×1¾ in. bolt with

$$\begin{aligned} L_{G,max} &= 1\tfrac{3}{4}\text{ in.} - 1\tfrac{1}{2}\text{ in.} \\ &= 0.25\text{ in.} \end{aligned}$$

$$\begin{aligned} L_{B,min} &= 0.25\text{ in.} - 0.28\text{ in.} \\ &= -0.03\text{ in.} \end{aligned}$$

Therefore use $L_{B,min}$ = 0.00 in.

Because $L_{B,min}$ is less than 2.5p, $L_{G,max}$ is taken as 2.5p = 0.28 in. Because the value of $L_{B,min}$ is less than 2.5p, this bolt, despite having a nominal thread length of L_T = 1½ in. and a nominal shank or body length of L_B = 1¾ in. – 1½ in. = ¼ in., would be considered fully threaded according to ASME B18.2.6 and, like Case 1, may indeed be produced as fully threaded, or it may have a small unthreaded and transition length that could be as long as 0.28 in. These two alternatives, both of which are in compliance with ASME B18.2.6, are shown as Figures 4(c) and 4(d), respectively.

Case 3

Next, consider a ⅞ in.-9×2 in. bolt with

$$\begin{aligned} L_{G,max} &= 2\text{ in.} - 1\tfrac{1}{2}\text{ in.} \\ &= 0.50\text{ in.} \end{aligned}$$

$$\begin{aligned} L_{B,min} &= 0.50\text{ in.} - 0.28\text{ in.} \\ &= 0.22\text{ in.} \end{aligned}$$

Because $L_{B,min}$ is less than 2.5p, $L_{G,max}$ is taken as 2.5p = 0.28 in. Because this value of $L_{B,min}$ is less than 2.5p, this bolt, despite having a nominal thread length of L_T = 1½ in. and a nominal shank or body length of L_B = 2 in. – 1½ in. = ½ in., would be considered fully threaded according to ASME B18.2.6. Like Cases 1 and 2, the ⅞-in.-9×2-in. bolt may be produced as fully threaded, or it may have a small unthreaded and transition length that could be as long as 0.28 in. These two alternatives, both of which are in compliance with ASME B18.2.6, are shown in Figures 4(e) and 4(f), respectively.

Three different variants of ⅞-in.-9×2-in. bolts made by different manufacturers are shown in Figure 5. The bolt on the left has a short unthreaded body with a diameter that is less than the nominal diameter of the bolt, the bolt in the middle is basically all transition up to the body diameter, and the bolt on the right has a short body with a diameter equal to the nominal diameter of the bolt and a short transition. All three bolts are in compliance with ASME B18.2.6.

Case 4

Now, consider a ⅞-in.-9×2¼ in. bolt with

$$L_{G,max} = 2\tfrac{1}{4} \text{ in.} - 1\tfrac{1}{2} \text{ in.}$$
$$= 0.75 \text{ in.}$$

$$L_{B,min} = 0.75 \text{ in.} - 0.28 \text{ in.}$$
$$= 0.47 \text{ in.}$$

Because this value of $L_{B,min}$ is greater than 2.5p, this bolt will have a shank that is at least 0.47 in. long with a diameter equal to the nominal body diameter (±tolerances) and a transition region that may be as long as 0.28 in. This bolt, which is in compliance with ASME B18.2.6, is shown as Figure 4(g).

It should be noted that the ⅞-in.-9×2-in. is the shortest ⅞-in.-diameter high-strength bolt that is routinely produced. Because the ⅞-in. × 2-in. is considered to be fully threaded, there is little demand for a ⅞-in.-diameter bolt shorter than 2 in., although shorter ones are occasionally manufactured upon request. The ⅞-in. × 2¼-in. is the shortest ⅞-in.-diameter high-strength bolt that is guaranteed to have a shank.

The case of the ⅞-in.-9×2 in. represents a potentially serious design issue. A structural engineer or connection detailer would likely review tables in the RCSC *Specification* (RCSC, 2015) or AISC *Manual* (AISC, 2017), see that the thread length is listed as $L_T = 1\frac{1}{2}$ in. for a bolt that is 2 in. long, and, expecting the bolt to have a ½-in.-long shank, may design the bolt as if the threads are excluded from the shear plane, as is shown in Figure 6(a). The ⅞-in.-9×2 in. bolt as supplied by the manufacturer in full compliance with ASME B18.2.6 may however have a shank much shorter than ½ in. or have no shank at all, resulting in a significant deviation from the engineer's or detailer's expectations, as is shown in Figure 6(b).

The shortest length bolts that are routinely produced for each diameter are shown in Table 3. With the exceptions of 1⅜-in.- and 1½-in.-diameter bolts, bolts with the lengths shown in the table may or may not have a shank or unthreaded body depending on the tooling and preferences of the manufacturer. Bolts with lengths greater than those shown in the table will have a shank.

A direct application of the formulas for L_b and L_g in ASME B18.2.6 to 1⅜-in.- and 1½-in.-diameter bolts would show that 1⅜-in.- and 1½-in.-diameter bolts with lengths

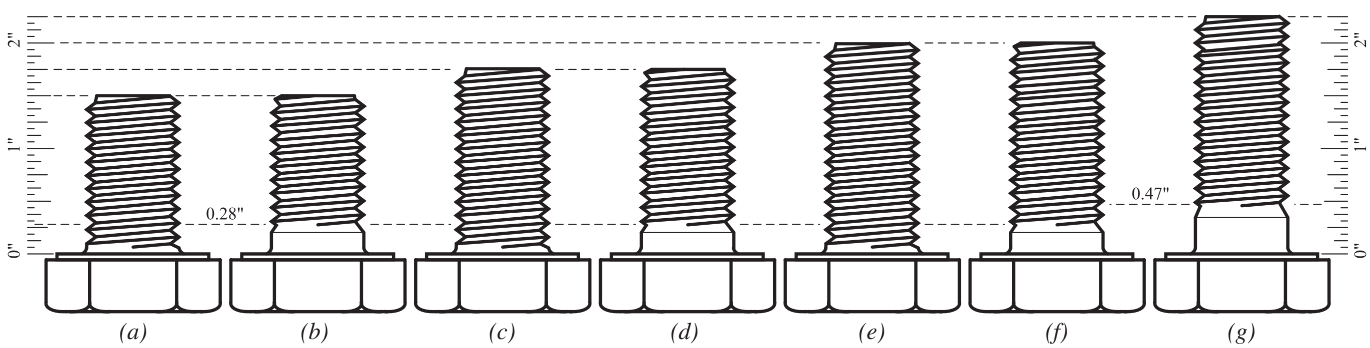


Fig. 4. Conforming variants of ⅞-in.-9×1½-in., ⅞-in.-9×1¾-in., ⅞-in.-9×2-in., and ⅞-in.-9×2¼-in. bolts.

Fig. 5. Three ⅞-in.-9 × 2-in. bolts manufactured by three different manufacturers.

3¼ in. and shorter would be fully threaded. In Table 2 of ASME B18.2.6-10 (2010), however, 1⅜-in.- and 1½-in.-diameter bolts with lengths 3 in. and shorter were both shown as fully threaded. Going back to the 2006 edition, in Table 2 of ASME B18.2.6-06 (2006), 1⅜-in.-diameter bolts with lengths 3 in. and shorter were shown as fully threaded but 1½-in.-diameter bolts with lengths 3¾ in. and shorter were shown as fully threaded. This discrepancy has been resolved in ASME B18.2.6-19 (2019), and the corresponding table has been updated to be consistent with the equations. As was stated earlier, 1⅜-in.-diameter bolts are not routinely produced and are available by special order only; they are included in this discussion for the sake of completeness only.

Case 5

Finally consider a ⅞-in.-9×4-in. bolt, where

$$L_{G,max} = 4 \text{ in.} - 1\tfrac{1}{2} \text{ in.}$$
$$= 2.50 \text{ in.}$$

$$L_{B,min} = 2.50 \text{ in.} - 0.28 \text{ in.}$$
$$= 2.22 \text{ in.}$$

Note that the transition length can vary from manufacturer to manufacturer; thus, depending on the actual values of Y and L (including tolerances for L), the thread length, L_T, for this bolt could range anywhere from L_T = (4.00 in. − 0.19 in.) − 2.50 in. = 1.31 in. to L_T = (4.00 in. − 0.00 in.) − 2.22 in. = 1.78 in., despite having a nominal reference value of L_T = 1½ in. In all cases, though, the body of the bolt will be at least 2.22 in. long, and the grip gage length will not exceed 2.50 in.

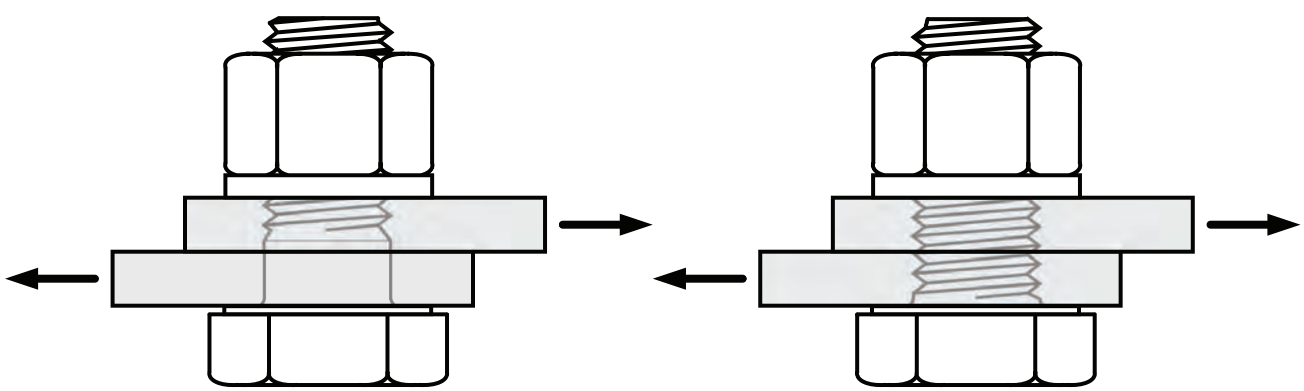
Four different variations of the ⅞-in.-9×4-in. bolt are shown in Figure 7. Figures 7(a) and 7(b) show the bolt with its maximum permitted length, L = 4.00 in., and Figures 7(c) and 7(d) show the bolt with its minimum permitted length L = 4.00 − 0.19 in. = 3.81 in. Additionally, Figures 7(a) and 7(c) show the bolt with its minimum body length of $L_B = L_{B,min}$ = 2.22 in., while Figures 7(b) and 7(d) show the bolt with a slightly longer body length of L_B = 2.38 in. that could result with a shorter transition length of Y = 0.22 in. instead of the reference value of Y = 0.28 in. All four variations are within the ASME B18.2.6 tolerances for a ⅞-in.-9×4-in. bolt, and consideration of these variations is essential when selecting a bolt with appropriate length for a joint.

Nut and Washer Thickness Dimensions

ASME B18.2.6 also includes dimensions for heavy hex nuts and hardened washers. While dimensions and tolerances are given for the width across the flats and the width across the corners for nuts and for outer and inner diameters for washers, the tolerances on thickness for both the nuts and washers are of primary concern for this paper and are shown in Table 4. Nominal thicknesses are not provided for washers in ASME B18.2.6, but 5⁄32 in. is often used as the nominal thickness for all diameter washers (RCSC, 2015).

BOLT LENGTH DETERMINATION

Two design criteria that must be satisfied in selecting the length of a bolt for a given joint are (1) the bolt must be short enough that the nut can be either snug tightened or pretensioned without the threads of the nut running out onto the transition region of the bolt (“shanking out”), and (2) the bolt must be long enough that the nut can be threaded completely onto the bolt (zero or positive stick-out). These two cases are illustrated in Figures 8(a) and 8(b), which show a bolting assembly consisting of a 1-in.-8×6-in. bolt, a nut, and



(a) (b)

Fig. 6. Illustration of a joint with ⅞-in.-9×2-in. bolts (a) as might be expected by the engineer of record and (b) as provided.

Table 4. Nut and Washer Thicknesses					
Nominal Bolt Diameter (in.)	Heavy Hex Nut Thickness			Washer Thickness	
	Maximum (in.)	Nominal (in.)	Minimum (in.)	Minimum (in.)	Maximum (in.)
½	0.504	31⁄64	0.464	0.097	0.177
⅝	0.631	39⁄64	0.587	0.122	0.177
¾	0.758	47⁄64	0.710	0.122	0.177
⅞	0.885	55⁄64	0.833	0.136	0.177
1	1.012	63⁄64	0.956	0.136	0.177
1⅛	1.139	1 7⁄64	1.079	0.136	0.177
1¼	1.251	1 7⁄32	1.187	0.136	0.177
1⅜	1.378	1 11⁄32	1.310	0.136	0.177
1½	1.505	1 15⁄32	1.433	0.136	0.177

two F436 washers.[2] Figure 8(a) illustrates the first criterion, which leads to the minimum grip for a given bolt length, while Figure 8(b) illustrates the second criterion, which leads to the maximum grip for a given bolt length.[3] These two criteria can be written mathematically as

$$\text{Minimum grip} = L_{G,max} - \Sigma t_{washers} - \delta_{pretension} \quad (3)$$

$$\text{Maximum grip} = L - \Sigma t_{washers} - t_{nut} \quad (4)$$

Considering a 1-in.-8×6-in. bolt, the maximum grip gage length can be computed as

$$L_{G,max} = 6 \text{ in.} - 1\tfrac{3}{4} \text{ in.}$$
$$= 4.25 \text{ in.}$$

For this bolt, with $L < 8d_b$, the change in length during pretensioning, based on a half turn past snug being required, can be estimated as

$$\delta_{pretension} = 0.5/8$$
$$= 0.0625 \text{ in.}$$

It should be noted that most of this elongation is expected to occur within the threaded region of the bolt. Thus,

2 The case of a bolting assembly with two washers—one under the head and one under the nut—has been used in this paper because it represents a situation that is useful for illustrating the calculations that are presented. It should be noted, however, that while the use of two washers is necessary in some situations, it is more common to use just a single washer.

3 Care should be taken to not confuse the "grip gage length" of a bolt with the "grip" of a joint. The former is a control dimension used in the manufacturing of bolts while the latter is the total thickness of a joint between the bearing surfaces of the bolt and nut, which, in this work, excludes the thickness of F436 washers included with the bolting assembly. This is consistent with the definition of found in the RCSC *Specification* (2015) but not with the definition found in the AISC *Specification* (2016).

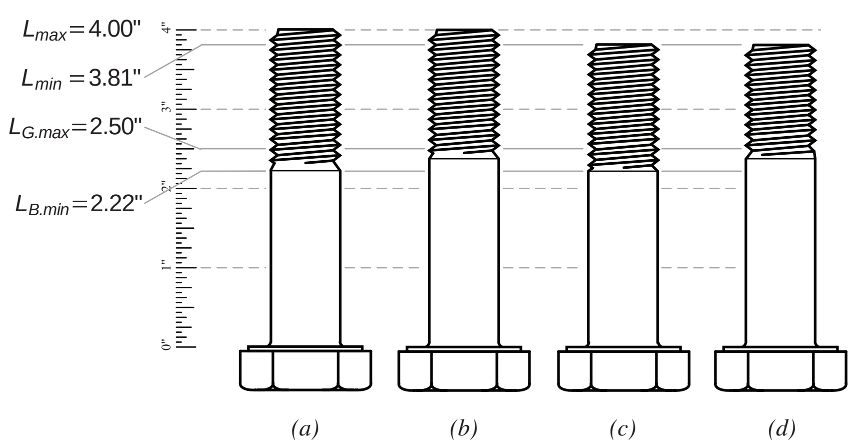


Fig. 7. Different variations of ⅞-in.-9×4-in. high-strength bolts per ASME B18.2.6-19.

assuming that two washers are used and using the minimum washer thickness permitted in ASME B18.2.6, the minimum grip can be determined using Equation 3:

$$\begin{aligned}\text{Minimum grip} &= L_{G,max} - \Sigma t_{washers} - \delta_{pretension} \quad (3)\\ &= 4.25\text{ in.} - (2)(0.136\text{ in.}) - 0.0625\text{ in.}\\ &= 3.92\text{ in.}\end{aligned}$$

Again, assuming that two washers are used, but using the maximum washer thickness, maximum nut thickness, and minimum overall bolt length permitted in ASME B18.2.6, the maximum grip can be determined using Equation 4:

$$\begin{aligned}\text{Maximum grip} &= L - \Sigma t_{washers} - t_{nut} \quad (4)\\ &= (6.00\text{ in.} - 0.19\text{ in.})\\ &\quad - (2)(0.177\text{ in.}) - 1.012\text{ in.}\\ &= 4.44\text{ in.}\end{aligned}$$

Values for the minimum and maximum grip for all common diameter and length high-strength bolts are tabulated in Table A2. The values shown in Table A2 for the minimum grip include an elongation during pretensioning based on a half-turn of the nut for bolts where $L \le 8d_b$ and two-thirds turn of the nut for bolts where $L > 8d_b$. This is largely consistent with the RCSC *Specification* for joints where outer faces of the gripped material are normal to the axis of the bolt but is slightly conservative for bolts where $L \le 4d_b$ where one-third turn would be required.

A third design criterion that is implemented in some cases is that the bolt should be long enough so as to exclude the threads of the bolt from the shear plane(s) of the joint. Considering the joint shown in Figure 9, where four plies are joined, the bolt can be designed with the threads excluded from the shear plane so long as none of the faying surfaces between plies intersects the threaded portion of the bolt. This is true when

$$\Sigma t_{(n-1)} + \Sigma t_{washers\ under\ head} \le L_{B,min} + f(Y) \quad (5)$$

where $\Sigma t_{(n-1)}$ is used as short hand for $\sum_{i=1}^{n-1} t_i$ and $f(Y)$ is some fraction of thread transition length, Y. The threads can be excluded from the shear plane as long as

$$\Sigma t_{(n-1)} \le L_{B,min} - \Sigma t_{washers\ under\ head} + f(Y) \quad (6)$$

A fraction of the thread transition length, $f(Y)$, is included in Equations 5 and 6 because commentary to Section 2.3 of the RCSC *Specification* (RCSC, 2015) can be interpreted as permitting a bolt to be considered in the threads excluded or X condition when the shear plane passes through the transition region of the bolt, though this section of the RCSC *Specification* is under review as of the writing of this paper. Knowing that the actual value of Y may vary from one manufacturer to another, however, there is merit to simply and conservatively taking $f(Y) = 0$.

If the $\Sigma t_{(n-1),max}$ is defined as

$$\Sigma t_{(n-1),max} = L_{B,min} - \Sigma t_{washers\ under\ head} + f(Y) \quad (7)$$

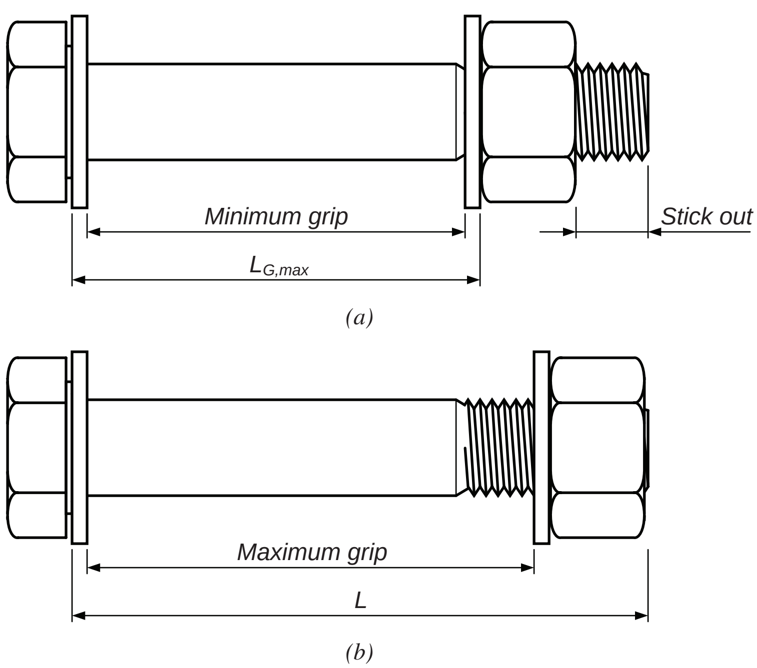


Fig. 8. Minimum and maximum grip for a given bolt length.

then it can be stated that when $\Sigma t_{(n-1)}$ for a joint is less than $\Sigma t_{(n-1),max}$ for a bolting assembly, then the bolt can be treated as if it is in the threads excluded or X condition. Again, considering a bolting assembly consisting of a 1-in.-8×6-in. bolt, a nut, and two F436 washers—one under the head and one under the nut—the $\Sigma t_{(n-1),max}$ can be computed as

$$\Sigma t_{(n-1),max} = L_{B,min} - \Sigma t_{washers\ under\ head} + f(Y) \tag{7}$$

$$= 3.94 \text{ in.} - 0.177 \text{ in.} + 0 \text{ in.}$$

$$= 3.76 \text{ in.}$$

where the maximum thickness of the washer was used and $f(Y)$ was conservatively taken as zero. Values of $\Sigma t_{(n-1),max}$ for common configurations of bolting assemblies computed with $f(Y)$ conservatively taken as zero are provided in Table A2.

In the past, other authors have related the location of the shear plane relative to the thread transition as a function of the bolt length and the thickness of the last ply in the joint, t_n (i.e., the ply closest to the nut). Because the thread length, L_T, is a reference dimension, however, there is little assurance that the transition will be where it is expected when its location is based on L_T. In this work, the location of the thread transition is determined as a function of the minimum body length, $L_{B,min}$, which is a control dimension, and is thus more reliable than the thread length.

Use of Washers

The AISC *Specification* (AISC, 2016) refers to the RCSC *Specification* regarding the use of washers. According to the RCSC *Specification* (RCSC, 2015), washers are not required for snug-tightened joints except when a slotted hole is used, in which case an F436 washer or a 5⁄16-in.-thick common plate washer is required to completely cover the hole, or when sloping surfaces are joined, where a beveled washer is required. When pretensioned or slip-critical joints are employed, an F436 washer or a 5⁄16-in.-thick common plate washer is again required to completely cover slotted holes, a beveled washer is required for sloping surfaces, and an F436 washer is required under both the head and the nut when material with a yield strength of less than 40 ksi is joined using Grade A490 or F2280 bolts, except that a washer is not needed under the head of an F2280 when it has a button head like that shown in Figure 1(c). Further, when twist-off bolts are used in a pretensioned or slip-critical joint, an F436 washer is required under the nut of the assembly, and when the calibrated wrench method of installation is used, an F436 washer must be used under the turned element (either the nut or head of the bolt), regardless of bolt grade or material strength. Finally, there are additional washer requirements when direct-tension-indicators are used for pretensioning.

Given these requirements for washers, common bolting assembly configurations include no washers, a single washer under either the nut or head, or two washers with one under the nut and another under the head. Occasionally, multiple washers are used to allow more margin for nut rotation and thread engagement during pretensioning. In those cases, the washers are generally added under the nut. With few exceptions, there is little to be gained from using washers under the head of the bolt, since doing so makes it more challenging to exclude the threads of the bolt from the shear plane. One exception is when the head of the bolt is the turned element during tightening using the calibrated wrench method of installation, in which case a single washer under the head will suffice. Another exception is when Grade A490 or hex headed Grade F2280 bolts are used to connect material with a yield stress less than 40 ksi. In other cases, washers other than F436 are used, including 5⁄16-in.-thick common washers, plate washers, direct-tension-indicating washers, or shims. In these cases, the thicknesses of those washers, shims, and/or washer plates must be included in the bolt length calculations.

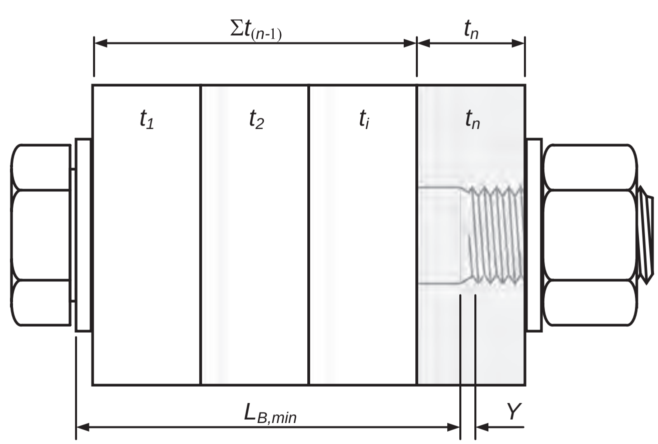


Fig. 9. Illustration of plies and threads in the grip of a bolt.

Procedure for Determination of Bolt Length

A procedure for the determination of bolt length for a joint is summarized here.

Step 1. Determine the type and location of washer(s) required for the particular joint specified, taking care to consider washer requirements in the RCSC and AISC *Specifications* related to hole type, location, material strength, and method of installation.

Step 2. Compute the required grip for the joint. The required grip should include all materials between the bearing surfaces of the bolt head and the nut except for F436 hardened washers addressed in Step 1.

Step 3. Unless bolts are not subject to shear or are predetermined to be designed in the threads not excluded or N condition, determine the cumulative thickness of all but the last plate within the grip of the bolt, $\Sigma t_{(n-1)}$ (i.e., the thickness of all of the plates within the grip except for the plate closest to the nut). This quantity may depend on the way in which the bolt is installed in joints that are not symmetric about the mid-length of the joint grip and should include all materials between the bearing surface of the nut and last faying surface of the joint except for F436 washers, which are included in Table A2.

Step 4. Enter Table A2 on the appropriate page for the bolt diameter specified, and using the appropriate columns for the washer configuration determined in Step 1, select a bolt length that has both (1) a minimum grip that is shorter than the required grip and (2) a maximum grip that is longer than the required grip. Rarely, a bolt cannot be identified that satisfies both criteria. In that case, try either adding washer(s) to the bolting assembly or selecting a bolt that meets only the first criterion (i.e., a bolt that has a minimum grip that is shorter than the required grip).

Step 5. Unless bolts are not subject to shear or are predetermined to be designed in the threads not excluded or N condition, determine the maximum cumulative thickness of all but the last plate within the grip of the bolt, $\Sigma t_{(n-1),max}$, and compare this value to $\Sigma t_{(n-1)}$ that was determined in Step 3. If $\Sigma t_{(n-1)}$ is not greater than $\Sigma t_{(n-1),max}$ then the bolt can be designed in the threads excluded or X condition.

Note that there is sometimes more than one bolt length that will satisfy the design criteria for a given joint. It is up to the engineer to decide which one is desired. On one hand, choosing a longer bolt will result in more thread stick out past the exposed face of the nut and will make it more likely that the threads will be excluded from the shear plane, resulting in higher strength from the fastener. On the other hand, though, choosing a shorter bolt will result in less thread stick out and more threads within the grip of the bolt, which can improve the rotational capacity of the bolt during pretensioning and can improve the ductility of the bolt under tension. Note that the values shown for minimum grip in Table A2 include the *minimum* required turn of nut for most cases. As such, when the required grip for a joint is close to the minimum grip for a bolt shown in Table A2, an engineer may wish to select the next shorter bolt, if possible, to avoid potential problems with the nut "shanking out" on the transition region due to over-rotation of the nut during pretensioning. Alternatively, an additional washer can often be added to the bolting assembly to reduce the likelihood of shanking out.

DESIGN EXAMPLES

Six examples are presented in this section to illustrate the use of the proposed procedure for determining the appropriate length of bolts. Examples 1–4 are adapted from examples that were presented by Carter (1996). Examples 5 and 6 are intended to illustrate less commonly encountered design issues.

Example 1

Determine the bolt length for ¾-in.-diameter ASTM F3125 Grade A325 snug-tight bolts in standard holes in a ⅜-in. single-plate connection supporting a W21×50 beam (nominal t_w = ⅜ in.)

Recommended Solution

The required grip for the joint is ⅜ in. + ⅜ in. = ¾ in. Because the thickness of either of the plies joined is t = ⅜ in., $\Sigma t_{(n-1)}$ = ⅜ in. for the joint regardless of whether the bolt is installed through the single plate first or through the beam web first. No washers are required for a snug-tightened joint using Grade A325 fasteners. Table A2 is used to select ¾-in.-diameter bolts with a minimum grip less than or equal to ¾ in. and a maximum grip greater than or equal to ¾ in. The following options are available:

1. ¾-in.-10×1¾-in. with no washers
   Min grip = 0.20 in.
   Max grip = 0.80 in.
   Fully threaded

2. ¾-in.-10×2-in. with no washers
   Min grip = 0.57 in.
   Max grip = 1.05 in.
   $\Sigma t_{(n-1),max}$ = 0.37 in.

3. ¾-in.-10×2-in. with one washer
   Min grip = 0.45 in.
   Max grip = 0.88 in.
   $\Sigma t_{(n-1),max}$ = 0.37 in.

4. ¾-in.-10×2¼-in. with one washer
   Min grip = 0.70 in.
   Max grip = 1.13 in.
   $\Sigma t_{(n-1),max}$ = 0.62 in.

5. ¾-in.-10×2¼-in. with two washers[4]
   Min grip = 0.58 in.
   Max grip = 0.95 in.
   $\Sigma t_{(n-1),max}$ = 0.44 in.

Based on the options available for this joint, a ¾-in.-10×2-in. bolt is recommended because it would be of an acceptable length to work either without washers or with a single washer. Because $\Sigma t_{(n-1),max}$ = 0.37 in. in either case, which is smaller than $\Sigma t_{(n-1)}$ = ⅜ in., the bolt would need to be designed in the threads not excluded or N condition regardless of whether it is inserted through the shear plate first or through the beam web first.

If it is required to exclude the threads from the shear plane, then a ¾-in.-10×2¼ in. bolt would work, though this assembly would need at least one washer included to avoid shanking out the nut during installation (even for a snug-tight installation).

RCSC Method

Because washers are not required for this joint, the grip + washer thickness is ¾ in. Per RCSC *Specification* Table C2.2, the length of the bolt is determined by adding 1 in. to the grip + washer thickness. Thus, the length of the bolt can be determined as

$$L_{req} = ¾\text{ in.} + 1\text{ in.}$$
$$= 1¾\text{ in.}$$

Therefore, use L_{req} = 1¾ in.

For the ¾-in.-diameter bolt, L_T = 1⅜-in., thus the shank would be expected to be 1¾-in. − 1⅜-in. = ⅜ in. long, and the engineer may expect the bolt to be in the threads excluded or X condition. Note, however, that according to Table A1, the ¾-in.-10×1¾-in. bolt is considered to be fully threaded and would have an unthreaded and transition length that would be no longer than 0.25 in.

4 In the design examples presented in this work, when one washer is included, it will be under the nut, and when two washers are included, one will be under the nut and one will be under the head.

Example 2

Determine the bolt length for ¾-in.-diameter ASTM F3125 Grade A325 snug-tightened bolts in standard holes in a double-angle connection with 2L5×3×5⁄16-in. angles supporting a W27×84 beam (nominal t_w = 7⁄16-in.).[5]

Recommended Solution

The required grip for this joint is 5⁄16 in. + 7⁄16 in. + 5⁄16 in. = 11⁄16 in. Regardless of which direction the bolt is installed, $\Sigma t_{(n-1)}$ for the joint would be 5⁄16 in. + 7⁄16 in. = ¾ in. Washers are not required for a snug tightened joint. Table A2 is used to select ¾-in.-diameter bolts with a minimum grip less than or equal to 11⁄16 in. and a maximum grip greater than or equal to 11⁄16 in. The following options are available:

1. ¾-in.-10×2¼-in. with no washers

   Min grip = 0.82 in.

   Max grip = 1.30 in.

   $\Sigma t_{(n-1),max}$ = 0.62 in.

2. ¾-in.-10×2¼-in. with one washer

   Min grip = 0.70 in.

   Max grip = 1.13 in.

   $\Sigma t_{(n-1),max}$ = 0.62 in.

3. ¾-in.-10×2½-in. with one washer

   Min grip = 0.95 in.

   Max grip = 1.38 in.

   $\Sigma t_{(n-1),max}$ = 0.87 in.

4. ¾-in.-10×2½-in. with two washers

   Min grip = 0.83 in.

   Max grip = 1.20 in.

   $\Sigma t_{(n-1),max}$ = 0.69 in.

Based on the options available for this joint, a ¾-in.-10×2¼-in. bolt is recommended because it would be of an acceptable length either without washers or with a single washer. Because $\Sigma t_{(n-1),max}$ = 0.62 in. in either case, which is smaller than $\Sigma t_{(n-1)}$ = ¾ in., the bolt would need to be designed in the threads not excluded or N condition.

If a threads excluded or X condition is required, a ¾-in.-10×2½-in. bolt would be acceptable, though this assembly would require one washer under the nut to avoid shanking out the nut during installation (even for a snug-tight installation). Putting the washer under the head, however, may cause the threads to be not excluded from the shear plane.

RCSC Method

When no washers are required, as would be the case for snug-tightened Grade A325 bolts, the grip + washer thickness is 5⁄16 in. + 7⁄16 in. + 5⁄16 in. = 11⁄16 in. Thus, the length of the bolt can be determined using RCSC *Specification* Table C2.2 as

$$L_{req} = 1\tfrac{1}{16}\text{ in.} + 1\text{ in.}$$
$$= 2\tfrac{1}{16}\text{ in.}$$

Therefore, use L_{req} = 2¼ in.

5 It is noted that 2L4×32×5⁄16 angles may be more commonly used than 2L5×3×5⁄16. However, the 2L5×3×5⁄16 angles were selected for this example to maintain consistency with the paper by Carter (1996), where it first appeared.

For the ¾-in.-diameter bolt, L_T = 1⅜ in.; thus, the shank would be expected to be 2¼ in. − 1⅜ in. = ⅞ in. long, and the engineer would expect the bolt to be in the threads excluded or X condition. Note from Table A1, however, that the shank or body length of a ¾-in.-10×2¼-in. bolt is guaranteed only to be 0.62 in. As a result, the 2¼-in.-long bolt might actually be in the threads not excluded or N condition. If the engineer considers a bolt with the shear plane passing through the thread transition to be treated as if it is in the threads excluded or X condition, then from Table A1 it can be observed that the maximum grip gage length of a 2¼-in.-long bolt is 0.87 in, and the bolt could be treated as if the threads were excluded. This is a maximum length, however, and the actual grip gage length may be anywhere between 0.62 in. and 0.87 in.

