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## ABSTRACT

Load and Resistance Factor Design Criteria are developed in this report for the proportioning of connections and connectors in steel building structures. In particular, LRFD criteria are developed for fillet welds and high-strength steel bolts.

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# 1. INTRODUCTION

This report presents Load and Resistance Factor Design (LRFD) criteria for connectors and connections in steel building structures. The presented material is part of a project in which design criteria are developed for structural steel elements using first-order probabilistic methods.* The basic development of the method is given in Ref. 1, and LRFD criteria for beams (1,2), plate girders (3) and beam-columns (4) are derived in subsequent reports. The principal feature of the LRFD method is the design formula

$$\phi R_n \geq \gamma_A [\gamma_D c_D D_m + \gamma_L c_L L_m + \dots ] \tag{1}$$

where the right side represents the factored load effects ($\gamma_A$ : load factor reflecting the uncertainties of analysis; $\gamma_D$, $\gamma_L$, etc.: dead, live, etc., load factors; $D_m$, $L_m$, etc.: mean dead, live, etc., load intensities; $c_D$, $c_L$, etc.: influence factors translating load intensities into load effects), and the left side is the factored nominal resistance ($R_n$: nominal resistance; $\phi$ : resistance factor accounting for the uncertainties of the resistance). The principal purpose of this report is to present derivations for $\phi$ and $R_n$ for connectors and connections.

Connection design is a vital part in the structural design process because improperly designed connections may fail prematurely or they may deform excessively under low loads, thereby rendering the structure unfit for use. Conversely, overdesigned connections may be grossly inefficient and expensive.

Current design specifications for steel construction usually handle connection proportioning by giving general requirements rather than detailed rules for each of the many conceivable types of connections. A good example of this is Sec. 2.8 of the 1969 AISC Specification, where a few short paragraphs concisely present the requirements. There are many texts, handbooks, papers, design aids and internal office procedures which are at the disposal of the designer, who then can provide connections in accordance with the general requirements of the specification.

There are two fundamental structural requirements which must be met for a well designed connection:

1) Connections should usually be stronger than the parts they connect, so that the forces are transmitted through the connection without undue distortion and without failing it. In general, then, the members themselves should fail rather than the connections.

2) Connections should be ductile, so that "brittle" failure, either through material fracture or instability, is not initiated by anything that happens in the connection.

Thus "strength" and "ductility" are the structural bases for connection design, and these are coupled with two other desirable features: "economy" and "simplicity".

In present practice, as illustrated in the AISC and AASHTO Specification, these criteria are achieved by specifying allowable stresses for connectors and geometric limits for connection plates, flanges and stiffeners. Some of the forces used in computing stresses in connection elements relate to the actual forces acting on the connection, while some are determined on the basis of full plastic capacity.

## 2. CALIBRATION OF CONNECTOR DESIGN REQUIREMENTS

The load factors $\gamma$ and the resistance factor $\phi$ in Eq. 1 depend on a "safety index" $\beta$ (Ref. 1) which is obtained by "calibration" to existing standard designs. In the case of beams and columns (Ref. 1) it was found that $\beta$ = 3 was a good estimate of the reliability inherent in current design, and thus this value was taken also as the basis for LRFD criteria for all other types of structural members. In view of the requirement that connections should be more reliable than the members they connect, it is necessary that the safety index $\beta$ should be larger for connections than $\beta$ = 3.0 which was used for members. In order to decide on the proper $\beta$ to be used it is desirable to go through a calibration process for connectors.

The calibration procedure follows the same procedure as that presented in Ref. 1 for beams and columns. The purpose of such a calibration is to determine the value of the safety index $\beta$ inherent in current design as characterized by Part 2 of the AISC Specification. As in Ref. 1, calibration will be performed for the combination of dead and live loads for connectors in connections located in beam-type members. The calibrations will be performed for fillet welds and high-strength steel bolts.

The safety index $\beta$ is defined as (Ref. 1)

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

where $R_m$ is the mean and $V_R$ the coefficient of variation of the resistance, while $Q_m$ and $V_Q$ are the corresponding quantities for the load effects. These latter quantities are defined in Ref. 1 as

$$Q_m = c\ (D_m + L_m) \tag{3}$$

$$V_Q^2 = V_E^2 + \frac{D_m^2 (V_A^2 + V_D^2) + L_m^2 (V_B^2 + V_L^2)}{(D_m + L_m)^2}$$

where c is an influence coefficient translating load intensity to load effect; $D_m$ is the mean dead load intensity which is assumed to be equal to the code value ($D_m = D_c$); $L_m$ is the mean live load intensity for office buildings

$$L_m = 14.9 + \sqrt{\frac{762}{2\ A_T}} \tag{5}$$

in psf, $A_T$ being the tributary area in sq. ft.; $V_E$ is the C.O.V. of the uncertainties of the force computation; and $V_A$ and $V_D$, and $V_B$ and $V_L$, are the C.O.V.'s associated with the uncertainties of the dead load and the live load, respectively. In Ref. 1 the following values are given for these: $V_E = 0.05$ ; $V_A = V_D = 0.04$, $V_B = 0.2$ and $V_L = 0.13$.

Fillet Welds

The mean resistance of fillet welds is obtained experimentally. Fisher gives the following data (5) from the research on which the 1969 AISC allowable stresses for welds were derived:

$$\left(\frac{\tau_u}{\sigma_u}\right)_{mean} = 0.73 \qquad V = 0.08$$

Kulak (6) gives values derived from a set of several other experiments as

$$\left(\frac{\tau_u}{\sigma_u}\right)_{mean} = 0.77 \qquad V = 0.13$$

Both sets of values will be used in determining $\beta$. In this data $\sigma_u$ is the tensile strength of the weld metal and $\tau_u$ is the shear strength. This tensile strength is related to the nominal weld tensile strength ($F_{Exx}$) by the statistics (from Fisher, Ref. 5):

$$\left(\frac{\sigma_u}{F_{Exx}}\right)_{mean} = 1.20\ ; \qquad V = 0.04$$

The mean shear strength of fillet welds is thus equal to

$$(\tau_u)_m = \left(\frac{\tau_u}{\sigma_u}\right)_m \left(\frac{\sigma_u}{F_{Exx}}\right)_m F_{Exx} \tag{6}$$

With

$$V_w = \sqrt{V^2_{\tau_u/\sigma_u} + V^2_{\sigma_u/F_{Exx}}} \qquad (7)$$

the statistical properties from the two date sets are as follows:

Ref. 5: $(\tau_u)_m = (0.73)(1.20\ F_{Exx}) = 0.88\ F_{Exx}$ ; $V_w = \sqrt{0.08^2 + 0.04^2} = 0.09$

Ref. 6: $(\tau_u)_m = (0.73)(1.20\ F_{Exx}) = 0.92\ F_{Exx}$ ; $V_w = \sqrt{0.13^2 + 0.04^2} = 0.14$

The coefficient of variation of the resistance, $V_R$ in Eq. 2, also contains contributions which were termed "professional", $V_p$, and "fabrication", $V_F$, in Ref. 1. The variation in the "professional assumptions" reflect the accuracy with which the forces acting on the fasteners are estimated. The exact determination of these forces is highly complex, especially for complicated joints, and they are usually assigned according to a distribution which fulfills the equilibrium (statics) requirements only. However, for a ductile structure, which well constructed welded joints are made to be, the principles of the lower bound theorem of plasticity are valid. Thus as long as no error is made in statics and weld material is provided to resist the assigned forces, the joint will be safe. There is thus no variability of the professional assumptions: the assigned statically correct forces will be resisted. The variation in fabrication reflects the variation of the weld length and the weld area. There does not appear to be a quantitative way of obtaining $V_F$, due to lack of suitable data, and thus a value of $V_F = 0.15$ will be assumed for fillet welds. This is a high scatter ($V_F = 0.15$ implies that there is a 50% probability that the dimension will be within $\pm$ 10% of the design values), and it is probably conservative.

