Bolt Shear Design Considerations

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Abstract

In this paper, bolt shear capacities are reviewed using the Load and Resistance Factor Design (LRFD) philosophy. Only bolt-shear limit states are addressed, although one aspect of slip critical limit states is addressed incidentally. This paper does not consider bolt bearing limit states. Test data used to justify the adoption of ASTM A325 and A490 high-strength bolts was obtained from previous research programs. The data also included various types of rivets and Huck bolts for general comparison. First, the test data are used to evaluate the current American Institute of Steel Construction (AISC, 2005) and Research Council on Structural Connections (RCSC, 2004) bolt shear provisions and to determine the current reliability, β, which is found to be conservative when based on a resistance factor, ϕ, of 0.75. The appropriateness of the ϕ-factor for bolt shear is addressed. Canadian (CSA S16-01) and Eurocode (EN 1993) provisions are also evaluated and shown not to be compatible with the test results. Two design equations are developed—one linear, one a step function—that result in a β value slightly greater than 3.0, appropriate for a manufactured product. The single-step function (with a step at 38 in.) is recommended for inclusion in updated design specifications. This design provision increases the design strength by 12.5% for short connections and by 17.2% for long connections. The test data indicate that there is no need for a bolt strength reduction due to the length of the connection, provided that the connection material gross and net section areas exceed certain ratios. That ratio is a function of the connection material yield and tensile strength, the total bolt shear area and the bolt tensile strength.

Keywords: bolt shear, reliability, resistance factor, connection length factor.

BACKGROUND

The current shear strength of a high strength bolt may be expressed by the following equation:

$$P_n = P_u A_b R_1 R_2 R_3 R_4 \tag{1}$$

where

P_u = ultimate tensile strength of bolt, ksi
R_1 = 0.625, shear-to-tension ratio
R_2 = 0.80, connection length reduction factor for $L \le 50$ in.
R_3 = 1.00 if threads are excluded from the shear plane
= 0.80 if threads are included in the shear plane
R_4 = 0.80, additional connection length reduction factor for $L > 50$ in.
L = connection length, in.
A_b = nominal unthreaded body area of bolt, in.^{2}

The design shear values for ASTM A325 and A490 bolts are given in RCSC Specification Table 5.1 (RCSC, 2004). The design values, for other fasteners, such as ASTM A307 bolts and threaded material, are given in AISC *Specification for Structural Steel Buildings* (hereafter AISC *Specification,* 2005) Table J3.2. In Load Resistance and Factor Design (LRFD(terms, the design shear strength of a bolt is ϕR_n, with $\phi = 0.75$ and $R_n = P_n$.

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The design values are based on an extensive research program conducted by the steel industry at the Fritz Engineering Laboratory at Lehigh University from the 1950s through the early 1970s. As was the custom at the time, the high-strength bolts were fully pretensioned. Bolt threads were excluded from the shear plane. In addition, an earlier research investigation at the University of Illinois and at University of California by Davis et al. (1940) was reviewed concerning riveted connections for the San Francisco–Oakland Bay Bridge. All of the data was summarized in the *Guide to Design Criteria of Bolted and Riveted Joints* (the *Guide*) by Kulak et al. (1987). The roles of 12 basic variable groups resulted in approximately 45 test variables that are described in the *Guide* and will not be repeated in this paper. However, three of the basic variable groups will be subsequently examined and used to develop a proposed design procedure.

The types of connections tested were the basic lap splice, the butt splice, the open shingle splice, and the closed shingle splice, as shown in Figure 1. A review of the literature revealed that the data from each test series were not uniformly reported. As a result, the original research reports were used to augment the background data. The connection length for a lap splice is the distance between the centerlines of the extreme end bolts. The connection length for a butt splice is the distance from the centerline of the bolt at one end of the connection to the centerline of the bolt closest to the overall connection centerline. Fortunately, large quantities of bolts were obtained from production lots, so essentially identical bolts were used in several test programs. Each lot of bolts was tested to determine both the tensile and shear strength.

A literature search identified 11 papers and reports in addition to several supporting reports that resulted in data from 119 tests. Because of incomplete background information, the 40 tests from Davis et al. (1940) were not used other than to document its test results. Of the remaining 79 tests, the breakdown is as follows: 54 used ASTM A325 bolts, 18 used ASTM A490 bolts, 5 used rivets, and 2 used Huck bolts.