Example 3

Determine the bolt length for pretensioned ¾-in.-diameter ASTM F3125 Grade A490 bolts connecting a ½-in.-thick angle to a W14×500 column flange (nominal t_w = 3½ in.). The calibrated wrench method will be used to pretension the bolts.

Recommended Solution

The required grip for this joint is 3½ in. + ½ in. = 4 in. If the bolt is installed through the column flange first, then $\Sigma t_{(n-1)}$ = 3½ in. but if the bolt is installed through the angle leg first, then $\Sigma t_{(n-1)}$ = ½ in. Because the calibrated wrench method of installation is to be used, at least one F436 washer is required under the turned element. A Grade A490 bolt was specified, so an F436 washer is also required between the angle and the bearing surface of the head or nut since the angle would be made of 36-ksi material unless otherwise designated.

Assuming that the bolt is installed through the column flange first and that the nut is the turned element, then a single washer under the nut is required and from Table A2 a 5½-in.-long bolt can be selected. For a ¾-in.-10×5½ in. bolting assembly with one washer under the nut, the minimum grip is 3.95 in. and the maximum grip is 4.38 in. In that case, $\Sigma t_{(n-1),max}$ = 3.87 in. and because this is larger than $\Sigma t_{(n-1)}$ = 3½ in., the bolt can be considered to be in the threads excluded or X condition. Note that a ¾-in.-10×5¼-in. bolting assembly with one washer under the nut could be specified, but this would likely be available only by special order.

Alternatively, assuming that the bolt is installed through the angle leg first and the nut is the turned element, then two washers are required—one under the nut and one under the head of the bolting assembly. A 5½-in.-long bolt can again be selected, but in this case, the minimum grip would be 3.83 in.; the maximum grip would be 4.20 in.; the $\Sigma t_{(n-1),max}$ would be 3.69 in.; and because this is larger than $\Sigma t_{(n-1)}$ = ½ in., the bolt can be considered to be in the threads excluded or X condition.

RCSC Method

With a washer under the turned element, the grip + washer thickness is 3½ in. + ½ in. + 5⁄32 in. = 4 5⁄32 in. Thus, the length of the bolt can be determined as

$$
\begin{aligned}
L_{req} &= 4\tfrac{5}{32} \text{ in.} + 1 \text{ in.} \\
&= 5\tfrac{5}{32} \text{ in.}
\end{aligned}
$$

Therefore, use L_{req} = 5½ in.

For the ¾-in.-diameter bolt, L_T = 1⅜ in.; thus, the shank would be expected to be 5½ in. − 1⅜ in. = 4⅛ in. long, and the engineer would expect the bolt to be in the threads excluded or X condition.

Example 4

Determine the bolt length for ⅞-in.-diameter ASTM F3125 Grade F2280 bolts in standard holes in an extended endplate moment connection (1-in.-thick plate) to a W14×132 column flange (nominal t_f = 1 in.) The column and endplate are both made of material with F_y = 50 ksi.

Recommended Solution

The required grip for this joint is 1 in. + 1 in. = 2 in. Because both plates joined are 1 in. thick, the $\Sigma t_{(n-1)}$ is 1 in. regardless of whether the bolt is installed through the column flange first or through the endplate first. Grade F2280 bolts are to be used, so an F436 washer is required under the nut. Because 50-ksi material is gripped by the F2280 bolts, a washer is not required under the head. Table A2 is used to select ⅞-in.-diameter bolts with a minimum grip less than or equal to 2 in. and a maximum grip greater than or equal to 2 in. The following options are available:

1. ⅞-in.-9×3¼-in. with one washer
   Min grip = 1.56 in.
   Max grip = 2.00 in.
   $\Sigma t_{(n-1),max}$ = 1.47 in.

2. ⅞-in.-9×3½-in. with one washer
   Min grip = 1.81 in.
   Max grip = 2.25 in.
   $\Sigma t_{(n-1),max}$ = 1.72 in.

3. ⅞-in.-9×3½-in. with two washers
   Min grip = 1.67 in.
   Max grip = 2.07 in.
   $\Sigma t_{(n-1),max}$ = 1.54 in.

4. ⅞-in.-9×3¾-in. with two washers
   Min grip = 1.92 in.
   Max grip = 2.32 in.
   $\Sigma t_{(n-1),max}$ = 1.79 in.

Based on the options available for this joint, a ⅞-in.-9×3½-in. bolt is recommended because it would be of an acceptable length either with one washer or with two washers. Because the smallest value of $\Sigma t_{(n-1),max}$ for this bolt is 1.54 in., which is greater than $\Sigma t_{(n-1)}$ = 1 in., the bolt can be designed in the threads excluded or X condition, in either case.

RCSC Method

With a washer under the nut of the bolting assembly, the grip + washer thickness is 1 in. + 1 in. + 5⁄32 in. = 2 5⁄32 in. Thus, the length of the bolt can be determined as

$$L_{req} = 2\tfrac{5}{32}\text{ in.} + 1\tfrac{1}{8}\text{ in.}$$
$$= 3\tfrac{9}{32}\text{ in.}$$

Therefore, use L_{req} = 3½ in.

For the ⅞-in.-diameter bolt, L_T = 1½ in.; thus the shank would be expected to be 3½ in. − 1½ in. = 2 in. long, and the engineer would expect the bolt to be in the threads excluded or X condition.

Example 5

Determine the length for 1-in.-diameter ASTM F3148 bolts in standard holes connecting a flange plate (1 in. thick) to the flange of a W24×76 beam (nominal t_f = 11⁄16 in.) in a moment connection. The beam and flange plate are both made of material with F_y = 50 ksi.

Recommended Solution

F3148 bolts ship as matched bolting assemblies with one washer that is to be used under the nut. The required grip for this joint is 1 in. + 11⁄16 in. = 1 11⁄16 in. To account for the tolerances in the depth of the beam, however, which are ±⅛ in. per ASTM A6 (AISC, 2017), the connection will be detailed allowing for the beam depth plus ⅛ in. Shims will be provided to accommodate a gap of up to (2)(⅛ in.) between one beam flange and the adjacent flange plate. Thus the required grip for the bolts could be as large as 1 in. + 11⁄16 in. + (2)(⅛ in.) = 1 15⁄16 in. If the bolts are installed though the beam flange first, the $\Sigma t_{(n-1)}$ would be 11⁄16 in. without shim plates and could be as large as 15⁄16 in. with shim plates. If the bolts are installed through the flange plate first, the

$\Sigma t_{(n-1)}$ would be 1 in. without shim plates and could be as large as 1¼ in. with shim plates.

Table A2 is used to select 1-in.-diameter bolts with a minimum grip less than or equal to 1$^{11}\!/_{16}$ in. and a maximum grip greater than or equal to 1$^{15}\!/_{16}$ in. Based on these criteria, the following options are available:

1. 1-in.-8×3½-in. with one washer
   Min grip = 1.55 in.
   Max grip = 2.12 in.
   $\Sigma t_{(n-1),max}$ = 1.44 in.

2. 1-in.-8×3½-in. with two washers
   Min grip = 1.42 in.
   Max grip = 1.94 in.
   $\Sigma t_{(n-1),max}$ = 1.26 in.

3. 1-in.-8×3¾-in. with two washers
   Min grip = 1.67 in.
   Max grip = 2.19 in.
   $\Sigma t_{(n-1),max}$ = 1.51 in.

Based on the options available for this joint, a 1-in.-8×3½-in. bolting assembly with one washer is selected. This assembly will accommodate a grip ranging from 1.55 in. to 2.12 in. with either one washer or two and will work in the joint either with or without shims. Using just one washer would eliminate the need for and the added cost of the second washer. It can be further noted that the minimum value of $\Sigma t_{(n-1),max}$ for the options shown is 1.26 in. Thus, because this is greater than the maximum considered $\Sigma t_{(n-1)}$, the bolts can be designed in the threads excluded or X condition regardless of which of the options is selected, regardless of the direction in which the bolts are installed, and regardless of whether or not shim plates are used. If for some reason the 1-in.-8×3½-in. bolting assembly was unavailable, the 1 in.-8×3¾ in. bolting assembly with two washers would also work, though this would require additional washers to be provided.

Example 6

Determine the appropriate length for ⅝-in.-diameter ASTM F3125 Grade F1852 bolts that are used in a double lap splice shear joint where two 1½-in.-thick splice plates are used to connect a 2½-in. main member.

Recommended Solution

The required grip for this joint is 1½ in. + 2½ in. + 1½ in. = 5½ in. The $\Sigma t_{(n-1)}$ is 1½ in. + 2½ in. = 4 in. regardless of the direction in which the bolts are installed. Because Grade F1852 bolts are specified, one hardened washer is required under the nut of the bolting assembly.

Using a required grip of 5½ in., Table A2 is used and it is noted that there is not a solution for a ⅝-in.-diameter assembly with one washer tabulated. Despite this, there are several options for this joint.

Option 1. From Table A2, select a ⅝-in.-11×7-in. bolting assembly with two washers. From the table, the minimum grip is 5.45 in. and the maximum grip is 5.77 in. If both washers are installed under the nut of the assembly, $\Sigma t_{(n-1),max}$ would be 5.53 in. Alternatively if one washer was installed under the head and one washer under the nut, then $\Sigma t_{(n-1),max}$ would be 5.35 in. In either case, $\Sigma t_{(n-1),max}$ would be larger than $\Sigma t_{(n-1)}$, indicating that the bolt could be designed in the threads excluded or X condition.

Option 2. Investigate the use of an assembly with tighter overall length tolerances: Using a required grip of 5½ in., Table A2 is used to select a bolt with one washer that has a minimum grip that is smaller than the required grip. A ⅝-in.-11×6½-in. bolting assembly is selected with a minimum grip of 5.07 in. and from Equation 4, the required length of the bolt can be determined as

$$L_{Rqd} = \text{Maximum grip} + \Sigma t_{washers} + t_{nut} \tag{8}$$

$$L_{Rqd} = 5\tfrac{1}{2}\text{ in.} + (1)(0.177\text{ in.}) + 0.631\text{ in.}$$
$$= 6.31\text{ in.}$$

As long as the bolt has an actual length of at least 6.31 in., even with a washer and nut that are both as thick as they can be while still within tolerance, the bolting assembly will work. The standard tolerance for overall length for this bolting assembly is −0.25 in. To be acceptable, the tolerance would need to be 6.31 in. − 6½ in. = −0.19 in. This is certainly achievable and can be accommodated by discussing needs with a reputable bolt supplier. In this case, $\Sigma t_{(n-1),max}$ would be 5.03 in., and because this is larger than $\Sigma t_{(n-1)}$ = 4.00 in., the bolt could again be designed in the threads excluded or X condition.

Option 3. Investigate the use of a ⅝-in.-11×6¾-in. bolting assembly with one washer under the nut: Though this bolt length is not included in Table A2, it would likely be available by special order given proper coordination with a bolt supplier. For the ⅝-in.-11×6¾-in. bolt,

$$\begin{aligned} L_{G,max} &= L_{nom} - L_T \\ &= 6\tfrac{3}{4} \text{ in.} - 1\tfrac{1}{4} \text{ in.} \\ &= 5.50 \text{ in.} \end{aligned} \tag{1}$$

$$\begin{aligned} L_{B,min} &= L_{G,max} - Y \\ &= 5.50 \text{ in.} - 0.22 \text{ in.} \\ &= 5.28 \text{ in.} \end{aligned} \tag{2}$$

$$\begin{aligned} \text{Minimum grip} &= LG,max - \Sigma t_{washers} - \delta_{pretension} \\ &= 5.50 \text{ in.} - (1)(0.177 \text{ in.}) - (\tfrac{1}{2} \text{ turn})/(11) \\ &= 5.28 \text{ in.} \end{aligned} \tag{3}$$

$$\begin{aligned} \text{Maximum grip} &= L - \Sigma t_{washers} - t_{nut} \\ &= (6.75 \text{ in.} - 0.25 \text{ in.}) - (1)(0.177 \text{ in.}) - 0.631 \text{ in.} \\ &= 5.69 \text{ in.} \end{aligned} \tag{4}$$

$$\begin{aligned} \Sigma_{t(n-1)} &\le L_{B,min} - \Sigma t_{washers\ under\ head} + f(Y) \\ &= 5.28 \text{ in.} - (0 \text{ in.}) + (0 \text{ in.}) \\ &= 5.28 \text{ in.} \end{aligned} \tag{6}$$

With these values, it can be seen that that the ⅝-in.-11×6¾-in. assembly with one washer under the nut would be acceptable and could be designed in the threads excluded or X condition.

SUMMARY AND CONCLUSIONS

The dimensional tolerances in ASME B18.2.6, to which F3125 and F3148 bolts must conform, consist of control dimensions and reference dimensions. Control dimensions and their tolerances are those that bolt manufacturers must meet in order for their product to be in conformance with published ASME and, by extension, ASTM standards. Resources that structural engineers and detailers commonly have available, however, typically provide reference dimensions for bolts. This inconsistency can lead to variances between the expectations of the engineer or detailer and what is actually built, possibly in ways that are unconservative.

It should be noted that this inconsistency has existed for nearly four decades. Prior to the introduction of ASME B18.2.6, the dimensions of high-strength fasteners were maintained in ASME B18.2.1 (1981). The dimensional requirements in ASME B18.2.619 can be traced back to at least the 1981 edition of ASME B18.2.1. Despite the fact that this misunderstanding has occurred in the design of numerous structures since at least 1981, the authors are not aware of a structural failure that has resulted from this specific issue. While this misunderstanding can certainly lead to unconservative designs, it might be considered only less conservative compared to actual demand. It should be noted that the application of resistance factors, documented overstrength of fasteners (Moore et al., 2010), and other factors mitigate the risk associated with these incorrect design assumptions.

Based on an analysis of available bolt diameters and lengths, a series of tables was generated to aid in the length determination of bolts for joints, considering the most punitive combination of dimensional tolerances of high-strength bolts, hardened washers, and heavy hex nuts. The tables present ranges of grip lengths that bolting assemblies can accommodate based on bolt diameter, bolt length, and washer configuration and also provide a tool for quickly determining whether the threads in a bolting assembly can be considered as "excluded" from the shear plane or whether they should be considered as "not excluded." Several design examples are presented demonstrating the use of these tables in determining the length of bolts and comparing the results with traditional methods presented in engineering and detailing resources.

The proposed design tables represent a departure from the approach presented by Carter (1996), where acceptable bolt lengths were tabulated for given grips. A similar approach was considered in the current work, but the resulting tables were substantially larger than those proposed, including some redundancies. Instead of centering on the required grip, the tables proposed in this work focus on acceptable grips for practically all available bolting assemblies. One notable difference between the tables presented by Carter and those proposed herein is that Carter's tables provide the minimum ply thickness closest to the nut ($t_{n,min}$) that is required to ensure that the threads are excluded from the shear plane. The tables proposed herein instead provide the maximum thickness of the plies closest to the head ($\Sigma t_{(n-1),max}$) that can be used to ensure that the threads are excluded from the shear plane. This latter approach was chosen because $\Sigma t_{(n-1),max}$ is not dependent on the number of washers used under the nut. For the example joints presented, bolt lengths determined using Carter's method match those determined using the method proposed herein, for the most part. In some cases, however, such as for ¾-in.-10×1¾-in. and ⅞-in.-9×2-in. bolts, Table 2 in Carter indicates that the bolts can be designed in the threads excluded or X condition for some ply thickness when, in fact, the bolts would be considered as fully threaded according to ASME B18.2.6.

It was observed that when the most punitive combination of tolerances for the bolt, washer, and nut were assumed to occur simultaneously, the minimum grip for a given bolt length was actually larger than the maximum grip for the next shorter bolt of the same diameter and washer configuration. This generally occurred for bolting assemblies ranging from ½ in. to ⅞ in. in diameter that are longer than 6-in. nominal length, although cases were observed for bolting assemblies of all diameters with multiple washers. Bolts longer than 6 in. have looser tolerances for nominal length than bolts 6 in. long and shorter. This, combined with the shorter threaded length of smaller-diameter bolts and the standard practice of producing bolts longer than 5 in. in ½-in.-long increments, leads to a situation where it may be difficult to determine an appropriate bolt length for a given joint. This potential situation is not likely to lead to too many difficulties in the field, but it could be eliminated if the rather permissive lower tolerance on bolt length was tightened up.

The design examples included illustrate potential situations where bolts selected using traditional methods of length determination might be designed in the threads excluded or X condition when they may actually be in the threads not excluded or N condition. Examples were also presented illustrating situations where an engineer or detailer may expect bolts to have a small, but predictable, shank when, in fact, they may—in full compliance with ASME B18.2.6—be manufactured fully threaded with no shank at all. It was shown that this second situation generally occurs for the shortest length of each diameter bolt that is routinely produced. While this represents a small percentage of the bolt lengths available, it should be noted that approximately 90% of bolts sold have a length less than four times their diameter, and bolts that are considered to be fully threaded per ASME B18.2.6 represent a disproportionately high percentage—possibly in excess of 35% for some diameters—of bolts sold.[6]

The ambiguous case of the ⅞-in.-9×2-in. bolt is encountered quite commonly. If ⅞-in.-diameter bolts were used to join two ⅜-in.-thick plates, a bolt with a length of 2 in. would be appropriate. The engineer, based on tables in resources readily available to him or her, may expect that the bolts would be in the threads excluded condition, as is shown in Figure 5(a). The bolts, however, would be considered as fully threaded according to ASME B18.2.6 and would actually be in the threads not excluded condition, as is shown in Figure 5(b). Note that selecting a ⅞-in.-9×2¼-in. bolt instead of a ⅞-in.-9×2-in. bolt would also be appropriate, but since this bolt is not considered to be fully threaded, it would have a minimum body length of $L_{B,min} = 0.47$ in. and could reliably be considered to be in the threads excluded condition.

Based on the analysis and examples included in this work, the authors recommend that bolts that are considered to be fully threaded per ASME B18.2.6 be designed for shear strength only in the threads not excluded or N condition. Bolts in this category are those that are the shortest produced for each diameter as is shown in Table 3 (in addition to 1⅜-in.- and 1½-in.-diameter bolts up to and including 3¼ in. long), which are those bolts shown above the solid lines in Tables A1 and A2.

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APPENDIX

Table A1. Maximum Grip Gage Length and Minimum Body Length for Common Structural Bolt Sizes

Bolt Length	Nominal Diameter and Threads per Inch																	
	½ in.-13		⅝ in.-11		¾ in.-10		⅞ in.-9		1 in.-8		1⅛ in.-7		1¼ in.-7		1⅜ in.-6		1½ in.-6	
L (in.)	$L_{G,max}$ (in.)	$L_{B,min}$ (in.)	$L_{G,max}$ (in.)	$L_{B,min}$ (in.)	$L_{G,max}$ (in.)	$L_{B,min}$ (in.)	$L_{G,max}$ (in.)	$L_{B,min}$ (in.)	$L_{G,max}$ (in.)	$L_{B,min}$ (in.)	$L_{G,max}$ (in.)	$L_{B,min}$ (in.)	$L_{G,max}$ (in.)	$L_{B,min}$ (in.)	$L_{G,max}$ (in.)	$L_{B,min}$ (in.)	$L_{G,max}$ (in.)	$L_{B,min}$ (in.)
1¼	0.19	0.06	0.23	0.00	*0.25*	*0.00*	*0.28*	*0.00*	*0.31*	*0.00*	*0.50*	*0.00*	*0.50*	*0.00*	*0.58*	*0.00*	*0.58*	*0.00*
1½	0.50	0.31	0.23	0.03	0.25	0.00	*0.28*	*0.00*	*0.31*	*0.00*	*0.50*	*0.00*	*0.50*	*0.00*	*0.58*	*0.00*	*0.58*	*0.00*
1¾	0.75	0.56	0.50	0.28	0.25	0.12	0.28	0.00	*0.31*	*0.00*	*0.50*	*0.00*	*0.50*	*0.00*	*0.58*	*0.00*	*0.58*	*0.00*
2	1.00	0.81	0.75	0.53	0.62	0.37	0.28	0.22	0.31	0.00	*0.50*	*0.00*	*0.50*	*0.00*	*0.58*	*0.00*	*0.58*	*0.00*
2¼	1.25	1.06	1.00	0.78	0.87	0.62	0.75	0.47	0.31	0.19	*0.50*	*0.00*	*0.50*	*0.00*	*0.58*	*0.00*	*0.58*	*0.00*
2½	1.50	1.31	1.25	1.03	1.12	0.87	1.00	0.72	0.75	0.44	0.50	0.16	0.50	0.12	*0.58*	*0.00*	*0.58*	*0.00*
2¾	1.75	1.56	1.50	1.28	1.37	1.12	1.25	0.97	1.00	0.69	0.50	0.41	0.50	0.37	*0.58*	*0.06*	*0.58*	*0.06*
3	2.00	1.81	1.75	1.53	1.62	1.37	1.50	1.22	1.25	0.94	1.00	0.66	1.00	0.62	*0.58*	*0.31*	*0.58*	*0.31*
3¼	2.25	2.06	2.00	1.78	1.87	1.62	1.75	1.47	1.50	1.19	1.25	0.91	1.25	0.87	*0.58*	*0.56*	0.58	0.56
3½	2.50	2.31	2.25	2.03	2.12	1.87	2.00	1.72	1.75	1.44	1.50	1.16	1.50	1.12	*1.25*	*0.81*	1.25	0.81
3¾	2.75	2.56	2.50	2.28	2.37	2.12	2.25	1.97	2.00	1.69	1.75	1.41	1.75	1.37	*1.50*	*1.06*	1.50	1.06
4	3.00	2.81	2.75	2.53	2.62	2.37	2.50	2.22	2.25	1.94	2.00	1.66	2.00	1.62	*1.75*	*1.31*	1.75	1.31
4¼	3.25	3.06	3.00	2.78	2.87	2.62	2.75	2.47	2.50	2.19	2.25	1.91	2.25	1.87	*2.00*	*1.56*	2.00	1.56
4½	3.50	3.31	3.25	3.03	3.12	2.87	3.00	2.72	2.75	2.44	2.50	2.16	2.50	2.12	*2.25*	*1.81*	2.25	1.81
4¾	3.75	3.56	3.50	3.28	3.37	3.12	3.25	2.97	3.00	2.69	2.75	2.41	2.75	2.37	*2.50*	*2.06*	2.50	2.06
5	4.00	3.81	3.75	3.53	3.62	3.37	3.50	3.22	3.25	2.94	3.00	2.66	3.00	2.62	*2.75*	*2.31*	2.75	2.31
5½	4.50	4.31	4.25	4.03	4.12	3.87	4.00	3.72	3.75	3.44	3.50	3.16	3.50	3.12	*3.25*	*2.81*	3.25	2.81
6	5.00	4.81	4.75	4.53	4.62	4.37	4.50	4.22	4.25	3.94	4.00	3.66	4.00	3.62	*3.75*	*3.31*	3.75	3.31
6½	5.50	5.31	5.25	5.03	5.12	4.87	5.00	4.72	4.75	4.44	4.50	4.16	4.50	4.12	*4.25*	*3.81*	4.25	3.81
7	6.00	5.81	5.75	5.53	5.62	5.37	5.50	5.22	5.25	4.94	5.00	4.66	5.00	4.62	*4.75*	*4.31*	4.75	4.31
7½	6.50	6.31	6.25	6.03	6.12	5.87	6.00	5.72	5.75	5.44	5.50	5.16	5.50	5.12	*5.25*	*4.81*	5.25	4.81
8	7.00	6.81	6.75	6.53	6.62	6.37	6.50	6.22	6.25	5.94	6.00	5.66	6.00	5.62	*5.75*	*5.31*	5.75	5.31
8½	7.50	7.31	7.25	7.03	7.12	6.87	7.00	6.72	6.75	6.44	6.50	6.16	6.50	6.12	*6.25*	*5.81*	6.25	5.81
9	8.00	7.81	7.75	7.53	7.62	7.37	7.50	7.22	7.25	6.94	7.00	6.66	7.00	6.62	*6.75*	*6.31*	6.75	6.31
9½	8.50	8.31	8.25	8.03	8.12	7.87	8.00	7.72	7.75	7.44	7.50	7.16	7.50	7.12	*7.25*	*6.81*	7.25	6.81
10	9.00	8.81	8.75	8.53	8.62	8.37	8.50	8.22	8.25	7.94	8.00	7.66	8.00	7.62	*7.75*	*7.31*	7.75	7.31

Notes: Cells above the solid line represent bolt lengths that are considered to be fully threaded per ASME B18.2.6. These bolts may or may not have a shank.
Shaded cells represent bolt lengths that are rarely produced. Italics values represent bolt lengths that are available by special order only.

Table A2. Minimum Grip, Maximum Grip, and Thread Condition for High-Strength Bolts								
	½-in.-Diameter Bolts							
	0 Washers		1 Washer		2 Washers		$\Sigma t_{(n-1),max}$	
Bolt Length (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	0 Washers (in.)	1 Washer (in.)
1¼	0.15	0.63	0.06	0.45	0.00	0.27	N	N
1½	0.46	0.88	0.36	0.70	0.27	0.52	0.31	0.13
1¾	0.71	1.13	0.61	0.95	0.52	0.77	0.56	0.38
2	0.96	1.38	0.86	1.20	0.77	1.02	0.81	0.63
2¼	1.21	1.63	1.11	1.45	1.02	1.27	1.06	0.88
2½	1.46	1.88	1.36	1.70	1.27	1.52	1.31	1.13
2¾	1.71	2.13	1.61	1.95	1.52	1.77	1.56	1.38
3	1.96	2.38	1.86	2.20	1.77	2.02	1.81	1.63
3¼	2.21	2.63	2.11	2.45	2.02	2.27	2.06	1.88
3½	2.46	2.88	2.36	2.70	2.27	2.52	2.31	2.13
3¾	2.71	3.13	2.61	2.95	2.52	2.77	2.56	2.38
4	2.96	3.38	2.86	3.20	2.77	3.02	2.81	2.63
4¼	3.20	3.63	3.10	3.45	3.00	3.27	3.06	2.88
4½	3.45	3.88	3.35	3.70	3.25	3.52	3.31	3.13
4¾	3.70	4.13	3.60	3.95	3.50	3.77	3.56	3.38
5	3.95	4.38	3.85	4.20	3.75	4.02	3.81	3.63
5½	4.45	4.88	4.35	4.70	4.25	4.52	4.31	4.13
6	4.95	5.38	4.85	5.20	4.75	5.02	4.81	4.63
6½	5.45	5.81	5.35	5.63	5.25	5.45	5.31	5.13
7	5.95	6.31	5.85	6.13	5.75	5.95	5.81	5.63
7½	6.45	6.81	6.35	6.63	6.25	6.45	6.31	6.13
8	6.95	7.31	6.85	7.13	6.75	6.95	6.81	6.63
8½	7.45	7.81	7.35	7.63	7.25	7.45	7.31	7.13
9	7.95	8.31	7.85	8.13	7.75	7.95	7.81	7.63
9½	8.45	8.81	8.35	8.63	8.25	8.45	8.31	8.13
10	8.95	9.31	8.85	9.13	8.75	8.95	8.81	8.63

Notes: For Min Grip and Max Grip, consider the total number of F436 washers used under the head and under the nut.
For the $\Sigma t_{(n-1),max}$, consider only the number of F436 washers under the head of the bolt.
"N" indicates bolts that should be designed in the "threads not excluded" or "N" condition only.
Cells above the heavy line represent bolt lengths that are considered to be fully threaded per ASME B18.2.6.

Table A2. Minimum Grip, Maximum Grip, and Thread Condition for High-Strength Bolts (continued)								
	⅝-in.-Diameter Bolts							
	0 Washers		1 Washer		2 Washers		$\Sigma t_{(n-1),max}$	
Bolt Length (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	0 Washers (in.)	1 Washer (in.)
1¼	0.18	0.50	0.06	0.32	0.00	0.15	N	N
1½	0.18	0.75	0.06	0.57	0.00	0.40	N	N
1¾	0.45	1.00	0.33	0.82	0.21	0.65	0.28	0.10
2	0.70	1.25	0.58	1.07	0.46	0.90	0.53	0.35
2¼	0.95	1.50	0.83	1.32	0.71	1.15	0.78	0.60
2½	1.20	1.75	1.08	1.57	0.96	1.40	1.03	0.85
2¾	1.45	2.00	1.33	1.82	1.21	1.65	1.28	1.10
3	1.70	2.25	1.58	2.07	1.46	1.90	1.53	1.35
3¼	1.95	2.50	1.83	2.32	1.71	2.15	1.78	1.60
3½	2.20	2.75	2.08	2.57	1.96	2.40	2.03	1.85
3¾	2.45	3.00	2.33	2.82	2.21	2.65	2.28	2.10
4	2.70	3.25	2.58	3.07	2.46	2.90	2.53	2.35
4¼	2.95	3.50	2.83	3.32	2.71	3.15	2.78	2.60
4½	3.20	3.75	3.08	3.57	2.96	3.40	3.03	2.85
4¾	3.45	4.00	3.33	3.82	3.21	3.65	3.28	3.10
5	3.70	4.25	3.58	4.07	3.46	3.90	3.53	3.35
5½	4.19	4.75	4.07	4.57	3.95	4.40	4.03	3.85
6	4.69	5.25	4.57	5.07	4.45	4.90	4.53	4.35
6½	5.19	5.62	5.07	5.44	4.95	5.27	5.03	4.85
7	5.69	6.12	5.57	5.94	5.45	5.77	5.53	5.35
7½	6.19	6.62	6.07	6.44	5.95	6.27	6.03	5.85
8	6.69	7.12	6.57	6.94	6.45	6.77	6.53	6.35
8½	7.19	7.62	7.07	7.44	6.95	7.27	7.03	6.85
9	7.69	8.12	7.57	7.94	7.45	7.77	7.53	7.35
9½	8.19	8.62	8.07	8.44	7.95	8.27	8.03	7.85
10	8.69	9.12	8.57	8.94	8.45	8.77	8.53	8.35

Notes: For Min Grip and Max Grip, consider the total number of F436 washers used under the head and under the nut.
For the $\Sigma t_{(n-1),max}$, consider only the number of F436 washers under the head of the bolt.
"N" indicates bolts that should be designed in the "threads not excluded" or "N" condition only.
Cells above the heavy line represent bolt lengths that are considered to be fully threaded per ASME B18.2.6.
Shaded cells represent bolt lengths that are rarely produced.

Table A2. Minimum Grip, Maximum Grip, and Thread Condition for High-Strength Bolts (continued)								
	¾-in.-Diameter Bolts							
	0 Washers		1 Washer		2 Washers		$\Sigma t_{(n-1),max}$	
Bolt Length (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	0 Washers (in.)	1 Washer (in.)
1¼	*0.20*	*0.30*	*0.08*	*0.13*	—	—	*N*	*N*
1½	0.20	0.55	0.08	0.38	0.00	0.20	N	N
1¾	0.20	0.80	0.08	0.63	0.00	0.45	N	N
2	0.57	1.05	0.45	0.88	0.33	0.70	0.37	0.19
2¼	0.82	1.30	0.70	1.13	0.58	0.95	0.62	0.44
2½	1.07	1.55	0.95	1.38	0.83	1.20	0.87	0.69
2¾	1.32	1.80	1.20	1.63	1.08	1.45	1.12	0.94
3	1.57	2.05	1.45	1.88	1.33	1.70	1.37	1.19
3¼	1.82	2.30	1.70	2.13	1.58	1.95	1.62	1.44
3½	2.07	2.55	1.95	2.38	1.83	2.20	1.87	1.69
3¾	2.32	2.80	2.20	2.63	2.08	2.45	2.12	1.94
4	2.57	3.05	2.45	2.88	2.33	2.70	2.37	2.19
4¼	2.82	3.30	2.70	3.13	2.58	2.95	2.62	2.44
4½	3.07	3.55	2.95	3.38	2.83	3.20	2.87	2.69
4¾	3.32	3.80	3.20	3.63	3.08	3.45	3.12	2.94
5	3.57	4.05	3.45	3.88	3.33	3.70	3.37	3.19
5½	4.07	4.55	3.95	4.38	3.83	4.20	3.87	3.69
6	4.57	5.05	4.45	4.88	4.33	4.70	4.37	4.19
6½	5.05	5.49	4.93	5.32	4.81	5.14	4.87	4.69
7	5.55	5.99	5.43	5.82	5.31	5.64	5.37	5.19
7½	6.05	6.49	5.93	6.32	5.81	6.14	5.87	5.69
8	6.55	6.99	6.43	6.82	6.31	6.64	6.37	6.19
8½	7.05	7.49	6.93	7.32	6.81	7.14	6.87	6.69
9	7.55	7.99	7.43	7.82	7.31	7.64	7.37	7.19
9½	8.05	8.49	7.93	8.32	7.81	8.14	7.87	7.69
10	8.55	8.99	8.43	8.82	8.31	8.64	8.37	8.19

Notes: For Min Grip and Max Grip, consider the total number of F436 washers used under the head and under the nut.
For the $\Sigma t_{(n-1),max}$, consider only the number of F436 washers under the head of the bolt.
"N" indicates bolts that should be designed in the "threads not excluded" or "N" condition only.
Cells above the heavy line represent bolt lengths that are considered to be fully threaded per ASME B18.2.6.
Shaded cells represent bolt lengths that are rarely produced.
Italics values indicate bolt lengths that are available by special order only.
"—" indicates bolt assemblies where the nut may not be able to be threaded fully onto the bolt.