The next item required for the calibration is the weld size required by the 1969 AISC Specification. According to Part 2 of the specification the design criterion for a load combination of dead and live loads is

$$1.7\ A_w \times 0.3\ F_{Exx} = 1.7\ c(D_c + L_{rc}) \tag{8}$$

where $A_w$ is the throat area of the weld, and $D_c$ and $L_{rc}$ are the code-specified dead and live load intensities. As in Ref. 1, the code live load intensity $L_{rc}$ is reduced according to the tributary area (ANSI 58-1, 1972). The mean resistance of a fillet weld designed according to the 1969 AISC Specification is thus

$$R_m = A_w\ (\tau_u)_m = \frac{c(D_c + L_{rc})(\tau_u)_m}{0.3\ F_{Exx}} \tag{9}$$

and the value of the C.O.V. is equal to

$$V_R^2 = V_w^2 + V_F^2 = V_w^2 + 0.15^2 \tag{10}$$

Substitution of $R_m$ (Eq. 9), $V_R$ (Eq. 10), $Q_m$ (Eq. 3) and $V_Q$ (Eq. 4) into Eq. 2 gives $\beta$. Values of $\beta$ for fillet welds are listed in Table 1 for a basic code live load of $L_c$ = 50 psf and for dead load intensities of 50, 75 and 100 psf for the range of tributary areas from 200 to 1000 $ft.^2$ Values of $\beta$ are given for the weld data from Ref. 5 and Ref. 6. A plot of $\beta$ versus tributary area is shown in Fig. 1. From Table 1 and Fig. 1 it is apparent that $\beta$ for the whole domain of variables does not change much, the range of $\beta$ being from a low of $\beta$ = 4.1 to a high of 5.0.

High-Strength Steel Bolts in Direct Tension

The mean resistance of high-strength steel bolts in direct tension is

$$R_m = \left(\frac{\sigma_u}{F_u}\right)_m A_{SA}\ F_u \tag{11}$$

In this equation $\sigma_u$ is the ultimate tensile strength of the bolts, $F_u$ is the specified minimum tensile strength for the type bolt and $A_{SA}$ is the stress area of the threaded part of the bolts. The following experimental data is given in Ref. 5:

$$\left(\frac{\sigma_u}{F_u}\right)_m = 1.20 \; ; \quad V_{\sigma_u/F_u} = 0.07 \quad \text{for A325 bolts}$$

$$\left(\frac{\sigma_u}{F_u}\right)_m = 1.07 \; ; \quad V_{\sigma_u/F_u} = 0.02 \quad \text{for A490 bolts}$$

If it is assumed that $V_P = 0$ (as for fillet welds) and $V_F = 0.05$ (reflecting good control characteristic of bolt manufacturing quality control),

$$\left.\begin{aligned} R_m &= 1.20\ A_{SA}\ F_u \\ V_R &= \sqrt{0.07^2 + 0.05^2} = 0.09 \end{aligned}\right\} \text{for A325 bolts}$$

$$\left.\begin{aligned} R_m &= 1.07\ A_{SA}\ F_u \\ V_R &= \sqrt{0.02^2 + 0.05^2} = 0.05 \end{aligned}\right\} \text{for A490 bolts}$$

The bolt tensile strength requirements in the 1969 AISC Specification are

$$1.7\left[A_G \times \frac{40}{120} \times F_u\right] = 1.7\left[c_D\ D_c + c_L\ L_c\right] \tag{12}$$

for A325 bolts and

$$1.7\left[A_G \times \frac{54}{150} \times F_u\right] = 1.7\left[c_D\ D_c + c_L\ L_c\right] \tag{13}$$

for A490 bolts. In these equations $A_G$ is the shank (gross) area, and 40 and 54 ksi, respectively, are the allowable tensile stresses $F_t$ (AISC, Sec. 1.5.2), while 120 and 150 ksi are the ASTM specified minimum tensile strengths for the two grades of bolts. The stress area is related to the gross area and its ratio varies upwards from 0.723 for a 1/2 inch bolt. For example, for a 7/8 inch bolt the ratio $A_{SA}/A_G = 0.768$. Using a 1/2 and 7/8 inch bolt for the purpose of calibration, values of $\beta$ have been determined from Eq. 2 for

$$R_m = \left( \frac{\sigma_u}{F_u} \right)_m F_u \left( \frac{A_{SA}}{A_G} \right) \left\{ \frac{c\ (D_c + L_{rc})}{F_t} \right\} \tag{14}$$

and these are tabulated in Table 1 and plotted in Fig. 1. The $\beta$'s are generally higher than the $\beta$'s for fillet welds, the range of variation is greater, and they increase with an increase in the bolt diameter; $\beta$ for A490 bolts is also lower than for A325 bolts.

<u>High-Strength Steel Bolts in Shear</u>

For high-strength steel bolts in joints where failure is due to bolt shear after slip when the bolts act in bearing ("bearing" - type joints), the mean resistance is equal to

$$R_m = \left( \frac{\tau_u}{\sigma_u} \right)_m \left( \frac{\sigma_u}{F_u} \right)_m A_{SA}\ F_u \tag{15}$$

where $\tau_u$ is the shear strength, $\sigma_u$ is the tensile strength, $F_u$ is the nominal (specified minimum) tensile strength and $A_{SA}$ is the stress area equal to the shank area if the shear plane passes through the shank, and it is the area of the threaded portion of the bolt if the shear plane passes through the threads. The following statistics are given in Ref. 5 from experimental data:

$$(\tau_u/\sigma_u)_m = 0.625 \quad ; \quad V = 0.05$$

The means and the C.O.V.'s of the resistance are thus:

$$\left.\begin{aligned} R_m &= 0.625 \times 1.2 \times A_{SA} F_u = 0.75 A_{SA} F_u \\ V_R &= \sqrt{0.05^2 + 0.07^2 + 0.05^2} = 0.10 \end{aligned}\right\} \text{A325 bolts}$$

$$\left.\begin{aligned} R_m &= 0.625 \times 1.07 \times A_{SA} F_u = 0.67 A_{SA} F_u \\ V_R &= \sqrt{0.05^2 + 0.02^2 + 0.05^2} = 0.07 \end{aligned}\right\} \text{A490 bolts}$$

The stress area is determined for the requirements of the 1969 AISC Specification, and the resulting values of $\beta$ are tabulated in Table 1 and plotted in Fig. 1. The $\beta$'s for the case where the shear plane passes through the threads was determined for a 1/2" bolt diameter. The $\beta$'s are considerably higher than the safety indices for fillet welds, indicating that the reliability level of high-strength bolts in shear is considerably above that of fillet welds in the current AISC Specification (1969).

<u>Friction Joints</u>

The mean slip force $P_S$ in a friction-type joint with uniform diameter high-strength steel bolts is (7)

$$P_s = (m\ n\ K_s\ T_i)_m \tag{16}$$

where m is the number of slip planes per bolt, n is the number of bolts, $K_s$ is the slip coefficient and $T_i$ is the clamping force.

The statistics of the slip coefficient have been determined from experiments (7), and for clean mill scale surfaces $(K_s)_m = 0.33$. The C.O.V. is V = 0.21.

The clamping force $T_i$ depends on the type of bolt tightening: turn-of-the-nut method or the calibrated wrench method. The mean clamping force and its C.O.V. for bolts tightened by the turn-of-the-nut clamping force are equal to:

$$(T_i)_m = A_{SA}(\sigma_i)_m = 1.20 \quad (0.7\ F_u)\left(\frac{1.20}{1.03}\right) A_{SA} = 0.98\ A_{SA}\ F_u$$

$$V = \sqrt{0.08^2 + 0.07^2 + 0.05^2} = 0.12$$

for A325 bolts and

$$(T_i)_m = 1.26\ (0.7\ F_u)\left(\frac{1.07}{1.10}\right) A_{SA} = 0.86\ A_{SA}\ F_u$$

$$V = \sqrt{0.08^2 + 0.02^2 + 0.05^2} = 0.10$$

for A490 bolts.

The numbers in these calculations have the following meaning: 1.20 and 1.26 are the mean ratios of measured-to-required clamping forces in friction-grip joints for A325 and A490 bolts, respectively, and V = 0.08 is the corresponding coefficient of variation (Fig. 5.7 in Ref. 7); $0.7\ F_u$ is the minimum specified clamping stress; and 1.20/1.03 and 1.07/1.10 are the ratios of the mean tensile strength of all bolts to the tensile strength of the bolts used for the samples in Ref. 7. The corresponding C.O.V.'s are 0.07 and 0.02, respectively, for A325 and A490 bolts. The C.O.V. of 0.05 is the assumed C.O.V. of fabrication uncertainties.