The CSA S16-01 standard, as supplemented in 2005, adopted the 2003 Eurocode EN 1993, Clause 3.8, which is shown as Equation 2, converted to U.S. customary units:

$$V_R = 0.60\phi F_u A_b \tag{2}$$

where

ϕ = 0.80, resistance factor
F_u = nominal bolt tensile stress, ksi
A_b = bolt area, in.^{2}

The R_1 factor of 0.60 represents the shear-to-tension ratio used in the CSA S16 document. When the threads are included in the shear plane, an R_3 factor of 0.70 is used. Of greater significance, V_R is valid up to a connection length, L, of 15d, where d is the bolt diameter in inches. When the connection length exceeds 15d, V_R is reduced by the factor $(1.075 - 0.005L/d)$ but is not taken as less than 0.75 times the original value given in Equation 2.

TEST DATA

As previously indicated, test data were obtained from 11 papers and reports: Bendigo et al. (1963), Davis et al. (1940), Fisher et al. (1963), Fisher and Kulak (1968), Fisher and Yoshida (1970), Foreman and Rumpf (1961), Kulak and Fisher (1968), Power and Fisher (1972), Rivera and Fisher (1970), and Sterling and Fisher (1965, 1966). Because of the various reporting formats and test parameters, it was not possible to directly compare the results. The published test ultimate shear strength of each connection was reduced to an average ultimate shear strength, P_{TEST}, of a single connector, bolt or rivet, loaded on two shear planes (double shear). The predicted ultimate shear strength of the same connector was computed using the reported appropriate single shear connector test data times two, P_{PRED}, for each lot of bolts or rivets.

The ratio P_{TEST}/P_{PRED} was then computed to compare the results, with connection length as the only independent variable. All of the reconfigured test data are given in Table 1 and plotted in Figure 2. The solid line represents the current step function for the length reduction factor. The dotted line represents the same equation multiplied by the current AISC ϕ for bolts, 0.75. The plotted data are in a non-dimensional form, eliminating the independent variables of bolt diameter, material type and connection configuration. Although this

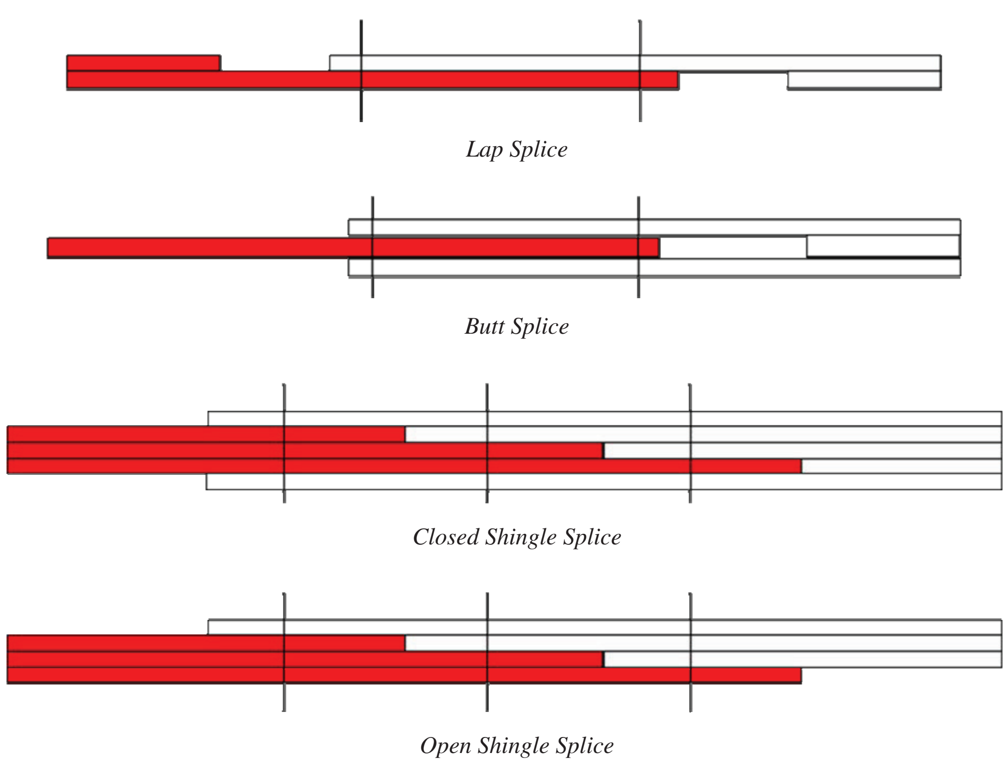

Fig. 1. Four types of connections.