Table A2. Minimum Grip, Maximum Grip, and Thread Condition for High-Strength Bolts (continued)								
	⅞-in.-Diameter Bolts							
	0 Washers		1 Washer		2 Washers		$\Sigma t_{(n-1),max}$	
Bolt Length (in.)	**Min Grip (in.)**	**Max Grip (in.)**	**Min Grip (in.)**	**Max Grip (in.)**	**Min Grip (in.)**	**Max Grip (in.)**	**0 Washers (in.)**	**1 Washer (in.)**
1¼	*0.22*	*0.18*	—	—	—	—	*N*	*N*
1½	*0.22*	*0.43*	*0.09*	*0.25*	*0.00*	*0.07*	*N*	*N*
1¾	0.22	0.68	0.09	0.50	0.00	0.32	N	N
2	0.22	0.93	0.09	0.75	0.00	0.57	N	N
2¼	0.69	1.18	0.56	1.00	0.42	0.82	0.47	0.29
2½	0.94	1.43	0.81	1.25	0.67	1.07	0.72	0.54
2¾	1.19	1.68	1.06	1.50	0.92	1.32	0.97	0.79
3	1.44	1.93	1.31	1.75	1.17	1.57	1.22	1.04
3¼	1.69	2.18	1.56	2.00	1.42	1.82	1.47	1.29
3½	1.94	2.43	1.81	2.25	1.67	2.07	1.72	1.54
3¾	2.19	2.68	2.06	2.50	1.92	2.32	1.97	1.79
4	2.44	2.93	2.31	2.75	2.17	2.57	2.22	2.04
4¼	2.69	3.18	2.56	3.00	2.42	2.82	2.47	2.29
4½	2.94	3.43	2.81	3.25	2.67	3.07	2.72	2.54
4¾	3.19	3.68	3.06	3.50	2.92	3.32	2.97	2.79
5	3.44	3.93	3.31	3.75	3.17	3.57	3.22	3.04
5½	3.94	4.43	3.81	4.25	3.67	4.07	3.72	3.54
6	4.44	4.93	4.31	4.75	4.17	4.57	4.22	4.04
6½	4.94	5.37	4.81	5.19	4.67	5.01	4.72	4.54
7	5.44	5.87	5.31	5.69	5.17	5.51	5.22	5.04
7½	5.93	6.37	5.79	6.19	5.65	6.01	5.72	5.54
8	6.43	6.87	6.29	6.69	6.15	6.51	6.22	6.04
8½	6.93	7.37	6.79	7.19	6.65	7.01	6.72	6.54
9	7.43	7.87	7.29	7.69	7.15	7.51	7.22	7.04
9½	7.93	8.37	7.79	8.19	7.65	8.01	7.72	7.54
10	8.43	8.87	8.29	8.69	8.15	8.51	8.22	8.04

Notes: For Min Grip and Max Grip, consider the total number of F436 washers used under the head and under the nut.
For the $\Sigma t_{(n-1),max}$, consider only the number of F436 washers under the head of the bolt.
"N" indicates bolts that should be designed in the "threads not excluded" or "N" condition only.
Cells above the heavy line represent bolt lengths that are considered to be fully threaded per ASME B18.2.6.
Shaded cells represent bolt lengths that are rarely produced.
Italics values indicate bolt lengths that are available by special order only.
"—" indicates bolt assemblies where the nut may not be able to be threaded fully onto the bolt.

Table A2. Minimum Grip, Maximum Grip, and Thread Condition for High-Strength Bolts (continued)								
	1-in.-Diameter Bolts							
	0 Washers		1 Washer		2 Washers		$\Sigma t_{(n-1),max}$	
Bolt Length (in.)	**Min Grip (in.)**	**Max Grip (in.)**	**Min Grip (in.)**	**Max Grip (in.)**	**Min Grip (in.)**	**Max Grip (in.)**	**0 Washers (in.)**	**1 Washer (in.)**
1¼	—	—	—	—	—	—	—	—
1½	—	—	—	—	—	—	—	—
1¾	*0.25*	*0.55*	*0.11*	*0.37*	*0.00*	*0.19*	*N*	*N*
2	0.25	0.80	0.11	0.62	0.00	0.44	N	N
2¼	0.25	1.05	0.11	0.87	0.00	0.69	N	N
2½	0.69	1.30	0.55	1.12	0.42	0.94	0.44	0.26
2¾	0.94	1.55	0.80	1.37	0.67	1.19	0.69	0.51
3	1.19	1.80	1.05	1.62	0.92	1.44	0.94	0.76
3¼	1.44	2.05	1.30	1.87	1.17	1.69	1.19	1.01
3½	1.69	2.30	1.55	2.12	1.42	1.94	1.44	1.26
3¾	1.94	2.55	1.80	2.37	1.67	2.19	1.69	1.51
4	2.19	2.80	2.05	2.62	1.92	2.44	1.94	1.76
4¼	2.44	3.05	2.30	2.87	2.17	2.69	2.19	2.01
4½	2.69	3.30	2.55	3.12	2.42	2.94	2.44	2.26
4¾	2.94	3.55	2.80	3.37	2.67	3.19	2.69	2.51
5	3.19	3.80	3.05	3.62	2.92	3.44	2.94	2.76
5½	3.69	4.30	3.55	4.12	3.42	3.94	3.44	3.26
6	4.19	4.80	4.05	4.62	3.92	4.44	3.94	3.76
6½	4.69	5.24	4.55	5.06	4.42	4.88	4.44	4.26
7	5.19	5.74	5.05	5.56	4.92	5.38	4.94	4.76
7½	5.69	6.24	5.55	6.06	5.42	5.88	5.44	5.26
8	6.19	6.74	6.05	6.56	5.92	6.38	5.94	5.76
8½	6.67	7.24	6.53	7.06	6.39	6.88	6.44	6.26
9	7.17	7.74	7.03	7.56	6.89	7.38	6.94	6.76
9½	7.67	8.24	7.53	8.06	7.39	7.88	7.44	7.26
10	8.17	8.74	8.03	8.56	7.89	8.38	7.94	7.76

Notes: For Min Grip and Max Grip, consider the total number of F436 washers used under the head and under the nut.
For the $\Sigma t_{(n-1),max}$, consider only the number of F436 washers under the head of the bolt.
"N" indicates bolts that should be designed in the "threads not excluded" or "N" condition only.
Cells above the heavy line represent bolt lengths that are considered to be fully threaded per ASME B18.2.6.
Shaded cells represent bolt lengths that are rarely produced.
Italics values indicate bolt lengths that are available by special order only.
"—" indicates bolt assemblies where the nut may not be able to be threaded fully onto the bolt.

Table A2. Minimum Grip, Maximum Grip, and Thread Condition for High-Strength Bolts (continued)								
	1⅛-in.-Diameter Bolts							
	0 Washers		1 Washer		2 Washers		$\Sigma t_{(n-1),max}$	
Bolt Length (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	0 Washers (in.)	1 Washer (in.)
1¼	—	—	—	—	—	—	—	—
1½	—	—	—	—	—	—	—	—
1¾	—	—	—	—	—	—	—	—
2	*0.43*	*0.61*	*0.29*	*0.43*	*0.16*	*0.26*	*N*	*N*
2¼	*0.43*	*0.86*	*0.29*	*0.68*	*0.16*	*0.51*	*N*	*N*
2½	0.43	1.11	0.29	0.93	0.16	0.76	N	N
2¾	0.43	1.36	0.29	1.18	0.16	1.01	N	N
3	0.93	1.61	0.79	1.43	0.66	1.26	0.66	0.48
3¼	1.18	1.86	1.04	1.68	0.91	1.51	0.91	0.73
3½	1.43	2.11	1.29	1.93	1.16	1.76	1.16	0.98
3¾	1.68	2.36	1.54	2.18	1.41	2.01	1.41	1.23
4	1.93	2.61	1.79	2.43	1.66	2.26	1.66	1.48
4¼	2.18	2.86	2.04	2.68	1.91	2.51	1.91	1.73
4½	2.43	3.11	2.29	2.93	2.16	2.76	2.16	1.98
4¾	2.68	3.36	2.54	3.18	2.41	3.01	2.41	2.23
5	2.93	3.61	2.79	3.43	2.66	3.26	2.66	2.48
5½	3.43	4.11	3.29	3.93	3.16	3.76	3.16	2.98
6	3.93	4.61	3.79	4.43	3.66	4.26	3.66	3.48
6½	4.43	5.11	4.29	4.93	4.16	4.76	4.16	3.98
7	4.93	5.61	4.79	5.43	4.66	5.26	4.66	4.48
7½	5.43	6.11	5.29	5.93	5.16	5.76	5.16	4.98
8	5.93	6.61	5.79	6.43	5.66	6.26	5.66	5.48
8½	6.43	7.11	6.29	6.93	6.16	6.76	6.16	5.98
9	6.93	7.61	6.79	7.43	6.66	7.26	6.66	6.48
9½	7.40	8.11	7.27	7.93	7.13	7.76	7.16	6.98
10	7.90	8.61	7.77	8.43	7.63	8.26	7.66	7.48

Notes: For Min Grip and Max Grip, consider the total number of F436 washers used under the head and under the nut.
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"N" indicates bolts that should be designed in the "threads not excluded" or "N" condition only.
Cells above the heavy line represent bolt lengths that are considered to be fully threaded per ASME B18.2.6.
Shaded cells represent bolt lengths that are rarely produced.
Italics values indicate bolt lengths that are available by special order only.
"—" indicates bolt assemblies where the nut may not be able to be threaded fully onto the bolt.

Table A2. Minimum Grip, Maximum Grip, and Thread Condition for High-Strength Bolts (continued)								
	1¼-in.-Diameter Bolts							
	0 Washers		1 Washer		2 Washers		$\Sigma t_{(n-1),max}$	
Bolt Length (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	0 Washers (in.)	1 Washer (in.)
1¼	—	—	—	—	—	—	—	—
1½	—	—	—	—	—	—	—	—
1¾	—	—	—	—	—	—	—	—
2	*0.43*	*0.50*	*0.29*	*0.32*	—	—	*N*	*N*
2¼	*0.43*	*0.75*	*0.29*	*0.57*	*0.16*	*0.40*	*N*	*N*
2½	0.43	1.00	0.29	0.82	0.16	0.65	N	N
2¾	0.43	1.25	0.29	1.07	0.16	0.90	N	N
3	0.93	1.50	0.79	1.32	0.66	1.15	0.62	0.44
3¼	1.18	1.75	1.04	1.57	0.91	1.40	0.87	0.69
3½	1.43	2.00	1.29	1.82	1.16	1.65	1.12	0.94
3¾	1.68	2.25	1.54	2.07	1.41	1.90	1.37	1.19
4	1.93	2.50	1.79	2.32	1.66	2.15	1.62	1.44
4¼	2.18	2.75	2.04	2.57	1.91	2.40	1.87	1.69
4½	2.43	3.00	2.29	2.82	2.16	2.65	2.12	1.94
4¾	2.68	3.25	2.54	3.07	2.41	2.90	2.37	2.19
5	2.93	3.50	2.79	3.32	2.66	3.15	2.62	2.44
5½	3.43	4.00	3.29	3.82	3.16	3.65	3.12	2.94
6	3.93	4.50	3.79	4.32	3.66	4.15	3.62	3.44
6½	4.43	5.00	4.29	4.82	4.16	4.65	4.12	3.94
7	4.93	5.50	4.79	5.32	4.66	5.15	4.62	4.44
7½	5.43	6.00	5.29	5.82	5.16	5.65	5.12	4.94
8	5.93	6.50	5.79	6.32	5.66	6.15	5.62	5.44
8½	6.43	7.00	6.29	6.82	6.16	6.65	6.12	5.94
9	6.93	7.50	6.79	7.32	6.66	7.15	6.62	6.44
9½	7.43	8.00	7.29	7.82	7.16	7.65	7.12	6.94
10	7.93	8.50	7.79	8.32	7.66	8.15	7.62	7.44

Notes: For Min Grip and Max Grip, consider the total number of F436 washers used under the head and under the nut.
For the $\Sigma t_{(n-1),max}$, consider only the number of F436 washers under the head of the bolt.
"N" indicates bolts that should be designed in the "threads not excluded" or "N" condition only.
Cells above the heavy line represent bolt lengths that are considered to be fully threaded per ASME B18.2.6.
Shaded cells represent bolt lengths that are rarely produced.
Italics values indicate bolt lengths that are available by special order only.
"—" indicates bolt assemblies where the nut may not be able to be threaded fully onto the bolt.

Table A2. Minimum Grip, Maximum Grip, and Thread Condition for High-Strength Bolts (continued)								
	1⅜-in.-Diameter Bolts							
	0 Washers		1 Washer		2 Washers		$\Sigma t_{(n-1),max}$	
Bolt Length (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	0 Washers (in.)	1 Washer (in.)
1¼	—	—	—	—	—	—	—	—
1½	—	—	—	—	—	—	—	—
1¾	—	—	—	—	—	—	—	—
2	—	—	—	—	—	—	—	—
2¼	*0.50*	*0.62*	*0.36*	*0.45*	*0.23*	*0.27*	*N*	*N*
2½	*0.50*	*0.87*	*0.36*	*0.70*	*0.23*	*0.52*	*N*	*N*
2¾	*0.50*	*1.12*	*0.36*	*0.95*	*0.23*	*0.77*	*N*	*N*
3	*0.50*	*1.37*	*0.36*	*1.20*	*0.23*	*1.02*	*N*	*N*
3¼	*0.50*	*1.62*	*0.36*	*1.45*	*0.23*	*1.27*	*N*	*N*
3½	*1.17*	*1.87*	*1.03*	*1.70*	*0.89*	*1.52*	*0.81*	*0.63*
3¾	*1.42*	*2.12*	*1.28*	*1.95*	*1.14*	*1.77*	*1.06*	*0.88*
4	*1.67*	*2.37*	*1.53*	*2.20*	*1.39*	*2.02*	*1.31*	*1.13*
4¼	*1.92*	*2.62*	*1.78*	*2.45*	*1.64*	*2.27*	*1.56*	*1.38*
4½	*2.17*	*2.87*	*2.03*	*2.70*	*1.89*	*2.52*	*1.81*	*1.63*
4¾	*2.42*	*3.12*	*2.28*	*2.95*	*2.14*	*2.77*	*2.06*	*1.88*
5	*2.67*	*3.37*	*2.53*	*3.20*	*2.39*	*3.02*	*2.31*	*2.13*
5½	*3.17*	*3.87*	*3.03*	*3.70*	*2.89*	*3.52*	*2.81*	*2.63*
6	*3.67*	*4.37*	*3.53*	*4.20*	*3.39*	*4.02*	*3.31*	*3.13*
6½	*4.17*	*4.87*	*4.03*	*4.70*	*3.89*	*4.52*	*3.81*	*3.63*
7	*4.67*	*5.37*	*4.53*	*5.20*	*4.39*	*5.02*	*4.31*	*4.13*
7½	*5.17*	*5.87*	*5.03*	*5.70*	*4.89*	*5.52*	*4.81*	*4.63*
8	*5.67*	*6.37*	*5.53*	*6.20*	*5.39*	*6.02*	*5.31*	*5.13*
8½	*6.17*	*6.87*	*6.03*	*6.70*	*5.89*	*6.52*	*5.81*	*5.63*
9	*6.67*	*7.37*	*6.53*	*7.20*	*6.39*	*7.02*	*6.31*	*6.13*
9½	*7.17*	*7.87*	*7.03*	*7.70*	*6.89*	*7.52*	*6.81*	*6.63*
10	*7.67*	*8.37*	*7.53*	*8.20*	*7.39*	*8.02*	*7.31*	*7.13*

Notes: For Min Grip and Max Grip, consider the total number of F436 washers used under the head and under the nut.
For the $\Sigma t_{(n-1),max}$, consider only the number of F436 washers under the head of the bolt.
“N” indicates bolts that should be designed in the “threads not excluded” or “N” condition only.
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Shaded cells represent bolt lengths that are rarely produced.
Italics values indicate bolt lengths that are available by special order only.
“—” indicates bolt assemblies where the nut may not be able to be threaded fully onto the bolt.

Table A2. Minimum Grip, Maximum Grip, and Thread Condition for High-Strength Bolts (continued)								
	1½-in.-Diameter Bolts							
	0 Washers		1 Washer		2 Washers		$\Sigma t_{(n-1),max}$	
Bolt Length (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	Min Grip (in.)	Max Grip (in.)	0 Washers (in.)	1 Washer (in.)
1¼	—	—	—	—	—	—	—	—
1½	—	—	—	—	—	—	—	—
1¾	—	—	—	—	—	—	—	—
2	—	—	—	—	—	—	—	—
2¼	—	—	—	—	—	—	—	—
2½	*0.50*	*0.75*	*0.36*	*0.57*	*0.23*	*0.39*	*N*	*N*
2¾	*0.50*	*1.00*	*0.36*	*0.82*	*0.23*	*0.64*	*N*	*N*
3	*0.50*	*1.25*	*0.36*	*1.07*	*0.23*	*0.89*	*N*	*N*
3¼	0.50	1.50	0.36	1.32	0.23	1.14	N	N
3½	1.17	1.75	1.03	1.57	0.89	1.39	0.81	0.63
3¾	1.42	2.00	1.28	1.82	1.14	1.64	1.06	0.88
4	1.67	2.25	1.53	2.07	1.39	1.89	1.31	1.13
4¼	1.92	2.50	1.78	2.32	1.64	2.14	1.56	1.38
4½	2.17	2.75	2.03	2.57	1.89	2.39	1.81	1.63
4¾	2.42	3.00	2.28	2.82	2.14	2.64	2.06	1.88
5	2.67	3.25	2.53	3.07	2.39	2.89	2.31	2.13
5½	3.17	3.75	3.03	3.57	2.89	3.39	2.81	2.63
6	3.67	4.25	3.53	4.07	3.39	3.89	3.31	3.13
6½	4.17	4.75	4.03	4.57	3.89	4.39	3.81	3.63
7	4.67	5.25	4.53	5.07	4.39	4.89	4.31	4.13
7½	5.17	5.75	5.03	5.57	4.89	5.39	4.81	4.63
8	5.67	6.25	5.53	6.07	5.39	5.89	5.31	5.13
8½	6.17	6.75	6.03	6.57	5.89	6.39	5.81	5.63
9	6.67	7.25	6.53	7.07	6.39	6.89	6.31	6.13
9½	7.17	7.75	7.03	7.57	6.89	7.39	6.81	6.63
10	7.67	8.25	7.53	8.07	7.39	7.89	7.31	7.13

Notes: For Min Grip and Max Grip, consider the total number of F436 washers used under the head and under the nut.
For the $\Sigma t_{(n-1),max}$, consider only the number of F436 washers under the head of the bolt.
"N" indicates bolts that should be designed in the "threads not excluded" or "N" condition only.
Cells above the heavy line represent bolt lengths that are considered to be fully threaded per ASME B18.2.6.
Shaded cells represent bolt lengths that are rarely produced.
Italics values indicate bolt lengths that are available by special order only.
"—" indicates bolt assemblies where the nut may not be able to be threaded fully onto the bolt.

A Reliability Study of Joints With Bolts Designed With Threads Excluded but Installed With Threads Not Excluded

JAMES A. SWANSON, GIAN ANDREA RASSATI, and CHAD M. LARSON

ABSTRACT

This paper presents a reliability and probability study focusing on connections using relatively short bolts that have been shown in a companion paper to have the potential to have been designed with threads excluded from the shear plane and then subsequently installed with the threads not excluded from the shear plane. After an introduction outlining the background of the shear strength and associated design of joints in various editions of the AISC *Specification*, the paper presents a structural reliability analysis as well as a probability study using Monte Carlo simulations; finally, the paper discusses additional considerations and mitigating factors associated with this potential problem. Calculated reliability coefficients and probabilities of failure are tabulated for joints using two diameter groups of 120-ksi bolts (from ⅝ in. to 1 in. and from 1⅝ in. to 1¼ in.) and for joints using 150-ksi bolts. The paper provides an evaluation of the reliability of joints with bolts that have been designed with the threads excluded from the shear plane but installed with the threads not excluded from the shear plane. Although it is recommended that future designs involving short bolts be based on the assumption that the threads are not excluded from the shear plane, this study provides a measure of the reliability of structures that have already been constructed with bolts designed assuming that the threads were excluded but installed with the threads not excluded. The results show that the reliability of joints in this class is dependent on the grade and size of the bolts used, on the length of the joint, and on which edition of the AISC *Specification* was used for design. It was found that some joints in this class still meet the AISC target reliability for connections and that many joints meet the AISC target reliability for members.

Keywords: structural bolt, high-strength bolt, fastener, A325, A490, F1852, F2280, F3125, threads excluded, threads not excluded, shear.

INTRODUCTION

When designing a bolted joint for shear forces, one of the first determinations that is made is whether the bolts will be designed with the shear plane of the joint passing through the shank or through the threads of the bolts. In the former case where the threads are excluded from the shear plane (the X condition), the bolts have an available strength that is higher than in the latter case where the threads are not excluded from the shear plane (the N condition). To determine whether or not threads will be excluded from the shear plane, engineers and detailers often consult Table 7-14 of the AISC *Steel Construction Manual* (AISC, 2017), hereafter referred to as the AISC *Manual*, or Tables C-2.1 or C-2.2 of the Research Council on Structural Connections (RCSC) *Specification* (RCSC, 2015). In these tables, the thread lengths of structural bolts are presented as constant for each given bolt diameter. Structural bolts, however, are manufactured to conform to the American Society of Mechanical Engineers (ASME) B18.2.6 Standard (ASME, 2019) where the thread length is presented as a reference dimension and not a control dimension, which means that the thread length may not be exactly equal to the values shown in tables.

This situation often manifests itself in structural bolts that are relatively short but still longer than the reference thread length for their diameter, where the bolt may not have an unthreaded shank at all despite what is implied in some design aids. Figure 1 shows three different ⅞-in. × 2-in. bolts that were made by three different producers, all in compliance with ASME standards. Structural bolts that are ⅞ in. in diameter have a published thread length of 1½ in., which leads to the assumption that a bolt that is 2 in. long should have an unthreaded shank that is ½ in. long. Provisions in the ASME standard, however, require this bolt to be manufactured as fully threaded, with a short unthreaded body with a diameter smaller than the nominal bolt diameter, or with an unthreaded shank with a diameter equal to the full nominal diameter of the bolt. This issue was addressed in detail in a previous work (Swanson et al., 2019); Table 1 lists bolts that may be affected by this issue.

In addition to the issues noted for relatively short bolts, it is possible for bolts of any length to have geometric

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Table 1. Bolt Lengths Considered to Be Fully Threaded Per ASME B18.2.6	
Nominal Bolt Diameter, d_b	Fully Threaded Bolt Lengths
½ in.	$L \leq$ 1¼ in.
⅝ in.	$L \leq$ 1½ in.
¾ in.	$L \leq$ 1¾ in.
⅞ in.	$L \leq$ 2 in.
1 in.	$L \leq$ 2¼ in.
1⅛ in.	$L \leq$ 2¾ in.
1¼ in.	$L \leq$ 2¾ in.
1⅜ in.	$L \leq$ 3¼ in.
1½ in.	$L \leq$ 3¼ in.

deviations from nominal reference dimensions. A common misunderstanding is that the length of the unthreaded shank or body of a bolt can be computed simply by subtracting the reference thread length from the overall length of the bolt. This is incorrect, though, because of tolerances on the length and transition length of the bolt. A footnote to Table 7-14 of the AISC *Manual* provides a reference to tolerances in ASME B18.2.6, where minimum body lengths (as well as other dimensions) are found. The minimum body length, as a control dimension, is not permitted to be smaller than published values. A ⅞-in. × 2½-in. bolt, for example, might be expected to have a shank length of 2½ in. − 1½ in. = 1 in. computed by subtracting the thread length from the overall length. Consultation with Table 2 of ASME B18.2.6, however, shows a minimum body length, $L_{B,min}$, of 0.72 in. for that bolt. While it is sometimes argued that a bolt sheared through its transition region demonstrates behavior comparable to a bolt sheared through its body; that may not actually be the case.

Fig. 1. Three ⅞-in.-9×2-in.-bolts made by three different manufacturers.

This situation presents a potential problem for engineers and owners with structures already constructed with short bolts that have been designed in the threads excluded condition. Bolts that were designed to carry shear forces through their shanks may actually be carrying shear forces through their threads or transitions, meaning that they will likely have an available strength that is lower than expected. Both the AISC *Specification for Structural Steel Buildings* (AISC, 2016) and the RCSC *Specification* include a factor of 0.80 for bolts designed in the threads not excluded condition, thus bolts designed as X but installed as N may have 20% less strength than expected. While this difference is less than the margin afforded by the resistance factor of 0.75 that is used in the design of bolted joints,[1] this may not be a sufficient remedy in some circumstances.

It should be noted that this issue has existed for nearly four decades; it can be traced back to at least the 1981 edition of ASME B18.2.1 (ASME, 1981). Bolts designed with threads excluded from the shear plane have been discovered during installation to be fully threaded on numerous occasions, often resulting in replacement of the fasteners with longer ones, often at great expense. While it is suspected that this variation has likely gone undiscovered in the design and construction of numerous other structures, the authors are not aware of a structural failure that has resulted from this specific issue. While this issue can certainly result in designs that are unconservative, those designs might actually be only less conservative but still adequate when compared to actual demand. Application of resistance factors, documented overstrength of fasteners, conservatism built into design equations, and other factors mitigate the risk

1 Note that there are several differences between the AASHTO LRFD specification (AASHTO, 2017) relative to the AISC and RCSC specifications, including the use of a resistance factor of 0.80 instead of 0.75. These issues will be addressed briefly later in the paper.

associated with these incorrect design assumptions. The following sections aim to quantify the reliability and risk associated with this issue.

BACKGROUND OF BOLT SHEAR STRENGTH

The most fundamental form of the design equation for a connection, written in terms of load and resistance factor design (LRFD), is

$$\phi R_n \geq \Sigma \gamma Q \tag{1}$$

where

Q = applied load or load effect, kips
R_n = nominal resistance, kips
γ = load factor
ϕ = resistance factor

On the left-hand side of the equation, we find the factored resistance, ϕR_n, and this is a logical place to start. There are numerous limit states for a bolted joint where the bolts are subjected to shear, including the shear strength of the bolt, the bearing and/or tear-out strength of the material around the bolt, block shear, a net section fracture, and possibly joint slip depending on the connection type and the requirements of the engineer of record.

Bolt Shear Strength

The nominal shear strength of a bolt, r_{nv}, in its fundamental form can be shown as (Kulak et al., 1987; Tide, 2010)

$$r_{nv} = A_b F_{u,bolt} R_v R_{nx} R_j R_{lj} \tag{2}$$

where:

A_b = nominal unthreaded body area of the bolt, in.^{2}
$F_{u,bolt}$ = ultimate tensile strength of the bolt material, ksi
R_j = length reduction factor for joints with $L \leq 38$ in.
R_{lj} = length reduction factor for joints with $L > 38$ in.
R_{nx} = thread condition factor (X vs N)
R_v = ratio of shear strength to tensile strength

This form of the nominal strength equation includes the area of the shank of the bolt, the ultimate tensile strength of the bolt material, and four reduction factors. A value of 0.625 is used for the first reduction factor, R_v, which is the ratio of the shear strength of the bolt material to the tensile strength of the bolt material (AISC, 2016). The second reduction factor, R_{nx}, accounts for the thread condition of the bolts in the joint; a value of $R_{nx} = 1.00$ is used if the threads are excluded (X) from the shear plane of the joint, and a value of $R_{nx} = 0.80$ is used when the threads are not excluded (N) from the shear plane. A value of 0.90 is used for the third factor, R_j, which represents a reduction in the strength of the bolts due to unequal distribution of shear force among the bolts in typical joints having lengths up to 38 in. Finally, R_{lj} is taken as 1.00 for joints with a length up to and including 38 in., but a value of $R_{lj} = 0.80$ is used for joints longer than 38 in. The factor R_{lj} is similar to the factor R_j but accounts for a more pronounced inequality in the distribution of shear forces among the bolts in longer joints.

According to the 2016 edition of the AISC *Specification*, for bolts in a joint with a length not exceeding 38 in., where the threads are excluded from the shear plane (X), $R_v = 0.625$, $R_{nx} = 1.00$, $R_j = 0.90$, and $R_{lj} = 1.00$. For bolts in a joint that has a length not exceeding 38 in., where the threads are not excluded from the shear plane (N), $R_v = 0.625$, $R_{nx} = 0.80$, $R_j = 0.90$, and $R_{lj} = 1.00$.

Threads excluded, X:

$$r_{nv} = 0.563 A_b F_{u,bolt} \tag{3}$$

Threads not excluded, N:

$$r_{nv} = 0.450 A_b F_{u,bolt} \tag{4}$$

All bolts:

$$r_{nv} = A_b F_{u,bolt} \tag{5}$$

where

F_{nv} = the nominal strength per unit area of the bolt in shear, ksi

$$= R_v R_{nx} R_j R_{lj} F_{u,bolt} \tag{6}$$

The first two factors, R_v and R_{nx}, address the strength of individual bolts and the variability of those strengths, whereas the third and fourth factors, R_j and R_{lj}, address the performance of bolt groups in joints and the variability of those joints.

The Strength of Individual Bolts in Shear with Threads Not Excluded

Three research studies have been conducted at the University of Cincinnati since 2008 wherein the strength of individual bolts was measured (Moore et al., 2008; Taylor et al., 2008; Roenker et al., 2017). These studies included ASTM A325, A325T, A490, F1852, and F2280 bolts of various diameters manufactured by approximately a dozen different producers.[2] A summary of the grades and diameters of bolts tested is shown in Table 2. All bolts considered herein were tested in single shear with the shear plane passing through the threads. For the sake of clarity, A325, A325T, and F1852 bolts are treated as equivalent and are collectively referred to as Group 120. Similarly, A490 (ASTM, 2014b) and F2280 (ASTM, 2014d) bolts are also treated as equivalent and are collectively referred to as Group 150.

The results of the bolt tests are summarized in Figures 2 through 5 where the strength of bolts divided by the

2 Test results that are referenced in this work were obtained for bolts that were produced before the ASTM F3125 (2018) standard was officially adopted.

Table 2. Number of Each Grade and Diameter of Bolts Tested with Threads Not Excluded					
	Bolt Grade				
Diameter, in.	A325	A325T	F1852	A490	F2280
½	0	0	0	0	0
⅝	20	15	5	30	0
¾	86	50	10	64	15
⅞	100	15	5	104	15
1	79	20	5	92	15
1⅛	21	10	5	25	5
1¼	20	25	0	30	0
1⅜	0	0	0	0	0
1½	0	0	0	0	0

nominal area of their unthreaded shanks is shown as histograms. Figure 2 shows data for Group 120 bolts sized ⅝ in. through 1¼ in. It was observed, however, that the strength of Group 120 bolts exhibited a bimodal distribution where the strength of smaller diameter bolts sized ⅝ in. through 1 in., shown in Figure 3, was noted to be higher than that of larger diameter bolts sized 1⅛ in. through 1¼ in., shown in Figure 4. When the larger-diameter Group 120 bolts were considered separately as a subset of the complete dataset, a lower mean strength and lower standard deviation were observed for this class of fasteners. This difference may be related to challenges associated with hardening the larger diameter bolts and may also be a reflection of the fact that tests were performed on bolts manufactured prior to the adoption of ASTM F3125 (2018), which requires that Grade A325 and A325TC bolts larger than 1 in. in diameter have a minimum strength of F_u = 120 ksi. Prior to the adoption of ASTM F3125, ASTM A325 (2014a) and ASTM F1852 (2014c) required that bolts larger than 1 in. in diameter have a minimum tensile strength of only F_u = 105 ksi. A bimodal distribution was not observed in the data for Group 150 bolts, shown in Figure 5.

It was also noted that the mean shear strength of Group 150 bolts, 74.6 ksi, was slightly lower than the minimum required value of $F_{u,bolt}R_vR_{nx}$ = (150 ksi)(0.625)(0.80) = 75.0 ksi. In fact, more than half of the Group 150 bolts that were tested (211 of 395) had a shear strength less than 75.0 ksi. In contrast, the average shear strength of the Group 120 bolts was greater than the minimum required value of (120 ksi)(0.625)(0.80) = 60.0 ksi, and only 10 of 491 Group 120 bolts tested

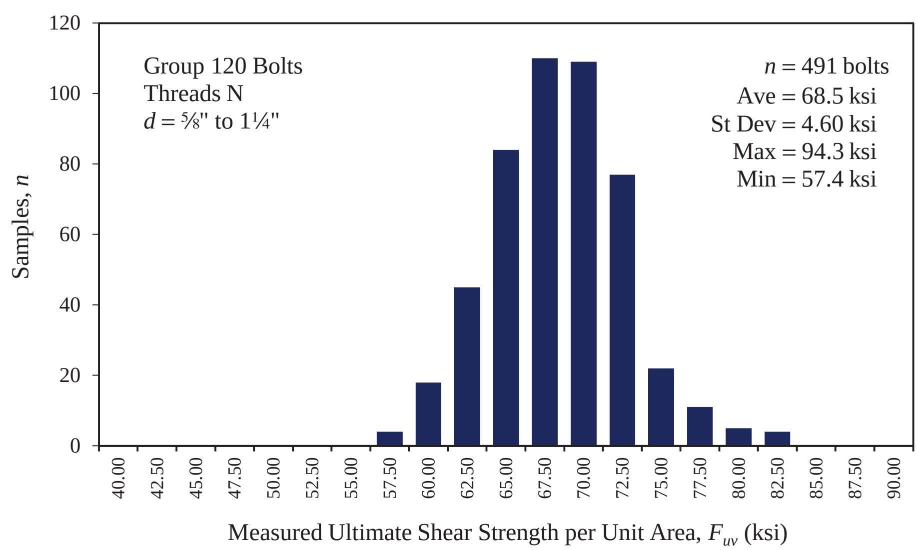


Fig. 2. Shear strength per unit area of Group 120 bolts tested with threads not excluded.

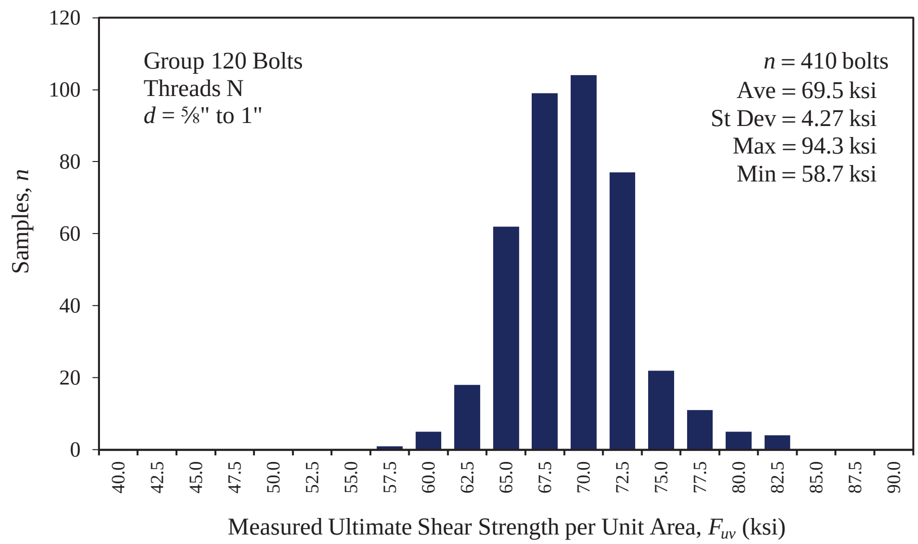


Fig. 3. Shear strength per unit area of smaller-diameter Group 120 bolts tested with threads not excluded.

