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 1⅛ 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

James A. Swanson, PhD, Associate Professor, Department of Civil and Architectural Engineering and Construction Management, University of Cincinnati, Cincinnati, Ohio. Email: swansojs@ucmail.uc.edu (corresponding)

Gian Andrea Rassati, PhD, Associate Professor, Department of Civil and Architectural Engineering and Construction Management, University of Cincinnati, Cincinnati, Ohio. Email: rassatga@ucmail.uc.edu

Chad M. Larson, President, LeJeune Bolt Co., Burnsville, Minn. Email: clarson@lejeunebolt.com

Paper No. 2019-15

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 ≤ 1¼ in.
⅝ in.	L ≤ 1½ in.
¾ in.	L ≤ 1¾ in.
⅞ in.	L ≤ 2 in.
1 in.	L ≤ 2¼ in.
1⅛ in.	L ≤ 2¾ in.
1¼ in.	L ≤ 2¾ in.
1⅜ in.	L ≤ 3¼ in.
1½ in.	L ≤ 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

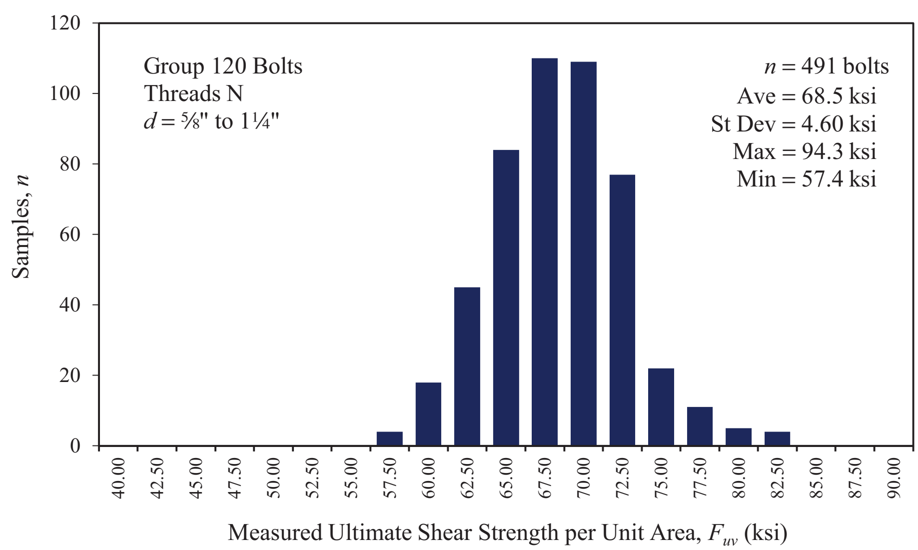


Fig. 2. Shear strength per unit area of Group 120 bolts tested with threads not excluded.

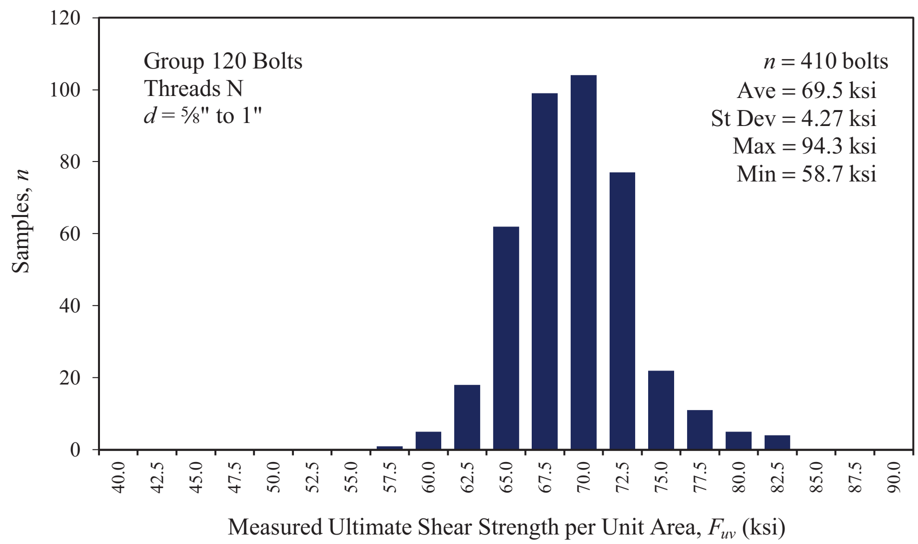


Fig. 3. Shear strength per unit area of smaller-diameter Group 120 bolts tested with threads not excluded.