The mean clamping stress for bolts tightened by the calibrated wrench method is given in Ref. 7 as 0.796 times the minimum specified tensile strength, and its C.O.V. is 0.05. Thus

$$\left.\begin{aligned} (T_i)_m &= 0.796 \times F_u \times A_{SA} = 0.80\ A_{SA}\ F_u \\ V &= \sqrt{0.05^2 + 0.05^2} = 0.07 \end{aligned}\right\}\ \text{A325 bolts}$$

$$\left.\begin{array}{l} (T_i)_m = 0.796 \times F_u \times A_{SA} = 0.80\, A_{SA}\, F_u \\ V = \sqrt{0.05^2 + 0.05^2} = 0.07 \end{array}\right\} \text{A490 bolts.}$$

The stress area $A_{SA}$ required by the 1969 AISC Specification is

$$A_{SA} = \frac{c\ (D_c + L_{rc})}{mn\ F_v} \left( \frac{A_{SA}}{A_G} \right) \qquad (17)$$

The corresponding values of $\beta$, computed from Eq. 2, are listed in Table 1 for A325 and A490 bolts of 1/2 inch diameter tightened by the turn-of-the-nut method. Somewhat lower $\beta$'s apply for joints with bolts tightened by the calibrated wrench method. As expected, the $\beta$'s are low ($1.2 \leq \beta \leq 2.2$ for the domain of the parameters considered) compared to $\beta$ for fillet welds and bolts in bearing type joints because of the less severe consequence of slip-versus-joint failure by bolt rupture or shear.

The calibration exercise presented above has indicated that a wide range of reliability exists in the different fastener provisions of the 1969 AISC Specification, since $\beta$ varies from a low of about 1.2 to a high of about 10.6. Two separate values of $\beta$ will be selected for use in the Load and Resistance Factor Design Criteria: one value of $\beta$ for the strength limit states (fillet welds, bolts in bearing type joints) and one for the serviceability limit state of slip.

The safety index $\beta = 4.5$ will be selected for the strength limit state, reflecting essentially the values of $\beta$ obtained for fillet welds (Fig. 1). With this value of $\beta = 4.5$ fillet welds and high-strength steel bolts in tension under high live-to-dead load ratios (which gave the low values of $\beta$ in the calibration) should give essentially the same size welds and bolts as the 1969 AISC Specification. Since the allowable stresses in shear are overly conservative in this specification (giving the high values of $\beta$ in the calibration), $\beta = 4.5$ will provide bolts of smaller size in the LRFD criteria.

The safety index of $\beta = 2.0$ will be selected for bolts in friction joints, reflecting the fact that this is a serviceability criterion. For these bolts it will be necessary to also check the ultimate limit state with $\beta = 4.5$.

The selected $\beta$'s, i.e., 4.5 for ultimate limit states of fasteners and 2 for serviceability limit states, compare with $\beta = 3$ for the safety index of the members connected (Ref. 1). Thus the fasteners are indeed designed for a higher reliability than the members they connect, while the reliability under a serviceability limit state is lower.

## 3. DETERMINATION OF THE RESISTANCE FACTOR

The resistance factor $\phi$ (Eq. 1) is expressed as follows (Ref. 1):

$$\phi = \frac{R_m}{R_n} \exp(-\alpha \beta V_R) \tag{18}$$

In this equation $R_m$ is the mean resistance and $R_n$ is the nominal resistance (i.e., the expression for the resistance in the design criteria); $\alpha$ is a numerical factor equal to 0.55.

Following are the various $\phi$-factors for fasteners.

Fillet Welds

$$\beta = 4.5$$

$$R_n = A_w (0.6 F_{Exx}) \tag{19}$$

where the more convenient 0.6 rather than the usual $1/\sqrt{3} = 0.577$ factor was selected to represent the relationship between tensile strength and shear strength. $A_w$ is the throat area of the fillet weld.

$$R_m = A_w \, (\tau_u)_m \tag{20}$$

For the experimental data from Ref. 5:

$$(\tau_u)_m = 0.88 \, F_{Exx} \, , \quad V_R = 0.17 \quad \text{and} \quad \phi = 0.96$$

For the experimental data from Ref. 6:

$$(\tau_u)_m = 0.92 \, F_{Exx} \, , \quad V_R = 0.21 \quad \text{and} \quad \phi = 0.91$$

High-Strength Steel Bolts in Direct Tension

$$\beta = 4.5$$

$$R_n = A_{SA} \, F_u \tag{21}$$

where $A_{SA}$ is the stress area of the threaded part of the bolt and $F_u$ is the specified minimum tensile strength.

For A325 bolts

$$R_m = 1.20 \, A_{SA} \, F_u \, , \quad V_R = 0.09 \, , \quad \phi = 0.96$$

For A490 bolts

$$R_m = 1.07 \, A_{SA} \, F_u \, , \quad V_R = 0.05 \, , \quad \phi = 0.94$$

High-Strength Steel Bolts in Shear

$$\beta = 4.5$$

$$R_n = A_{SA} \, (0.6 \, F_u) \tag{22}$$

where $A_{SA}$ is the stress area equal to the shank area if the shear plane passes through the shank and it is the area of the threaded portion of the bolt if the shear plane passes through the threads.

For A325 bolts

$$R_m = 0.75 \, A_{SA} \, F_u \, , \quad V_R = 0.10 \, , \quad \phi = 0.98$$

For A490 bolts

$$R_m = 0.67\ A_{SA}\ F_u\ , \quad V_R = 0.07\ , \quad \phi = 0.93$$

The shear capacity is reduced for long joints, and it is recommended that $R_n$ from Eq. 22 be reduced to 80% of this value when the joint length exceeds 50 in (Ref. 7); the same $\phi$ applies.

High-Strength Steel Bolts in Combined Tension and Shear

The following interaction equation has been recommended by Fisher (5,7) for the case when the fastener is subjected to both tension and shear:

$$S^2 + (0.6\ T^2) = \phi\ (0.6\ A_{SA}\ F_u)^2 \tag{23}$$

In this equation S and T are the factored design shear force and tensile force, respectively, and $A_{SA}$ is the stress area with its magnitude dependent on the position of the shear plane.

The resistance factor $\phi$ can be determined as follows:

$$\frac{R_m}{R_n} = \left(\frac{R_{exp}}{R_n}\right)_m \times \left(\frac{\sigma_u}{F_u}\right)_m \tag{24}$$

and

$$V_R = \sqrt{V^2_{R_{exp}/R_n} + V^2_{\sigma_F/F_u} + V_P^2 + V_F^2} \tag{25}$$

In Fig. 5 of Fisher's report (Ref. 5) there are a number of bolt and rivet test results from experiments where the ratio of tension-to-shear was varied. The ratio $R_{exp}/R_n$ is the ratio of the experimental strength to the nominal strength according to the interaction equation

$$S^2 + (0.6\ T)^2 = (0.6\ A_{SA}\ F_u)^2 \tag{26}$$

when the resistance is measured along a radius from the origin to the test point. The statistics of this ratio are

$$\left(\frac{R_{exp}}{R_n}\right)_m = 1.05 \; ; \qquad V_{R_{exp/R_n}} = 0.10$$

Noting that $V_P = 0$ (as explained previously) $V_F = 0.05$, and $(\sigma_f/F_u)_m = 1.20$ and $V_{\sigma_u/F_u} = 0.07$ for A325 bolts, while $(\sigma_f/F_u)_m = 1.07$ and $V_{\sigma_u/F_u} = 0.02$ for A490 bolts (this data was presented in the previous section) $\phi$ can be determined as being equal to the following values:

<u>A325 bolts</u>: $V_R = \sqrt{0.07^2 + 0.10^2 + 0.05^2} = 0.13$

$\phi = 1.05 \times 1.20 \exp(-0.55 \times 4.5 \times 0.13) = 0.91$

<u>A490 bolts</u>: $V_R = \sqrt{0.02^2 + 0.10^2 + 0.05^2} = 0.11$

$\phi = 1.05 \times 1.07 \exp(-0.55 \times 4.5 \times 0.11) = 0.86$

<u>High-Strength Steel Bolts in Friction-Grip Joints</u>

$$\beta = 2.0$$

$$R_n = mn\, K_s\, (A_{SA} \times 0.7\, F_u) \tag{27}$$

where m is the number of slip planes per bolt, n is the number of bolts, $A_{SA}$ is the thread area and $K_s$ is the friction coefficient. For clean mill scale contact surfaces $K_s = 0.33$*.