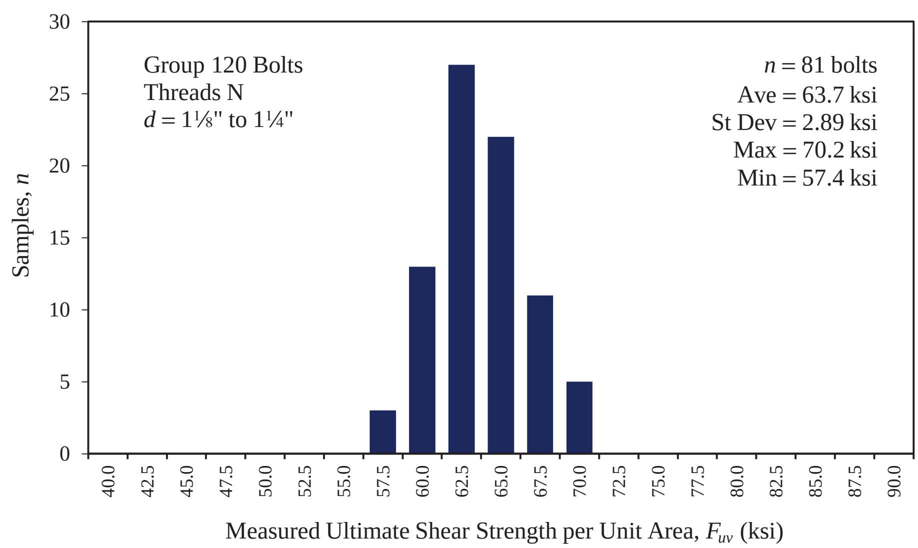


Fig. 4. Shear strength per unit area of larger-diameter Group 120 bolts tested with threads not excluded.

had a strength less than 60.0 ksi. This is thought to be a reflection of the fact that, unlike Group 120 bolts, Group 150 bolts have a maximum specified tensile strength, which results in Group 150 bolts being manufactured with a strength that is closer to their minimum specified strength than Group 120 bolts. This is also evidence that the value of $R_{nx} = 0.80$ might possibly be too large. This reduction factor was proposed as 0.75 by Fisher and Struik (1974), was then proposed as 0.70 by Kulak et al. (1987), was reported as 0.833 by Frank and Yura (1981), and was then reported as 0.762 by Moore et al. (2008; 2010). Early versions of the RCSC *Specification* incorporated a value of approximately 0.68 (RCSC, 1964) but this was increased slightly to 0.70 in the 1976 edition (RCSC, 1976). The value of 0.80 was adopted with the release of the LRFD version of the RCSC *Specification* in 1988 (RCSC, 1988), though the value of 0.70 was retained in the 1985 and 1994 editions of the ASD version of the RCSC *Specification* (RCSC, 1985). This evolution was mirrored in the AISC *Specification* development with the exception that a value of 0.75 was adopted in the first edition of the AISC LRFD *Specification* (AISC, 1986), but this was increased to 0.80 in the second edition (AISC, 1993).

Consideration of the Effect of Joint Length

In most designs, the strength of a group of bolts is taken as the sum of the individual strengths of each bolt in the group. It has been shown through testing of end-loaded bolted shear joints, however, that the load transferred through the joint may not be equally shared among the bolts in the joint. Kulak et al. (1987) reported that the bolts near the beginning and end of a shear joint tend to experience loads that are higher than the average, while bolts near the middle of the joint tend to experience loads that are lower than the average. Kulak et al. further reported that the extent of the nonuniform load distribution was more severe in longer joints than in shorter joints. Kulak et al. recommended that a reduction factor, referred to as R_j herein, of 0.80 be applied to the computed strength of bolts in all joints and that a second reduction factor, referred to as R_{lj} herein, of 0.80 be applied to the computed strength of bolts in joints that have a length greater than 50 in.

The recommendations of Kulak et al. are reflected in AISC *Specifications* up to and including the 2005 edition (AISC, 2005). The reduction factors were modified in the 2010 edition of the AISC *Specification* (AISC, 2010), though, based in part on work by Tide (2010), such that the factor R_j of 0.90 is applied to joints of all lengths and the factor R_{lj} of 0.833 is applied to joints exceeding 38 in. in length, as shown in Figure 6. The limit of $L > 38$ in. was chosen based on the notion that few joints have a length equal to 38 in. using typical bolt spacings.

It should be noted, however, that Kulak et al. (1987) reported that the distribution of load within a bolt group is approximately uniform for short or compact bolt groups that have a length up to approximately 10 in. The commentary to the 2016 AISC *Specification* states that, "[i]n connections consisting of only a few fasteners and length not exceeding approximately 16 in. the effect of differential strain on the shear in bearing fasteners is negligible." Further, the commentary goes on to say that, "[f]or shear-type connections used in beams and girders with lengths greater than 38 in., there is no need to make the second reduction," and provides examples shown in Figure 7 as references.

In this paper, compact joints will be defined as joints with a length, L, less than or equal to 16 in. Conversely, long

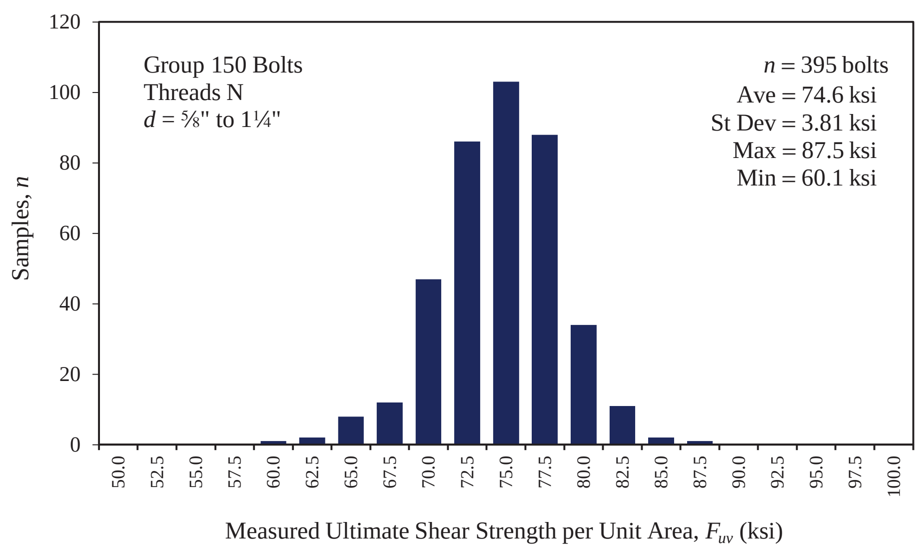


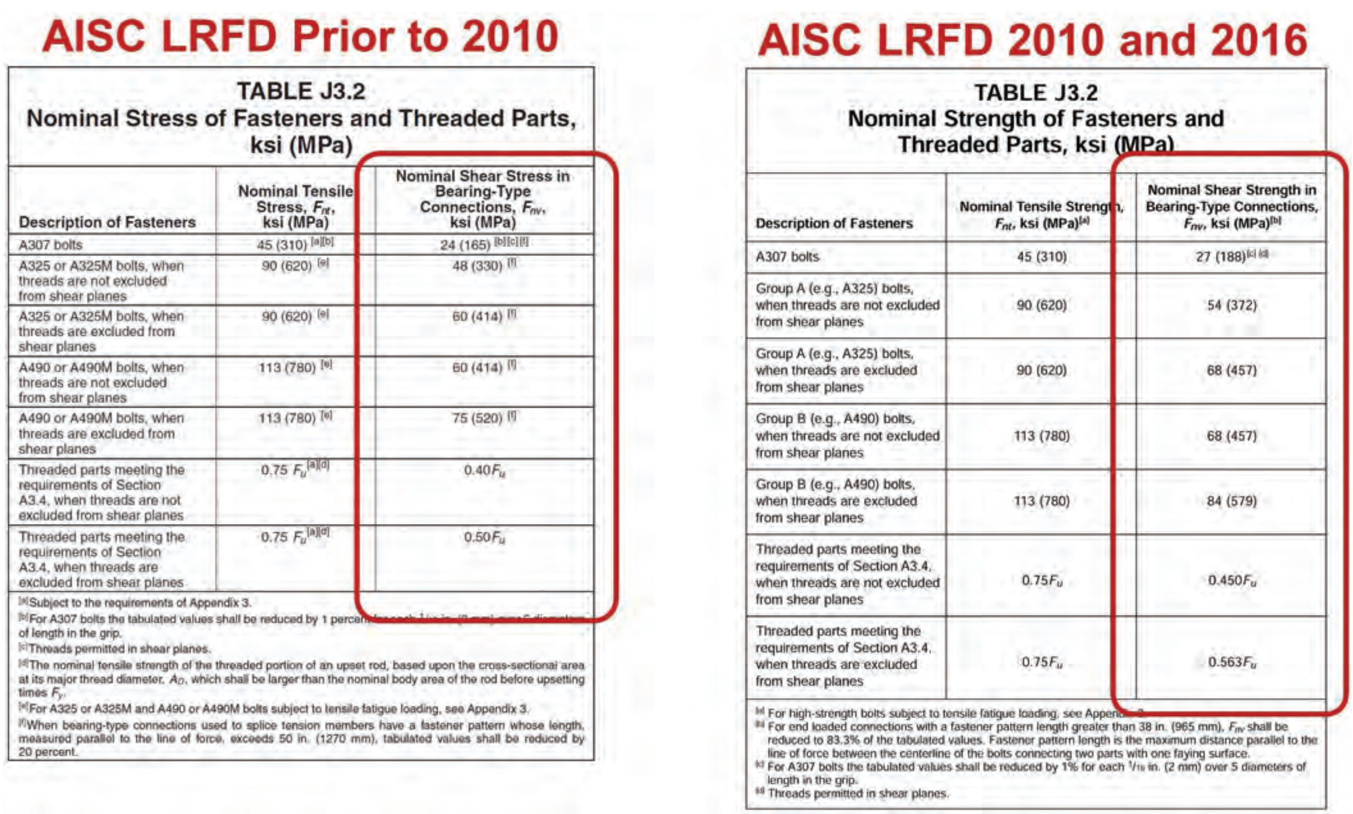
Fig. 5. Shear strength per unit area of Group 150 bolts tested with threads not excluded.

Table 3. Joint Efficiency, R_jR_{lj}, Based on Distribution of Bolt Forces in Shear Joints (Tide, 2010)				
	AISC *Specification* Prior to 2010		AISC *Specification* 2010 and 2016	
	Mean	Standard Deviation	Mean	Standard Deviation
Compact	0.970	0.045	0.970	0.045
Intermediate	0.831	0.122	0.834	0.101
Long	0.820	0.160	0.821	0.163
All	0.881	0.598	0.881	0.598

joints will be defined herein as those joints with a length greater than 50 in. when taken in the context of AISC *Specifications* prior to 2010 and joints with a length greater than 38 in. when taken in the context of the 2010 or 2016 AISC *Specifications*. Joints that are neither compact nor long are defined as intermediate length.

In 2010, Tide (2010) presented a discussion of the effects of nonuniform loading of bolts in joints. Tide summarized 72 experiments wherein the value of R_jR_{lj} was measured for joints ranging in length from 3.50 in. to 94.0 in., shown in Figure 8. Twenty-seven of those experiments were of joints with lengths up to and including 14 in., comprising from two to five rows of bolts arranged perpendicular to the applied load. For those compact joints, the average value of R_jR_{lj} was found to be 0.970 with a standard deviation of 0.045. These values are shown in Table 3 along with means and standard deviations for compact, intermediate, and long joints as the joint efficiency, which is defined within this context as the ratio of the joint strength to the sum of the strengths of the individual bolts in the joint.

Tide presented an equation for R_jR_{lj}, shown in Equation 7 and Figure 8, that is a linear function of the joint length, L.



AISC LRFD Prior to 2010

TABLE J3.2 Nominal Stress of Fasteners and Threaded Parts, ksi (MPa)		
Description of Fasteners	Nominal Tensile Stress, F_{nt}, ksi (MPa)	Nominal Shear Stress in Bearing-Type Connections, F_{nv}, ksi (MPa)
A307 bolts	45 (310) [a][b]	24 (165) [b][c][f]
A325 or A325M bolts, when threads are not excluded from shear planes	90 (620) [e]	48 (330) [f]
A325 or A325M bolts, when threads are excluded from shear planes	90 (620) [e]	60 (414) [f]
A490 or A490M bolts, when threads are not excluded from shear planes	113 (780) [e]	60 (414) [f]
A490 or A490M bolts, when threads are excluded from shear planes	113 (780) [e]	75 (520) [f]
Threaded parts meeting the requirements of Section A3.4, when threads are not excluded from shear planes	$0.75\ F_u$ [a][d]	$0.40F_u$
Threaded parts meeting the requirements of Section A3.4, when threads are excluded from shear planes	$0.75\ F_u$ [a][d]	$0.50F_u$

[a] Subject to the requirements of Appendix 3.
[b] For A307 bolts the tabulated values shall be reduced by 1 percen[illegible] of length in the grip.
[c] Threads permitted in shear planes.
[d] The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, A_D, which shall be larger than the nominal body area of the rod before upsetting times F_y.
[e] For A325 or A325M and A490 or A490M bolts subject to tensile fatigue loading, see Appendix 3.
[f] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in. (1270 mm), tabulated values shall be reduced by 20 percent.

AISC LRFD 2010 and 2016

TABLE J3.2 Nominal Strength of Fasteners and Threaded Parts, ksi (MPa)		
Description of Fasteners	Nominal Tensile Strength, F_{nt}, ksi (MPa)[a]	Nominal Shear Strength in Bearing-Type Connections, F_{nv}, ksi (MPa)[b]
A307 bolts	45 (310)	27 (188)[c] [d]
Group A (e.g., A325) bolts, when threads are not excluded from shear planes	90 (620)	54 (372)
Group A (e.g., A325) bolts, when threads are excluded from shear planes	90 (620)	68 (457)
Group B (e.g., A490) bolts, when threads are not excluded from shear planes	113 (780)	68 (457)
Group B (e.g., A490) bolts, when threads are excluded from shear planes	113 (780)	84 (579)
Threaded parts meeting the requirements of Section A3.4, when threads are not excluded from shear planes	$0.75F_u$	$0.450F_u$
Threaded parts meeting the requirements of Section A3.4, when threads are excluded from shear planes	$0.75F_u$	$0.563F_u$

[a] For high-strength bolts subject to tensile fatigue loading, see Appen[illegible]
[b] For end loaded connections with a fastener pattern length greater than 38 in. (965 mm), F_{nv} shall be reduced to 83.3% of the tabulated values. Fastener pattern length is the maximum distance parallel to the line of force between the centerline of the bolts connecting two parts with one faying surface.
[c] For A307 bolts the tabulated values shall be reduced by 1% for each $^1/_{16}$ in. (2 mm) over 5 diameters of length in the grip.
[d] Threads permitted in shear planes.

Fig. 6. Comparison of design shear strength in the 2005 ($R_j = 0.80$*) and 2010 (*$R_j = 0.90$*) AISC* Specifications.

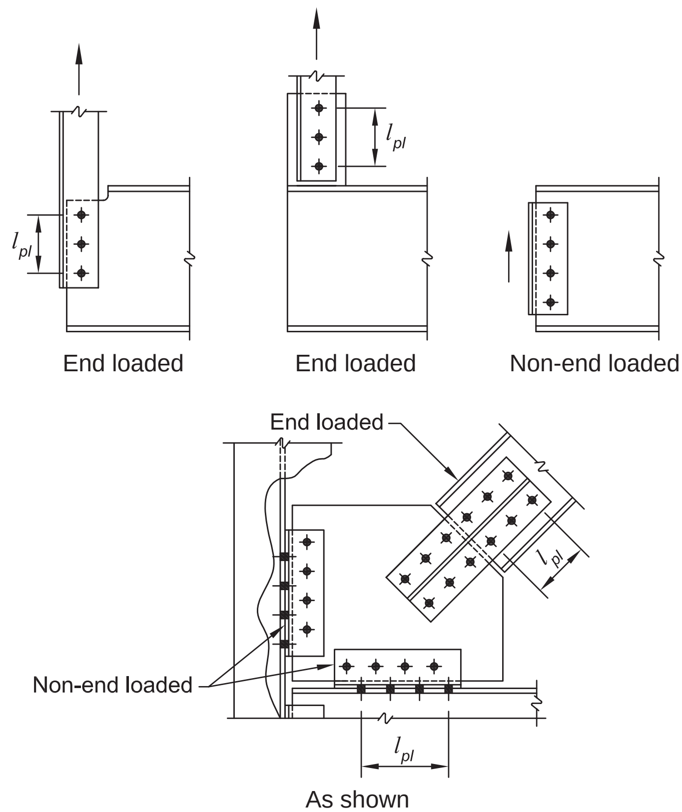


Fig. 7. Examples of joints that are end-loaded and joints that are non-end-loaded (AISC, 2016).

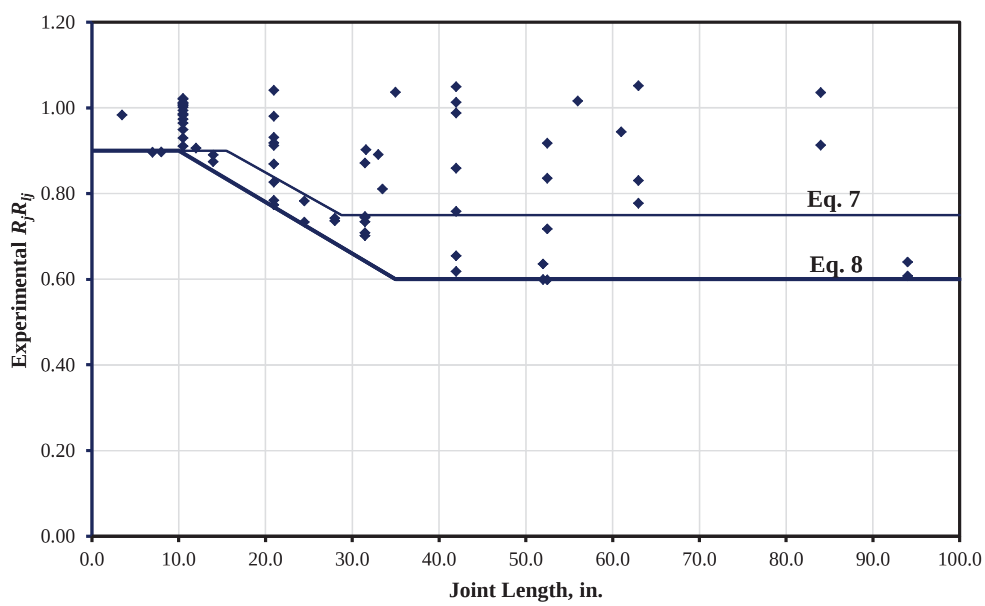


Fig. 8. Experimentally measured joint efficiency, R_jR_{lj} *(Tide, 2010).*

When evaluated against the 72 data points, the ratio of the experimental data to the value predicted by the equation is on average 1.05 with a standard deviation 0.146. The equation proved to match the experimental data very well for compact joints but demonstrated more deviation for longer joints, particularly for joints in excess of approximately 40 in. long, where the ratio was as low as 0.80 for some joints and as high as 1.40 for others.

$$R_l R_{lj} = 1.075 - 0.0113L \tag{7}$$

where $0.75 \le R_l R_{lj} < 0.90$

Recognizing the iterative nature and challenges associated with designing a joint where the strength is a function of both the number of bolts and the length of the joint, Tide ultimately recommended that $R_j R_{lj}$ be taken as 0.90 for joints not longer than 38 in. and that $R_j R_{lj}$ be taken as 0.75 for joints longer than 38 in.

A second equation proposed herein can be used to represent the lower bound of the data in Tide (2010). This equation, shown in Equation 8 and Figure 8, provides a lower bound of the measured data within 1%.

$$R_l R_{lj} = 1.02 - 0.0120L \tag{8}$$

where $0.60 \le R_l R_{lj} < 0.90$

DISCUSSION

For a bolt in a joint designed assuming that the threads would be excluded but installed with the threads not excluded, the ratio of the as-installed strength to the as-designed strength would be

$$\frac{\text{Strength N}}{\text{Strength X}} = \frac{R_v(0.80)R_j R_{lj} A_b F_{u,bolt}}{R_v(1.00)R_j R_{lj} A_b F_{u,bolt}} = 0.80 \tag{9}$$

This represents a difference of 20%, and while at first glance it might seem that bolted joints designed as X but installed as N must always be understrength, this is not necessarily the case. While the location of the shear plane is one of the more critical parameters affecting the strength of bolts in shear joints, other factors may partially or completely offset the effect of shear plane location. These other factors include inherent overstrength of bolt materials, conservative approaches used to account for the distribution of forces among the bolts in a joint, rounding up the number of bolts required in a joint, and designing for connection forces that are conservative.

As an example, the ultimate shear strength per unit area of a bolt sheared through its shank, not considering joint length effects, would be

$$\begin{aligned} F_{nv} &= (0.625)(1.00)F_{u,bolt} \\ &= 0.625F_{u,bolt} \end{aligned} \tag{10}$$

For a smaller-diameter Group 120 bolt, this would be (0.625)(120 ksi) = 75.0 ksi. The average measured shear strength per unit area from Figure 3 is 69.5 ksi, which is approximately 93% of the strength assumed during design. Of the 410 bolts in the category that were tested, 33 had a measured strength greater than or equal to 75 ksi. It is also noted, however, that there were two bolts with a measured strength below 60 ksi, which is the nominal design strength of a single Group 120 bolt with the threads not excluded.

As a second example, based on the rationale for compact joints described earlier, the strength of bolts in compact joints (joints with $L \le 16$ in.), where the threads are not excluded from the shear plane (N), taking $R_v = 0.625$, $R_{nx} = 0.80$, $R_j = 1.00$, and $R_{lj} = 1.00$, can be expected to be approximately

$$r_{nv} = A_b F_{u,bolt} R_v R_{nx} R_j R_{lj} \tag{2}$$

$$= A_b F_{u,bolt}(0.625)(0.80)(1.00)(1.00)$$

$$= 0.500 A_b F_{u,bolt} \tag{11}$$

For joints designed using the AISC *Specification* prior to 2010, this is the same as the strength of bolts designed as X. For joints designed using the 2010 or 2016 AISC *Specification*, this is approximately 89% of the nominal strength of bolts designed as X. In the former case, there would be no deficiency in the design of a joint designed as X but installed as N. In the latter case, the 11% difference in nominal strength would be offset by the resistance factor to be applied to the joint.

Finally, considering that the resistance factor assigned to the bolt fracture limit state is 0.75, the ratio of nominal strength in the N condition to design strength in the X condition is 1.07, suggesting that the resistance factor may offset a strength deficiency due to inaccurate assumptions regarding thread condition.

$$\frac{\text{Nominal strength N}}{\text{Design strength X}} = \frac{R_v(0.80)R_j R_{lj} A_b F_{u,bolt}}{(0.75)R_v(1.00)R_j R_{lj} A_b F_{u,bolt}} = 1.07 \tag{12}$$

While these anecdotal examples show that a bolt designed as X but installed as N may have sufficient strength to be considered adequate without the need for remedial action, basing decisions on anecdotal evidence alone may not be appropriate in all cases. A more rational approach to quantifying the safety of joints with bolts designed as X but installed as N is to conduct a structural reliability analysis.

A Structural Reliability Analysis

It can be shown that there is a high likelihood that joints with bolts designed as X but installed as N will actually have a strength lower than expected during design. However, that alone does not mean that the joint will fail. In order for a failure to occur, the available strength of the joint has to be lower than the demand on the joint.

Equation 13 has been a generally accepted fundamental basis of the LRFD approach for steel structures for many years (Fisher et al., 1978):

$$\beta = \frac{\ln\left(\dfrac{R_m}{Q_m}\right)}{\sqrt{V_R^2 + V_Q^2}} \tag{13}$$

where

Q_m = mean value of load, kips

R_m = mean value of resistance, kips

V_R = coefficient of variation of resistance

V_Q = coefficient of variation of load

β = reliability index

In the following example, results of 410 tests of smaller-diameter (⅝ in. to 1 in.) Group 120 bolts sheared in the N condition conducted at the University of Cincinnati are used to illustrate the procedure followed to calculate the reliability indices as part of this study. Based on the measured shear strengths, the mean value, standard deviation, and coefficient of variation can be calculated as

$$R_B = \left(\frac{V_{ult}}{A_{b,nom}}\right)_{mean} \tag{14}$$
$$= 69.5 \text{ ksi}$$

$$\sigma_B = 4.63 \text{ ksi}$$

$$V_B = 0.061$$

For example, during one experiment, a ⅞-in.-diameter A325 bolt was tested in single shear with the threads not excluded and failed at a load of 41.8 kips. For this bolt

$$A_{b,nom} = \left(\frac{\pi}{4}\right)\left(\frac{7}{8}\text{ in.}\right)^2$$
$$= 0.601 \text{ in.}^2$$

$$F_{uv} = \frac{41.8 \text{ kips}}{0.601 \text{ in.}^2}$$
$$= 69.6 \text{ ksi}$$

where F_{uv} is the measured shear strength per unit area, based on the nominal area of the unthreaded body of the bolt.

Considering compact joints where $L \leq 16$ in., the joint efficiency, its standard deviation, and its coefficient of variation are

$$R_p = \left(\frac{V_{ult}}{V_{exp}}\right)_{mean} \tag{15}$$
$$= 0.970$$

$$\sigma_p = 0.045$$

$$V_p = 0.046$$

Thus, the resistance, referred to as R_m when taken as the mean of several instances, and the coefficient of variation can be computed as

$$R_m = R_B R_p \tag{16}$$
$$= (69.5 \text{ ksi})(0.970)$$
$$= 67.4 \text{ ksi}$$

$$V_R = \sqrt{V_B^2 + V_P^2} \tag{17}$$
$$= \sqrt{(0.061)^2 + (0.046)^2}$$
$$= 0.076$$

For this analysis, it will be assumed that the mean dead load effect is 1.05 times the nominal value with a coefficient of variation of 0.10 ($D_m = 1.05D_{nom}$ with $V_D = 0.10$) and that the mean live load effect is equal to the nominal value with a coefficient of variation of 0.25 ($L_m = 1.00L_{nom}$ with $V_L = 0.25$). Using a ratio of live load to dead load of $L/D = 3.0$, the mean load effect can be computed as

$$Q_m = D_m + L_m \tag{18}$$
$$= 1.05D_{nom} + (1.00)(3.0D_{nom})$$
$$= 4.05D_{nom}$$

Similarly, the coefficient of variation for the load effect with a ratio of $L/D = 3.0$ can be computed as

$$V_Q = \frac{\sqrt{\left[(1.05D_{nom})(0.10)\right]^2 + \left[(3.00D_{nom})(0.25)\right]^2}}{4.05D_{nom}} \tag{19}$$
$$= 0.187$$

The basis for design is $\phi R_n \geq \Sigma\gamma Q$, where the design resistance according to the 2010 and 2016 AISC *Specifications*, assuming that the threads would be excluded from the shear plane, is

$$\phi R_n = \phi R_v R_{nx}(R_j R_{lj}) A_{b,nom} F_{u,nom} \tag{20}$$
$$= (0.75)(0.625)(1.00)(0.90) A_{b,nom} F_{u,nom}$$

For the load combination $\Sigma\gamma Q = 1.2D + 1.6L$ using a ratio of $L/D = 3.0$

$$\Sigma\gamma Q = (1.2)(D_{nom}) + (1.6)(3.0D_{nom}) \tag{21}$$
$$= 6.0D_{nom}$$

Thus

$$(0.75)(0.625)(1.00)(0.90) A_{b,nom} F_{u,nom} \geq 6.0D_{nom} \tag{22}$$

$$D_{nom} \leq 0.070 A_{b,nom} F_{u,nom}$$

Substituting this value into Equation 18,

$$Q_m = 4.05D_{nom} \tag{18}$$

Table 4. Reliability Indices for Bolted Shear Joints Designed as X but Installed as N Based on Reliability Analyses for *L/D* = 3.0						
	AISC *Specification* Prior to 2010			AISC *Specification* 2010 and 2016		
Grade	**120**	**120**	**150**	**120**	**120**	**150**
Diameter, in.	**⅝–1**	**1⅛–1¼**	**¾–1⅛**	**⅝–1**	**1⅛–1¼**	**¾–1⅛**
Compact	3.94	3.59	3.24	3.36	2.99	2.65
Intermediate	2.61	2.29	2.01	2.28	1.94	1.64
Long	3.07	2.79	2.54	2.48	2.19	1.95

Table 5. Approximate Probabilities of Failure for Bolted Shear Joints Designed as X but Installed as N Based on Reliability Analyses for *L/D* = 3.0						
	AISC *Specification* Prior to 2010			AISC *Specification* 2010 and 2016		
Grade	**120**	**120**	**150**	**120**	**120**	**150**
Diameter, in.	**⅝–1**	**1⅛–1¼**	**¾–1⅛**	**⅝–1**	**1⅛–1¼**	**¾–1⅛**
Compact	< 0.01%	0.02%	0.06%	0.04%	0.14%	0.41%
Intermediate	0.45%	1.10%	2.20%	1.12%	2.64%	5.01%
Long	0.11%	0.26%	0.55%	0.65%	1.41%	2.54%

$$= 4.05(0.070 A_{b,nom} F_{u,nom})$$

$$= 0.284 A_{b,nom} F_{u,nom} \quad (23)$$

Finally, the reliability index can be computed as

$$Reliability\ index = \frac{R_m}{Q_m} \quad (24)$$

$$= \frac{(67.4\text{ ksi})(A_{b,nom})}{0.284 A_{b,nom} F_{u,nom}}$$

$$= \frac{67.4\text{ ksi}}{(0.284)(120\text{ ksi})}$$

$$= \frac{67.4\text{ ksi}}{34.1\text{ ksi}}$$

$$\beta = \frac{\ln\left(\frac{R_m}{Q_m}\right)}{\sqrt{V_R^2 + V_Q^2}} \quad (13)$$

$$= \frac{\ln\left(\frac{67.4\text{ ksi}}{34.1\text{ ksi}}\right)}{\sqrt{(0.077)^2 + (0.187)^2}}$$

$$= 3.36$$

Considering the three classes of bolts (smaller Group 120, larger Group 120, and Group 150), the categories of joints based on joint length (compact, intermediate, and long), and the two slightly different approaches employed in the design specifications (AISC *Specifications* prior to 2010 versus the AISC 2010 and 2016 *Specifications*), a total of 18 reliability indices were computed for the ratio of live load to dead load of 3.0 and are shown in Table 4. Approximate probabilities of failure that correspond to these reliability indices are shown in Table 5. Reliability indices and probabilities of failure for other ratios of live load to dead load are shown in Appendix A as Tables A1 and A2, respectively.

Overstrength Due to Discretization

Most bolted joints are inherently overdesigned because of the practical necessity to select a discrete number of bolts for a given joint. Suppose, for example, that a joint is subjected to a factored design load of P_u = 95 kips and ⅞-in.-diameter Grade 120 bolts are being used. According to the 2010 or 2016 AISC *Specification*, the bolts would have a design strength of ϕr_{nv} = 30.7 kip/bolt in the X condition and ϕr_{nv} = 24.3 kip/bolt in the N condition. If the joint is designed as X, then 95 kips/30.7 kip/bolt = 3.07 bolts are required, and the engineer would likely use four bolts.[3] Thus, the bolts in this joint would have a design strength of (4 bolts)(30.7 kip/bolt) = 123 kips, which is 29% larger than the required strength. Suppose that after the joint with four bolts is erected, it is discovered that the bolts were actually installed as N. The design strength in that case would be

3 It is assumed in the analyses presented in this work that the theoretical number of required bolts is always rounded up to the next largest whole number. It is recognized, however, that some engineers might be inclined to round the number of required bolts down in some cases where it would be only slightly unconservative to do so.

(4 bolts)(24.3 kip/bolt) = 97.2 kips, which is still larger than the required strength of 95 kips. This situation is referred to herein as discretization overstrength.

Discretization overstrength is challenging to quantify deterministically, but there are a few factors that influence when discretization overstrength is present and how large it may be. The first factor is the number of rows of bolts that are in a joint parallel to the applied force. The incremental change in design strength is typically an integer multiple of the number of parallel rows in the joint. For example, if there is a single parallel row of bolts, then bolts are added one at a time until the design strength exceeds the required strength of the joint. On the other hand, if there are two parallel rows of bolts, then two bolts are generally added at a time. A second factor is the size and strength of an individual bolt. If small bolts are used in a joint, then incrementally increasing the number of bolts in the joint increases the strength by a smaller amount than if larger bolts had been used. Additionally, if bolts are deployed in double shear, then the incremental strength increase would be double relative to bolts deployed in single shear. While this is interesting in a general sense, it is not likely to have an influence on the fully threaded short bolt problem because those bolts are likely not long enough to be deployed in double shear. Finally, the size of the joint is also a consideration. When joints made of a larger number of bolts are designed, the incremental strength increase associated with adding one row of bolts is smaller relative to the overall strength of the joint made up of a smaller number of bolts. This behavior, and the fact that no fewer than two bolts can be used in any bolted joint, leads to a higher discretization overstrength for smaller joints than for larger joints.