Table 1. Connection Test and Computation Data

No.	ID	Dia. (in.)	Type	Bolts in Line	L (in.)	P_{TEST} (kip)	P_{PRED} (kip)	$\frac{P_{TEST}}{P_{PRED}}$	F_{yp} (ksi)	F_{up} (ksi)	F_v (ksi)	A_s (in.^{2})	A_n/A_s	A_g (in.^{2})	$\frac{0.90\,A_sF_v}{F_{yp}}$ (in.^{2})	A_n (in.^{2})	$\frac{0.90\,A_sF_v}{F_{up}}$ (in.^{2})	(1) R_2	(2) Mat.	Ref. No.
1	F42b	1⅛	A325	4	10.5	131.5	131.4	1.001	114.0	121.4	66.1	15.9	0.51	13.0	8.3	8.07	7.8	0.90C	A514	4
2	F42c	1⅛	A325	4	10.5	133.0	131.4	1.012	114.0	121.4	66.1	15.9	0.56	13.8	8.3	8.9	7.8	0.90C	A514	4
3	F42d	1⅛	A325	4	10.5	132.0	131.4	1.005	114.0	121.4	66.1	15.9	0.61	14.5	8.3	9.66	7.8	0.90C	A514	4
4	F42e	1⅛	A325	4	10.5	132.8	131.4	1.01	114.0	121.4	66.1	15.9	0.66	15.4	8.3	10.5	7.8	0.90C	A514	4
5	F42g	1⅛	A325	4	10.5	134.3	131.4	1.022	114.0	121.4	66.1	15.9	0.72	16.3	8.3	11.4	7.8	0.90C	A514	4
6	F111	1⅛	A325	11	35.0	167.8	131.4	1.036	114.0	119.8	81.5	21.8	0.74	18.9	14.0	15.5	13.3	0.90C	A514	4
7a	HJ131	⅞	A325	13	42.0	85.7	113.0	0.758	36.5	66.1	94.0	15.6	1.13	21.1	36.2	17.6	20.0	0.75S	A36M	4
7b									44.1	76.3	94.0	15.6	0.91	16.9	29.9	14.1	17.3	0.75S	A440L	4
8A	HJ132	⅞	A325	13	42.0	69.8	113.0	0.618	36.5	66.1	94.0	15.6	0.89	16.3	36.2	13.9	20.0	0.75S	A36M	4
8b									44.1	76.3	94.0	15.6	0.71	13.0	29.9	11.1	17.3	0.75S	A440L	4
9a	HJ133	⅞	A490	13	42.0	116.8	115.3	1.013	44.1	76.3	95.9	15.6	1.92	33.6	30.5	29.8	17.6	0.90C	A440M	4
9b									114.0	121.4	95.9	15.6	0.90	15.8	11.8	14.0	11.1	0.90C	A514L	4
10a	HJ135	⅞	A325	13	42.0	95.7	96.9	0.988	44.1	76.3	80.6	15.6	1.68	29.8	25.7	26.1	14.8	0.90C	A440M	4
10b									114.0	121.4	80.6	15.6	0.79	14.1	9.9	12.3	9.3	0.90C	A514L	4
11	J42a	1	A490	4	10.5	154.8	151.7	1.020	114.0	121.4	96.6	12.6	0.76	13.9	9.6	9.58	9.0	0.90C	A514	7
12	J42b	1	A490	4	10.5	153.5	151.7	1.012	114.0	121.4	96.6	12.6	0.82	14.6	9.6	10.3	9.0	0.90C	A514	7
13	J42c	1	A490	4	10.5	150.8	151.7	0.994	114.0	121.4	96.6	12.6	0.86	15.2	9.6	10.9	9.0	0.90C	A514	7
14	J42d	1	A490	4	10.5	152.5	151.7	1.006	114.0	121.4	96.6	12.6	0.92	16.0	9.6	11.6	9.0	0.90C	A514	7
15	J072	⅞	A490	7	21.0	121.4	116.6	1.041	101.6	111.9	97.0	8.41	0.91	9.58	7.2	7.66	6.6	0.90C	A514	7
16	J132	1⅛	A490	13	42.0	201.2	191.8	1.049	101.6	111.9	96.5	25.8	0.91	28.6	22.1	23.7	20.0	0.90C	A514	7
17	J172	⅞	A490	17	56.0	118.5	119.8	1.016	101.6	111.9	97.0	20.4	0.90	20.4	17.5	18.5	15.9	0.90C	A514	7
18	J251	⅞	A490	25	84.0	109.4	119.8	0.913	101.6	111.9	99.7	30.1	0.82	28.4	26.6	24.6	24.1	0.90C	A514	7
19	J252	⅞	A490	25	84.0	124.0	119.8	1.035	101.6	111.9	99.7	30.1	1.12	37.6	26.6	33.7	24.1	0.90C	A514	7
20	D71	⅞	A325	7	21.0	80.4	102.5	0.784	28.2	60.0	85.3	16.8	1.08	21.9	45.7	18.2	21.5	0.75S	A7	1
21	D81	⅞	A325	8	24.5	80.1	102.5	0.782	28.2	60.0	85.3	19.2	1.09	24.7	52.3	21.0	24.6	0.75S	A7	1
22	D91	⅞	A325	9	28.0	75.4	102.5	0.736	28.2	60.0	85.3	21.6	1.10	27.5	58.8	23.7	27.6	0.75S	A7	1
23	D101	⅞	A325	10	31.5	75.3	102.5	0.734	28.2	60.0	85.3	24.0	1.08	29.8	65.3	26.1	30.7	0.75S	A7	1
24	D701	⅞	A325	7	21.0	86.6	112.0	0.773	33.6	64.3	93.2	16.8	1.14	23.8	41.9	19.3	21.9	0.75S	A7	1