For the turn-of-the-nut method of tightening:

$R_m = mn\, K_s \times 0.96\, A_{SA}\, F_u$ , $V_R = 0.24$ , $\phi = 1.08$ : A325 bolts

$R_m = mn\, K_s \times 0.86\, A_{SA}\, F_u$ , $V_R = 0.23$ , $\phi = 0.95$ : A490 bolts

*Friction, or slip-coefficients, for other types of surfaces are given in Table 5.1 of Ref. 7.

For the calibrated wrench method of tightening:

$$R_m = mn\ K_s \times 0.80\ A_{SA}\ F_u\ ,\ V_R = 0.22\ ,\ \phi = 0.90\ :\quad \text{A325 bolts}$$

$$R_m = mn\ K_s \times 0.80\ A_{SA}\ F_u\ ,\ V_R = 0.22\ ,\ \phi = 0.90\ :\quad \text{A490 bolts}$$

The use of two different values of $\beta$ for the members ($\beta = 3$) and the fasteners ($\beta = 4.5$ or 2) introduces some operational difficulties, as detailed below.

From Eq. 1, the LRFD design criterion is expressed as follows:

$$\phi\, R_n \geq \gamma_A \left[ c_D\, \gamma_D\, D_m + c_L\, \gamma_L\, L_m \right] \tag{28}$$

where $\phi$ is the resistance factor and $\gamma_E$, $\gamma_D$ and $\gamma_L$ are load factors (1)

$$\gamma_A = \exp \alpha\, \beta\, V_E \tag{29}$$

$$\gamma_D = 1 + \alpha\, \beta \sqrt{V_A^2 + V_D^2} \tag{30}$$

$$\gamma_L = 1 + \alpha\, \beta \sqrt{V_B^2 + V_L^2} \tag{31}$$

Then, with $V_A = 0.04$, $V_D = 0.04$, $V_B = 0.2$, $V_L = 0.13$ and $V_E = 0.05$ (Ref. 1) the following values of $\gamma_A$, $\gamma_D$ and $\gamma_L$ are obtained for $\beta = 3$, $\beta = 4.5$ and $\beta = 2.0$

| $\beta$ | $\gamma_A$ | $\gamma_D$ | $\gamma_L$ |
|---|---|---|---|
| 3 | 1.09 | 1.09 | 1.39 |
| 2 | 1.06 | 1.06 | 1.26 |
| 4.5 | 1.13 | 1.14 | 1.59 |

For beams, columns and other main members $\beta = 3$, and the use of $\gamma_A = 1.1$, $\gamma_D = 1.1$ and $\gamma_L = 1.4$ has been recommended for the rounded off load factors to be used in LRFD criteria (Ref. 1). While $\gamma_A = \gamma_D = 1.1$ would still be valid for fasteners, $\gamma_L$ = 1.6 or 1.3 should be used for the live load factors, thus requiring two different load factors - one for the determination of the forces for the design of the main members, and one for the connections. While such a course of action would be rational, it would lead to unnecessary confusion and to greater chances of error in the design calculations. This confusion can be circumvented by throwing all the penalty for the increased $\beta$ on the $\phi$-factor, as follows:

$$\phi\, R_n \quad \frac{1.1\,[1.1\, c_D\, D_m + 1.4\, c_L\, L_m]}{1.1\,[1.1\, c_D\, D_m + 1.5\, c_L\, L_m]} \geq 1.1\,[1.1\, c_D\, D_m + 1.4\, c_L\, L_m]$$

The factor in the brackets varies from 0.92 to 0.88 as the ratio $c_L\, L_m / c_D\, D_m$ varies from 0.25 to 2. This variation is not large, and thus it is recommended that a reduction factor of 0.88 be applied to $\phi$ for connections*. Thus for the design of connectors and of other connection elements it is recommended that

$$\bar{\phi}\, R_n \geq \gamma_A\,[\gamma_D\, c_D\, D_m + \gamma_L\, c_L\, L_m + \gamma_W\, c_W\, W_m + \ldots] \qquad (32)$$

where the load factors $\gamma_A$, $\gamma_D$, $\gamma_L$, $\gamma_W$, etc., are determined, as before (see Ref. 1 for $\beta$ = 3, and $\phi$ is determined for $\beta$ = 4.5, or $\beta$ = 2.0, whichever is appropriate, and

$$\bar{\phi} = 0.88\,\phi \qquad \text{when } \beta = 4.5$$

$$\bar{\phi} = 1.10\,\phi \qquad \text{when } \beta = 2.0$$

---

* The corresponding variation of the correction factor is from 1.10 to 1.13 for $\beta$ = 2.0

The modified resistance factors are as follows:

| | | | |
|---|---|---|---|
| Fillet Welds: | $\phi = 0.91^*$ ; | $\bar{\phi} = 0.91 \times 0.88 = 0.80$ | |
| Bolts in Tension: | $\phi = 0.96$ , | $\bar{\phi} = 0.84$ (A325) | |
| | $\phi = 0.94$ , | $\bar{\phi} = 0.83$ (A490) | |
| Bolts in Shear: | $\phi = 0.98$ , | $\bar{\phi} = 0.86$ (A325) | |
| | $\phi = 0.93$ , | $\bar{\phi} = 0.82$ (A490) | |
| Combined Tension and Shear: | $\phi = 0.91$ , | $\bar{\phi} = 0.80$ (A325) | |
| | $\phi = 0.86$ , | $\bar{\phi} = 0.76$ (A490) | |
| Friction Joints: | $\phi = 1.06$ , | $\bar{\phi} = 1.08 \times 1.1 = 1.17$ (A325) | turn of nut |
| | $\phi = 0.95$ , | $\bar{\phi} = 1.05$ (A490) | |
| | $\phi = 0.90$ , | $\bar{\phi} = 0.90 \times 1.1 = 1.00$ (A325) | Calibrated wrench |
| | $\phi = 0.90$ , | $\bar{\phi} = 1.00$ (A490) | |

For the sake of simplicity it appears desirable not to have a multiplicity of resistance factors. It is thus suggested that a modified resistance factor of $\bar{\phi} = 0.80$ be used for the strength limit state of fillet welds and A325 high strength bolts, $\bar{\phi} = 0.75$ be used for the strength limit state of A490 high strength bolts, and $\bar{\phi} = 1.0$ be used for friction grip bolts.

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*The lower of the two values, representing the data from Ref. 6, was used.

## 4. DISCUSSION OF ADDITIONAL TOPICS ON CONNECTIONS

Experimental statistical data were presented in Sec. 2 on fillet welds and high-strength bolts. These data were utilized in a calibration process, using a first order probabilistic theory, in Sec. 3 to develop resistance factors ($\phi$). The field of connections includes many additional topics, and a number of these will be presented in this section.

### Welds

The previous discussion on welds considered the shear stress on the effective throat area of the weld metal in fillet welds, and $\bar{\phi} = 0.8$ was calculated as the resistance factor to be used with the weld shear stress capacity $F_w = 0.6\ F_{Exx}$. These results can be generalized to include the shear stress capacity of the weld metal for other similar conditions also, e.g., complete and partial penetration groove welds, and plug and slot welds.

The direct tensile or compressive stress capacity of groove welds is the specified yield stress $F_y$ of the base metal, and tensile specimens with groove welds may be expected to fail in the parent plate rather than in the filler metal (Sec. 6.11.1, Ref. 8). The resistance factor $\phi$ is then determined for the main member with $\beta = 3.0$. The value of $\phi$ is calculated as follows from the statistical data given in Ref. 1:

$$(F_y)_m = 1.05\ F_y\ ; \quad V_{F_y} = 0.10$$

$$\beta = 3.0,\ \alpha = 0.55,\ \ V_F = 0.05,\ \ V_R = \sqrt{0.10^2 + 0.05^2} = 0.11$$

$$\phi = 1.05 \exp(-0.55 \times 3 \times 0.11) = 0.88$$

According to Sec. 1.5.3 of the AISC Specification it is also required to check the shear stress on the base metal. The shear stress capacity is $F_y/\sqrt{3}$, and $\phi = 0.86$ (Ref. 2).

The requirements concerning "matching" weld metal in Tables 1.5.3 and 1.17.2 of the AISC Specification, as well as all other provisions regarding weld details given in Sec. 1.14.7, 1.15.6, 1.15.9, 1.15.10, 1.15.12, 1.17, 1.18.2.3 and 1.18.3 of the AISC Specification also apply to these LRFD Criteria.