Monte Carlo Simulations

Given the difficulty in deterministically quantifying the discretization overstrength, an indirect approach was used to assess the safety of bolted joints designed as "threads excluded" but erected as "threads not excluded." A series of six analyses was conducted for end-loaded joints corresponding to each of the three classes of bolts and two versions of the AISC *Specification*. Each of the analyses consisted of 100,000 Monte Carlo simulations. Sample calculations for one simulation are shown in Appendix B.

The nominal dead load in each simulation, D_n, was modeled as a uniformly distributed random variable ranging between approximately 0.50 times and 24 times the design strength of a single bolt in the joint being considered. The simulated dead load, D_{sim}, was modeled as a normally distributed random variable with a mean of 1.05 times the nominal dead load and a coefficient of variation of 0.10 (Nowak and Collins, 2013). The nominal live load, L_n, was determined by multiplying the nominal dead load by the ratio of live load to dead load that was considered for the simulation. The simulated live load, L_{sim}, was modeled using a Gumbel distribution (Extreme Type I) with a mean of 1.00 times the nominal live load and a coefficient of variation of 0.25 (Nowak and Collins, 2013). The required strength was calculated using the load combination $\Sigma\gamma Q = 1.2D_n + 1.6L_n$, and the total simulated load was calculated as $Q_{sim} = D_{sim} + L_{sim}$.

Bolt diameter was modeled as a uniform random variable over the range of ⅝ in. to 1 in. for smaller Group 120 bolts, 1⅛ in. to 1¼ in. for larger Group 120 bolts, or ¾ in. to 1⅛ in. for Group 150 bolts. The number of bolts was determined by dividing the required strength by the design strength per bolt in the X condition as $\Sigma\gamma Q/\phi r_{nv}$ and then rounding up to the next highest integer, but using no fewer than two bolts. Spacing of the bolts in the direction of load, s, was taken as the larger of 3 in. or three times the bolt diameter, and the length of the joint was calculated as $L = (n_{bolts} - 1)s$. The design strength and number of bolts was modified for joints whose length exceeded the transition values of 50 in. in the AISC *Specifications* prior to 2010 and 38 in. in the 2010 and 2016 AISC *Specifications*.

The simulated ultimate shear strength per unit area of the bolts, $F_{uv,sim}$, was modeled as a lognormal random variable using the means and standard deviations shown in Figures 3, 4, and 5. Figure 9 shows a histogram of simulated shear strength per unit area for smaller Group 120 bolts used in the analysis with AISC *Specifications* prior to 2010. The simulated bolt strength, R_{sim}, was calculated as $n_{bolts}R_jR_{lj}A_bF_{uv,sim}$, where R_jR_{lj} was modeled as a uniformly distributed random variable with a lower bound determined using Equation 8 and an upper bound of 1.00. Finally, the capacity-to-demand ratio was computed as the ratio of R_{sim} to Q_{sim}, and the probability of failure was taken as the number of simulations yielding a capacity-to-demand ratio less than unity divided by the number of simulations. The simulated values of joint efficiency, R_jR_{lj}, for the analysis of smaller Group 120 bolts designed using AISC *Specifications* prior to 2010 are shown in Figure 10.

Figure 11 shows an overview of the results of the analysis of smaller Group 120 bolts designed using AISC *Specifications* prior to 2010. Overall, the probability of failure for this analysis was 0.16%. The results were further subdivided based on the length of the joints in each of the 100,000 simulations and are shown in Table 6 along with the results from the other analyses. Results for ratios of live load to dead load other than 3.0 are shown in Appendix A as Table A3.

SUMMARY

A question that will undoubtedly arise regarding structures that were designed with short bolts assuming that threads were excluded from the shear plane is whether remedial measures need to be taken to ensure the safety of the structure. The results of the reliability analysis and Monte Carlo

Table 6. Approximate Probabilities of Failure for Bolts Designed as X but Installed as N Based on Monte Carlo Analyses for *L*/*D* = 3.0

	AISC *Specification* Prior to 2010			AISC *Specification* 2010 and 2016		
Grade	**120**	**120**	**150**	**120**	**120**	**150**
Diameter, in.	**⅝–1**	**1⅛–1¼**	**¾–1⅛**	**⅝–1**	**1⅛–1¼**	**¾–1⅛**
Compact	< 0.01%	<0.01%	< 0.01%	0.01%	0.05%	0.18%
Intermediate	0.34%	0.70%	1.45%	0.72%	1.72%	2.91%
Long	0.02%	0.07%	0.26%	0.29%	0.73%	1.48%

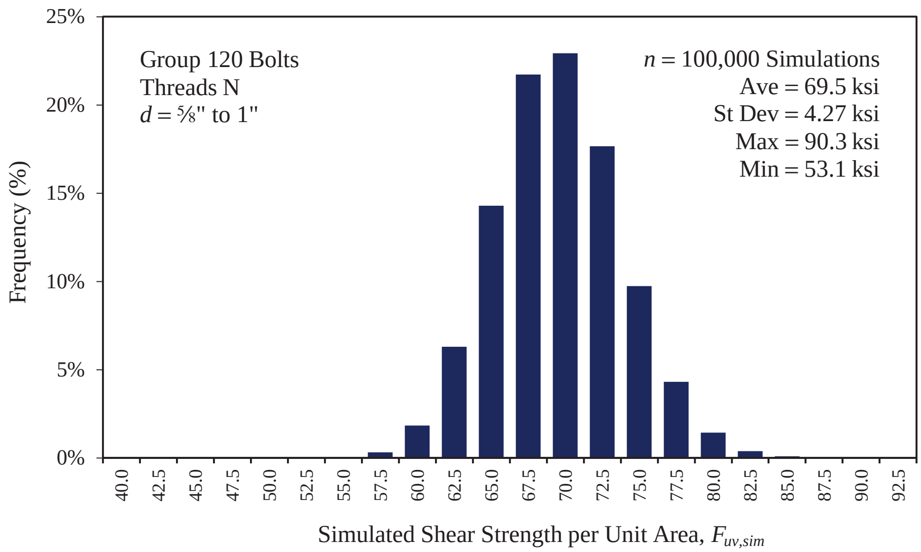


Fig. 9. Simulated shear strength per unit area of smaller Group 120 bolts with threads not excluded.

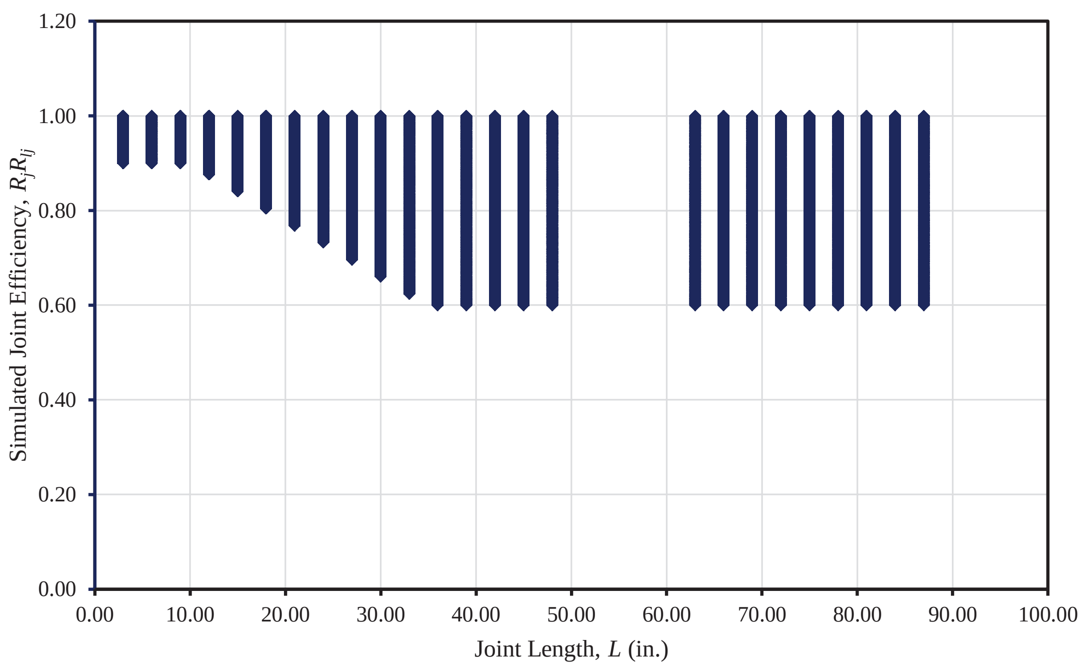


Fig. 10. Simulated joint efficiency used in the analysis of smaller Group 120 bolts per AISC Specifications *prior to 2010.*

Table 7. Relationship between Probability of Failure and Reliability Index for a Normally Distributed Dataset	
Reliability Index	**Probability of Failure**
1.0	15.9%
2.0	2.28%
2.5	0.621%
2.6	0.466%
3.0	0.135%
3.5	0.0233%
4.0	0.00317%
4.5	0.000340%

analyses described herein should help an engineer make that decision. The commentary to Chapter B of the 2016 AISC *Specification* (AISC, 2016) states that target reliability indices at $L/D = 3$ of approximately 2.6 for members and approximately 4.0 for connections were used in the development of the specification. The probabilities of failure associated with these reliability indices are approximately 0.466% and 0.00317%, respectively. Approximate probabilities of failure for these and additional values of the reliability index are presented in Table 7.

It can be observed from the analyses described herein that bolts designed as X but installed as N still have a substantial level of reliability, in some cases as high as target reliabilities cited in the commentary to the 2016 AISC *Specification*. The following conclusions can be made:

- Bolts in joints designed using AISC *Specifications* prior to 2010 have a higher reliability than those designed using the 2010 and 2016 AISC *Specifications*.
- Smaller Group 120 bolts demonstrated the highest level of reliability, and Group 150 bolts demonstrated the lowest reliability of the three classes of bolts considered in this study.
- Compact joints proved to be the most reliable, followed by long joints, and then intermediate-length joints.
- Based on the Monte Carlo analyses, it can be concluded that compact joints designed using AISC *Specifications* prior to 2010, regardless of bolt grade and diameter, have a reliability that approximately meets target reliabilities for connections in the 2016 AISC *Specification*.

It is evident that bolts designed as X but installed as N may not meet the target reliability index for connections in

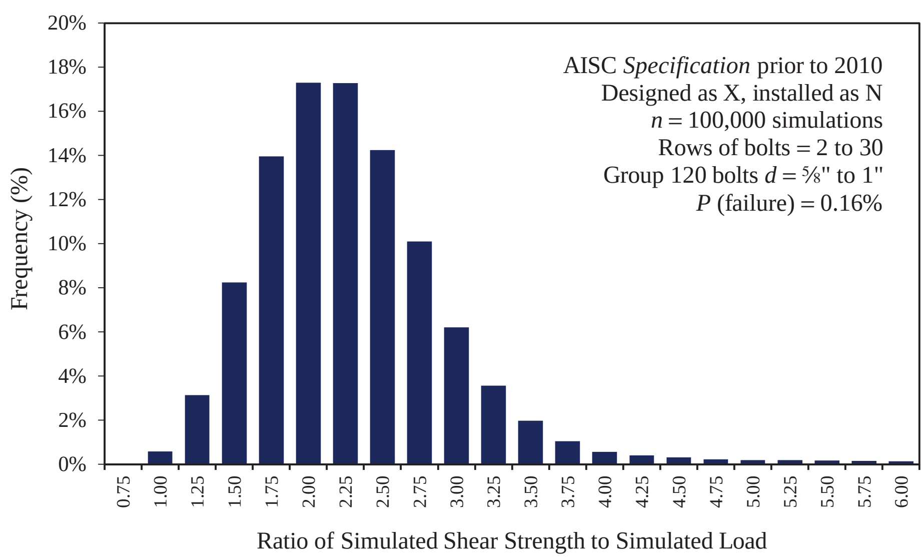


Fig. 11. Results of analysis of smaller Group 120 bolts per AISC Specifications *prior to 2010.*

many cases. However, a criterion that an engineer may consider imposing when deciding whether remedial measures are necessary is whether the connection has a reliability that is at least equal to that of the members that are joined by it. If that criterion is employed, then a maximum probability of failure of approximately 0.466% would be acceptable. In that case, it could be stated that all compact joints designed using any AISC *Specification* would satisfy that criterion. Further, it could be stated that all joints, regardless of length, designed in accordance with AISC *Specifications* prior to 2010 would also satisfy that criterion as long as Group 120 bolts not larger than 1 in. in diameter were used.

Additional Considerations

The analyses described herein were conducted under the assumption that the joints were designed for the loads to which they were actually subjected. If joints in this case were actually designed, as is often done in practice, for artificially large loads, such as those corresponding to the capacity of the member(s) being joined or a fraction of the load that would cause failure of the connected member(s) [e.g., ½ uniformly distributed load (UDL)], then the reliability of the joints would be increased, substantially in many cases.

Shear joints designed as slip critical will likely be unaffected by the issue described herein. While it is noted that slip-critical joints must also be checked to ensure that the strength of the bolts in bearing is satisfactory, a larger number of bolts is generally required in slip-critical joints than in bearing joints, and those additional bolts will likely provide enough added strength in shear to provide a satisfactory level of reliability.

Additionally, bolts deployed with more than one shear plane (e.g., double shear, triple shear, etc.) are less likely to be affected by the short bolt issue. Because bolts deployed with more than one shear plane tend to be longer, they are less likely to fall into the category of short fully threaded bolts. However, care should be taken with bolts of any length that are deployed in applications with more than one shear plane to ensure that the threads of the bolts are excluded from all shear planes, if that is the assumption made during design.

One aspect of joint design that is absent from the analyses presented herein is the possibility that failure modes other than bolt shear may govern the strength of the joint. For example, the strength of a joint with bolts designed as X but later discovered to actually be N may have been controlled by, for example, the bearing strength of the material around the bolt instead of by the strength of the bolts. Revising the shear strength of the bolts in that case may have no effect on the strength of the joint or may result in a reduction in strength less than the 20% implied by Equation 9. An analysis of bearing strength is possible because it depends only on the bolt diameter and the thickness and material strength of the connected plies. This is explored in Appendix C, where critical ply thicknesses are tabulated; joints with plies thinner than those tabulated will have bearing strengths that are lower than the shear strength of the associated bolt in the N condition.

The analyses presented herein focused on end-loaded joints to the exclusion of eccentrically loaded joints. However, because the strength of eccentrically loaded joints tends to depend mostly on the strength of the single most heavily loaded fastener in the bolt group, it could be postulated that the reliability of eccentrically loaded joints would be similar to the reliability of compact end-loaded joints. This is supported by additional Monte Carlo simulations that were conducted but are not described herein.

Finally, if after considering the analyses and discussion presented herein, an engineer decides that remedial action may be needed for a structure as a last resort, consideration should be given to the idea of removing a representative sample of the bolts in question to see if sufficient body length may actually exist before prescribing a more extensive and costly remediation solution.

It is noted that differences exist between the AISC and AASHTO-LRFD specifications—particularly differences in the resistance factors, loads, and load combinations—that make it difficult to directly apply the reliabilities and probabilities of failure presented herein to bridge structures. Still, it is expected that the analyses presented herein will be useful to bridge engineers nonetheless. It is noted that the change in design strength between the 2005 and 2010 AISC *Specifications* was approximately mirrored between the 7th and 8th editions of the AASHTO-LRFD Specifications. In the 7th and prior editions of the AASHTO-LRFD Specification, the design strengths were $r_{nv} = 0.48A_bF_{u,bolt}N_s$ and $r_{nv} = 0.38A_bF_{u,bolt}N_s$ for threads excluded and not excluded, respectively, whereas in the 8th edition, the design strengths were increased to $r_{nv} = 0.56A_bF_{u,bolt}N_s$ and $r_{nv} = 0.45A_bF_{u,bolt}N_s$, where N_s is the number of shear planes. The coefficients used in the 7th and prior editions of the AASHTO-LRFD Specifications are slightly lower than those in the AISC *Specifications* prior to 2010; thus, the reliability of joints designed per those specifications would be favorably affected. It is further noted that the commentary to the 8th edition of the AASHTO-LRFD Specification states that it has been calibrated with a target reliability index of 3.5 for main members under the Strength I load combination over a 75-year design life. In a separate section, the AASHTO commentary states that a reliability index of 4.5 was targeted for truss gusset plates under the Strength I load combination at a dead load to live load ratio of 6.0, which is outside of the range considered in this study (AASHTO, 2017).

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APPENDIX A

ADDITIONAL CALCULATED VALUES

Table A1. Reliability Indices for Bolted Shear Joints Designed as X but Installed as N Based on Reliability Analyses for Various Ratios of *L/D*

			Ratio of Live Load to Dead Load				
			1	2	3	4	5
AISC *Specification* Prior to 2010	Compact	Smaller Gr 120	4.68	4.18	3.94	3.81	3.72
		Larger Gr 120	4.27	3.80	3.59	3.47	3.39
		Gr 150	3.78	3.42	3.24	3.14	3.07
	Intermediate	Smaller Gr 120	2.71	2.66	2.61	2.58	2.55
		Larger Gr 120	2.33	2.32	2.29	2.27	2.25
		Gr 150	2.00	2.03	2.01	2.00	1.99
	Long	Smaller Gr 120	3.16	3.12	3.07	3.04	3.01
		Larger Gr 120	2.84	2.82	2.79	2.76	2.74
		Gr 150	2.56	2.57	2.54	2.52	2.51
AISC *Specification* 2010 or 2016	Compact	Smaller Gr 120	3.91	3.54	3.36	3.26	3.19
		Larger Gr 120	3.47	3.15	2.99	2.90	2.85
		Gr 150	2.99	2.76	2.65	2.58	2.53
	Intermediate	Smaller Gr 120	2.35	2.32	2.28	2.25	2.23
		Larger Gr 120	1.94	1.95	1.94	1.92	1.91
		Gr 150	1.58	1.64	1.64	1.64	1.64
	Long	Smaller Gr 120	2.49	2.50	2.48	2.46	2.45
		Larger Gr 120	2.16	2.20	2.19	2.19	2.18
		Gr 150	1.89	1.95	1.95	1.95	1.95

Table A2. Probabilities of Failure for Bolted Shear Joints Designed as X but Installed as N Based on Reliability Analyses for Various Ratios of *L/D*							
			Ratio of Live Load to Dead Load				
			1	**2**	**3**	**4**	**5**
AISC *Specification* Prior to 2010	**Compact**	**Smaller Gr 120**	< 0.01%	< 0.01%	< 0.01%	< 0.01%	< 0.01%
		Larger Gr 120	< 0.01%	< 0.01%	0.02%	0.03%	0.04%
		Gr 150	< 0.01%	0.03%	0.06%	0.09%	0.11%
	Intermediate	**Smaller Gr 120**	0.34%	0.39%	0.45%	0.50%	0.53%
		Larger Gr 120	0.98%	1.01%	1.10%	1.17%	1.22%
		Gr 150	2.27%	2.13%	2.20%	2.27%	2.32%
	Long	**Smaller Gr 120**	0.08%	0.09%	0.11%	0.12%	0.13%
		Larger Gr 120	0.22%	0.24%	0.26%	0.29%	0.31%
		Gr 150	0.52%	0.51%	0.55%	0.58%	0.61%
AISC *Specification* 2010 or 2016	**Compact**	**Smaller Gr 120**	< 0.01%	0.02%	0.04%	0.06%	0.07%
		Larger Gr 120	0.03%	0.08%	0.14%	0.19%	0.22%
		Gr 150	0.14%	0.29%	0.41%	0.50%	0.57%
	Intermediate	**Smaller Gr 120**	0.93%	1.01%	1.12%	1.21%	1.28%
		Larger Gr 120	2.62%	2.53%	2.64%	2.74%	2.82%
		Gr 150	5.72%	5.06%	5.01%	5.05%	5.09%
	Long	**Smaller Gr 120**	0.65%	0.62%	0.65%	0.69%	0.71%
		Larger Gr 120	1.53%	1.39%	1.41%	1.44%	1.47%
		Gr 150	2.95%	2.58%	2.54%	2.55%	2.57%

Table A3. Approximate Probabilities of Failure for Bolted Shear Joints Designed as X but Installed as N Based on Monte Carlo Analyses for Various Ratios of *L/D*							
			Ratio of Live Load to Dead Load				
			1	**2**	**3**	**4**	**5**
AISC *Specification* Prior to 2010	**Compact**	**Smaller Gr 120**	< 0.01%	< 0.01%	< 0.01%	< 0.01%	< 0.01%
		Larger Gr 120	< 0.01%	< 0.01%	< 0.01%	< 0.01%	< 0.01%
		Gr 150	< 0.01%	< 0.01%	< 0.01%	0.04%	0.04%
	Intermediate	**Smaller Gr 120**	0.21%	0.28%	0.34%	0.28%	0.31%
		Larger Gr 120	0.59%	0.70%	0.70%	0.76%	0.77%
		Gr 150	1.43%	1.40%	1.45%	1.40%	1.51%
	Long	**Smaller Gr 120**	< 0.01%	0.02%	0.02%	0.03%	0.03%
		Larger Gr 120	0.04%	0.07%	0.07%	0.11%	0.16%
		Gr 150	0.14%	0.21%	0.26%	0.29%	0.30%
AISC *Specification* 2010 or 2016	**Compact**	**Smaller Gr 120**	< 0.01%	< 0.01%	0.01%	0.02%	0.01%
		Larger Gr 120	0.02%	0.02%	0.05%	0.05%	0.04%
		Gr 150	0.06%	0.11%	0.18%	0.22%	0.25%
	Intermediate	**Smaller Gr 120**	0.56%	0.67%	0.72%	0.66%	0.74%
		Larger Gr 120	1.68%	1.66%	1.72%	1.74%	1.64%
		Gr 150	3.37%	2.99%	2.91%	3.03%	3.12%
	Long	**Smaller Gr 120**	0.15%	0.27%	0.29%	0.35%	0.39%
		Larger Gr 120	0.55%	0.58%	0.73%	0.79%	0.77%
		Gr 150	1.38%	1.43%	1.48%	1.45%	1.64%

APPENDIX B

SAMPLE CALCULATIONS FROM MONTE CARLO SIMULATIONS

Case Group 150 Bolts Conforming to AISC 2010 and 2016 *Specifications* with L/D = 3.0

Randomly select 1-in.-diameter bolts: A_b = 0.785 in.^{2}

The design strength of a single bolt with threads excluded is:

$$F_{nv} = R_v R_{nx} R_j R_{lj} F_{u,bolt} \qquad (6)$$
$$= (0.625)(1.00)(0.90)(1.00)(150 \text{ ksi})$$
$$= 84.4 \text{ ksi}$$

$$\phi r_{nv} = \phi A_b F_{nv} \qquad \text{(from Eq. 5)}$$
$$= (0.75)(0.785 \text{ in.}^2)(84.4 \text{ ksi})$$
$$= 49.7 \text{ kip/bolt}$$

Determine the design loads:

Set the minimum nominal dead load:

$$D_{n,min} = \frac{(0.50)(49.7 \text{ kip/bolt})}{[(1.2)(1.0)+(1.6)(3.0)]}$$
$$= 4.14 \text{ kips}$$

Set the maximum nominal dead load:

$$D_{n,max} = \frac{(24)(49.7 \text{ kip/bolt})}{[(1.2)(1.0)+(1.6)(3.0)]}$$
$$= 199 \text{ kips}$$

A nominal dead load of D_n = 136 kips is computed as a uniform random variable between 4.14 kips and 199 kips.

An "actual" dead load of D_{sim} = 134 kips is simulated as a normally distributed random variable with a mean of (1.05)(136 kips) = 143 kips and a standard deviation of (0.10)(136 kip) = 13.6 kips.

The nominal live load is computed as L_n = (3.0)(136 kips) = 408 kips.

An "actual" live load of L_{sim} = 529 kips is simulated as a random variable with Gumbel distribution using a mean of (1.00)(408 kips) = 408 kips and a standard deviation of (0.25)(408 kips) = 102 kips.

The total nominal load is

$$Q_n = D_n + L_n$$
$$= 136 \text{ kips} + 408 \text{ kips}$$
$$= 544 \text{ kips}$$

The total design load is

$$\Sigma\gamma Q = 1.2D_n + 1.6L_n$$
$$= (1.2)(136 \text{ kips}) + (1.6)(408 \text{ kips})$$
$$= 816 \text{ kips}$$

The total "actual" load is

$$Q_{sim} = D_{sim} + L_{sim}$$
$$= 134 \text{ kips} + 529 \text{ kips}$$
$$= 663 \text{ kips}$$

With a design load of $\Sigma\gamma Q = 816$ kips and a design strength of $\phi r_{nv} = 49.7$ kip/bolt, the trial number of required bolts is

$$\frac{816 \text{ kips}}{49.7 \text{ kip/bolt}} = 16.4 \text{ bolts} \quad \rightarrow \quad \text{try 17 bolts}$$

The spacing of the bolts is 3 in., thus the length of the joint would be $L = (17 - 1)(3 \text{ in.}) = 48.0$ in.

Because the length is greater than 38 in., the factor R_{lj} is taken as 0.833 instead of 1.00, thus,

$$F_{nv} = (0.625)(1.00)(0.90)(0.833)(150 \text{ ksi})$$
$$= 70.3 \text{ ksi}$$

$$\phi r_{nv} = (0.75)(0.7854 \text{ in.}^2)(70.3 \text{ ksi})$$
$$= 41.4 \text{ kip/bolt}$$

The number of required bolts is recomputed as

$$\frac{816 \text{ kips}}{41.4 \text{ kip/bolt}} = 19.7 \text{ bolts} \quad \rightarrow \quad \text{use 20 bolts}$$

and the length of the connection is recomputed as $L = (20 - 1)(3 \text{ in.}) = 57.0$ in.

The nominal shear strength per unit area of the bolt with the threads not excluded is simulated as a log-normally distributed random variable with a mean of 74.6 ksi and a standard deviation of 3.81 ksi and is determined to be $F_{uv,sim} = 78.0$ ksi for this simulation.

The factor R_jR_{lj} is simulated as a uniformly distributed variable with an upper bound of 1.00 and a lower bound of

$$R_jR_{lj,min} = 1.02\text{-}0.0120L \tag{8}$$
$$= 1.02 - (0.0120)(57.0 \text{ in.})$$
$$= 0.336$$

Because $R_jR_{lj,min} < 0.60$, use $R_jR_{lj,min} = 0.60$.

For this simulation, a value of $R_jR_{lj} = 0.619$ was used.

The "actual" strength of this joint is simulated as

$$R_{sim} = (20 \text{ bolts})(0.785 \text{ in.}^2/\text{bolt})(78.0 \text{ ksi})(0.619)$$
$$= 758 \text{ kips}$$

Finally, the capacity-to-demand ratio for the joint is then

$$\frac{R_{sim}}{Q_{sim}} = \frac{758 \text{ kips}}{663 \text{ kips}}$$
$$= 1.14$$

APPENDIX C

Table C1. Ply Thickness Below which Bearing Strength Will Govern over Bolt Shear Strength, *N*

Bolt Dia.	$F_{u,ply}$ = 58 ksi				$F_{u,ply}$ = 65 ksi			
	$F_{u,bolt}$ = 120 ksi		$F_{u,bolt}$ = 150 ksi		$F_{u,bolt}$ = 120 ksi		$F_{u,bolt}$ = 150 ksi	
	t_{crit}		t_{crit}		t_{crit}		t_{crit}	
(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)
½	0.152	⅛	0.190	3⁄16	0.136	⅛	0.170	3⁄16
⅝	0.190	3⁄16	0.238	¼	0.170	3⁄16	0.212	3⁄16
¾	0.228	¼	0.286	5⁄16	0.204	3⁄16	0.255	¼
⅞	0.267	¼	0.333	5⁄16	0.238	¼	0.297	5⁄16
1	0.305	5⁄16	0.381	⅜	0.272	¼	0.340	5⁄16
1⅛	0.343	5⁄16	0.428	7⁄16	0.306	5⁄16	0.382	⅜
1¼	0.381	⅜	0.476	½	0.340	5⁄16	0.425	7⁄16
1⅜	0.419	7⁄16	0.524	½	0.374	⅜	0.467	7⁄16
1½	0.457	7⁄16	0.571	9⁄16	0.408	7⁄16	0.510	½

Table C2. W-Shapes (A992 Steel) for which the Bearing Strength of the Web Will Govern over Bolt Shear Strength, *N*

Shape	Group 120 Bolts			Group 150 Bolts		
	d = ¾ in.	d = ⅞ in.	d = 1 in.	d = ¾ in.	d = ⅞ in.	d = 1 in.
W18	—	—	—	—	—	≤ W18×40
W16	—	—	≤ W16×26	W16×26	≤ W16×36	≤ W16×40
W14	—	≤ W14×22	≤ W14×30	≤ W14×26	≤ W14×34	≤ W14×48
W12	W12×14	≤ W12×19	≤ W12×30	≤ W12×19	≤ W12×35	≤ W12×45
W10	W10×12	≤ W10×15	≤ W10×26	≤ W10×22	≤ W10×26	≤ W10×39
W8	W8×10	≤ W8×13	≤ W8×24	≤ W8×24	≤ W8×31	≤ W8×35
W6	≤ W6×9	≤ W6×15	≤ W6×20	≤ W6×15	≤ W6×20	All

BEARING STRENGTH COMPARISON

For bolts in single shear, the bearing strength of a connected ply will govern when the bearing strength is less than or equal to the bolt shear strength, N:

$$\phi_{bearing}2.4t_{ply}d_bF_{u,ply} + \phi_{shear}0.450A_bF_{u,bolt} \tag{C1}$$

Thus, joints that have a ply thinner than shown in the following equation will be governed by bearing strength instead of the bolt shear strength.

$$t_{ply} \le \frac{d_bF_{u,bolt}}{6.791F_{u,ply}} \tag{C2}$$

Substituting common values of 58 ksi and 65 ksi for $F_{u,ply}$, and 120 ksi and 150 ksi for $F_{u,bolt}$ into this equation, the critical ply thickness can be found as a function of the bolt diameter. Joints with ply thicknesses less than this will have a bearing strength less than the bolt shear strength, calculated assuming that the threads are not excluded from the shear plane. These values are shown in Table C1, where both decimal values and the nearest 1⁄16th fractional values are tabulated. Additionally, note that the tabulated values are based on the case where deformation at the bolt hole at service load is a design consideration and that the bolt shear strength is calculated using the 2010 and 2016 AISC *Specifications*.

When bolts are used to connect the webs of wide-flange shapes, shapes can be identified that will satisfy the minimum ply thickness shown in Table C1. Those shapes are tabulated in Table C2 for Group 120 and Group 150 bolts for the common bolt diameters of ¾ in., ⅞ in., and 1 in.

Reexamination of Shear Lag in HSS Tension Members with Side Gusset Plate Connections

AKASHDEEP A. BHAT and PATRICK J. FORTNEY
In memory of Patrick J. Fortney, who passed away in October, 2019.

ABSTRACT

This paper presents an evaluation of the shear lag factor for HSS tension members connected with two side plate gussets with longitudinal welds as given in AISC *Specification* Table D3.1, Case 6b. The current AISC *Specification* for Case 6b does not permit weld lengths less than the perpendicular distance between the welds and has the potential of producing negative shear lag factors. Similar issues previously existed for members given in Case 4 of Table D3.1. However, the AISC *Specification* has adopted a mathematical model proposed by Fortney and Thornton for Case 4 of Table D3.1. The work presented in this paper (1) offers a mathematical model for calculating the shear lag factor for Case 6b derived by repurposing the model adopted by AISC for Case 4 of Table D3.1, (2) offers the results of a parametric study comparing the results of the new mathematical model to the results using the current AISC method, and (3) discusses the protocols developed for use in finite element analysis to evaluate the effectiveness of the proposed mathematical model. The proposed new mathematical model will permit longitudinal weld lengths less than the perpendicular distance between the welds and removes the possibility of calculating a negative shear lag factor, while better representing the redistribution of cross-sectional stress near the connection region.

Keywords: Shear lag, HSS, longitudinal welds, gusset plates.

BACKGROUND

The performance of a tension member is influenced by factors like size, material, type of connection, and the like. The manner in which the tension member is connected has a direct influence on shear lag. Tension members are generally connected to elements such as gusset plates using bolts or welds. When some, but not all the elements of the cross section of a tension member (e.g., flange, web, leg, etc.) are used to transfer the axial load to the connection, a nonuniform stress distribution occurs in the tension member adjacent to the connection, this phenomenon is commonly referred to as shear lag (Fortney and Thornton, 2012). Due to this effect, the entire cross section is not fully effective to carry the load at the critical section, resulting in reduced design strength of the member.

In an HSS tension member connected with two side gusset plates on opposite faces, the longitudinal welds receive the load from the gusset plate through shear. The shear in the welds is then transferred to the HSS section as an axial load. The transition from this localized stress to a uniformly distributed stress over the tension member cross section away from the connected region is a region thought to be subjected to the shear lag phenomenon. Along this critical length of the tension member, this shear lag effect can have a detrimental effect on the tensile capacity of the tension member.

Generally, the reduced capacity of a tension member is addressed through the use of a shear lag factor, U_A, applied to the gross area of the tension member. AISC *Specification for Structural Steel Buildings* (AISC, 2016), Table D3.1, requires an accounting for shear lag for various types of end connections. For HSS members connected with two side gusset plates with longitudinal welds (of length l), and considering the perpendicular distance between the welds as H, the shear lag factor is calculated using Case 6b of Table D3.1 as shown in Table 1.

INTRODUCTION

The current AISC *Specification* procedure for determining the shear lag factor for Case 6b assumes that the welds will transfer the entire member load to the two walls parallel with the gusset plates and assumes the walls that are perpendicular to the gusset plates are unconnected elements. An additional assumption made for the current Case 6b is that the perpendicular distance between the welds is equal to the width of the H side (which is parallel to the gusset plate). However, the authors propose that all four walls are connected by the welds, and as such, the perpendicular distance between the welds could be taken as either H or B, depending on which sides of the HSS the gusset plates are attached to. With this, one could argue that the eccentricity, $\bar{x}$, could

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Table 1. Shear Lag Factor Case 6b

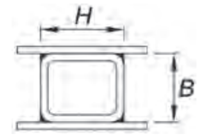
Case	Description of Element		Shear Lag Factor, UA	Example
6	Rectangular HSS	With two side gusset plates	$l \geq H ... U_A = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{B^2}{4(B+H)}$	

Table 2. Connection Eccentricity $\bar{x}$

Perpendicular Distance between Welds	$\bar{x}$
H	$\bar{x} = \frac{B^2}{4(B+H)}$
B	$\bar{x} = \frac{H^2}{4(B+H)}$

be calculated in two different ways (as shown in Table 2) for a rectangular HSS, giving two possible shear lag factors.