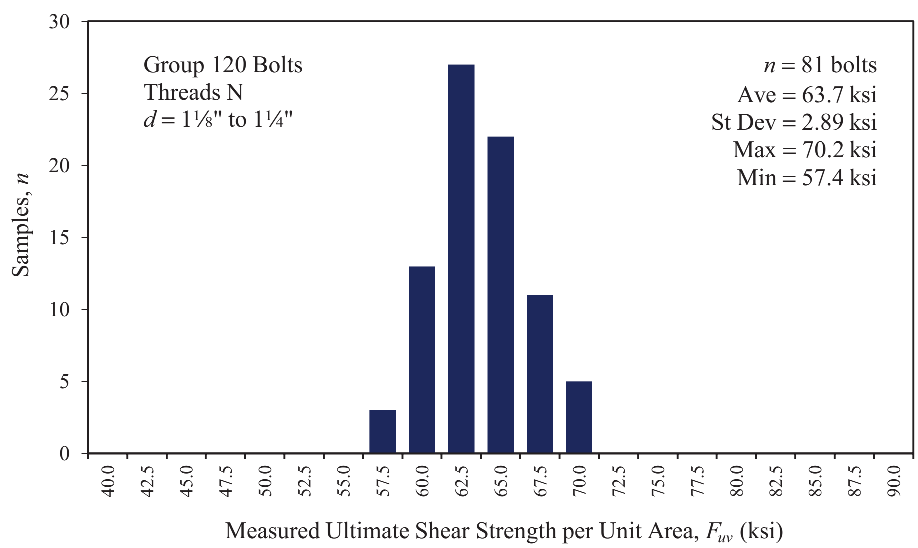


Fig. 4. Shear strength per unit area of larger-diameter Group 120 bolts tested with threads not excluded.