Rivets

The following experimentally obtained statistics are reported in Ref. 9 for A502 rivets:

Mean ultimate tensile capacity: $M_R\ D_R\ F_u$

Mean ultimate shear capacity: $0.70\ M_R\ D_R\ F_u$

C.O.V. of resistance, $V_R = 0.11$

where $F_u$ is the specified tensile strength ($F_u$ = 60 Ksi for Grade 1 A502 hot driven rivets and $F_u$ = 80 Ksi for Grade 2 A502 rivets) and $M_R$ and $D_R$ are equal to the values given below. The resistance factor $\phi$ is determined from the relationships

$$\bar{\phi} = 0.88\ \phi \quad , \quad \phi = [M_R\ D_R\ F_u/F_u]\ \exp\ (-\alpha\ \beta\ V_R)$$

$$\alpha = 0.55 \quad , \quad \beta = 4.5$$

| Type Rivet | $M_R$ | $D_R$ | $\bar{\phi}$ |
|---|---|---|---|
| A502 Grade 1 | 1.21 | 1.24 | 1.00 |
| A502 Grade 2 | 1.07 | 1.24 | 0.89 |

For the sake of simplicity it is recommended that $\bar{\phi}$ = 0.89 be used for both types of rivets, and that the shear capacity be equal to 0.6 $F_u$, the same as the shear capacity of bolts.

Bolts

It is recommended, in the absence of data, that A307 bolts have the same value of $\bar{\phi}$ as A490 high-strength bolts (e.g., $\bar{\phi}$ = 0.83 in tension,

$\bar{\phi} = 0.82$ in shear, and $\bar{\phi} = 0.76$ in combined bending and shear).

The threaded parts of steel meeting the requirements of Sec. 1.4.1 of the AISC Specification are to be considered as connection elements, and thus their resistance factors should be based on $\beta = 4.5$ and $\bar{\phi} = 0.88\ \phi$, i.e.,

$$\bar{\phi} = 0.88 \times 1.05 \exp(-0.55 \times 4.5 \times 0.11) = 0.70$$

<u>High-Strength Bolt Friction-Grip Joints</u>

High-strength bolted joints are usually still held together by friction under the service load conditions. Only when the force on the joint increases beyond the service loads will the bolts go into bearing, and at ultimate loading these joints will fail in bearing. Design criteria were developed previously for the limit state of bearing failure (Eq. 22), and the forces on the joint for this limit state are to be determined by the load factors for the strength limit state given in Ref. 10 and for the mean maximum loads characterizing the strength limit state. It is often necessary, however, that joints should not slip under service loading, and so the design must also consider this limit state. The design capacity is given by Eq. 27, and the corresponding value of $\bar{\phi} = 1.0$ was derived in the previous section. The forces on such joints are to be determined from the service loads using the load factors given in Ref. 10. It is important to realize that the design for one of the two limit states, e.g., strength (bearing failure) or serviceability (slip), does not automatically insure that the other is provided for, and so both must be considered. The following example illustrates this point:

Determine the number of 7/8 inch diameter A325 bolts needed to support the following forces:

1) <u>Strength limit state</u>:

mean dead load: 75 Kip

mean maximum lifetime wind load: 100 Kip

2) <u>Serviceability limit state</u>:

mean dead load: 75 Kip

<u>Load factors</u>: $\gamma_D = 1.2$ , $\gamma_w = 1.6$ (Ref. 10)

The load factors were derived for $\beta = 3.0$, but adjustment has been made through the correction factor discussed previously in this report for the fact that $\beta$ is 4.5 and 2 for the strength and the serviceability limit states, respectively.

<u>Factored design loads</u>:

Strength: $1.2 \times 75 + 1.6 \times 100 = 250$ Kip

Serviceability: $1.2 \times 75 + 1.6 \times 42 = 157$ Kip

<u>Nominal Resistance</u>

Bolt diameter: 7/8 inch

Gross area: 0.601 in$^2$ ; Stress area: $0.601 \times 0.768 = 0.462$ in$^2$

$F_u = 120$ Ksi for a 7/8 inch A325 bolt.

Two shear planes per bolt, $m = 2$

Strength limit state:

$$\bar{\phi} R_u = \bar{\phi}_m A_{SA} (0.6 F_u) = 0.86 \times 2 \times 0.462 \times 0.6 \times 120 = 57.2 \text{ Kip/bolt}$$

Required number of bolts: $\frac{250}{57.2} <$ 5 bolts

Serviceability limit state:

$$\bar{\phi} R_u = \bar{\phi}\, mK_s A_{SA}(0.7 F_u) = 1.0 \times 2 \times 0.33 \times 0.462 \times 0.7 \times 120 = 25.6 \text{ Kip/bolt}$$

Required number of bolts: $\frac{157}{25.6} <$ 7 bolts

Seven bolts are thus required, and serviceability controls the design.

When a friction type joint is loaded by a tensile component it is necessary to reduce the nominal capacity determined by Eq. 27 by multiplying it by the factor.

$$1 - \frac{T_D}{(T_i)_m} \quad (33)$$

where $T_D$ is the factored tensile design force and $T_i$ is the mean clamping force. It was shown previously that $(T_i) = 0.8\ A_{SA}\ F_u$, and, therefore, the reduction factor to be applied to $R_u$ in the presence of a tensile force is

$$1 - \frac{T_D}{0.8\ A_{SA}\ F_u} \quad (34)$$

Bearing Capacity of Bolt and Rivet Holes

The nominal resistance of bolt and rivet holes in bearing is given in Chapter 5 of Ref. 7 as

$$R_n = 1.4\ (L - \frac{d}{2})\ t\ F_u \quad (35)$$

but not greater than 3 dt $F_u$. This equation applies provided L/d is not less than 1.5. The terms in this equation are defined as follows:

- $L$ = distance from the center of the hole to the edge of the plate or to the edge of the next hole, measured parallel to the direction of the load
- $d$ = hole diameter
- $t$ = plate thickness
- $F_u$ = specified tensile strength of the plate material.

Test data presented in Fig. 5.52 of Ref. 7 give the following statistical data when test capacity is compared to prediction from Eq. 35:

Number of test data used: 27

Mean Test-to-Prediction ratio: 0.99

Coefficient of variation: 0.11

The statistics on $F_u$ (see following section of this report) are:

Ratio of mean-to-specified $F_u$: 1.10 ; V = 0.11

The modified resistance factor ($\bar{\phi} = 0.88\ \phi$) is, therefore, equal to

$$\bar{\phi} = 0.88 \times 0.99 \times 1.10 \exp(-\alpha\ \beta\ V_R)$$

Using $\alpha = 0.55$, $\beta = 4.5$ and $V_R = \sqrt{0.11^2 + 0.11^2} = 0.16$

$$\bar{\phi} = 0.64$$

An alternate expression is also given in Ref. 7 for $R_n$, and it would result in a somewhat simpler design equation:

$$R_n = Lt\ F_u \qquad (36)$$

but not greater than 3 dt $F_u$. This latter equation is recommended for use in LRFD with $\bar{\phi} = 0.65$.

Tension Members

Two strength limit states apply to tension members: 1) yielding and 2) fracture at the net section. The AISC Specification (Sec. 1.5.1.1) recognizes the differences in the consequences of yielding versus fracture by assigning a factor of safety of 5/3 to the former and 2 for the latter. This distinction will also be recognized here in that a safety index $\beta = 3$ will be assigned to yielding (i.e., "member" characterization) and $\beta = 4.5$ will be used for the limit state of fracture (i.e., "connection" characterization).

Yielding: The following statistics are given for the yield stress of steel in Ref. 1:

Ratio of mean-to-minimum specified:

1.05 for flanges, (V = 0.1)

1.10 for webs (V = 0.11)

If a coefficient of variation for "fabrication" $V_F = 0.05$ is assumed, and using $\beta = 3$ and $\alpha = 0.55$, the following two values of $\phi$ are obtained: for the flange material, $\phi = 0.88$; for the web material: 0.90. The smaller value is recommended for use.