Furthermore, the current AISC *Specification* Case 6b does not permit weld lengths less than the perpendicular distance between the welds. When the length of the connection, l, is less than the connection eccentricity, $\bar{x}$, and the gusset plates are attached to the B side, the equation produces a negative shear lag factor. For example, for an HSS 20×4×½ with the gusset plate attached to the B side (4-in. side) with welds of length l = 4 in., the connection eccentricity and the shear lag factor, using the current AISC *Specification* Case 6b, can be calculated as:

$$\begin{aligned}\bar{x} &= \frac{H^2}{4(B+H)} \\ &= \frac{(20 \text{ in.})^2}{4(20 \text{ in.} + 4 \text{ in.})} \\ &= 4.17 \text{ in.}\end{aligned}$$

$$\begin{aligned}U_A &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{4.17 \text{ in.}}{4 \text{ in.}} \\ &= -0.04\end{aligned}$$

This is problematic because any connection, even one with a relatively short connection length, will provide some level of force transfer.

To overcome these shortcomings, a mathematical model was developed by repurposing the mathematical model developed by Fortney and Thornton (2012) in AISC *Specification* Table D3.1, Case 4 (AISC, 2016). Subsequent to the development of the mathematical model, a parametric study was performed comparing the shear lag factor calculated using the current AISC *Specification* to that calculated using the proposed new mathematical model. A series of finite element models were then developed in ABAQUS (2013) to evaluate the validity of the mathematical model. For the new model proposed, the welds are considered to connect all four of the HSS walls, and the weld length need not necessarily be equal to or greater than the perpendicular distance between the welds. The following presents the new proposed model.

MATHEMATICAL MODEL

Fortney and Thornton (2012) proposed a new method for evaluating the shear lag factor for Case 4 as given in Table D3.1 of the 2010 AISC *Specification* (AISC, 2010) addressing the longitudinally welded end-connected members in a more general way. The objective of that work was to develop a generalized procedure for calculating shear lag in plates, angles, channels, and tee members connected with longitudinal welds. Consideration was given to connections with weld lengths less than the perpendicular distance between the welds and also to address the condition where connections have unequal weld lengths. Fortney and Thornton compared three models for evaluating the shear lag factor: (1) the AISC model (prior to 2016), (2) the Canadian Standards Association (CSA) model, and (3) a fixed-fixed beam model.

The fixed-fixed beam model proposed by Fortney and Thornton captures the biplanar shear lag effect due to the connected and unconnected elements of a section. At the time of that study, the current AISC *Specifications* considered only uniplanar shear lag effects. The recommendation proposed by Fortney and Thornton was adopted by the AISC *Specification* for Case 4 of Table D.3.1 with the by-product effect that certain shapes moved from Case 2 to Case 4. Based on the generalized biplanar model proposed, a similar approach may be taken that will address issues with Case 6b of Table D3.1 for HSS tension members connected with two side gusset plates.

It is postulated that the longitudinal welds connect all four walls of the HSS and each wall acts similarly to plates with longitudinal welds (similar to plates as shown for Case 4 in Table D3.1). Thus, only in-plane shear lag exists, eliminating the connection eccentricity ($\bar{x} = 0$). Figure 1 shows the migration of the load in the HSS walls to the four weld lines. As can be seen in the Figure 1, each weld line attracts load from both walls connected by the weld.

The total tension load carried by each wall is proportional to the relative areas of each of the four HSS walls as given by Equations 1 and 2.

$$P_B = P_u\left(\frac{B}{B+H}\right) \tag{1}$$

$$P_H = P_u\left(\frac{H}{B+H}\right) \tag{2}$$

Note that the shear lag factor will be different for the B and H walls since the l/w ratio can be different for B and H.

The generalized shear lag factor given for Case 4 is:

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right) \tag{3}$$

The shear lag factor for the B walls is given by:

$$\begin{aligned} U_B &= \frac{1}{1+\frac{1}{3}\left(\frac{w}{l_w}\right)^2}\left(1-\frac{\bar{x}}{l_w}\right) \\ &= \frac{1}{1+\frac{1}{3}\left(\frac{B}{l_w}\right)^2}\left(1-\frac{0}{l_w}\right) \\ &= \frac{3l_w^2}{3l_w^2 + B^2} \end{aligned} \tag{4}$$

The shear lag factor for the H walls is given by:

$$\begin{aligned} U_H &= \frac{1}{1+\frac{1}{3}\left(\frac{w}{l_w}\right)^2}\left(1-\frac{\bar{x}}{l_w}\right) \\ &= \frac{1}{1+\frac{1}{3}\left(\frac{H}{l_w}\right)^2}\left(1-\frac{0}{l_w}\right) \\ &= \frac{3l_w^2}{3l_w^2 + H^2} \end{aligned} \tag{5}$$

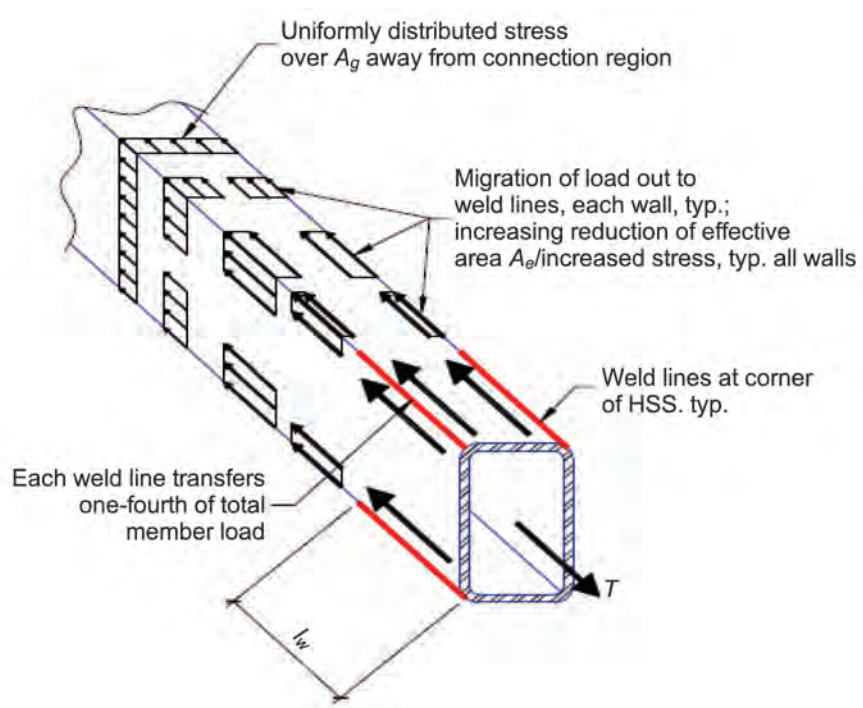


Fig. 1. In-plane shear lag in rectangular HSS connected with two side gusset plates with longitudinal welds.

Table 3. Parametric Study Matrix		
Configuration	Perpendicular Distance between Welds	Length of Weld
Gusset plates on *H* side	*H*	Varied from 0.25*H* up to 2*H* with an increment of 0.25*H*
Gusset plates on *B* side	*B*	Varied from 0.25*B* up to 2*B* with an increment of 0.25*B*
Gusset plates on *H* side	*H*	Varied from $0.25\left(\frac{H+B}{2}\right)$ up to $2\left(\frac{H+B}{2}\right)$ with an increment of $0.25\left(\frac{H+B}{2}\right)$
Gusset plate on *B* side	*B*	Varied from $0.25\left(\frac{H+B}{2}\right)$ up to $2\left(\frac{H+B}{2}\right)$ with an increment of $0.25\left(\frac{H+B}{2}\right)$

Combining the effects of both walls gives a total shear lag factor as given in Equation 6:

$$U_M = \frac{BU_B + HU_H}{B+H} \tag{6}$$

Equation 6 can then be used for rectangular and square HSS as follows

- Rectangular HSS

$$U_H = \frac{3l^2}{3l^2 + H^2} \tag{7}$$

$$U_B = \frac{3l^2}{3l^2 + B^2} \tag{8}$$

$$U_M = \frac{BU_B + HU_H}{B+H} \tag{9}$$

- Square HSS

$$H = B,\ U_H = U_B \tag{10}$$

$$U_M = \frac{3l^2}{3l^2 + B^2} \tag{11}$$

Parametric Study

A parametric study was performed based on the proposed equations for rectangular and square HSS sections; equations 9 and 11, respectively, for AISC *Specification* Case 6b. A total of 76 HSS sections were evaluated, where the thickest walls were randomly selected from each HSS family. For each specimen, a configuration of gusset plates on the *H* side and *B* side and weld lengths varying from 0.25*H* up to 2*H*, with an increment size of 0.25*H*, was considered. Various such configurations were considered as illustrated in Table 3. A comparison of the shear lag factors calculated based on current AISC *Specification* Case 6b and the new equation is presented. Bar charts are plotted to compare the shear lag factors calculated from the two methods. Comparisons for HSS 12×10×½ (*H*×*B*×*t*) and HSS 20×4×½ (*H*×*B*×*t*) are provided in Figures 2 and 3, respectively. Refer to the trends observed in the following section for discussion of the ratios shown above the bars in Figures 2 and 3. Note that

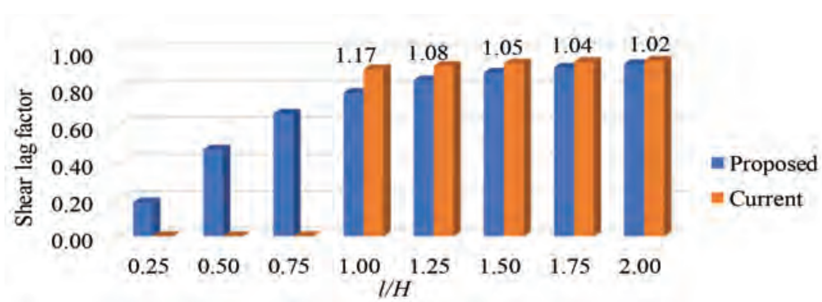
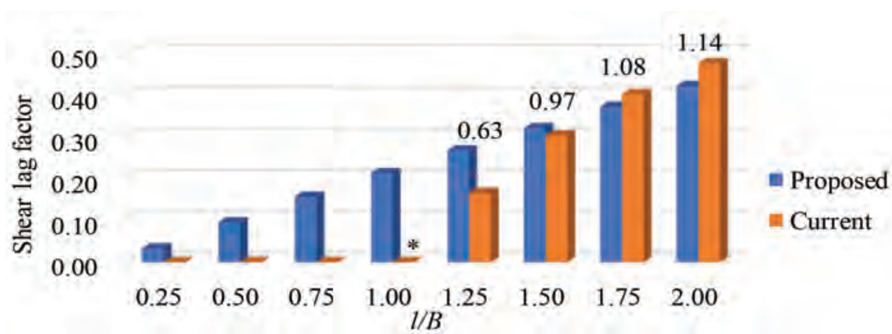


Fig. 2. Comparison of shear lag factor for the proposed equation with the current AISC Specification *Case 6b for* HSS 12×10×½ *(gusset plate on H side).*

the are no bars for the l/H and l/B less than 1, and no ratios are provided because the current AISC *Specification* Case 6b does not permit connections with l/H and l/B less than 1.

Parametric Study

- For HSS sections that are almost square or square—for example, a HSS 12×10×½—when the l/B ratio is greater than 1, the difference in the shear lag factor calculated from the current AISC *Specification* Case 6b procedure compared to the new proposed equation is negligible as seen in Figure 2 for the currently permitted cases where the weld length is at least as long as the width of the H dimension. However, the shear lag factor calculated from the proposed mathematical model is conservative when l/B ratio is equal to 1.
- Referring to Figure 2, another observation can be made that as the weld length is increased, the ratio of $U_{current}/U_{proposed}$ continually decreases from 1.17 to 1.02
- For gusset plates on the B side of the rectangular HSS member (refer to Figure 3), the shear lag factor calculated using the proposed model is higher as compared to the current Case 6b. This was observed when the l/B ratio is 1.25 and 1.5. When the ratio is 1.75 and 2.00, the proposed model predicts a smaller factor relative to the current AISC model.
- When using the current AISC model, HSS sections with high H/B ratios can result in negative shear lag values (i.e., when $l < \bar{x}$). This can be observed in Figure 3 for the HSS 20×4×½ with l/B equal to 1.00. In this case, the shear lag factor calculated using the proposed model is 0.27 and, from current Case 6b, is negative 0.04 (represented as zero in Figure 3). When l/B = 1.25, the proposed model predicts a value higher than the current Case 6b with the $U_{current}/U_{proposed}$ ratio being 0.63.
- For square HSS sections when l/B is 1.0, the ratio of $U_{current}/U_{proposed}$ is 1.17, and it continuously decreases to a final value of 1.02 when the l/B ratio is increased to 1.25, 1.5, up to 2, as seen in Figure 4.
- In general, it can be concluded that the shear lag factors calculated using the new proposed equations are slightly conservative as compared to current AISC *Specification* Case 6b except for the cases when H/B ratio is quite high. The proposed model also provides a non-zero shear lag factor for all cases and will not produce a negative value.



* *For l/B =1.00, U is negative using the current AISC* Specification *Case 6b.*

Fig. 3. Comparison of shear lag factor from the proposed equation with the current AISC Specification *Case 6b for* HSS 20×4×½ *(gusset plate on B side).*

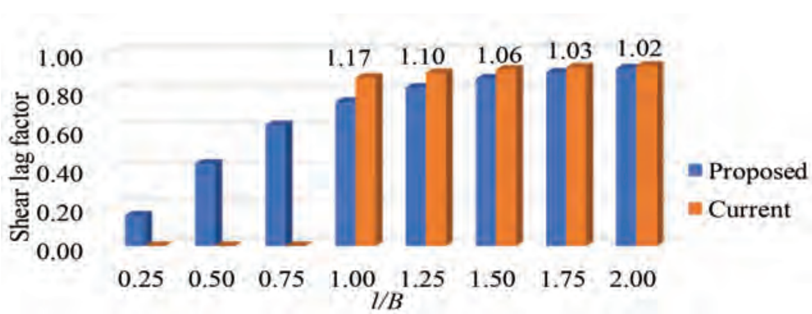


Fig. 4. Comparison of shear lag factor from the proposed equation with the current AISC Specification *Case 6b for* HSS 12×12×⅝ *(gusset plate on B).*

- The new proposed model permits weld lengths less than the perpendicular distance between the welds whereas the current procedure does not permit this.

FINITE ELEMENT ANALYSIS

The ABAQUS software package, using "explicit" analysis, was selected to perform the finite element analysis (FEA). A total of 31 members were modeled and evaluated in ABAQUS. The HSS members were modeled with square corners in lieu of rounded corners, and fillet welds were used in place of flare bevel groove welds, which would typically be required for the rounded corners of an HSS. One model was developed using rounded corners and flare bevel groove welds, and it was determined that the analysis is not sensitive to this particular detail. As such, for simplicity, square corners with fillets welds were used for this study. The preprocessing of the model was done in ABAQUS/CAE, and the ABAQUS dynamic explicit solver was selected to run all of the finite element analyses. The explicit procedure performs a large number of small time increments efficiently. It uses a form of the central difference time integration method to satisfy the dynamic equilibrium equations.

Material Properties

The modulus of elasticity was taken as 29,000 ksi, and Poisson's ratio was taken as 0.30 in accordance with industry standards. The material for the gusset plate was defined as elastic perfectly plastic with a yield stress of 50 ksi. A similar material type was defined for the weld material with an ultimate stress of 70 ksi. The stress strain curve for ASTM A500 Grade B, as shown in Figure 5, was used for the HSS steel material and was scaled to R_yF_y and R_tF_u to account for expected yield and tensile strengths, respectively.

The "ductile damage initiation criterion" was assigned to the HSS material. The ductile damage criterion deals with predicting the initiation of damage due to growth of voids and nucleation and provides a relationship among fracture strain, stress triaxiality, and strain rate. A reasonable value of 0.1995 was used for the fracture strain based on the stress strain curve used for the HSS material. When dynamic explicit analysis is performed, it is required to provide proper definition of density for all the materials. The density of the materials was taken as 0.1 kip/in^3 as recommended by Utsab Dhungana (2014). Table 4 shows the dimensions of the specimens as modeled in ABAQUS. At the connections, the weld lengths considered were 0.5H, 1.0H, 2.0H, and 1.0B, where H is the longer side of the HSS member. The parameters shown in Table 4 are calculated using the following equations:

Length of HSS section,

$$L_H = l + 4H + 20 \tag{12}$$

Length of gusset plate,

$$L_G = l + 2\frac{D}{16} \tag{13}$$

Width of gusset plate,

$$W = H + 2\tan(30)l \tag{14}$$

Thickness of the gusset plate,

$$T_G = \frac{\text{Applied load}}{F_yW} \tag{15}$$

Size of weld,

$$D = \frac{0.75\left(\text{Applied load}\right)}{1.392ln} \tag{16}$$

Expected ultimate tensile capacity of the HSS section

$$= R_tF_uA_g \tag{17}$$

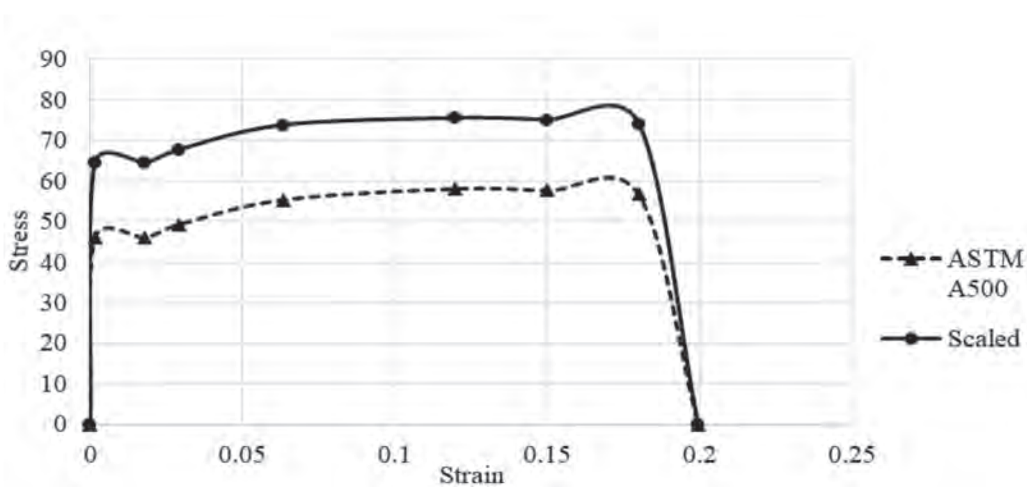


Fig. 5. Stress-strain curve for ASTM A500 Grade B.

Table 4. Model Details								
	HSS Section Details			Gusset Plate Details			Weld Details	
ID	Member	L_H (in.)	$R_tF_uA_g$ (kips)	L_G (in.)	W (in.)	T_G (in.)	D (16th of an inch)	l (in.)
1a*	4×4×½	38	528	6.50	11.3	0.50	36	2.0
1b		40		6.25	9.00	0.60	18	4.0
1c		44		9.13	13.5	0.40	9.0	8.0
2a*	6×6×⅝	47	1015	8.75	17.5	0.60	46	3.0
2b		50		8.88	13.0	0.80	23	6.0
2c		56		13.5	20.0	0.51	12	12
3a	8×8×⅝	56	1390	9.88	19.8	0.75	47	4.0
3b		60		11.0	17.0	0.80	24	8.0
3c		58		17.5	26.5	0.55	12	16
4a	12×6×⅝	74	1580	10.5	21.0	0.80	36	6.0
4b		80		14.3	26.0	0.65	18	12
4c		92		21.1	40.0	0.40	9.0	24
4d		50		10.5	15.0	1.10	36	6.0
5a	12×12×⅝	74	2140	12.1	24.3	0.90	49	6.0
5b		80		15.1	26.0	0.85	25	12
5c		92		25.6	40.0	0.55	13	24
6a	16×4×⅝	92	1770	11.8	26.0	0.70	30	8.0
6b		100		17.9	35.0	0.51	15	16
6c		116		33.0	53.0	0.35	8.0	32
6d		40		11.5	19.0	1.00	60	4.0
7a	16×16×⅝	92	2900	14.1	28.3	1.10	49	8.0
7b		100		19.1	34.5	0.85	25	16
7c		116		33.6	53.0	0.55	13	32
8a	20×4×½	110	1730	13.0	32.0	0.55	24	10
8b		120		21.5	23.0	0.40	12	20
8c		140		40.8	66.2	0.27	6.0	40
8d		40		9.88	15.8	1.20	47	4.0
9a	20×12×⅝	110	2900	15.0	31.6	0.95	40	10
9b		120		22.5	43.1	1.00	20	20
9c		140		41.3	66.2	0.45	10	40
9d		80		16.1	26	1.15	33	12

* Specimens were not used because the required weld sizes exceeded the width of the HSS wall.

For specimen 1a, the weld size required to transfer the expected tensile capacity of the HSS section was calculated to be 2.25 in. Two such welds would be required on each wall for the desired geometry of the analytical specimen, resulting in a dimension of 4.5 in., which is greater than the width of the HSS. A similar comment can be made for specimen 2a. Although a calculation can be made for the shear lag using the mathematical model, for the reasons previously mentioned, specimens 1a and 2a were not able to be modeled in ABAQUS.

The interaction of welds with the gusset plates and the HSS member was defined using tie constraints; tie constraints are found to be effective for welded connections (Ruffley, 2010). A fixed boundary condition was applied to the gusset plates attached to one end of the HSS member, and load was applied on the gusset plates attached to the other end of the HSS using kinematic coupling interactions. Figure 6 shows the assembly configuration, partitioning of the members, kinematic coupling interactions, and the welds attached to the gusset plate and HSS member via tie constraints for specimen HSS 12×12×⅝.

RESULTS AND DISCUSSION

All of the HSS specimens fractured at a section outside of the connection region. It was observed that the section fractured at a load smaller than the calculated expected tensile capacity of the section. All of the specimens yielded, and necking was clearly visible, followed by complete rupture of the HSS section. The reactions were taken at the gusset plate at the fixed boundary end of the specimen. The maximum value of the reaction at the fixed end is taken as the fracture load. The shear lag coefficient, U_F, was calculated as the ratio of this reaction force to the calculated expected tensile capacity $R_tF_uA_g$ of the HSS section. The FEA results for all the specimens are provided in Table 5.

The development of stresses in specimen 5b is shown in Figures 7(a), (b), (c), and (d). Figure 7(a) shows the specimen when there is zero load. Figure 7(b) shows the stress distribution when 85% of the applied load is transferred by the HSS section to the fixed end. At section *b-b* in Figure 7(b), it can be observed that there is a uniform distribution of stresses, whereas away from section *b-b* and closer to the connection region, there is nonuniform stress distribution. Similar stress distribution patterns were observed on each wall because of symmetry. In Figure 7(c) at section *c-c*, necking was observed, and the load transferred by the HSS section at that time was 86% of the applied load. Figure 7(d) shows fracture initiation at section *d-d* when the load transferred by the HSS section is equal to 87% of the applied load. Figure 8 shows an axonometric view of specimen 5b after fracture.

Table 6 shows a comparison between shear lag factors calculated from the three methods: AISC *Specification* Case 6b (U_A), mathematical model (U_M), and FEA (U_F).

Using the values provided in Table 6, graphs were generated, as shown in Figure 9, to compare the values of the shear lag factor from the three methods. Figure 9 compares the shear lag values for HSS 16×4×⅝ (specimen 6). It can be observed that when l/H is less than 1.0, current AISC *Specification* Case 6b does not permit such a connection, but the mathematical model gives a value of $U_M = 0.53$, which is conservative compared to the value obtained from FEA ($U_F = 0.76$). When l/H is 2.0, using the current AISC *Specification* Case 6b, the HSS section is 99% effective to carry or transfer load thus giving a shear lag factor of $U_A = 0.99$. However, the mathematical model and the FEA predict a shear lag value of $U_M = U_F = 0.94$.

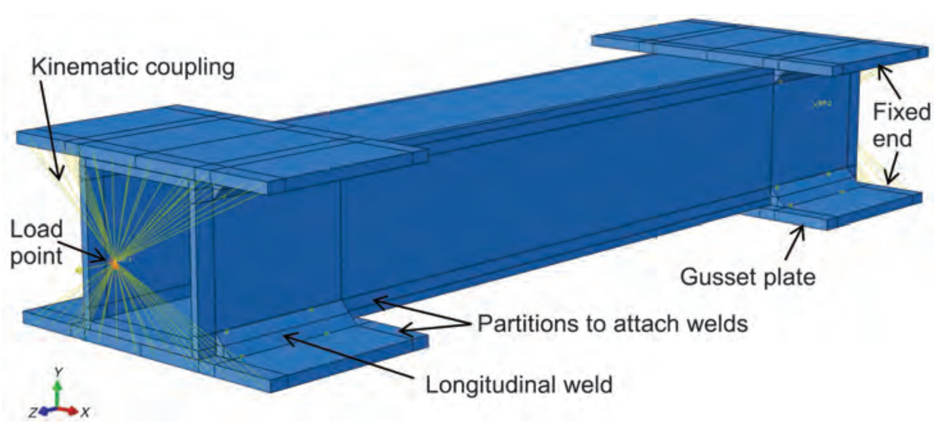
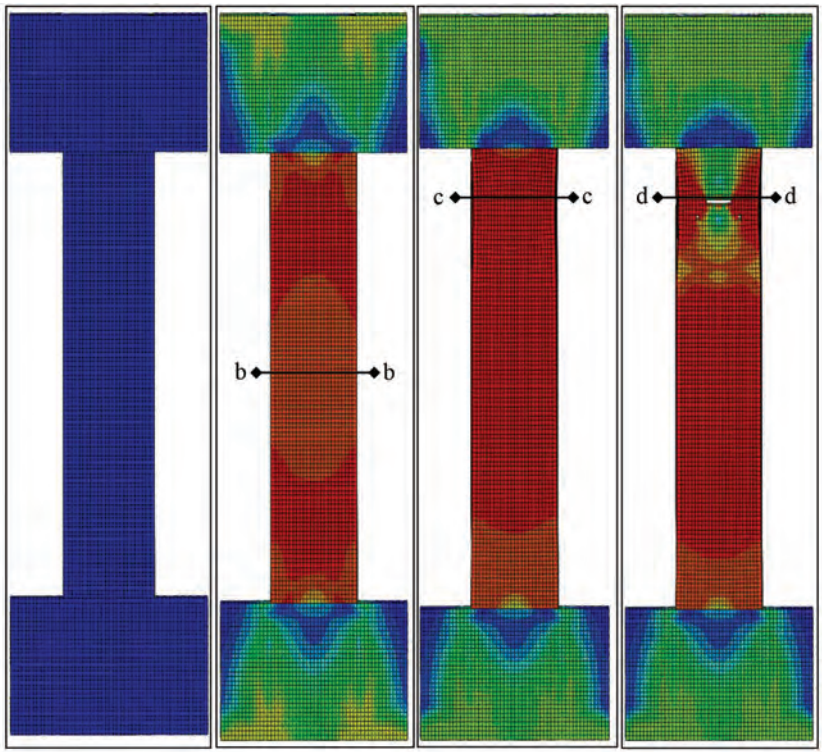


Fig. 6. Assembly configuration for HSS 12×12×⅝.

Table 5. FEA Results						
HSS Section Details			Weld Configuration		Failure Load (kips)	Shear Lag Factor, U_F
Specimen Designation	Member	$R_t F_u A_g$ (kips)	l (in.)			
1a	4×4×½	528	0.5H	2.0	NA	NA
1b			1.0H	4.0	485	0.92
1c			2.0H	8.0	460	0.87
2a	6×6×⅝	1010	0.5H	3.0	NA	NA
2b			1.0H	6.0	866	0.85
2c			2.0H	12	965	0.95
3a	8×8×⅝	1390	0.5H	4.0	1160	0.83
3b			1.0H	8.0	1190	0.86
3c			2.0H	16	1310	0.94
4a	12×6×⅝	1580	0.5H	6.0	1350	0.85
4b			1.0H	12	1370	0.86
4c			2.0H	24	1470	0.93
4d			1.0B	6.0	1480	0.94
5a	12×12×⅝	2140	0.5H	6.0	1670	0.78
5b			1.0H	12	1870	0.87
5c			2.0H	24	2010	0.94
6a	16×4×⅝	1770	0.5H	8.0	1340	0.76
6b			1.0H	16	1510	0.85
6c			2.0H	32	1660	0.94
6d			1.0B	4.0	1330	0.75
7a	16×16×⅝	2900	0.5H	8.0	2080	0.72
7b			1.0H	16	2480	0.86
7c			2.0H	32	2730	0.94
8a	20×4×½	1730	0.5H	10	1160	0.67
8b			1.0H	20	1619	0.93
8c			2.0H	40	1590	0.92
8d			1.0B	4.0	949	0.55
9a	20×12×⅝	2900	0.5H	10	2290	0.79
9b			1.0H	20	2650	0.91
9c			2.0H	40	2680	0.92
9d			1.0B	12	2440	0.84



(a) Unloaded (necking) *(b) 85% load* *(c) 86% load (fracture)* *(d) 87% load*

Fig. 7. Development of stresses and fracture initiation in specimen 5b.

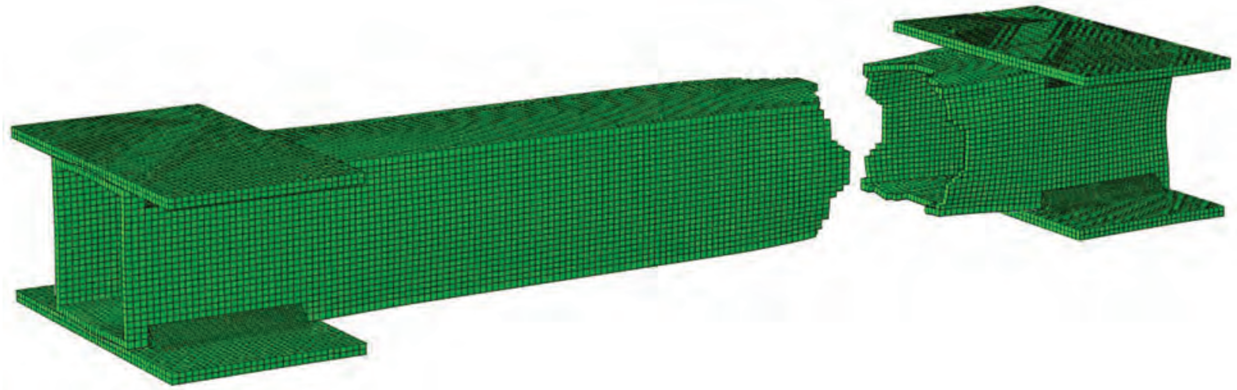


Fig. 8. Axonometric view of specimen 5b after fracture.

Table 6. Shear Lag Factors Comparison				
HSS Section Details		AISC *Specification*	Mathematical Model	FEA
Specimen Designation	Section	U_A^*	U_M	U_F
1a	4×4×½	NA	0.43	NA
1b		0.88	0.75	0.92
1c		0.94	0.92	0.87
2a	6×6×⅝	NA	0.16	NA
2b		0.88	0.75	0.85
2c		0.94	0.92	0.95
3a	8×8×⅝	NA	0.43	0.83
3b		0.88	0.75	0.86
3c		0.94	0.92	0.94
4a	12×6×⅝	NA	0.54	0.85
4b		0.96	0.81	0.86
4c		0.98	0.94	0.93
4d		0.67	0.54	0.94
5a	12×12×⅝	NA	0.43	0.78
5b		0.875	0.75	0.87
5c		0.94	0.92	0.94
6a	16×4×⅝	NA	0.53	0.76
6b		0.99	0.8	0.85
6c		0.99	0.94	0.94
6d		0.2	0.28	0.75
7a	16×16×⅝	NA	0.43	0.72
7b		0.88	0.75	0.86
7c		0.94	0.92	0.94
8a	20×4×½	NA	0.52	0.67
8b		0.99	0.79	0.93
8c		0.99	0.93	0.92
8d		NA	0.21	0.55
9a	20×12×⅝	NA	0.52	0.79
9b		0.94	0.8	0.91
9c		0.97	0.94	0.92
9d		0.74	0.61	0.84

* N/A denotes cases not currently permitted in AISC *Specification* for weld lengths less than *H* or *B*.

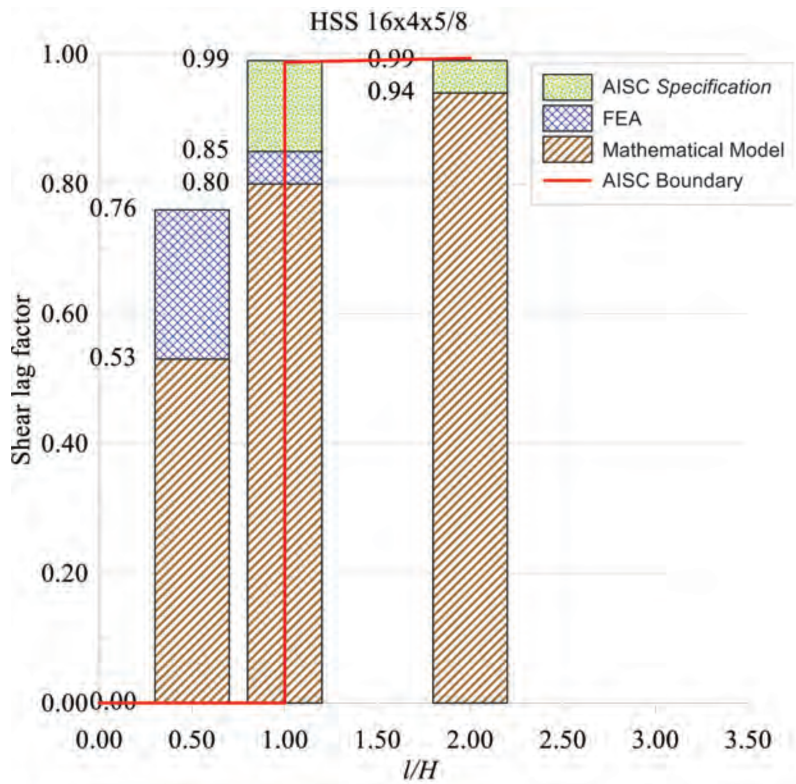


Fig. 9. Shear lag factor comparison.