Notes:
(1) 0.90 indicates data represented by a circle (C) in Figure 3;
0.75 indicates data represented by a square (S) in Figure 3;
0.75* indicates data represented by a triangle (T) in Figure 3.
(2) L and M refer to lap plates and main member plates in hybrid connections.

Table 1. Connection Test and Computation Data (Cont.)

No.	ID	Dia. (in.)	Type	Bolts in Line	L (in.)	P_{TEST} (kip)	P_{PRED} (kip)	$\frac{P_{TEST}}{P_{PRED}}$	F_{yp} (ksi)	F_{up} (ksi)	F_v (ksi)	A_s (in.^{2})	A_n/A_s	A_g (in.^{2})	$\frac{0.90 A_s F_v}{F_{yp}}$ (in.^{2})	A_n (in.^{2})	$\frac{0.90 A_s F_v}{F_{up}}$ (in.^{2})	(1) R_2	(2) Mat.	Ref. No.
25	D801	⅞	A325	8	24.5	82.1	112.0	0.733	33.6	64.3	93.2	19.2	1.10	26.0	47.9	21.1	25.0	0.75S	A7	1
26	D901	⅞	A325	9	28.0	83.2	112.0	0.742	33.6	64.3	93.2	21.6	1.12	30.0	53.9	24.3	28.2	0.75S	A7	1
27	D1001	⅞	A325	10	31.5	83.4	112.0	0.744	33.6	64.3	93.2	24.0	1.12	33.1	59.9	26.8	31.3	0.75S	A7	1
28	D10	⅞	A325	10	31.5	77.2	109.0	0.708	28.2	60.0	90.7	24.0	1.09	33.7	69.5	26.2	32.7	0.75S	A7	1
29	D13A	⅞	A325	13	31.5	76.5	109.0	0.701	28.2	60.0	90.7	31.3	1.09	41.7	90.6	34.2	42.6	0.75S	A7	1
30	D13	⅞	A325	13	42.0	71.3	109.0	0.654	28.2	60.0	90.7	31.3	1.09	41.7	90.6	34.2	42.6	0.75S	A7	1
31	D16	⅞	A325	16	52.5	65.2	109.0	0.598	28.2	60.0	90.7	38.5	1.09	49.4	111.4	41.9	52.4	0.75S	A7	1
32	L2	⅞	A325	2	3.5	98.5	100.2	0.983	28.2	60.0	83.4	2.40	1.65	5.76	6.4	3.96	3.0	0.75*T	A7	1
33	L5	⅞	A325	5	14.0	89.2	100.2	0.89	28.2	60.0	83.4	6.01	1.32	9.63	16.0	7.92	7.5	0.75*T	A7	1
34	L7	⅞	A325	7	21.0	91.4	100.2	0.912	28.2	60.0	83.4	8.41	1.57	14.9	22.4	13.2	10.5	0.75*T	A7	1
35	L10	⅞	A325	10	31.5	74.8	100.2	0.746	28.2	60.0	83.4	12.0	1.10	14.9	31.9	13.2	15.0	0.75S	A7	1
36	DR71	⅞	A141R	7	21.0	52.7	66.5	0.793	28.2	60.0	55.3	16.8	0.785	16.9	29.7	13.2	13.9	0.75S	A7	1