Fracture: Data on the tensile strength of steel is given in Refs. 11 and 12, and Table 2 is a summary of information obtained from these references by analysis. From an examination of Table 2 it is apparent that assumed values of

$$\frac{(F_u)_{mean}}{F_u} = 1.10 \qquad \text{and} \qquad V = 0.1$$

are reasonable and conservative. The resistance factor will be determined with these values and for $\bar{\phi} = 0.88\ \phi$, $\beta = 4.5$, $V_F = 0.05$, $V_R = \sqrt{0.1^2 + 0.05^2} = 0.11$ and $\alpha = 0.55$.

$$\bar{\phi} = 0.88 \times 1.10 \exp(-0.55 \times 4.5 \times 0.11) = 0.74$$

Tension members are thus designed for the two limit states as follows:

$$\text{Yielding:} \quad R_n = A_n F_y, \quad \phi = 0.88 \tag{37}$$

$$\text{Fracture:} \quad R_n = A_n F_u, \quad \phi = 0.74 \tag{38}$$

Fracture controls when $F_u/F_y < 0.88/0.74 = 1.19$, and this is about the same as in Sec. 1.5.1.1 of the AISC Specification where fracture controls when $F_u/F_y < 1.2$.

In order to avoid localized plastic deformations caused by stress concentrations at the sides of the hole, $R_n = 0.75\ A_n F_y$ for pin-holes in eyebars, pin-connected plates or built-up members. This, too, is in accordance with the provisions of Sec. 1.5.1.1 of the AISC Specification.

## 4.7 Bearing on Contact Areas

In the absence of sufficient statistical data the provisions of the AISC Specification will be translated into an LRFD format as follows:

Sec. 1.5.1.5. of the AISC Specification states that the allowable bearing stress is

$$F_p = 0.90\ F_y \tag{39}$$

for milled surfaces, including bearing stiffeners and pins in reamed, drilled or bored holes, and

$$F_p = \left( \frac{F_y - 13}{20} \right) (0.66\ d) \tag{40}$$

for expansion rollers and rockers, in ksi, where d is the diameter of the roller or rocker in inches. Assuming that the basic allowable stress in the AISC Specification is 0.6 $F_y$, Eqs. 39 and 40 are translated into an LRFD format by multiplying each equation by 1/0.6:

The nominal stress capacity is thus

$$R_n = 1.5\ F_y \tag{41}$$

for milled surfaces and

$$R_n = 1.1\ d \left( \frac{F_y - 13}{20} \right) \tag{42}$$

for expansion rollers and rockers. The modified resistance factor for $\beta = 4.5$ ("connection" characterization) is $\bar{\phi} = 0.88 \times 0.88 = 0.77$ for both types of bearing surfaces.

A similar approach will be used in translating the provisions of AISC Sec. 1.5.5 Masonry Bearing. It will be assumed that the bearing capacity on sandstone and limestone and on brick in cement mortar is at least twice the allowable stress given in the AISC Specification. For bearing on concrete the provisions of the ACI Standard 318-71 (Sec. 10.14) will be used. The value of $\phi = 0.70$ from the ACI Standard will be adopted for all categories.

1) On sandstone and limestone, $R_n = 0.8$ ksi

2) On brick in cement mortar, $R_n = 0.5$ ksi

3) On the full area of concrete support, $R_n = 0.85\ f'_c$, where $f'_c$ is the specified compressive strength of concrete. When the supporting concrete surface is wider on all sides than the loaded area, the value of $R_n$ may be multiplied by $\sqrt{A_2/A_1}$ but not more than 2, where $A_1$ is the bearing area and $A_2$ is the area of the concrete.

## 5. SUMMARY

This report presented the background for the development of Load and Resistance Factor Design Criteria for connections. A tentative set of connection design criteria, based on the derivations in the report, is given in the Appendix. These criteria concern not only the design of fasteners, but they also make reference to the provisions relating to connection details in the current (1975) AISC Specification which remain valid for the LRFD Criteria.

## 5. ACKNOWLEDGMENTS

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## 7. NOMENCLATURE

| | | |
|---|---|---|
| $A_G$ | : | Gross area of bolt |
| $A_{SA}$ | : | Stress area of bolt |
| $A_T$ | : | Tributary floor area |
| $A_w$ | : | Effective area of throat of weld |
| $c$, $c_D$, $c_L$ | : | Coefficients translating load into force; subscripts D and L denote dead and live load, respectively |
| $D_m$, $D_c$ | : | Dead load intensity; subscripts m and c denote mean and code specified, respectively |
| $d$ | : | Bolt or rivet hole diameter |
| $F_{Exx}$ | : | Specified ultimate tensile strength of weld metal |
| $F_u$ | : | Specified ultimate tensile strength of bolt or plate material |
| $F_t$, $F_v$ | : | Allowable tensile and shear stress of bolts, respectively, according to AISC Specification |
| $K_s$ | : | Friction coefficient |
| $L$ | : | Length between center of bolt or rivet hole and the edge of the plate or the next adjacent hole |
| $m$ | : | Number of shear planes |
| $n$ | : | Number of bolts or rivets |
| $P_s$ | : | Capacity of a friction joint |
| $Q_m$ | : | Mean load effect |
| $R_n$, $R_m$ | : | Nominal and Mean Resistance, respectively |
| $S$, $S_D$ | : | Factored design shear force on bolt or rivet |
| $T$, $T_D$ | : | Factored design tensile force on bolt or rivet |

| | | |
|---|---|---|
| $T_i$ | : | Clamping force in high strength bolt |
| $t$ | : | Plate thickness |
| $V$ | : | Coefficient of variation, subscripts denoting various types according to load, material, fabrication, etc. |
| $\alpha$ | : | A coefficient equal to 0.55 |
| $\beta$ | : | Safety index |
| $\gamma$ | : | Load factor, subscripts denoting type according to loads |
| $\phi$ | : | Resistance Factor |
| $\sigma_u$ | : | Specified tensile strength of weld metal |
| $\tau_u$ | : | Specified shear strength of weld metal |

TABLE 1: SAFETY INDEX β FOR HIGH-STRENGTH BOLTS AND FILLET WELDS
(Code Specified Live Load Intensity, $L_c$ = 50 psf)

| $D_c$ (Dead Load) | $A_T$ (Trib. Area) | β (1) | β (2) | β (3) | β (4) | β (5) |
|---|---|---|---|---|---|---|
| 50 psf | 200 $ft^2$ | 4.3 | 4.1 | 5.1 | 5.4 | 4.4 |
| | 400 | 4.7 | 4.3 | 5.8 | 6.2 | 5.2 |
| | 575 | 4.5 | 4.2 | 5.5 | 5.9 | 4.9 |
| | 800 | 4.8 | 4.4 | 6.1 | 6.5 | 5.6 |
| | 1000 | 4.9 | 4.5 | 6.3 | 6.7 | 5.8 |
| 75 psf | 200 | 4.7 | 4.9 | 5.8 | 6.2 | 5.2 |
| | 400 | 4.9 | 4.5 | 6.2 | 6.7 | 5.7 |
| | 719 | 4.7 | 4.3 | 6.1 | 6.5 | 5.6 |
| | 1000 | 4.8 | 4.4 | 6.3 | 6.7 | 5.9 |
| 100 psf | 200 | 5.0 | 4.6 | 6.5 | 7.0 | 6.2 |
| | 400 | 5.0 | 4.7 | 6.7 | 7.1 | 6.3 |
| | 600 | 5.0 | 4.6 | 6.7 | 7.4 | 6.6 |
| | 750 | 4.9 | 4.5 | 6.6 | 7.1 | 6.3 |
| | 1000 | 5.0 | 4.6 | 6.8 | 7.3 | 6.5 |

(1) Fillet welds, data from Ref. 5

(2) Fillet welds, data from Ref. 6

(3) A325 Bolts, 1/2" Dia., direct tension

(4) A325 Bolts, 7/8" Dia., direct tension

(5) A490 Bolts, 1/2" Dia., direct tension

TABLE 1, Continued

| $D_c$ | $A_T$ | β (6) | β (7) | β (8) | β (9) | β (10) |
|---|---|---|---|---|---|---|
| 50 psf | 200 $ft^2$ | 7.6 | 8.0 | 6.4 | 1.8 | 1.2 |
| | 400 | 8.2 | 8.6 | 7.5 | 2.0 | 1.3 |
| | 575 | 8.0 | 8.4 | 7.3 | 1.9 | 1.2 |
| | 800 | 8.8 | 9.2 | 8.1 | 2.0 | 1.3 |
| | 1000 | 9.0 | 9.4 | 8.3 | 2.1 | 1.4 |
| 75 psf | 200 | 8.2 | 8.6 | 7.5 | 2.0 | 1.3 |
| | 400 | 8.9 | 9.3 | 8.2 | 2.1 | 1.4 |
| | 719 | 9.1 | 9.5 | 8.4 | 1.9 | 1.2 |
| | 1000 | 9.3 | 9.7 | 8.6 | 2.0 | 1.3 |
| 100 psf | 200 | 9.5 | 9.9 | 8.9 | 2.1 | 1.4 |
| | 400 | 9.7 | 10.1 | 9.0 | 2.2 | 1.5 |
| | 600 | 10.2 | 10.6 | 9.6 | 2.1 | 1.4 |
| | 750 | 10.0 | 10.4 | 9.3 | 2.0 | 1.3 |
| | 1000 | 10.1 | 10.6 | 9.5 | 2.0 | 1.3 |