CONCLUSIONS

Based on the presented results, the following can be concluded.

1. From the parametric study, the proposed mathematical model is conservative as compared to the current AISC *Specification* Table D3.1, Case 6b, except for some cases when the gusset plate is attached on the B side of the HSS.
2. The proposed model allows for weld lengths less than the distance between the welds.
3. The stress distribution pattern on which the mathematical model is based has good correlation with the FEA results.
4. The results from the finite element analyses are consistent with the values of the shear lag factors calculated using the proposed mathematical model.

RECOMMENDATIONS

1. It is recommended that AISC adopt the mathematical model proposed here and revise Case 6b of Table D3.1 of the AISC *Specification*.
2. Experimental testing should be performed to validate the results of the finite element analysis and provide more insights on some of the conclusions mentioned earlier.
3. The FEA models can be further refined by using actual material properties, measured from coupon tests, for the HSS ASTM A500 Grade B material.

SYMBOLS

A_g	Gross area of the HSS section, in.^{2}
B	Shorter side of the HSS section, in.
D	Weld size in sixteenths of an in.
F_y	Yield stress, ksi
F_u	Ultimate tensile stress of the HSS section, ksi
H	Longer side of the HSS section, in.
L_H	Length of the HSS section, in.
L_G	Length of the gusset plate, in.
P_B	Tension load carried by B wall, kips
P_H	Tension load carried by H wall, kips
P_u	Total tension load, kips
R_t	Correction factor
T_G	Thickness of the gusset plate, in.
U	Shear lag factor
U_A	Shear lag factor calculated using the current AISC Case 6b
U_B	Shear lag factor for B walls
$U_{Current}$	Shear lag factor calculated using the current AISC Case 6b
U_F	Shear lag factor obtained from the finite element analysis
U_H	Shear lag factor for H walls
U_M	Shear lag factor calculated using the mathematical model
$U_{Proposed}$	Shear lag factor calculated using the proposed mathematical model
W	Width of the gusset plate, in.
l	Length of the weld, in.
n	Number of welds
$\bar{x}$	Connection eccentricity, in.
w	Width of the HSS, in.

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Continuous Bracing Requirements for Constrained-Axis Torsional Buckling

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ABSTRACT

The design of floor and roof framing members is typically controlled by flexural demands; however, if a member serves as a chord or collector, it can also be subjected to significant axial compression. Continuous restraint provided by the floor or roof diaphragm is commonly assumed in design to provide adequate bracing of connected wide-flange members against minor-axis flexural buckling; however, these members are still susceptible to major-axis flexural buckling and potentially to torsional buckling about a constrained axis located at the top flange. In addition to the lateral restraint, floor and roof decking systems can also provide continuous torsional restraint through their flexural stiffness and strength. This restraint can be used to increase the calculated constrained-axis torsional buckling strength or inhibit the mode altogether. In this paper, the specific case of a wide-flange steel beam-column with both lateral and torsional restraint located at the top flange is investigated, and torsional bracing requirements are derived. The focus of the study is on continuous torsional bracing and its effect on the constrained-axis torsional buckling mode. The requirements are illustrated through a design example, and a parametric study is performed examining typical floor and roof decking system configurations, identifying cases where improved design efficiency can be achieved.

Keywords: Axial strength, constrained-axis torsional buckling, stability, wide-flange shapes.

INTRODUCTION

In building structures, floor or roof framing members that serve as chords or collectors can accumulate significant axial load as they transfer loads from the diaphragm to the vertical elements of the lateral-force-resisting system. The strength of these members in compression can be governed by a number of different buckling modes. Figure 1 depicts the potential member buckling modes that might control the axial strength. Although designers are familiar with the flexural buckling modes for wide-flange columns as shown in Figures 1(a) and 1(b), these members are also susceptible to torsional buckling when the bracing does not prevent twist and the unbraced length for torsional buckling is larger than the unbraced length for minor-axis flexural buckling. If the lateral bracing is located at the shear center (which coincides with the centroid for wide-flange sections), the torsional buckling mode as depicted in Figure 1(c) is possible. The strength for torsional buckling about the shear center can be predicted using the expressions in AISC *Specification* Section E4 (AISC, 2016). However, if the lateral bracing is offset from the centroid, the member can fail by constrained-axis torsional buckling as depicted in Figure 1(d) (for a case where the lateral bracing is located at the top of the top flange of the member), and the expressions provided in the Commentary on Section E4 are necessary as outlined in the next section of this paper.

When wide-flange beams are laterally restrained by a decking system, the limit states of minor-axis flexural buckling and torsional buckling [twisting about the shear center as shown in Figure 1(c)] are not applicable. As stated in AISC *Seismic Design Manual* (AISC, 2018), Table 8-1, this restraint occurs for bare steel deck (either steel roof deck or composite steel deck prior to placement of the concrete) when the ribs of the deck are perpendicular to the beam and for composite slabs (i.e., composite steel deck with concrete fill) in any orientation. The axial strength of the member for these cases is calculated from the limit states of major-axis flexural buckling and constrained-axis torsional buckling. Bare steel deck with ribs parallel to the beam is commonly assumed to provide inadequate lateral restraint and thus is not addressed in this paper.

The discussion thus far has focused on cases in which the decking system only restrains lateral movement. However, decking systems also provide torsional restraint to the beam. While it is conservative to neglect this restraint, in many cases the restraint is sufficient to brace the beam against constrained-axis torsional buckling or otherwise significantly increase the calculated strength. In this paper, bracing requirements for constrained-axis torsional buckling are developed. A detailed example is presented that illustrates

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the use of these requirements for common applications such as the design of collector beams or other members subjected to large axial forces. Finally, the results of a broad parametric study are presented with observations that highlight configurations where the benefits of the torsional restraint are most effective.

Although the "beams" in the flooring or roofing system that are discussed in this paper are actually "beam-columns" due to the combined axial force and bending moment, the term "beams" is used throughout the paper for convenience. The focus of the bracing requirements in the paper is the stiffness and strength requirements necessary to control the constrained-axis torsional buckling mode from the axial force component.

CONSTRAINED-AXIS TORSIONAL BUCKLING STRENGTH

The design of members for compression is governed by the provisions of Chapter E in the AISC *Specification* (AISC, 2016). The nominal axial compressive strength in the elastic and inelastic range is determined from the column curve expressions given in Section E3:

$$P_n = F_{cr} A_g \tag{1}$$

$$F_{cr} = \begin{cases} 0.658^{\left(F_y/F_e\right)} F_y & \text{when } F_y/F_e \le 2.25 \\ 0.877 F_e & \text{when } F_y/F_e > 2.25 \end{cases} \tag{2}$$

where

A_g = gross cross-sectional area, in.^{2}
F_{cr} = critical stress, ksi
F_e = elastic buckling stress, ksi
F_y = yield stress, ksi
P_n = nominal compressive strength, kips

If a section has slender elements, the provisions of Section E7 can be used to determine the effective area, A_e, that is used in place of A_g in Equation 1. The methodology for accounting for slender elements is covered in more detail later in this section as well as in the example problem that is presented in this paper.

For constrained-axis torsional buckling with bracing offset along the minor axis, as shown in Figure 2, the elastic buckling stress is given by AISC *Specification* (2016) Commentary Equation C-E4-1, shown here as Equation 3. The fundamental equation for this mode was developed by Timoshenko and Gere (1961). The expression presented in the Commentary is slightly modified as recommended by Errera and Apparao (1976) and Helwig and Yura (1999) to account for practical limitations that often occur in practice.

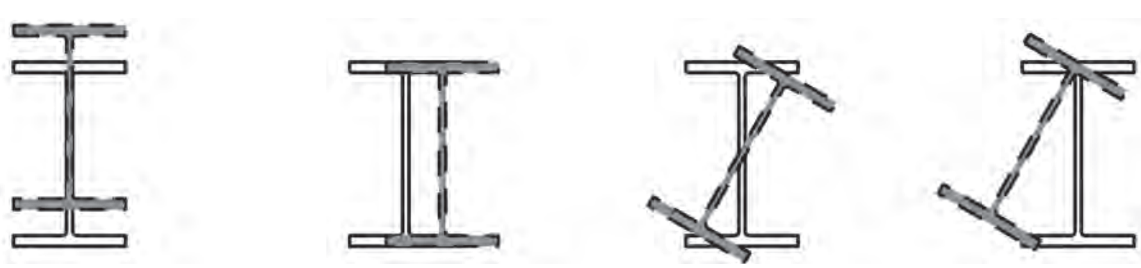
$$F_e = \omega \left[\frac{\pi^2 E I_y}{L_{cz}^2} \left(\frac{h_o^2}{4} + a^2 \right) + GJ \right] \frac{1}{A_g r_o^2} \tag{3a}$$

$$r_o^2 = r_x^2 + r_y^2 + a^2 \tag{3b}$$

where

E = modulus of elasticity of steel = 29,000 ksi
G = shear modulus of steel = 11,200 ksi
I_y = minor-axis moment of inertia, in.^{4}
J = torsional constant, in.^{4}
L_{cz} = effective torsional length, in.
a = distance from centroid to brace point, in.
h_o = distance between flange centroids, in.
r_x = major-axis radius of gyration, in.
r_y = minor-axis radius of gyration, in.
ω = finite brace stiffness factor = 0.9

Although wide-flange sections with slender elements are not typically used as columns, many beam-type sections are slender for axial compression. As a result, a wide-flange section that might be used in a flooring system may possess slender elements for compression. In such a case, the interaction of constrained-axis torsional buckling and local buckling should be accounted for in accordance with AISC *Specification* Section E7. The effective area, A_e, is computed as a function of the critical buckling stress, F_{cr}. For the case



Major-axis flexural buckling (a) | *Minor-axis flexural buckling (b)* | *Torsional buckling (c)* | *Constrained-axis torsional buckling (d)*

Fig. 1. Buckling modes.

of wide-flange steel members, A_e is computed as shown in Equation 4, where the effective widths of the flange and web are computed using Equations 5 and 6, respectively.

$$A_e = A_g - 4(b_f/2 - b_e)t_f - (h - h_e)t_w \tag{4}$$

$$b_e = \begin{cases} b_f/2 & \text{when } \lambda_f \le \lambda_{rf}\sqrt{F_y/F_{cr}} \\ \left(1-0.33\dfrac{\lambda_{rf}}{\lambda_f}\sqrt{\dfrac{F_y}{F_{cr}}}\right)1.49\dfrac{\lambda_{rf}}{\lambda_f}\sqrt{\dfrac{F_y}{F_{cr}}}\dfrac{b_f}{2} & \text{when } \lambda_f > \lambda_{rf}\sqrt{F_y/F_{cr}} \end{cases} \tag{5a}$$

$$\lambda_{rf} = 0.56\sqrt{\frac{E}{F_y}} \tag{5b}$$

$$\lambda_f = \frac{b_f}{2t_f} \tag{5c}$$

$$h_e = \begin{cases} h & \text{when } \lambda_w \le \lambda_{rw}\sqrt{F_y/F_{cr}} \\ \left(1-0.24\dfrac{\lambda_{rw}}{\lambda_w}\sqrt{\dfrac{F_y}{F_{cr}}}\right)1.31\dfrac{\lambda_{rw}}{\lambda_w}\sqrt{\dfrac{F_y}{F_{cr}}}h & \text{when } \lambda_w > \lambda_{rw}\sqrt{F_y/F_{cr}} \end{cases} \tag{6a}$$

$$\lambda_{rw} = 1.49\sqrt{\frac{E}{F_y}} \tag{6b}$$

$$\lambda_w = \frac{h}{t_w} \tag{6c}$$

Further discussion of constrained-axis torsional buckling strength including design tables and example calculations are provided in Liu et al. (2013). AISC *Design Guide* 25 (Kaehler et al., 2011) describes recommendations for computing the constrained-axis torsional buckling strength for singly symmetric and tapered members.

BRACING REQUIREMENTS

Effective stability bracing must possess adequate stiffness and strength. Brace stiffness and strength requirements to control torsional buckling and constrained-axis torsional buckling were developed by Helwig and Yura (1999), based upon the results of a parametric finite element study. For the case of discrete bracing and considering inelastic buckling, the required torsional brace stiffness is given by Equation 7.

$$\beta_T = A\frac{\left\{P_u r_o^2 - P_{ny}^*\left[\left(h_o^{\,2}/4\right)+a^2\right]\right\}^2}{\left(n_b 4\tau EI_y/L\right)\left[\left(h_o^{\,2}/4\right)+a^2\right]} \tag{7a}$$

$$A = 4 - \frac{2a}{h_o} \ge 2.0 \tag{7b}$$

$$P_{ny}^* = 0.877\tau\frac{\pi^2 EI_y}{L^2} \tag{7c}$$

where

L = beam span, in.

P_u = required axial compressive strength, kips

n_b = number of intermediate braces

β_T = required brace stiffness

τ = stiffness reduction factor

The stiffness reduction factor, τ, is applied to account for the reduced stiffness and strength due to inelasticity and local buckling and has been modified from the original presentation given in Helwig and Yura (1999) to reflect the provisions in the current AISC *Specification* (2016), as shown in Equation 8.

$$\tau = \begin{cases} -2.724\left(P_u/P_y\right)\ln\left(P_u/xP_y\right) & \text{when } P_u/xP_y > 0.39 \\ x & \text{when } P_u/xP_y \le 0.39 \end{cases} \tag{8a}$$

$$x = \frac{A_e|_{F_{cr}=P_u/A_e}}{A_g} \tag{8b}$$

where

P_y = axial yield strength, kips

$= F_y A_g$

x = ratio of the effective area (calculated iteratively for a critical stress equal to the required axial strength divided by the effective area) to the gross area

Alternatively, for previous versions of the AISC *Specification* (e.g., 2010) where the strength of members with slender elements was calculated using the net reduction factor, Q, the stiffness reduction factor should be taken as Equation 9, where Q is calculated at the required axial strength.

$$\tau = \begin{cases} -2.724\left(P_u/QP_y\right)\ln\left(P_u/QP_y\right) & \textit{when } P_u/QP_y > 0.39 \\ 1.0 & \textit{when } P_u/QP_y \le 0.39 \end{cases} \tag{9}$$

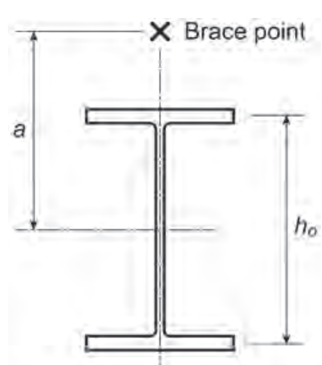


Fig. 2. Bracing offset along minor axis.

The required brace stiffness (Equation 7a) is modified for the case of a member braced continuously at the top flange by (1) substituting in the location of the brace at the centroid of the top flange ($a = h_o/2$) and (2) converting from discrete to continuous bracing by setting the term n_b/L equal to unity (note that after this change, β_T is expressed as the torsional stiffness per unit length). Additionally, a resistance factor of $\phi = 0.75$ is applied according to AISC *Specification* Appendix 6 (2016). The resulting expression is given by Equation 10.

$$\beta_T = \frac{1.5}{\phi}\frac{\left(P_u r_o^2 - P_{ny}^* h_o^2/2\right)^2}{\tau E I_y h_o^2} \tag{10}$$

There are a number of factors that can affect the total brace stiffness, including the stiffness of the decking system as well as other stiffness components such as cross-sectional distortion due to flexibility in the beam web and connection stiffness between the decking system and beam. In the derivation presented herein, the total brace stiffness is based on the stiffness of the decking system acting in series with the distortional stiffness of the beam web, β_{sec}, given in Equation 11 (AISC *Specification* Equation A-6-13). In this paper, the connection between the decking system and the beam is assumed to be rigid. Accordingly, the required stiffness of the decking system, β_{Tb}, is given by Equation 12 (AISC *Specification* Equation A-6-10). Note that if the web distortional stiffness is less than or equal to the required total brace stiffness ($\beta_{sec} \le \beta_T$), then Equation 12 is negative and the required total brace stiffness cannot be achieved regardless of the stiffness provided by the decking system.

$$\beta_{sec} = \frac{3.3Et_w^3}{12h_0} \tag{11}$$

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \dfrac{\beta_T}{\beta_{sec}}\right)} \tag{12}$$

where β_{Tb} = required stiffness of the decking system.

As noted by Helwig and Yura (1999), the required brace stiffness limits the twist of the beam due to the applied loading to a value equal to the initial twist imperfection. Thus, the resulting brace strength requirement is the product of the required brace stiffness, β_T, and the initial twist imperfection θ_0, as shown in Equation 13. The assumed initial twist imperfection is based on a configuration where one flange is straight and the other flange has a lateral initial out-of-straightness with maximum lateral imperfection of $L/500$. This results in an assumed initial twist imperfection given by Equation 14.

$$M_{br} = \beta_T \theta_0 \tag{13}$$

$$\theta_0 = \frac{L}{500h_o} \tag{14}$$

Often, the stiffness provided by the decking system is more than sufficient to brace the beam, leaving strength as the limiting brace requirement. In these cases, the required moment strength can be reduced as a function of the ratio of required to provided stiffness as given by Equation 15 based on Equation C-A-6-2 from the AISC *Specification* Commentary (2016). The total provided stiffness, β_{prov}, is computed using an expression for springs in series given in Equation 16, where the stiffness provided by the deck, $\beta_{prov\text{-}b}$, is combined with the web distortional stiffness, β_{sec}.

$$M_{br} = \frac{\beta_T \theta_o}{\left(2 - \beta_T/\beta_{prov}\right)} \tag{15}$$

$$\frac{1}{\beta_{prov}} = \frac{1}{\beta_{prov-b}} + \frac{1}{\beta_{sec}} \tag{16}$$

The example outlined in the following section illustrates the use of these bracing provisions for details commonly found in practice.

DESIGN EXAMPLE

Given:

Consider a 24-ft-long W18×35 ASTM A992 beam supporting a composite slab that has a total depth of 6 in., normal weight concrete ($f_c' = 3$ ksi), and 3-in. deep, 20-ga. composite steel deck. The deck spans perpendicular to the beams that are spaced at 10 ft. Steel headed stud anchors of ASTM A108 material with a diameter of ¾ in. are provided at a spacing, *s*, of 1 ft. The beam is assumed to be simply supported with twist restrained but warping deformations permitted at the ends.

Solution:

The problem is worked in multiple steps to illustrate the various modes as well as the contributions of the bracing. Baseline calculations are first carried out to understand the flexural and torsional modes, neglecting the contributions of the composite slab. It should be understood that although the lateral stiffness of the composite slab is neglected in the calculation of the minor-axis flexural buckling mode, the lateral stiffness is required for the constrained-axis torsional buckling mode to occur. The shear stiffness of most typical decking systems (bare composite steel or roof deck with ribs perpendicular to the braced member or composite slabs in any orientation) is most often much larger than necessary to provide adequate lateral bracing.

Baseline Calculations

Determine the nominal strength for the limit state of constrained-axis torsional buckling neglecting the torsional stiffness provided by the decking system.

From AISC *Manual* Table 2-4, the beam yield stress is:

$F_y = 50$ ksi

From AISC *Manual* Table 1-1 (AISC, 2017a), the relevant section properties for the beam are:

$A_g = 10.3$ in.^{2}	$I_x = 510$ in.^{4}	$I_y = 15.3$ in.^{4}	$J = 0.506$ in.^{4}
$b_f = 6.00$ in.	$d = 17.7$ in.	$t_w = 0.300$ in.	$t_f = 0.425$ in.
$r_x = 7.04$ in.	$r_y = 1.22$ in.	$b_f/2t_f = 7.06$	$h/t_w = 53.5$
$h_o = 17.3$ in.	$h = 16.1$ in.	$k_1 = 0.75$ in.	

The effective length of the beam is taken as the full span because twist is restrained but warping deformations are permitted at the ends of the beam.

$$\begin{aligned} L_{cz} &= L \\ &= (24 \text{ ft})(12 \text{ in./ft}) \\ &= 288 \text{ in.} \end{aligned}$$

To determine the elastic buckling stress, F_e, using Equation 3a, first determine a and r_o^2.

$$\begin{aligned} a &= \frac{h_o}{2} \\ &= \frac{17.3 \text{ in.}}{2} \\ &= 8.65 \text{ in.} \end{aligned} \tag{3b}$$

$$\begin{aligned} r_o^2 &= r_x^2 + r_y^2 + a^2 \\ &= (7.04 \text{ in.})^2 + (1.22 \text{ in.})^2 + (8.65 \text{ in.})^2 \\ &= 126 \text{ in.}^2 \end{aligned}$$

$$F_e = \omega\left[\frac{\pi^2 EI_y}{L_{cz}^2}\left(\frac{h_o^2}{4} + a^2\right) + GJ\right]\frac{1}{A_g r_o^2} \tag{3a}$$

$$= 0.90\left[\frac{\pi^2(29{,}000 \text{ ksi})(15.3 \text{ in.}^4)}{(288 \text{ in.})^2}\left(\frac{(17.3 \text{ in.})^2}{4} + (8.65 \text{ in.})^2\right) + (11{,}200 \text{ ksi})(0.506 \text{ in.}^4)\right]\frac{1}{(10.3 \text{ in.}^2)(126 \text{ in.}^2)}$$

$$= 9.42 \text{ ksi}$$

Compute the critical stress, F_{cr}, using Equation 2.

$$\begin{aligned} \frac{F_y}{F_e} &= \frac{50 \text{ ksi}}{9.42 \text{ ksi}} \\ &= 5.31 > 2.25 \end{aligned}$$

Because $F_y/F_e > 2.25$, the critical stress is determined as:

$$\begin{aligned} F_{cr} &= 0.877F_e \\ &= 0.877(9.42 \text{ ksi}) \\ &= 8.26 \text{ ksi} \end{aligned} \tag{2}$$

Classify the flange for local buckling.

$$\lambda_{rf} = 0.56\sqrt{\frac{E}{F_y}} \tag{5b}$$

$$= 0.56\sqrt{\frac{(29{,}000\text{ ksi})}{(50\text{ ksi})}}$$

$$= 13.5$$

$$\lambda_f = \frac{b_f}{2t_f} \tag{5c}$$

$$= 7.06 < \lambda_{rf}$$

Thus, the flanges are nonslender, and $b_e = b_f/2$.

Classify the web for local buckling.

$$\lambda_{rw} = 1.49\sqrt{\frac{E}{F_y}} \tag{6b}$$

$$= 1.49\sqrt{\frac{(29{,}000\text{ ksi})}{(50\text{ ksi})}}$$

$$= 35.9$$

$$\lambda_w = \frac{h}{t_w} \tag{6c}$$

$$= 53.5 > \lambda_{rw}$$

Thus, the web is slender for axial compression.

To calculate the effective width, h_e, using Equation 6a, first determine the relationship between λ_w and $\lambda_{rw}\sqrt{\frac{F_y}{F_{cr}}}$.

$$\lambda_{rw}\sqrt{\frac{F_y}{F_{cr}}} = (35.9)\sqrt{\frac{(50\text{ ksi})}{(8.26\text{ ksi})}} \tag{6}$$

$$= 88.3 > \lambda_w = 53.5$$

Thus, the critical stress is low enough that no reduction is necessary.

$$h_e = h \tag{6a}$$

$$A_e = A_g \qquad \text{(from Eq. 4)}$$

Compute the axial compressive strength, P_n, using Equation 1.

$$P_n = F_{cr}A_e \tag{1}$$

$$= F_{cr}A_g$$

$$= (8.26\text{ ksi})(10.3\text{ in.}^2)$$

$$= 85.1\text{ kips}$$

The nominal strengths of the other modes of buckling assuming no bracing provided by the decking system are presented in Table 1, where P_{nx}, P_{ny}, P_{nz}, and P_{nca} are the computed strengths for the major-axis flexural, minor-axis flexural, torsional, and

Table 1. Axial Strength for Various Buckling Modes	
Buckling Mode	**Axial Strength**
Major-axis flexural buckling	P_{nx} = 408 kips
Minor-axis flexural buckling	P_{ny} = 46.4 kips
Torsional buckling	P_{nz} = 165 kips
Constrained-axis torsional buckling	P_{nca} = 85.1 kips

constrained-axis torsional buckling modes, respectively. A comparison of the buckling capacities provides some interesting insights into the behavior. The minor-axis flexural buckling strength is significantly smaller than the torsional buckling strength because the unbraced length is the same for these two modes. Torsional buckling will always yield a larger strength than minor-axis flexural buckling for wide-flange members of the same unbraced length. The table also demonstrates the significant reduction in the strength for the constrained-axis torsional buckling mode versus the torsional buckling mode, which will always be the case when the location of bracing is offset along the minor-axis of the section. These values vary with the unbraced length as shown in Figure 3.

Bracing Requirement Checks

For the configuration described previously, determine if the decking system is adequate to brace the beam against constrained-axis torsional buckling at a required axial load of $P_u = 250$ kips.

To determine if the decking system is adequate to brace the beam, both stiffness and strength checks are necessary. First, the required stiffness is calculated, followed by an evaluation of the available stiffness. The required strength is then calculated (including a reduction based on the ratio of required stiffness to provided stiffness) and finally the available strength.

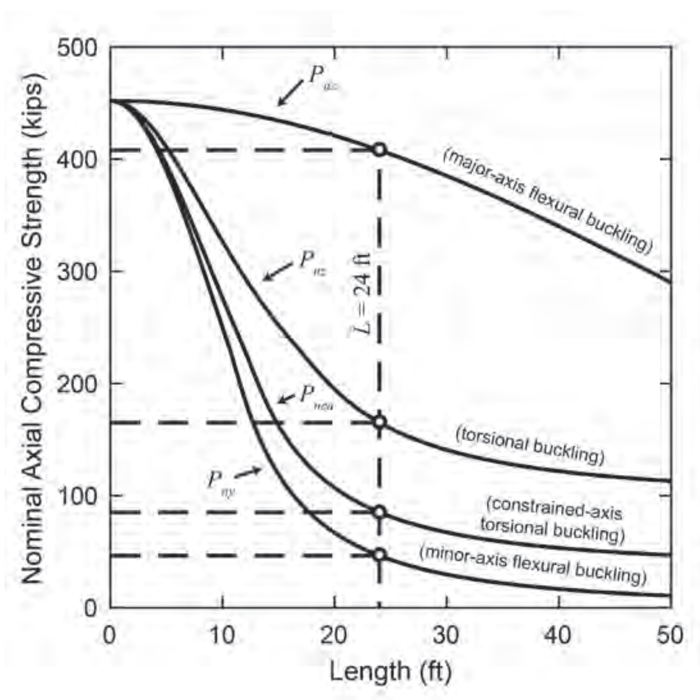


Fig. 3. Variation of axial strength with length.

Required Brace Stiffness

Recall that the value of x is the ratio of the effective area to the gross area at a critical stress equal to the required strength divided by the effective area. The x value needs to be determined iteratively. For this case, $x = 0.985$, as confirmed in the following calculations. A trial critical stress, F_{cr}, is calculated according to the definition of x given in Equation 8b.

$$A_e = xA_g \qquad \text{(from Eq. 8b)}$$
$$= (0.985)(10.3 \text{ in.}^2)$$
$$= 10.1 \text{ in.}^2$$

$$F_{cr} = \frac{P_u}{A_e}$$
$$= \frac{(250 \text{ kips})}{(10.1 \text{ in.}^2)}$$
$$= 24.8 \text{ ksi}$$

As demonstrated in the baseline calculations, the flanges are nonslender ($b_e = b_f/2$) and the web is slender. Calculate the effective width, h_e, using Equation 6.

$$\lambda_{rw}\sqrt{\frac{F_y}{F_{cr}}} = (35.9)\sqrt{\frac{(50 \text{ ksi})}{(24.8 \text{ ksi})}}$$
$$= 51.0 < \lambda_w$$

$$h_e = \left(1 - 0.24\frac{\lambda_{rw}}{\lambda_w}\sqrt{\frac{F_y}{F_{cr}}}\right)1.31\frac{\lambda_{rw}}{\lambda_w}\sqrt{\frac{F_y}{F_{cr}}}h \qquad (6a)$$
$$= \left(1 - 0.24\frac{(35.9)}{(53.5)}\sqrt{\frac{(50 \text{ ksi})}{(24.6 \text{ ksi})}}\right)1.31\frac{(35.9)}{(53.5)}\sqrt{\frac{(50 \text{ ksi})}{(24.6 \text{ ksi})}}(16.1 \text{ in.})$$
$$= 15.5 \text{ in.}$$

Compute the effective area, A_e, using Equation 4.

$$A_e = A_g - (h - h_e)t_w \qquad (4)$$
$$= (10.3 \text{ in.}^2) - \left[(16.1 \text{ in.}) - (15.5 \text{ in.})\right](0.300 \text{ in.})$$
$$= 10.1 \text{ in.}^2$$

Compute the value x using Equation 8b.

$$x = \frac{A_e}{A_g} \qquad (8b)$$
$$= \frac{(10.1 \text{ in.}^2)}{(10.3 \text{ in.}^2)}$$
$$= 0.981$$

The result is within the rounding error of the trial value; thus, $x = 0.985$ is confirmed.

Compute the stiffness reduction factor, τ, using Equation 8a.

$$P_y = A_g F_y$$
$$= (10.3\text{ in.}^2)(50\text{ ksi})$$
$$= 515\text{ kips}$$

$$\frac{P_u}{xP_y} = \frac{(250\text{ kips})}{(0.985)(515\text{ kips})}$$
$$= 0.493 > 0.39$$

Because $\frac{P_u}{xP_y} > 0.39$, τ is calculated as:

$$\tau = -2.724\left(\frac{P_u}{P_y}\right)\ln\left(\frac{P_u}{xP_y}\right) \tag{8a}$$
$$= -2.724\left[\frac{(250\text{ kips})}{(515\text{ kips})}\right]\ln\left[\frac{(250\text{ kips})}{(0.985)(515\text{ kips})}\right]$$
$$= 0.936$$

Compute P_{ny}^*, using Equation 7c.

$$P_{ny}^* = 0.877\tau\frac{\pi^2 EI_y}{L^2} \tag{7c}$$
$$= 0.877(0.936)\frac{\pi^2(29{,}000\text{ ksi})(15.3\text{ in.}^4)}{(288\text{ in.})^2}$$
$$= 43.3\text{ kips}$$

The value P_{ny}^* represents the minor-axis flexural buckling strength considering the full length of the beam and with the level of inelasticity expected at the required axial strength. The value varies as shown in Figure 4.

Compute the required total brace stiffness, β_T, using Equation 10.

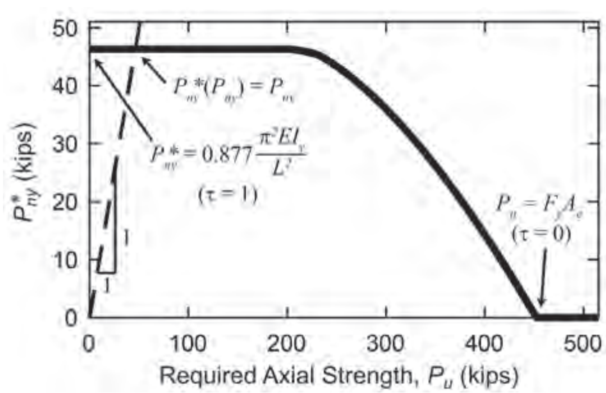


Fig. 4. Variation of P_{ny}^* *with required axial strength.*

$$\beta_T = \frac{1.5}{\phi}\frac{\left(P_u r_o^2 - P_{ny}^* {h_o}^2/2\right)^2}{\tau E I_y {h_o}^2} \tag{10}$$

$$= \frac{1.5}{(0.75)}\frac{\left[(250\text{ kips})(126\text{ in.}^2) - (43.3\text{ kips})(17.3\text{ in.})^2/2\right]^2}{(0.936)(29{,}000\text{ ksi})(15.3\text{ in.}^4)(17.3\text{ in.})^2}$$

$$= 10.1\text{ kip-in./rad/in.}$$

Compute the distortional stiffness of the beam web, β_{sec}, using Equation 11.

$$\beta_{\text{sec}} = \frac{3.3Et_w^3}{12h_0} = \frac{3.3(29{,}000\text{ ksi})(0.300\text{ in.})^3}{12(17.3\text{ in.})} \tag{11}$$

$$= 12.5\text{ kip-in./rad/in.}$$

Compute the required stiffness of the decking system, β_{Tb}, using Equation 12.

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{\text{sec}}}\right)} \tag{12}$$

$$= \frac{(10.1\text{ kip-in./rad/in.})}{\left[1 - \frac{(10.1\text{ kip-in./rad/in.})}{(12.5\text{ kip-in./rad/in.})}\right]}$$

$$= 52.6\text{ kip-in./rad/in.}$$

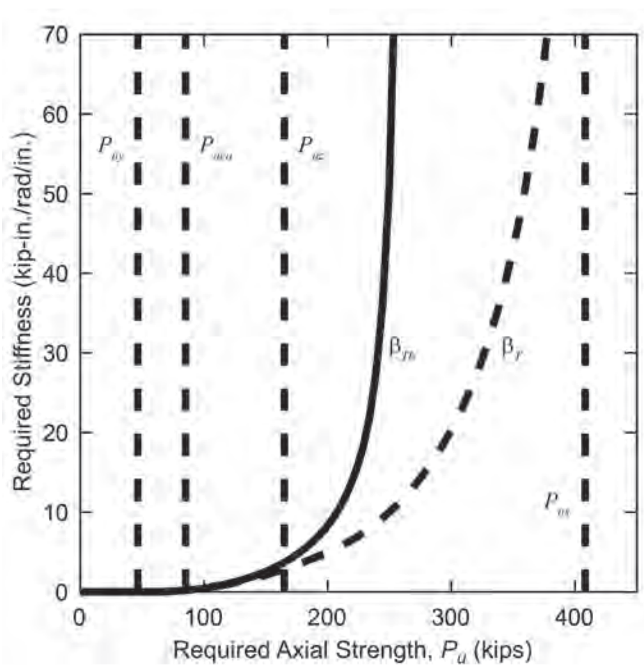


Fig. 5. Variation of required torsional bracing.