37	DR101	⅞	A141R	10	31.5	47.1	66.5	0.709	28.2	60.0	55.3	24.0	0.769	22.2	42.4	18.5	19.9	0.75S	A7	1
38	DR131	⅞	A141R	13	42.0	46.8	66.5	0.704	28.2	60.0	55.3	31.3	0.762	27.5	55.2	23.8	26.0	0.75S	A7	1
39	B3	⅞	A325	4	10.5	87.5	96.2	0.910	36.6	65.5	80.0	24.0	1.11	36.0	47.2	26.6	26.4	0.75*T	A7	6
40	B6	⅞	A325	3	7.0	86.1	96.2	0.896	36.6	65.5	80.0	21.6	1.15	36.0	42.5	24.8	23.7	0.75*T	A7	6
41	BR2	⅞	A141R	5	14.0	52.0	60.0	0.867	36.6	65.5	49.9	30.0	0.887	36.0	36.8	26.6	20.6	0.75*T	A7	6
42	A3	1	A325	4	12.0	113.8	125.6	0.906	36.6	65.5	80.0	25.1	1.10	36.0	49.4	27.5	27.6	0.75S	A7	6
43	G1	1⅛	A325	3	8.0	149.8	167.0	0.897	36.6	65.5	84.0	23.9	1.11	36.0	49.4	26.5	27.6	0.75S	A7	6
44	B5	1⅛	A325	5	14.0	84.0	96.2	0.874	36.6	65.5	80.0	24.0	1.11	36.0	47.2	26.6	26.4	0.75*T	A7	6
45	K42a	⅞	A490	4	10.5	122.5	124.4	0.985	43.0	76.0	103.5	9.62	1.22	15.5	20.8	11.7	11.8	0.75S	A440	10
46	K42b	⅞	A490	4	10.5	122.5	124.4	0.985	43.0	76.0	103.5	9.62	1.27	16.0	20.8	12.2	11.8	0.75*T	A440	10
47	K42c	⅞	A490	4	10.5	124.5	124.4	1.001	43.0	76.0	103.5	9.62	1.31	16.4	20.8	12.6	11.8	0.75*T	A440	10
48	K42d	⅞	A490	4	10.5	125.5	124.4	1.009	43.0	76.0	103.5	9.62	1.37	17.0	20.8	13.2	11.8	0.75*T	A440	10
49	K131	⅞	A490	13	31.6	109.6	121.5	0.902	43.0	76.0	101.1	15.6	1.30	24.2	33.0	20.4	18.7	0.75*T	A440	10
50	K132	⅞	A490	13	63.0	100.9	121.5	0.830	43.0	76.0	101.1	15.6	1.30	24.1	33.0	20.3	18.7	0.75*T	A440	10
51	K133	⅞	A490	13	63.0	127.7	121.5	1.051	43.0	76.0	101.1	15.6	1.92	33.8	33.0	30.0	18.7	0.90C	A440	10
52	K191	⅞	A490	19	63.0	94.4	121.5	0.777	43.0	76.0	101.1	22.8	1.30	33.4	48.2	29.7	27.3	0.75*T	A440	10

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Table 1. Connection Test and Computation Data (Cont.)