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

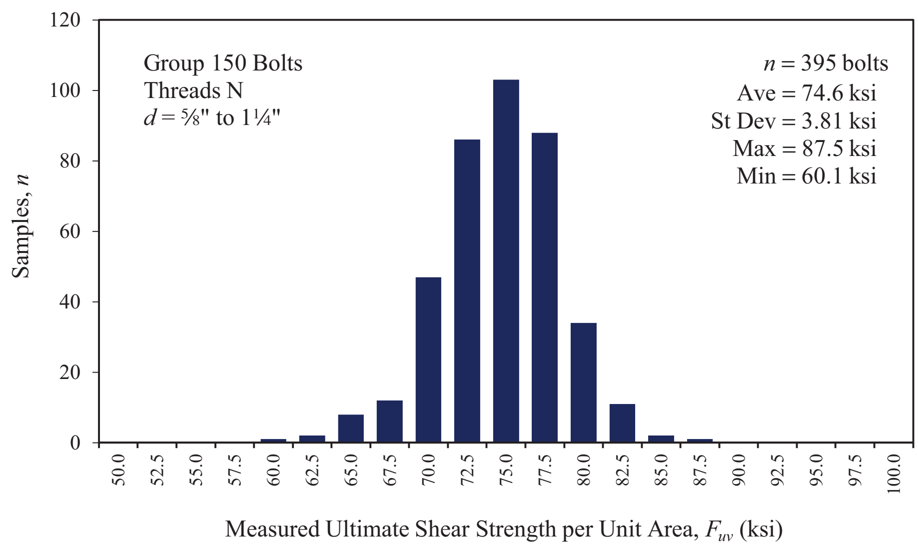


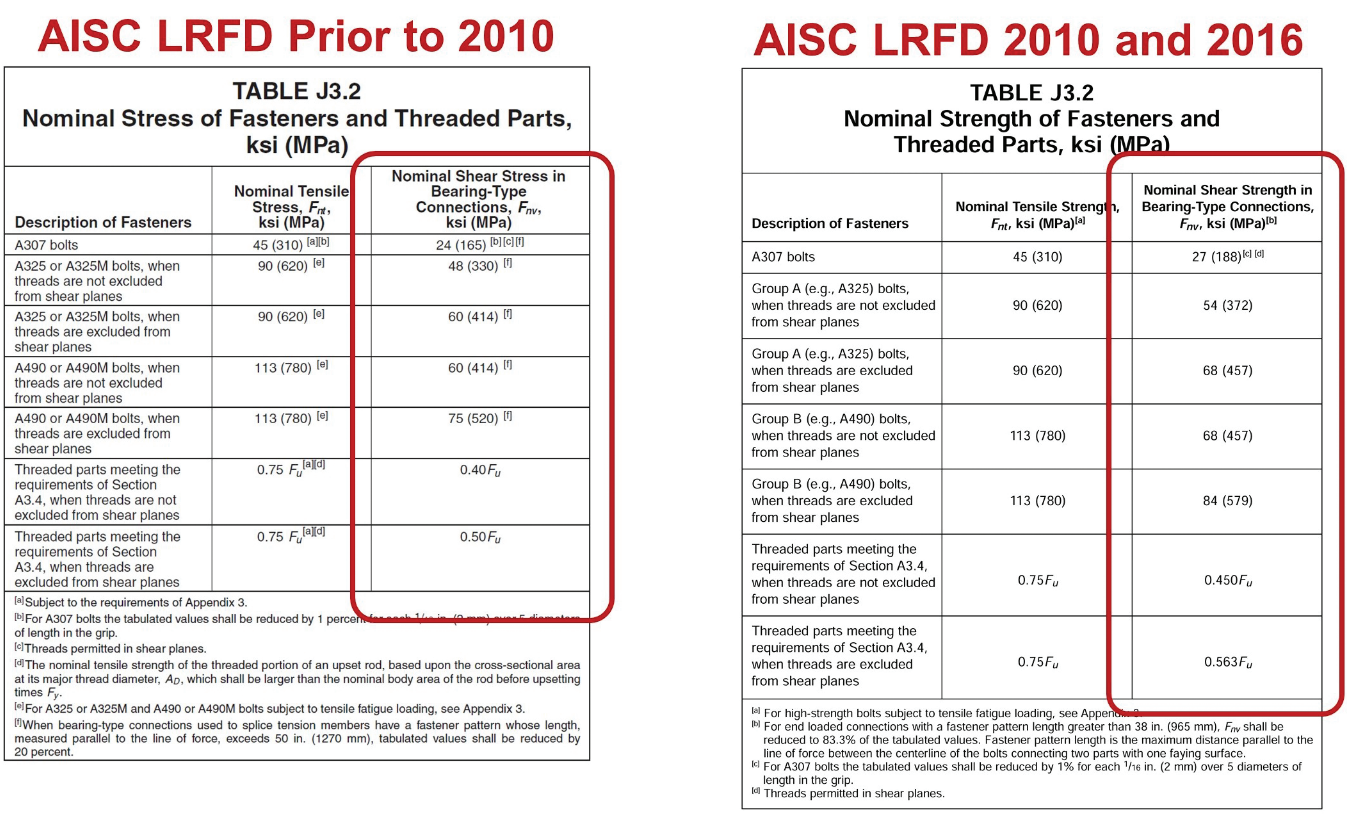
Fig. 5. Shear strength per unit area of Group 150 bolts tested with threads not excluded.

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 percent for each 1/16 in. (2 mm) over 5 diameters 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 Appendix 3.
[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.