(6) A325 Bolts, 1/2" Dia., shear through shank

(7) A325 Bolts, 1/2" Dia., shear through threads

(8) A490 Bolts, 1/2" Dia., shear through shank

(9) A325 Bolts, 1/2" Dia., friction joist, turn of nut

(10) A490 Bolts, 1/2" Dia., friction joist, turn of nut

TABLE 2: STATISTICAL DATA ON THE TENSILE STRENGTH

| Ref. | Type Steel | Type Section and Location | $F_u$ (Ksi) Minimum Specified | $\frac{(F_u)_{mean}}{F_u}$ | C.O.V=V | Number of Data |
|---|---|---|---|---|---|---|
| 11 | A7 | W-Web | 60 | 1.14 | 0.05 | 7 |
| 11 | A7 | W-Flange | 60 | 1.08 | 0.04 | 13 |
| 11 | A7 | W-Web | 60 | 1.08 | 0.04 | 13 |
| 12 | A36 | W-Web | 58 | 1.17 | 0.06 | 361 |
| 12 | A36 | W-Flange | 58 | 1.15 | 0.06 | 361 |
| 12 | A36 | Plates | 58 | 1.13 | 0.12 | 357 |
| 12 | HSS | Plates | 70 | 1.16 | 0.12 | 56 |

$D_c$ = 50psf:

A: A325 H.S. Bolts, direct tension, 1/2"$\phi$; A': 7/8"$\phi$

B: A490 H.S. Bolts, direct tension, 1/2"$\phi$

C: Fillet Welds, Data from Ref. 6.; C': Data from Ref. 5

D: A325 H.S. Bolts, shear through shank, 1/2"$\phi$

E: A325 H.S. Bolts, shear through thread, 1/2"$\phi$

F: A990 H.S. Bolts, shear through shank, 1/2"$\phi$

G: A325 H.S. Bolts, shear through shank, 1/2"$\phi$, $D_c$ = 100 psf

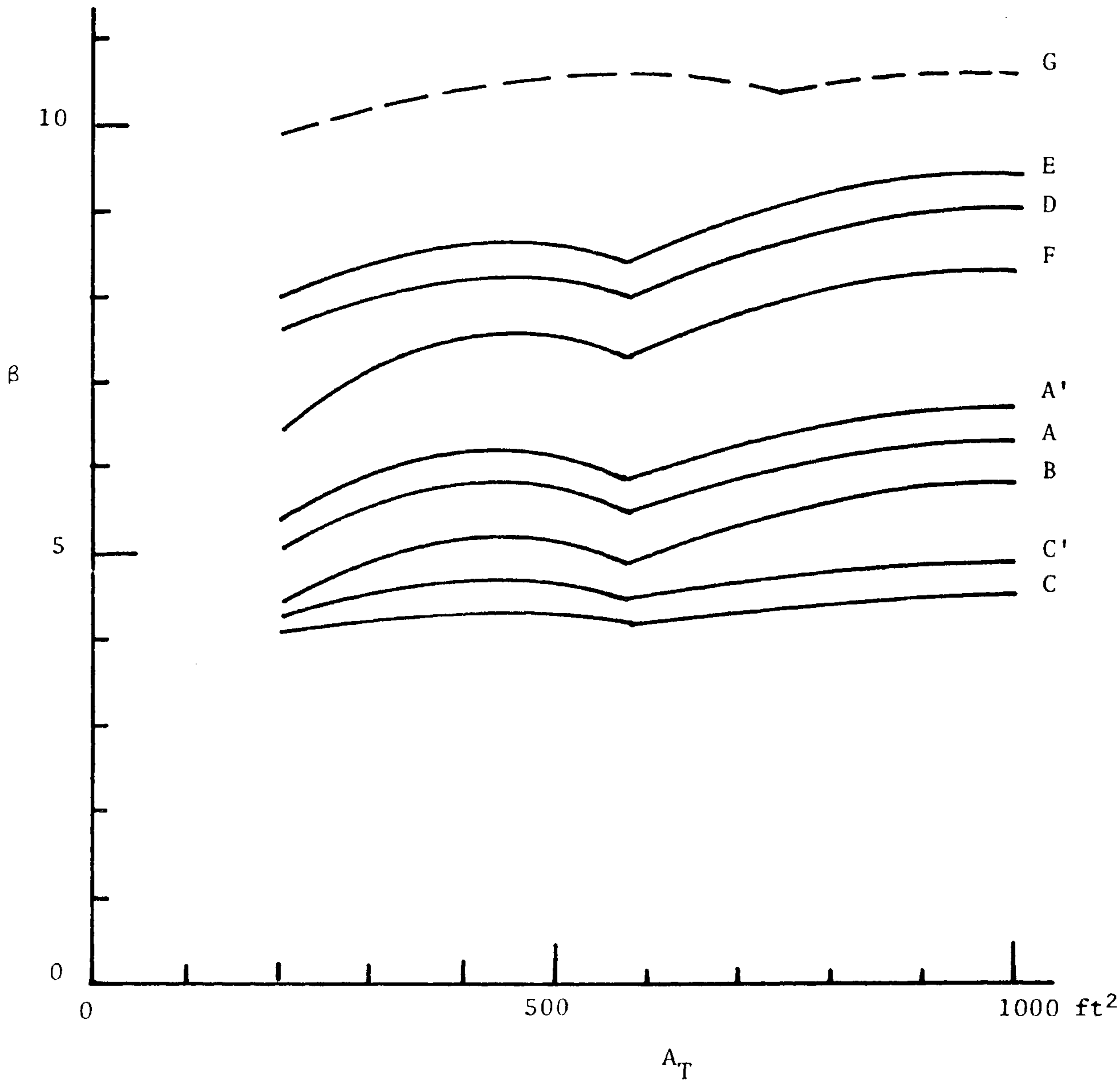


Fig. 1 Variation of the Safety Index β for Fasteners for a Basic Code Live Load Intensity of 50 psf.

## APPENDIX: TENTATIVE LRFD CRITERIA FOR CONNECTIONS

Following is an excerpt from Ref. 10; this section contains the tentative LRFD criteria provisions for connections. The section numbering in the criteria of this Appendix is the same as in Ref. 10.

## 2.4 The Design of Connections

### 2.4.1 Definition

Connections consist of connecting elements (e.g., stiffeners, plates, angles, brackets) and connectors (welds, bolts, rivets). Forces acting on the parts of the connections are the forces determined by structural analysis for the factored loads acting on the structure, or the forces necessary to develop part or all of the strength of the members, whichever is appropriate.

### 2.4.2 Design of Connecting Elements

The factored nominal strength $\phi R_n$ of connecting elements, such as shapes and plates (e.g., brackets, clip-angles, stiffeners, web plates, doubler plates, base plates) is to be determined for the appropriate limit state (e.g., yielding, plastification, buckling, rupture), using $\phi = 0.77$, to ascertain that $\phi R_n$ is larger than or equal to the forces to be resisted.

The provisions concerning details of the connections contained in Sec. 1.15 of the AISC Specification apply also for the connections designed according to these LRFD criteria.