No.	ID	Dia. (in.)	Type	Bolts in Line	L (in.)	P_{TEST} (kip)	P_{PRED} (kip)	$\frac{P_{TEST}}{P_{PRED}}$	F_{yp} (ksi)	F_{up} (ksi)	F_v (ksi)	A_s (in.^{2})	A_n/A_s	A_g (in.^{2})	$\frac{0.90 A_s F_v}{F_{yp}}$ (in.^{2})	A_n (in.^{2})	$\frac{0.90 A_s F_v}{F_{up}}$ (in.^{2})	(1) R_2	(2) Mat.	Ref. No.
53	E41b	⅞	A325	4	10.5	94.3	101.4	0.929	45.3	75.8	84.4	9.62	0.947	12.9	16.1	9.11	9.6	0.75S	A440	3
54	E41c	⅞	A325	4	10.5	96.3	101.4	0.949	45.3	75.8	84.4	9.62	0.981	13.3	16.1	9.49	9.6	0.75S	A440	3
55	E41e	⅞	A325	4	10.5	97.8	101.4	0.964	45.3	75.8	84.4	9.62	1.11	14.4	16.1	10.7	9.6	0.75*T	A440	3
56	E41f	⅞	A325	4	10.5	90.9	92.4	0.983	45.3	75.8	76.9	9.62	0.996	13.3	14.7	9.58	8.8	0.75*T	A440	3
57	E41g	⅞	A325	4	10.5	95.9	98.6	0.973	45.3	75.8	82.0	9.62	1.00	13.4	15.7	9.66	9.4	0.75*T	A440	3
58	E41	⅞	A325	4	10.5	91.0	92.4	0.985	45.3	75.8	76.9	9.62	1.01	13.5	14.7	9.70	8.8	0.75*T	A440	3
59	E71	⅞	A325	7	21.0	84.9	92.4	0.918	45.3	75.8	76.9	16.8	0.999	20.6	25.7	16.8	15.3	0.75*T	A440	3
60	E101	⅞	A325	10	31.5	80.5	92.4	0.871	45.3	75.8	76.9	24.0	1.00	27.8	36.7	24.0	21.9	0.75*T	A440	3
61	E131	⅞	A325	13	42.0	81.7	95.2	0.859	45.3	75.8	79.2	31.3	0.994	38.5	49.3	31.1	29.4	0.75*T	A440	3
62	E161	⅞	A325	16	52.5	79.5	95.2	0.835	45.3	75.8	79.2	38.5	0.994	45.7	60.6	38.2	36.2	0.75*T	A440	3
63	E46	⅞	A325	4	10.5	90.8	92.4	0.983	45.3	75.8	76.9	28.9	1.01	40.6	44.2	29.2	26.4	0.75*T	A440	3
64	E74	⅞	A325	7	21.0	86.1	92.4	0.931	45.3	75.8	76.9	33.7	0.995	41.0	51.5	33.5	30.8	0.75*T	A440	3
65	E741	⅞	A325	7	21.0	80.4	92.4	0.869	45.3	75.8	76.9	33.7	1.00	41.2	51.5	33.7	30.8	0.75*T	A440	3
66	Rivet	⅞	A502 Gr 1	32	94.0	43.8	54.1	0.809	59.0	88.0	45.0	76.9	0.660	57.5	52.8	50.8	35.4	0.90C	A572	9
67	Bolt	⅞	A325	32	94.0	55.5	91.4	0.607	59.0	88.0	76.0	76.9	0.660	57.5	89.2	50.8	59.8	0.75S	A572	9
68	1	⅞	A325	18	52.0	67.2	105.9	0.635	49.0	79.0	88.1	14.4	0.750	12.8	23.3	10.9	14.5	0.75S	A572	8
69	2	⅞	Huck	12	34.0	81.8	110.0	0.744	49.0	79.0	91.5	10.8	1.00	12.8	18.2	10.9	11.3	0.75S	A572	8
70	3A	⅞	A325	12	33.5	85.8	105.9	0.810	49.0	79.0	88.1	10.8	1.00	12.8	17.5	10.9	10.8	0.75*T	A572	8
71	3B	⅞	Huck	12	33.5	87.0	110.0	0.791	49.0	79.0	91.5	10.8	1.00	12.8	18.2	10.9	11.3	0.75S	A572	8
72	5	⅞	A325	18	52.0	63.4	105.9	0.599	49.0	79.0	88.1	16.8	0.646	12.8	27.2	10.9	16.9	0.75S	A572	8
73	6	⅞	A325	12	33.0	95.8	107.6	0.891	49.0	79.0	89.5	14.4	1.13	20.0	23.7	16.3	14.7	0.75*T	A572	8
74	7	⅞	A325	32	94.0	30.1	47.0	0.640	49.0	79.0	78.2	25.2	0.322	10.0	36.2	8.13	22.5	0.75S	A572	8
75	8	⅞	A325	21	61.0	44.4	47.0	0.944	49.0	79.0	78.2	18.6	0.436	10.0	26.7	8.13	16.6	0.75S	A572	8
76	E721	⅞	A325	7	21.0	76.4	92.6	0.826	45.3	76.0	77.0	16.8	0.805	34.4	25.7	13.6	15.3	0.75*T	A440	11
77	E722	⅞	A325	7	21.0	90.7	92.6	0.980	45.3	76.0	77.0	16.8	1.21	48.1	25.7	20.4	15.3	0.90C	A440	11
78	E163	⅞	A325	16	52.5	68.1	95.0	0.717	45.3	76.0	79.0	38.5	0.805	76.8	60.4	31.0	36.0	0.75*T	A440	11
79	E164	⅞	A325	16	52.5	87.0	95.0	0.917	45.3	76.0	79.0	38.5	1.20	107.4	60.4	46.2	36.0	0.90C	A440	11