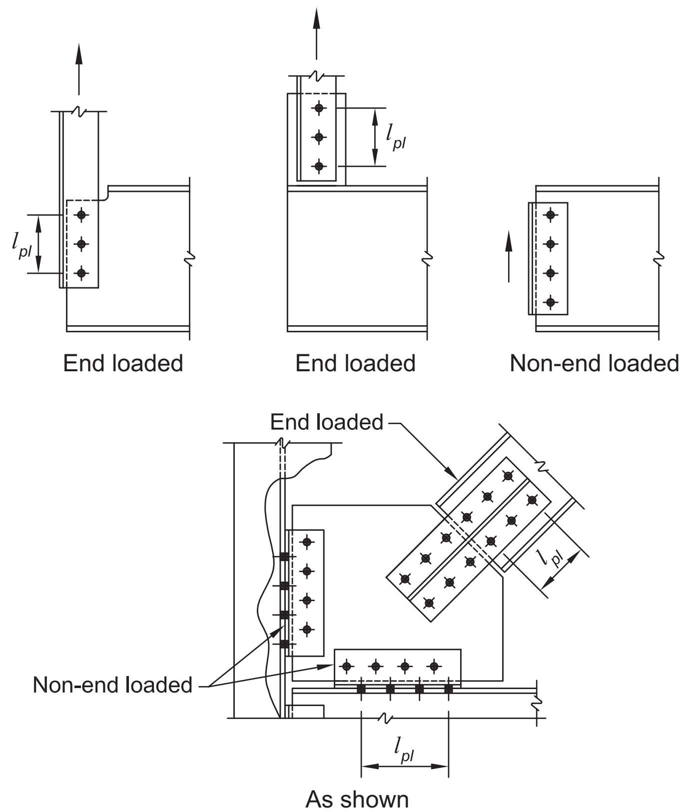


Fig. 7. Examples of joints that are end-loaded and joints that are non-end-loaded (AISC, 2016).

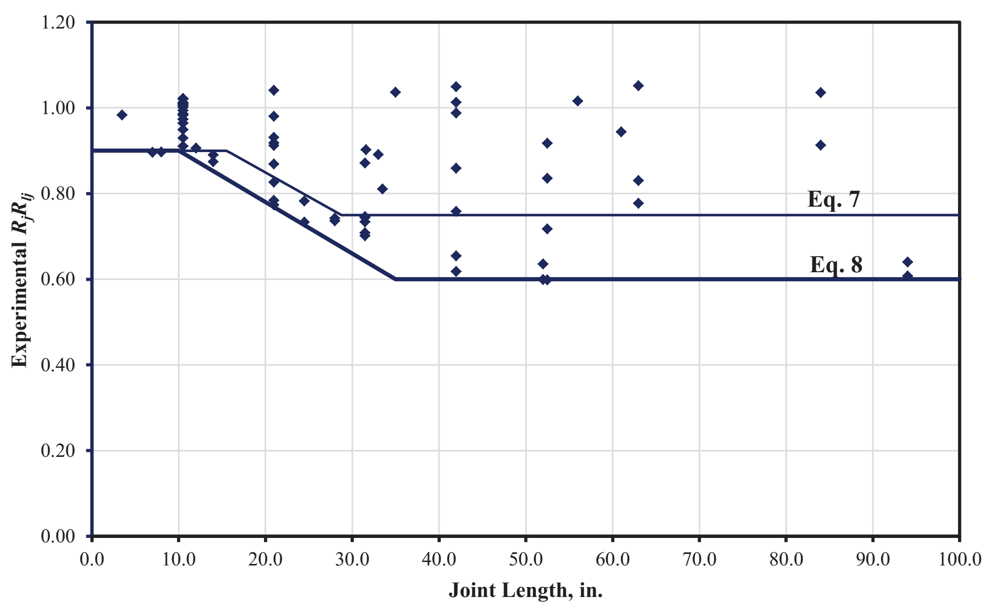


Fig. 8. Experimentally measured joint efficiency, R_jR_{lj} *(Tide, 2010).*

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

$$\begin{aligned} r_{nv} &= A_b F_{u,bolt} R_v R_{nx} R_j R_{lj} \qquad (2) \\ &= A_b F_{u,bolt}(0.625)(0.80)(1.00)(1.00) \\ &= 0.500 A_b F_{u,bolt} \end{aligned} \tag{11}$$

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.

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

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):

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

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

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

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

$$V_B = 0.061$$

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

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

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

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 $L \le 16$ in., the joint efficiency, its standard deviation, and its coefficient of variation are