### 2.4.3 Connectors

#### 2.4.3.1 Welds

The factored maximum strength $\phi F_w$ of welds is determined as follows:

Complete penetration groove welds

a) tension or compression normal to the effective area or parallel to the axis of the weld

$$\phi = 0.88 \ , \quad F_w = F_y \tag{2.4.3-1}$$

b) shear on the effective area

$$\left.\begin{aligned} &\phi = 0.80 \ , \quad F_w = 0.6 \ F_{EXX} \\ \text{and} \quad &\phi = 0.86 \ , \quad F_{BM} = F_y/\sqrt{3} \end{aligned}\right\} \tag{2.4.3-2}$$

Partial penetration groove welds

a) compression normal to effective area, tension or compression parallel to axis of the weld

$$\phi = 0.88 \ , \quad F_w = F_y \qquad (2.4.3\text{-}3)$$

b) shear parallel to axis of weld

$$\left.\begin{array}{ll} & \phi = 0.80 \ , \quad F_w = 0.6\ F_{EXX} \\ \text{and} & \phi = 0.86 \ , \quad F_{BM} = F_y/\sqrt{3} \end{array}\right\} \qquad (2.4.3\text{-}4)$$

c) tension normal to effective area

$$\left.\begin{array}{ll} & \phi = 0.80 \ , \quad F_w = 0.6\ F_{EXX} \\ \text{and} & \phi = 0.88 \ , \quad F_{BM} = F_y \end{array}\right\} \qquad (2.4.3\text{-}5)$$

Fillet welds

a) stress on effective area

$$\left.\begin{array}{ll} & \phi = 0.80 \ , \quad F_w = 0.6\ F_{EXX} \\ \text{and} & \phi = 0.86 \ , \quad F_{BM} = F_y/\sqrt{3} \end{array}\right\} \qquad (2.4.3\text{-}6)$$

b) tension or compression parallel to axis of weld

$$\phi = 0.88 \ , \quad F_w = F_y \qquad (2.4.3\text{-}7)$$

Plug and slot welds

Shear parallel to faying surfaces (on effective area)

$$\left.\begin{array}{l} \phi = 0.80 \ , \quad F_w = 0.6\ F_{EXX} \\ \phi = 0.86 \ , \quad F_{BM} = F_y/\sqrt{3} \end{array}\right\} \qquad (2.4.3\text{-}8)$$

In these equations $F_w$ is the nominal maximum stress capacity of the weld electrode material, $F_{EXX}$ is the specified tensile strength of the electrode material, $F_y$ is the specified yield stress of the base metal, and $F_{BM}$ is the nominal maximum stress capacity of the base metal.

The requirements regarding electrodes and matching base-metals given in Tables 1.5.3 and 1.17.3, as well as the provisions regarding welds given in Sec. 1.14.7, 1.15.6, 1.15.9, 1.15.10, 1.15.12, 1.17, 1.18.2.3 and 1.18.3 of the AISC Specification also apply to these LRFD criteria.

2.4.3.2 Bolts, Rivets and High-Strength Bolts

The factored maximum strength of bolts (ASTM-A307), rivets (ASTM-A502) and high-strength bolts (ASTM-A325 and A490) is $\phi\ R_n$, where $\phi$ and $R_n$ are defined as follows:

2.4.3.2.1 Tension

$$R_n = A_{SA}\ F_u \qquad (2.4.3\text{-}9)$$

$\phi = 0.89$ for A502 rivets

$\phi = 0.84$ for A325 high-strength bolts

$\phi = 0.83$ for A490 high-strength bolts and A307 bolts

$\phi = 0.77$ for threaded rods made from material meeting the requirements of Sec. 1.4.1 of the AISC Specification

where $F_u$ is the specified tensile strength* of the fastener material and $A_{SA}$ is the stress area (e.g., thread area for bolts and gross area for rivets).

2.4.3.2.2 Shear

$$R_n = m\ A_{SA}\ (0.6\ F_u) \qquad (2.4.3\text{-}10)$$

$\phi = 0.89$ for A502 rivets

$\phi = 0.86$ for A325 high-strength bolts

$\phi = 0.82$ for A490 high-strength bolts

* The specified tensile strength $F_u$ of the fasteners is: A502 grade 1 rivets: 60 Ksi; A502 grade 2 rivets: 80 Ksi; A307 bolts: 60 Ksi; A325 high-strength bolts: 120 Ksi for 1/2 through 1 inch diameters, 105 Ksi for 1-1/8 through 1-1/2 inch diameters; A490 bolts: 150 Ksi for 1/2 through 1-1/2 inch diameters. (These values are quoted from Ref. 7).

$\phi = 0.75$ for threaded bolts made from material meeting the requirements of Sec. 1.4.1 of the AISC Specification.

where m is the number of shear planes per bolt and $A_{SA}$ is the stress area, equal to the thread area if the shear plane passes through the threads, and the shank area if the shear plane passes through the shank.

2.4.3.2.3 Combined Tension and Shear

When a fastener is subject to forces producing both tension and shear, the following interaction equation must be satisfied:

$$S_D^2 + (m \times 0.6\ T_D)^2 \leq (\phi\ m\ A_{SA} \times 0.6\ F_u)^2 \qquad (2.4.3\text{-}11)$$

$\phi = 0.89$ for A502 rivets

$\phi = 0.80$ for A325 high-strength bolts

$\phi = 0.76$ for A490 high-strength bolts and A307 bolts

$\phi = 0.75$ for threaded bolts made from material meeting the requirements of Sec. 1.4.1 of the AISC Specification.

$S_D$ and $T_D$ are the factored design shear force and tension force, respectively, acting on the fastener.

2.4.3.2.5 Bearing Capacity of Bolt and Rivet Holes

The factored maximum strength of a bolt or rivet hole in bearing is $\phi\ R_n$, where $\phi = 0.65$ and

$$R_n = Lt\ F_u \qquad (2.4.3\text{-}12)$$

but not greater than $3\ dt\ F_u$

where $L$ = distance from plate edge to center of hole or to the edge of the next hole, measured parallel to the direction of the load

$d$ = hole diameter

$t$ = plate thickness

$F_u$ = specified tensile strength of plate material.

The ratio L/d may not be less than 1.5.

2.4.3.2.6 Bolt and Rivet Hole Details

The provisions concerning bolt and rivet hole details in Sec. 1.16.1 through 1.16.5 and 1.16.7 in the AISC Specification also apply to these LRFD criteria.

2.4.3.2.7 High-Strength Bolt Friction-Grip Joints*

The factored nominal strength of friction-grip joints is $\phi R_n$, where $\phi = 1.0$ and

$$R_n = mn K_s A_{SA} \times 0.7 F_u \tag{2.4.3-13}$$

where $m$ = number of slip planes

$n$ = number of bolts per joint

$K_s$ = friction coefficient**

$A_{SA}$ = thread area

$F_u$ = specified tensile strength of bolt material

The value of $R_n$ from Eq. 2.4.3-14 must be multiplied by the following reduction factor when a factored tensile force $T_D$ is present:

$$1 - \frac{T_D}{A_{SA} F_u} \tag{2.4.3-14}$$

The factored design forces for friction-grip joints are to be determined for the service loading. An additional check for maximum capacity must also be made for these joints for the factored maximum lifetime levels using the resistances determined from Sec. 2.4.3.2.1, 2.4.3.2.2 and 2.4.3.2.3.

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* Since slip is a serviceability limit state, serviceability load combinations are to be used in design (see Sec. C.1.2.2 in the Commentary).

** For clean mill-scale contact surfaces $K_s$ = 0.33. Values of $K_s$ for other types of surfaces are given in Chap. 12 of Ref. 7.

2.4.4 Bearing Stresses on Contact Area

The factored nominal stress capacity of surfaces in bearing is $\phi R_n$, which is defined below for various types of bearing:

2.4.4.1 Milled Surfaces

For milled surfaces, including bearing stiffeners and pins in reamed, drilled or bored holes

$$\phi = 0.77 \ , \quad R_n = 1.5 \ F_y \tag{2.4.3-15}$$

2.4.4.2 Expansion Rollers and Rockers

$$\phi = 0.77 \ , \quad R_n = \left( \frac{F_y - 13}{20} \right) \times 1.1 \ d \tag{2.4.3-16}$$

where $R_n$ is in kips per linear inch, and d is the diameter of the rocker in inches. When parts in contact have different yield stress values, the smaller value of $F_y$ is to be used in Eqs. 2.4.3-15 and 2.4.3-16.

2.4.4.3 Masonry Bearing

$\phi$ = 0.70 and

$R_n$ = 0.8 Ksi on sandstone or limestone

$R_n$ = 0.5 Ksi on brick in cement mortar

$R_n$ = 0.85 $f'_c$ on the full area of a concrete support

where $f'_c$ = specified compressive strength of concrete

When the supporting concrete area is wider on all sides than the loaded area, the value of $R_n = 0.85 \ f'_c$ may be increased by the factor $\sqrt{A_2/A_1}$ but not more than 2, where $A_1$ is the bearing area and $A_2$ is the concrete area.