Notes:
(1) 0.90 indicates data represented by a circle (C) in Figure 3;
0.75 indicates data represented by a square (S) in Figure 3;
0.75* indicates data represented by a triangle (T) in Figure 3.
(2) L and M refer to lap plates and main member plates in hybrid connections.

Table 2. Test Data for San Francisco–Oakland Bay Bridge Connections											
No.	ID	Dia. (in.)	Type*	Rivets in Line	L (in.)	P_{TEST} (kip)	P_{PRED} (kip)	$\frac{P_{TEST}}{P_{PRED}}$	F_v (ksi)	A_n/A_s	Ref. No.
80	CCC 7-1	1	R(c)	8	6	89.0	89.2	0.998	56.8	1.19	2
81	CCC 7-2	1	R(c)	8	6	90.0	89.2	1.009	56.8	1.19	2
82	DCC 7-1	1	R(c)	14	4	89.1	89.2	0.999	56.8	0.771	2
83	DCC 7-2	1	R(c)	14	4	88.9	89.2	0.997	56.8	0.771	2
84	ACM 12-1	1	R(m)	12	15.75	121.7	119.0	1.023	75.8	1.58	2
85	ACM 12-2	1	R(m)	12	15.75	116.5	119.0	0.979	75.8	1.58	2
86	ASM 12-1	1	R(m)	12	15.75	113.3	119.0	0.952	75.8	1.22	2
87	ASM 12-2	1	R(m)	12	15.75	113.8	119.0	0.956	75.8	1.22	2
88	ACC 18-1	1	R(c)	18	24.50	87.0	89.2	0.975	56.8	1.06	2
89	ACC 18-2	1	R(c)	18	24.50	87.8	89.2	0.984	56.8	1.06	2
90	ACC 36-1	1	R(c)	36	24.50	79.5	89.2	0.891	56.8	0.528	2
91	ACC 36-2	1	R(c)	36	24.50	81.1	89.2	0.909	56.8	0.528	2
92	ACC 54-1	1	R(c)	54	40.75	77.1	89.2	0.864	56.8	0.352	2
93	ACC 54-2	1	R(c)	54	40.75	79.9	89.2	0.896	56.8	0.352	2
94	ASC 18-1	1	R(m)	18	19.5	86.7	119.0	0.728	75.8	0.812	2
95	ASC 18-2	1	R(m)	18	19.5	85.4	119.0	0.718	75.8	0.812	2
96	ASC 36-1	1	R(m)	36	24.5	84.9	119.0	0.714	75.8	0.406	2
97	ASC 36-2	1	R(m)	36	24.5	84.4	119.0	0.710	75.8	0.406	2
98	ASC 54-1	1	R(m)	54	31.25	80.3	119.0	0.675	75.8	0.271	2
99	ASC 54-2	1	R(m)	54	31.25	80.1	119.0	0.673	75.8	0.271	2
100	ACM 24-1	1	R(m)	24	35	119.8	119.0	1.006	75.8	0.791	2
101	ACM 24-2	1	R(m)	24	35	123.6	119.0	1.039	75.8	0.791	2
102	ACM 36-1	1	R(m)	36	54.25	108.2	119.0	0.909	75.8	0.528	2
103	ACM 36-2	1	R(m)	36	54.25	115.8	119.0	0.973	75.8	0.528	2
104	ASM 24-1	1	R(m)	24	35	118.1	119.0	0.992	75.8	0.609	2
105	ASM 24-2	1	R(m)	24	35	126.8	119.0	1.066	75.8	0.609	2
106	ASM 36-1	1	R(m)	36	54.25	115.3	119.0	0.969	75.8	0.406	2
107	ASM 36-2	1	R(m)	36	54.25	112.9	119.0	0.949	75.8	0.406	2
108	ANM 12-1	1	R(m)	12	15.75	118.8	119.0	0.999	75.8	1.053	2
109	ANM 12-2	1	R(m)	12	15.75	115.8	119.0	0.973	75.8	1.053	2
110	ANM 24-1	1	R(m)	24	35	124.8	119.0	1.049	75.8	0.526	2
111	ANM 24-2	1	R(m)	24	35	119.2	119.0	1.001	75.8	0.526	2
112	ANM 36-1	1	R(m)	36	54.25	118.1	119.0	0.992	75.8	0.351	2
113	ANM 36-2	1	R(m)	36	54.25	119.7	119.0	1.006	75.8	0.351	2
114	BCC 20a-1	1	R(c)	22	15	86.6	89.2	0.971	56.8	1.031	2
115	BCC 20a-2	1	R(c)	22	15	86.6	89.2	0.971	56.8	1.031	2
116	BCC 20b-1	1	R(c)	22	22.5	91.2	89.2	1.022	56.8	1.031	2
117	BCC 20b-2	1	R(c)	22	22.5	93.0	89.2	1.043	56.8	1.031	2
118	BCC 20c-1	1	R(c)	22	30	90.6	89.2	1.016	56.8	1.031	2
119	BCC 20c-2	1	R(c)	22	30	89.0	89.2	0.998	56.8	1.031	2