The required stiffness of the decking system varies with the required axial strength as shown in Figure 5. As can be seen for this case, the required stiffness is zero (or near zero due to simplifications in the derivation of the stiffness requirement) at the constrained-axis torsional buckling strength. It is a fairly modest value at the torsional buckling strength. The required stiffness of the decking system increases asymptotically to infinity for a required axial strength less than the major-axis flexural buckling strength, indicating that the distortional stiffness of the web is insufficient to make major-axis buckling the controlling mode of failure. Although it would be possible to reduce or eliminate cross-sectional distortion by providing transverse web stiffeners or struts between the decking system and bottom flange at a few locations along the length of the beam, the fabrication costs are likely to make such detailing impractical. Selecting a beam with a stockier web would be more prudent. It should be noted that the W18×35 has a web slenderness of 53.5 and is among the most slender rolled W-shaped sections. Sections with stockier webs will have fewer issues with cross-sectional distortion.

Available Brace Stiffness

To determine the available stiffness, the decking system is assumed to act in single curvature on each side of the span, as shown in Figure 6, and the adjacent beams are assumed to have similar demands on the decking system as the example beam. Thus, the stiffness contribution from each side is $2EI/L$, where EI is the flexural rigidity of the decking system and L is the deck span (Figure 7). Rigid connections are assumed between the beam and decking system.

On one side of the beam (left side in Figure 6), the concrete is in tension, and thus only the moment of inertia of the composite steel deck is conservatively relied upon. The moment of inertia of 20-ga. composite steel deck with a 3-in. depth is 0.920 in.^{4}/ft (Sputo, 2014). On the other side of the beam (right side in Figure 6), the concrete is in compression, and the moment of inertia of the composite slab is considered. The design moment of inertia of a 6-in. total depth composite slab constructed with 20-ga. composite steel deck with a 3-in. depth and normal weight concrete is 13.34 in.^{4}/ft (Sputo, 2014). Perimeter beams or those adjacent to an opening should only consider the stiffness of the side where the decking system is present.

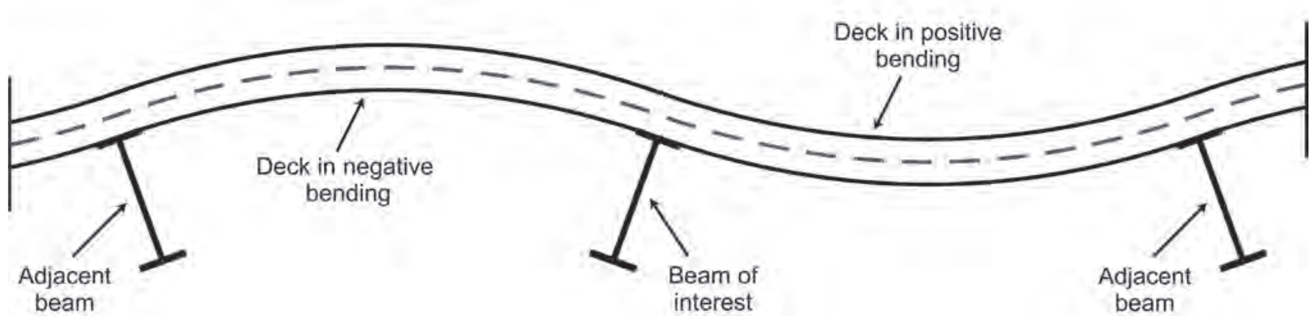


Fig. 6. Assumed deck bending mode.

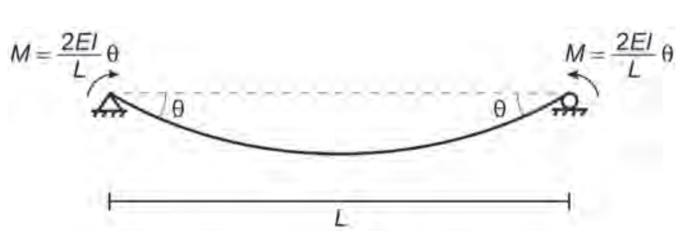


Fig. 7. Deck bending stiffness model.

Calculate the stiffness provided by the decking system, $\beta_{prov\text{-}b}$, as the sum of the contributions from both sides.

$$\beta_{prov-b} = \left(\frac{2EI}{L}\right)_{\text{left}} + \left(\frac{2EI}{L}\right)_{\text{right}}$$

$$= \frac{2(29{,}000 \text{ ksi})(0.920 \text{ in.}^4/\text{ft})\ 1 \text{ ft}/12 \text{ in.}}{(10 \text{ ft})(12 \text{ in./1 ft})} + \frac{2(29{,}000 \text{ ksi})(13.34 \text{ in.}^4/\text{ft})(1 \text{ ft}/12 \text{ in.})}{(10 \text{ ft})(12 \text{ in./1 ft})}$$

$$= 574 \text{ kip-in./rad/in.}$$

Because the stiffness provided by the decking system ($\beta_{prov\text{-}b}$ = 574 kip-in./rad/in.) is greater than the required stiffness of the decking system (β_{Tb} = 52.6 kip-in./rad/in.), the decking system has sufficient stiffness to brace the beam against constrained-axis torsional buckling at the required axial strength.

Required Brace Strength

The required brace strength is determined in accordance with Equation 15, which includes a reduction for surplus provided stiffness. Although the available stiffness of the decking system is substantially larger than the required stiffness, the effects of distortion need to be considered using Equation 16 to determine the provided total brace stiffness, β_{prov}.

$$\beta_{prov} = \left(\frac{1}{\beta_{prov-b}} + \frac{1}{\beta_{sec}}\right)^{-1} \qquad \text{(from Eq. 16)}$$

$$= \left[\frac{1}{(574 \text{ kip-in./rad/in.})} + \frac{1}{(12.5 \text{ kip-in./rad/in.})}\right]^{-1}$$

$$= 12.2 \text{ kip-in./rad/in.}$$

This calculation demonstrates that cross-sectional distortion associated with web flexibility dominates the stiffness of this bracing system.

Compute the initial twist imperfection, θ_o, using Equation 14.

$$\theta_0 = \frac{L}{500h_o} \qquad (14)$$

$$= \frac{(288 \text{ in.})}{500(17.3 \text{ in.})}$$

$$= 0.033 \text{ rad}$$

Compute the required brace strength, M_{br}, using Equation 15.

$$M_{br} = \frac{\beta_T \theta_o}{(2 - \beta_T/\beta_{prov})} \qquad (15)$$

$$= \frac{(10.1 \text{ kip-in./rad/in.})(0.033 \text{ rad})}{\left[2 - (10.1 \text{ kip-in./rad/in.})/(12.2 \text{ kip-in./rad/in.})\right]}$$

$$= 0.287 \text{ kip-in./in.}$$

Available Brace Strength

The calculation of the available strength will vary depending on the specific situation, but will generally include assessment of the strength of the decking system, the strength of the connection between the decking system and beam, and the strength of the beam web.

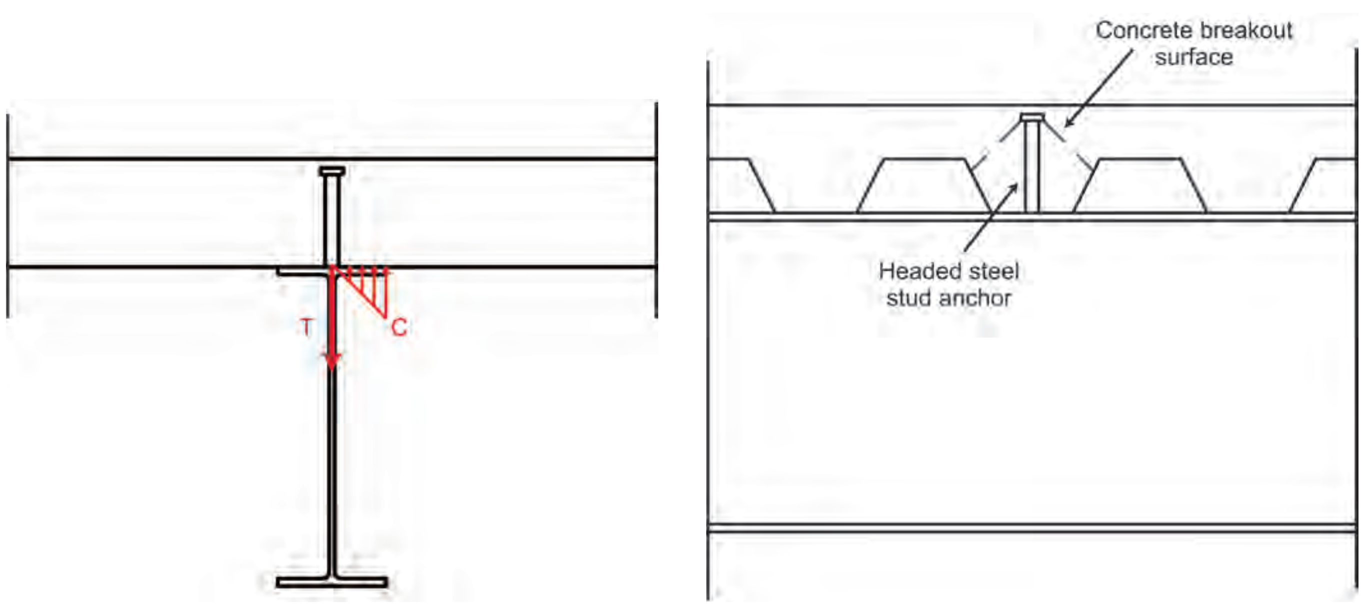
Deck Bending Strength

On one side of the beam (left side in Figure 6), the decking system is in negative bending, the concrete is assumed to be cracked, and thus only the strength of the bare composite steel deck remains. The design strength of 20-ga. deck that is 3 in. deep is $\phi M_{n,neg}$ = 1.72 kip-in./in. (Sputo, 2014). On the other side of the beam (right side in Figure 6), the decking system is in positive bending, and the composite slab can be relied upon. The design strength of 20-ga. composite steel deck that is 3 in. deep and supporting 6-in. total depth normal weight concrete is $\phi M_{n,pos}$ = 5.10 kip-in./in. (Sputo, 2014). These strengths are parallel and additive; thus, the strength of the decking system is calculated as:

$$\begin{aligned}\phi M_n &= \phi M_{n,neg} + \phi M_{n,pos} \\ &= (1.72 \text{ kip-in./in.}) + (5.10 \text{ kip-in./in.}) \\ &= 6.82 \text{ kip-in./in.}\end{aligned}$$

Connection Strength

The connection between the decking system and the beam is provided by bearing and through the steel headed stud anchor. The interface between steel beams and composite slab has been studied extensively in the past, but predominantly under shear loading. Little guidance is available in the literature for the calculation of the twisting moment strength of the connection. For the purposes of this work, the moment is assumed to be taken by a force couple formed through tension in each steel headed stud anchor and compression on the beam flange as shown in Figure 8(a). The moment strength per unit length along the beam is then computed as the product of the strength of the controlling limit state of the force couple and the lever arm [taken as $b_f/3$ for the triangular stress distribution shown in Figure 8(a)] divided by the stud spacing. The strength of the force couple is computed from the limit states of steel headed stud tensile rupture, concrete pullout, concrete breakout, concrete crushing, and beam flange yielding.



(a) Beam cross-section view *(b) Beam side view*

Fig. 8. Beam-to-deck connection.

Steel Tensile Strength

The strength of the steel headed stud anchor in tension is computed using the provisions of AISC *Specification* Section I8.3b (2016). The diameter of the shank of the steel headed stud anchor is 0.75 in. and the ultimate strength, F_u, is 65 ksi.

$$\phi R_n = \phi F_u A_s$$
$$= (0.75)(65\text{ ksi})\frac{\pi}{4}(0.75\text{ in.})^2$$
$$= 21.5\text{ kips}$$

Concrete Pullout Strength

Pullout of the headed stud anchor is determined using the provisions of ACI 318, Section 17.4.3 (ACI, 2014). For a ¾-in.-diameter steel headed stud anchor, the diameter of the head is 1.25 in.

Compute the bearing area, A_{brg}, as

$$A_{brg} = \frac{\pi}{4}d_{head}^2 - \frac{\pi}{4}d_{shank}^2$$
$$= \frac{\pi}{4}(1.25\text{ in.})^2 - \frac{\pi}{4}(0.75\text{ in.})^2$$
$$= 0.785\text{ in.}^2$$

Compute the pullout strength as

$$\phi R_n = \phi 8 A_{brg} f_c'$$
$$= (0.70)8(0.785\text{ in.}^2)(3\text{ ksi})$$
$$= 13.2\text{ kips}$$

Concrete Breakout Strength

Hawkins and Mitchell (1984) derived an equation for the breakout strength of steel headed stud anchors embedded in a composite slab with composite steel deck based on the height of the stud, H_s, and rib width at mid-height of the composite steel deck, w_r. The height of the stud is taken as the total depth of the composite slab (6 in.) minus the required cover (0.5 in.) minus the height of the head of the stud (0.375 in.). Thus, the height of the stud, H_s, is taken as 5.125 in. The rib width at mid-height of the composite steel deck, w_r. is taken as 6 in.

$$A_c = 2\sqrt{2}H_s w_r$$
$$= 2\sqrt{2}(5.125\text{ in.})(6.0\text{ in.})$$
$$= 87.0\text{ in.}^2$$
$$\phi R_n = \phi 4\sqrt{f_c'}A_c$$
$$= (0.75)4\sqrt{3{,}000\text{ psi}}(87.0\text{ in.}^2)$$
$$= 14{,}300\text{ lb}\left(\frac{1\text{ kip}}{1{,}000\text{ lb}}\right)$$
$$= 14.3\text{ kips}$$

As an alternative approach, Lawson and Hicks (2011) recommend taking the tensile strength of a headed stud anchor as 85% of its shear strength. This method yields results in strengths somewhat higher than by evaluating each limit state individually.

Concrete Crushing Strength

The bearing strength of the concrete is computed using the provisions of AISC *Specification* Section J8 (AISC, 2016).

Compute the bearing area, A_1, as the product of deck rib width (4.5 in.) and half the beam flange width [Figure 8(a)].

$$A_1 = (4.5\text{ in.})(b_f/2)$$
$$= (4.5\text{ in.})(6.00\text{ in.}/2)$$
$$= 13.5\text{ in.}^2$$

Compute the bearing strength as

$$\phi P_P = \phi 0.85 f'_c A_1$$
$$= (0.65)(0.85)(3\text{ ksi})(13.5\text{ in.}^2)$$
$$= 22.4\text{ kips}$$

Beam Flange Bending Strength

The strength of the beam flange in bending is computed based on the plastic moment strength of a length of the flange equal to the stud spacing at a distance k_1 from the beam centerline and assuming the compressive couple force acts at a distance $b_f/3$ from the beam centerline.

$$\phi M_{n,flange} = \phi F_y \frac{t_f^2 s}{4}$$
$$= (0.90)(50\text{ ksi})\frac{(0.425\text{ in.})^2(12\text{ in.})}{4}$$
$$= 24.4\text{ kip-in.}$$

$$\phi R_n = \frac{\phi M_{n,flange}}{(b_f/3 - k_1)}$$
$$= \frac{(24.4\text{ kip-in.})}{[(6.00\text{ in}/3) - (0.75\text{ in.})]}$$
$$= 19.5\text{ kips}$$

From the preceding calculations, the controlling limit state for the force within the couple is concrete pullout of the steel headed stud anchor. Combining that result with the lever arm of the couple and the spacing of the steel headed stud anchors, the controlling connection strength is calculated as:

$$\phi M_n = \frac{\phi R_n}{s}\left(\frac{b_f}{3}\right)$$
$$= \frac{(13.2\text{ kips})}{(12\text{ in.})}\left(\frac{6.00\text{ in.}}{3}\right)$$
$$= 2.20\text{ kip-in./in.}$$

Beam Web Bending Strength

It is expected that the out-of-plane bending demand on the beam web is less than the required strength of the connection; however, it is unclear how much less. For these calculations, the bending demand of the web is conservatively taken as the required brace strength. The strength of the beam web bending out of plane is computed as the plastic moment strength:

$$\phi M_n = \phi F_y \frac{t_w^2}{4}$$
$$= (0.90)(50\text{ ksi})\frac{(0.300\text{ in.})^2}{4}$$
$$= 1.01\text{ kip-in./in.}$$

Table 2. Parametric Study Properties

Property	Case A	Case B
Decking system stiffness, $\beta_{prov\text{-}b}$ (kip-in./rad/in.)	400	30
Decking system strength, ϕM_n (kip-in./in.)	5	1
Connection force couple strength, ϕR_n (kips)	10	0.4

Given that the strength of the beam web (ϕM_n = 1.01 kip-in./in.) is less than that of either the decking system or the connection, it is the controlling strength of the brace. Further, this strength is sufficient because it exceeds the required brace strength (M_{br} = 0.287 kip-in./in.).

Example Summary

Having met both the stiffness and strength requirements, the decking system is adequate to brace the beam against constrained-axis torsional buckling at the required axial strength.

The calculations presented in this example are not intended to cover every possible situation. Engineering judgment is necessary —especially when computing available braced stiffness and strength—to ensure that rational and reliable load paths exist and that all relevant sources of flexibility have been accounted for. Cases such as perimeter beams, where a decking system is present on only one side of the member, or bare composite steel or roof deck with a wide bottom flat, where the connection between the decking system and beam may be flexible, should be approached with special care.

Parametric Study

The calculations to check the bracing requirements, such as presented in the example, are quite involved and are generally impractical for typical designs. Noting that, in this section, a parametric study is presented with two goals: (1) to identify conditions that are most beneficial for relying upon the rotational bracing provided by the decking system and (2) to develop rules of thumb such that in specific typical cases, the bracing provided by the decking system can be taken advantage of without the need to perform full calculations.

Of interest in this study is the calculation of the axial strength of the beam for a given decking system configuration. The relevant parameters of the decking system configuration are the provided stiffness, $\beta_{prov\text{-}b}$, and the moment strength, ϕM_n (noting that the decking system, connection, or beam web may control strength). The two requirements can be stated as Equations 17 and 18.

$$\beta_{prov} \geq \beta_T \tag{17}$$

$$\phi M_n \geq M_{br} \tag{18}$$

Alternatively, noting the relationship between β_T and M_{br} in Equation 15, the requirements can be combined as in Equations 19 and 20, where the moment requirement has been converted into a stiffness requirement.

$$\beta_{limit} \geq \beta_T \tag{19}$$

$$\beta_{limit} = \min\left(\beta_{prov}, \frac{2\phi M_n}{\theta_o + \phi M_n / \beta_{prov}}\right) \tag{20}$$

The axial strength is calculated iteratively, not directly, as the maximum axial load for which the beam can be considered braced against constrained-axis torsional buckling (i.e., satisfying Equation 19). This axial load, termed $P_{u,braced}$, is analogous to the design constrained-axis torsional buckling strength and comparable to design strengths for other buckling modes.

For the parametric study, two generic bracing cases are defined. The first case, Case A, is representative of a composite slab similar to what was computed in the example presented in the previous section. The three defining factors are the decking system stiffness, $\beta_{prov\text{-}b}$, the decking system strength, ϕM_n, and the connection force couple strength, ϕR_n. Each of these values, given in Table 2, is marginally lower than what was calculated in the example so as to broaden the range of applicability. The second case, Case B, is representative of a steel roof deck (with ribs perpendicular to the beam). The strengths and stiffnesses are accordingly lower than that of the composite slab, including for the connection force couple strength which is controlled, for example, by the uplift strength of a spot weld.

The variation of $P_{u,braced}$ with length for a W18×35 beam is shown in Figure 9. The results in the figure show the two different cases (Table 2), an additional case of β_{limit} = 0, and the constrained-axis torsional buckling design strength computed without accounting for any bracing. It is expected, logically, that for $\beta_{limit} = 0$, the maximum permitted axial load will equal the constrained-axis torsional buckling strength from the code equations (i.e., $P_{u,braced} = \phi P_{nca}$). However, as can be seen in Figure 9, the two values differ. The differences are due to simplifications made in the derivation of the brace stiffness requirements and the fact that the derivation was not intended to be applicable for zero brace stiffness. It is expected that for practical values of β_{limit}, the method is accurate.

The parametric study was performed by computing $P_{u,braced}$ for each wide flange shape in the AISC Shapes Database (AISC, 2017b) with a weight less than or equal to 150 lb/ft, for both generic bracing configurations (Table 2) and for a range of lengths from $L/d = 5$ to $L/d = 50$, where d is the section depth. The results of the parametric study are presented in Figure 10. In each of the plots of Figure 10, the ratio of $P_{u,braced}$ to a design strength is plotted as a function of the unbraced length of the beam. Each line represents one cross section. The lines are shaded based on the cross-section web slenderness, h/t_w. The plots on the left-hand side are for Case A, while the plots on the right hand side are for Case B.

In Figures 10(a) and 10(b), $P_{u,braced}$ is plotted with respect to the design constrained-axis torsional buckling strength. These plots show most directly the increase in calculated strength by accounting for the rotational stiffness of the decking system. With the greater stiffness and strength of Case A, the calculated strength increases by as much as a factor of 6. Whereas for Case B, the calculated strength increases by as much as a factor of 3, but in some cases decreases, indicating that steel roof deck is not effective in providing bracing against constrained-axis torsional buckling. For both cases, the largest increases are for cross sections with higher web slenderness.

In Figures 10(c) and 10(d), $P_{u,braced}$ is plotted with respect to the design torsional buckling strength. Again, the greater stiffness and strength of Case A are demonstrated in the higher strength ratios. A key result shown in Figure 10(c) is that for Case A and for all wide flange cross sections weighing 150 lb/ft or less, the constrained-axis torsional buckling strength is nearly or at least equal to the calculated torsional buckling strength [the ratio $P_{u,braced}$ to ϕP_{nx} has a minimum value of 0.975 in Figure 10(c)]. For comparison, the ratio ϕP_{nca} to ϕP_{nz} has a minimum value of 0.371 over the same range (Figure 11). The lowest values in that ratio occur for cross sections with lower web slenderness.

In Figures 10(e) and 10(f), $P_{u,braced}$ is plotted with respect to the design major-axis flexural buckling strength. The second key result is shown in Figure 10(e). For Case A and for all wide flange cross sections weighing 150 lb/ft or less, the constrained-axis torsional buckling strength is at least half of the calculated major-axis flexural buckling strength [the ratio $P_{u,braced}$ to ϕP_{nx} has a minimum value of 0.577 in Figure 10(e)]. For comparison, the ratio ϕP_{nca} to ϕP_{nx} has a minimum value of 0.129 over the same range (Figure 12). The lowest values in that ratio occur for cross sections with higher web slenderness.

For the cases examined, the key results from Figure 10 are that the constrained-axis torsional buckling strength is nearly equal to or greater than either the torsional buckling strength or half the major-axis flexural buckling strength; these are potentially useful rules of thumb. These results can be conservatively used for cases within the range of the study—namely, wide flange shapes weighing 150 lb/ft or less, decking system properties meeting or exceeding those listed in Table 2 for Case A, and span lengths between 5 and 50 times the beam depth.

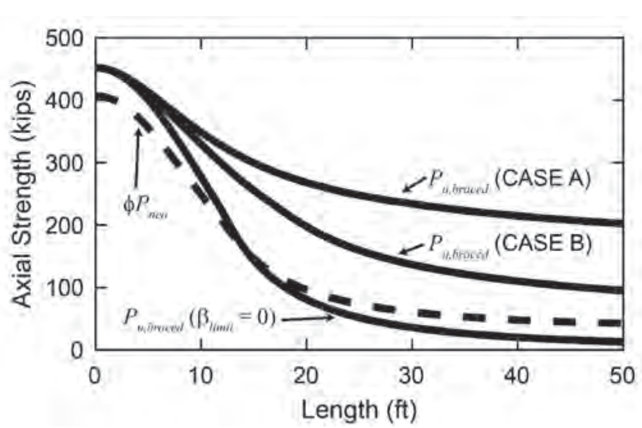


Fig. 9. Variation of axial strength with length.

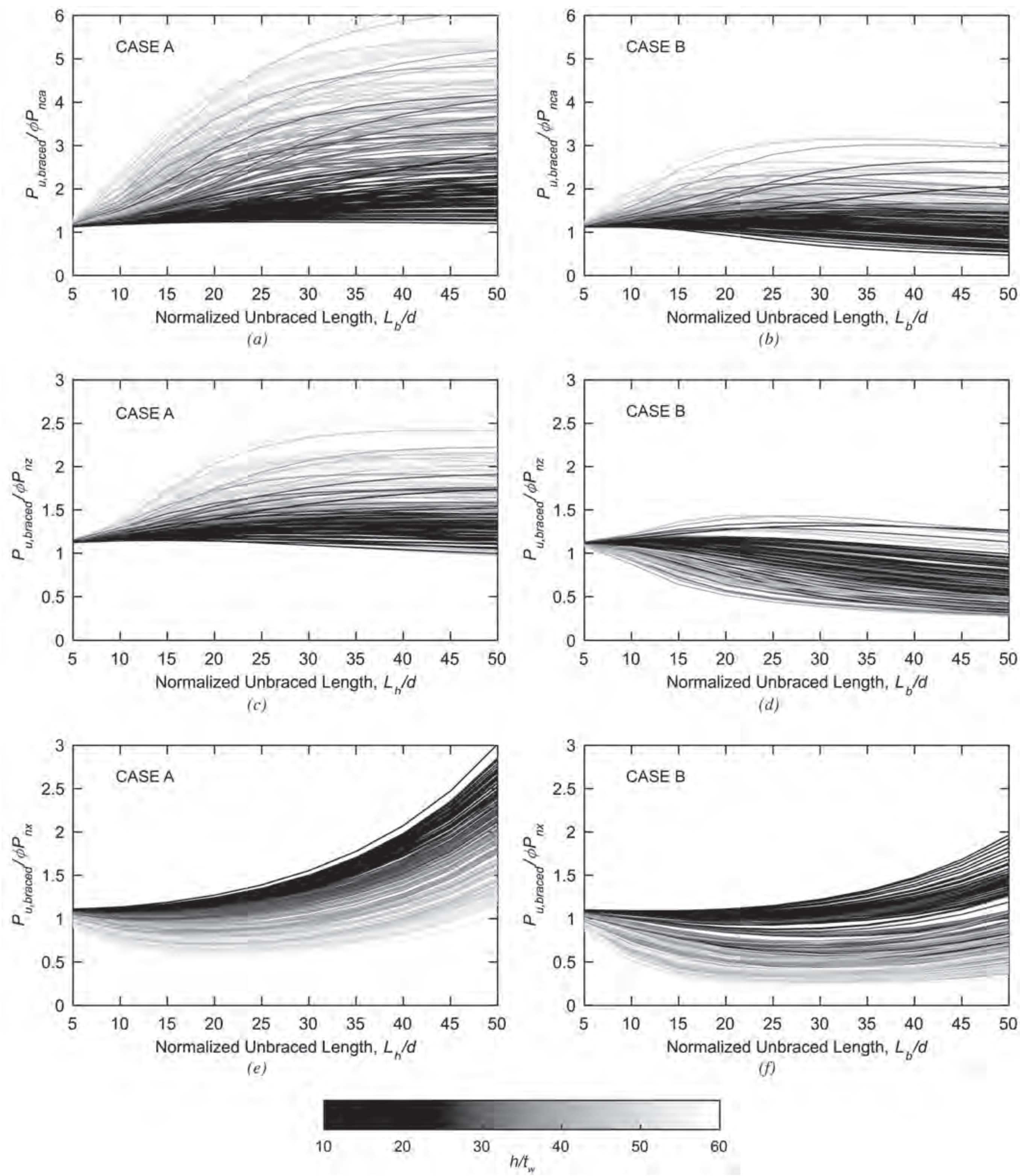


Fig. 10. Parametric study results.

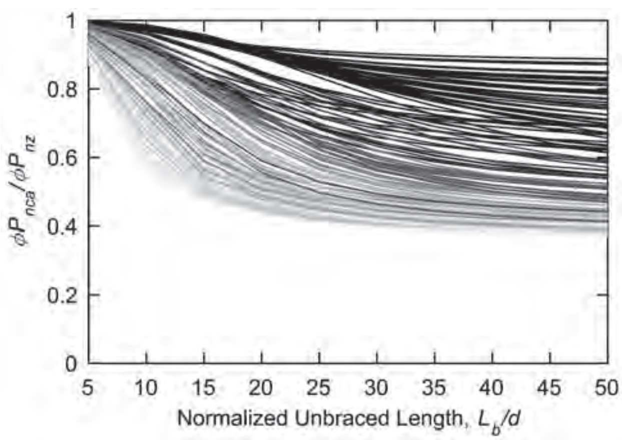


Fig. 11. Comparison of axial strength, P_{nca} *to* P_{nz}.

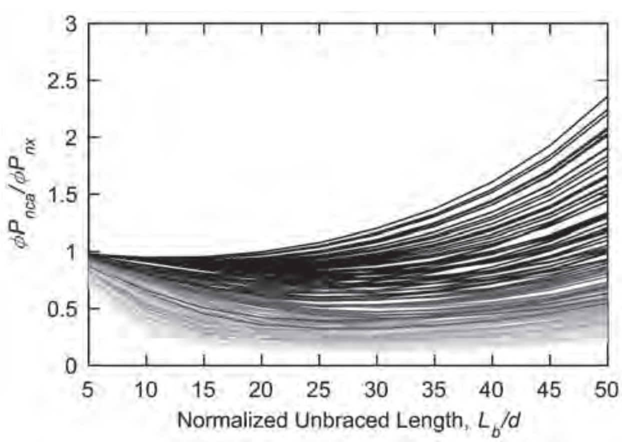


Fig. 12. Comparison of axial strength, P_{nca} *to* P_{nx}.

CONCLUSIONS

Accounting only for the lateral restraint provided by steel roof deck or composite floor deck may lead to a situation where the computed axial strength of a beam is controlled by constrained-axis torsional buckling. The stiffness and strength requirements developed in this paper allow for the inclusion of the rotational restraint provided by steel roof deck or composite floor deck. The inclusion of this restraint can lead to a significant increase in the calculated axial strength. The requirements are based on the results of prior studies, and an example was presented to illustrate their use. A parametric study was performed to identify cases when accounting for the rotational restraint was most beneficial and to develop rules of thumb, with the key result being that for wide-flange beams that weigh 150 lb/ft or less, the rotational restraint provided by typical composite deck to interior beams is sufficient to achieve strengths of at least 50% of the major-axis flexural buckling strength.

SYMBOLS

A Factor applied to ideal torsional brace stiffness to control deformations and brace moments

A_1 Loaded area of concrete, in.^{2}

A_{brg} Bearing area, in.^{2}

A_c Concrete breakout failure area, in.^{2}

A_e Effective area, in.^{2}

A_g Gross cross-sectional area, in.^{2}

A_s Cross-sectional area of steel headed stud anchor, in.^{2}

E Modulus of elasticity of steel = 29,000 ksi

EI Flexural rigidity of the decking system, kip-in.^{2}

F_{cr} Critical stress, ksi

F_e Elastic buckling stress, ksi

F_y Yield stress, ksi

F_u Tensile strength, ksi

G Shear modulus of steel = 11,200 ksi

H_s Height of steel headed stud anchor, in.

I_x Major-axis moment of inertia, in.^{4}

I_y Minor-axis moment of inertia, in.^{4}

J Torsional constant, in.^{4}

L Beam or deck span, in.

L_{cz} Effective torsional length, in.

M_{br} Required brace strength, kip-in.

M_n Nominal moment strength, kip-in.

$M_{n,flange}$ Nominal moment strength of flange, kip-in.

$M_{n,neg}$ Nominal negative moment strength of the decking system, kip-in.

$M_{n,pos}$ Nominal positive moment strength of the decking system, kip-in.

P_n Nominal compressive strength, kips

P_{nca} Nominal compressive strength for the limit state of constrained-axis torsional buckling, kips

P_{nx} Nominal compressive strength for the limit state of major-axis flexural buckling, kips

P_{ny} Nominal compressive strength for the limit state of minor-axis flexural buckling, kips

P_{ny}^* Minor-axis flexural buckling strength with full column length, kips

P_{nz} Nominal compressive strength for the limit state of torsional buckling, kips

P_p Nominal bearing strength, kips

P_u Required axial compressive strength, kips

$P_{u,braced}$ Maximum required axial compressive strength that can be considered braced, kips

P_y Axial yield strength, kips

Q Net reduction factor accounting for all slender compression elements

R_n Nominal strength, kips

a Distance from centroid to brace point, in.

b_f Width of flange, in.

b_e Reduced effective width of half-flange, in.

d Depth of section, in.

d_{head} Head diameter of steel headed stud anchor, in.

d_{shank} Shank diameter of steel headed stud anchor, in.

f_c' Specified compressive strength of concrete, ksi

h Web height, in.

h_e Reduced effective web height, in.

h_o Distance between flange centroids, in.

k_1 Distance from web center line to flange toe of fillet, in.

n_b Number of intermediate braces

r_o Polar radius of gyration, in.

r_x Major-axis radius of gyration, in.

r_y Minor-axis radius of gyration, in.

s Spacing of steel headed stud anchors, in.

t_f Thickness of flange, in.

t_w Thickness of web, in.

w_r Rib width at mid-height of composite steel deck, in.

x Ratio of the effective area to the gross area

β_{limit} Limiting provided brace stiffness including consideration of brace strength

β_{prov} Total provided brace stiffness

$\beta_{prov\text{-}b}$ Provided stiffness of the decking system

β_{sec} Web distortional stiffness

β_T Required brace stiffness

β_{Tb} Required stiffness of the decking system

λ_f Width-to-thickness ratio for flange

λ_{rf} Limiting width-to-thickness ratio for flange

λ_w Width-to-thickness ratio for web

λ_{rw} Limiting width-to-thickness ratio for web

ϕ Resistance factor

θ_0 Initial twist imperfection

τ Stiffness reduction factor

ω Finite brace stiffness factor = 0.90

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