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

$$\sigma_p = 0.045$$

$$V_p = 0.046$$

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

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

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

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

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

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

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

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

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

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

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

Thus

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

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

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%

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

$$= 0.284A_{b,nom}F_{u,nom} \tag{23}$$

Finally, the reliability index can be computed as

$$\textit{Reliability index} = \frac{R_m}{Q_m} \tag{24}$$

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

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

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

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

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

$$= 3.36$$

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%

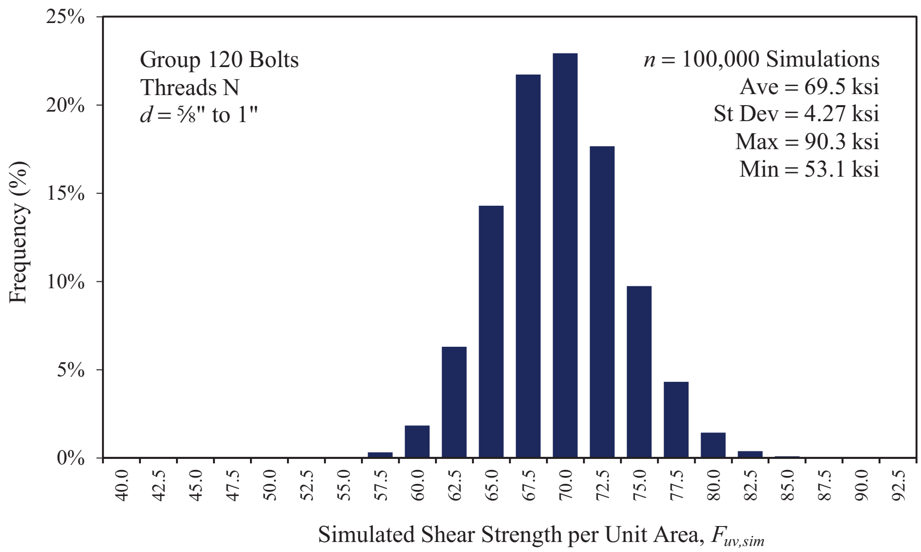


Fig. 9. Simulated shear strength per unit area of smaller Group 120 bolts with threads not excluded.

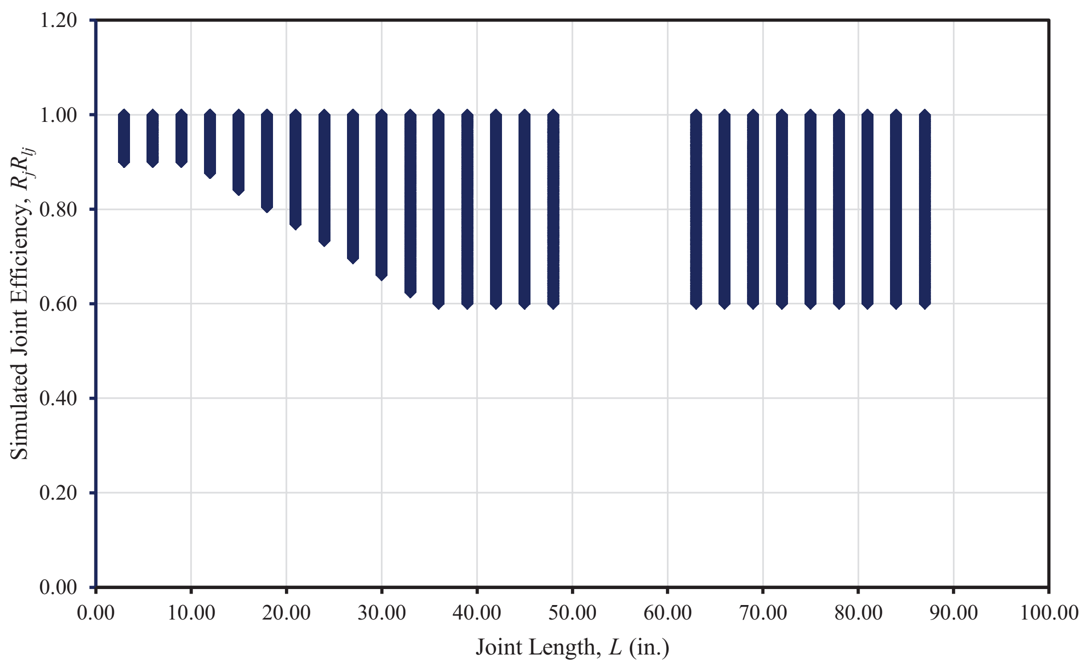


Fig. 10. Simulated joint efficiency used in the analysis of smaller Group 120 bolts per AISC Specifications *prior to 2010.*

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

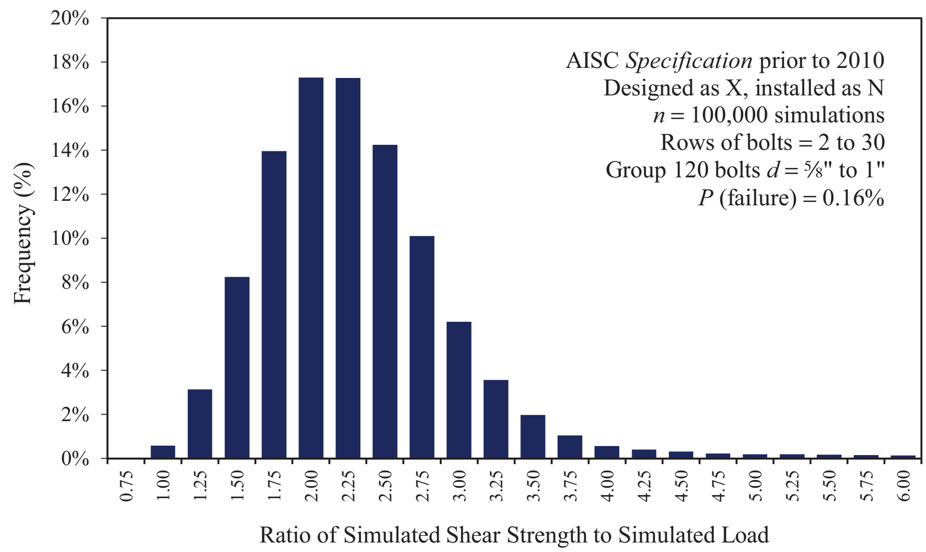


Fig. 11. Results of analysis of smaller Group 120 bolts per AISC Specifications *prior to 2010.*

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 $r_{nv} = 0.48 A_b F_{u,bolt} N_s$ and $r_{nv} = 0.38 A_b F_{u,bolt} N_s$ for threads excluded and not excluded, respectively, whereas in the 8th edition, the design strengths were increased to $r_{nv} = 0.56 A_b F_{u,bolt} N_s$ and $r_{nv} = 0.45 A_b F_{u,bolt} N_s$, 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).

REFERENCES

AASHTO (2017), *AASHTO LRFD Bridge Design Specifications,* 8th Ed., American Association of State Highway and Transportation Officials, Washington D.C.

AISC (1986), *Specification for Structural Steel Buildings—Load and Resistance Factor Design*, American Institute of Steel Construction, Chicago, Ill.

AISC (1993), *Load and Resistance Factor Design Specification for Structural Steel Buildings*, 2nd Ed., American Institute of Steel Construction, Chicago, Ill.

AISC (2005), *Specification for Structural Steel Buildings*, ANSI/AISC 360-05, American Institute of Steel Construction, Chicago, Ill.

AISC (2010), *Specification for Structural Steel Buildings*, ANSI/AISC 360-10, American Institute of Steel Construction, Chicago, Ill.

AISC (2016), *Specification for Structural Steel Buildings*, ANSI/AISC 360-16, American Institute of Steel Construction, Chicago, Ill.

AISC (2017), *Steel Construction Manual*, American Institute of Steel Construction, Chicago, Ill.

ASME (1981), *Square and Hex Bolts and Screws (In. Series)*, ASTM B18.2.1, American Society of Mechanical Engineers, New York, N.Y.

ASME (2019), *Fasteners for Use in Structural Applications*, ASTM B18.2.6, American Society of Mechanical Engineers, New York, N.Y., Adopted and In Press.

ASTM (2014a), *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*, ASTM A325, ASTM International, West Conshohocken, Pa. (withdrawn 2016).

ASTM (2014b), *Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength*, ASTM A490, ASTM International, West Conshohocken, Pa. (withdrawn 2016).

ASTM (2014c), *Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*, ASTM F1852, ASTM International, West Conshohocken, Pa. (withdrawn 2016).

ASTM (2014d), *Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 150 ksi Minimum Tensile Strength*, ASTM F2280, ASTM International, West Conshohocken, Pa. (withdrawn 2016).

ASTM (2018), *Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, In. and Metric Dimensions*, ASTM F3125, ASTM International, West Conshohocken, Pa.

Fisher, J.W., Galambos, T.V., Kulak, G.L., and Ravindra, M.K. (1978), "Load and Resistance Factor Design Criteria for Connectors," *Journal of the Structural Division*, ASCE, September.

Fisher, J.W. and Struik, J.H.A. (1974), *Guide to Design Criteria for Bolted and Riveted Joints*, 1st Ed., John Wiley and Sons, New York, N.Y.

Frank, K.H. and Yura, J.A. (1981), *An Experimental Study of Bolted Shear Connections*, FHWA/RD-81/148, Federal Highway Administration, Washington, D.C., December.

Kulak, G.L., Fisher, J.W., and Struik, J.H.A. (1987), *Guide Design Criteria for Bolted and Riveted Joints*, 2nd Ed., John Wiley and Sons, New York, N.Y.

Moore, A.M., Rassati, G.A., and Swanson, J.A. (2008), *Evaluation of the Current Resistance Factors for High-Strength Bolts*, Final Report Submitted to the Research Council on Structural Connections, January.

Moore, A.M., Rassati, G.A., and Swanson, J.A. (2010), "An Experimental Analysis of Strength and Ductility of High-Strength Fasteners," *Engineering Journal*, AISC, Vol. 47, No. 3.

Nowak, A.S. and Collins, K.R. (2013), *Reliability of Structures*, 2nd Ed., CRC Press, Boca Raton, Fla.

Roenker, A.T., Rassati, G.A., and Swanson, J.A. (2017), *Testing of Torque-and-Angle High Strength Fasteners*, A Technical Report Submitted to LeJeune Bolt Company, March.

RCSC (1964), *Specifications for Structural Joints Using ASTM A325 or A490 Bolts*, Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation.

RCSC (1976), *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation.

RCSC (1985), *Allowable Stress Design Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Research Council on Structural Connections of the Engineering Foundation.

RCSC (1988), *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Research Council on Structural Connections of the Engineering Foundation.

RCSC (2015), *2014 Specification for Structural Joints Using High-Strength Bolts with Errata*, Research Council for Structural Connections, Chicago, Ill.

Swanson, J.A., Rassati, G.A., and Larson, C.M. (2019), "Dimensional Tolerances and Length Determination of High-Strength Bolts," *Engineering Journal*, AISC, Vol. 57, No. 1.

Taylor, A.T., Rassati, G.A., and Swanson, J.A. (2008), *Evaluation of the Resistance Factor for Fully Threaded High Strength Fasteners*, Final Report Submitted to the Metal Building Manufacturers Association, September.

Tide, R.H.R. (2010), "Bolt Shear Design Considerations," *Engineering Journal*, AISC, Vol. 47, No. 1.

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:

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

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

Determine the design loads:

Set the minimum nominal dead load:

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

Set the maximum nominal dead load:

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

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

$$\begin{aligned} Q_n &= D_n + L_n \\ &= 136 \text{ kips} + 408 \text{ kips} \\ &= 544 \text{ kips} \end{aligned}$$

The total design load is

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

The total “actual” load is

$$
\begin{aligned}
Q_{sim} &= D_{sim} + L_{sim} \\
&= 134 \text{ kips} + 529 \text{ kips} \\
&= 663 \text{ kips}
\end{aligned}
$$

With a design load of $\Sigma\gamma Q$ = 816 kips and a design strength of ϕr_{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,

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

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

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

$$
\begin{aligned}
R_jR_{lj,min} &= 1.02\text{-}0.0120L \qquad (8) \\
&= 1.02 - (0.0120)(57.0 \text{ in.}) \\
&= 0.336
\end{aligned}
$$

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

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

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

$$
\begin{aligned}
\frac{R_{sim}}{Q_{sim}} &= \frac{758 \text{ kips}}{663 \text{ kips}} \\
&= 1.14
\end{aligned}
$$

APPENDIX C

Table C1. Ply Thickness Below which Bearing Strength Will Govern over Bolt Shear Strength, *N*								
	$F_{u,ply}$ = 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	
Bolt Dia.	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*						
	Group 120 Bolts			Group 150 Bolts		
Shape	*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*:

$$\phi_{bearing} 2.4 t_{ply} d_b F_{u,ply} + \phi_{shear} 0.450 A_b F_{u,bolt} \tag{C1}$$

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.

$$t_{ply} \leq \frac{d_b F_{u,bolt}}{6.791 F_{u,ply}} \tag{C2}$$

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.