Notes:
* R(c) = Carbon rivet; R(m) = Manganese rivet

did not completely eliminate the bending effect of the lap- and open-shingle splice, the quantity of these tests is small compared to the quantity of butt-splice connection tests, so that their overall effect is very limited. The available data for the 40 tests conducted by Davis et al. (1940) are in Table 2. As expected, there are a few data points that are randomly scattered throughout the plot. Because 22 data points are concentrated at 10.5 in., they have been distributed to 9.5 in., 10.5 in. and 11.5 in. for clarity. The bottom of the vertical scale is also truncated to spread out the data.

Figure 3 shows a plot of the test data, where the test data are identified relative to the connection's strength and quasi-stiffness characteristics (developed later in this paper). A review of Figure 3 indicates that the test results fall into groupings that suggest different design criteria for different connection lengths—as indicated by the earlier AISC and RCSC step function, which is shown in Figure 2. It appears that there is a band of data above the $0.90P_{TEST}/P_{PRED}$ level that extends across the full range of connection lengths. However, there is another group of data that slopes downwards between approximately 15 in. and approximately 40 in. After 40 in., the boundary line is a minimum of approximately $0.60P_{TEST}/P_{PRED}$.

The earlier research identified the connection net section, A_n, as a significant variable. Similarly, the total area, A_s, of all the bolt shear planes was also found significant. A ratio of A_n/A_s in some form was reported in each document. The test results, as clearly reported in the *Guide*, demonstrated that as A_n/A_s increased, the connection performance P_{TEST}/P_{PRED} also improved.

Recently, Moore et al. (2008) reported results that included tests on 1,533 high-strength bolts. The program included ASTM A325, A490, F1852 and F2280 bolts. The latter two are nominally referred to as tension control bolts and are comparable to A325 and A490 bolts, respectively. The program reported on both threads included in the shear plane as well as threads excluded from the shear plane. Tension tests were also performed to calibrate the various lots of bolts. Compared to the earlier Lehigh tests (e.g., Fisher et al., 1963), these bolts were tested in the snug-tight condition and not fully pretensioned. The results indicate that manufactured bolts have reasonably uniform properties as compared to an assembled connection and therefore warrant consideration of a lower target reliability, β.

Based on the data for bolt shear with threads excluded, a β of 4.0 was obtained by Moore et al. (2008) for a live to dead load ratio of 3.0. A resistance factor, ϕ, of 0.85 was obtained for the same condition. Thus, the current AISC/RCSC ϕ of 0.75 appears to be conservative. This observation was considered, along with other factors, when proposed revisions to the AISC and RCSC provisions were developed.

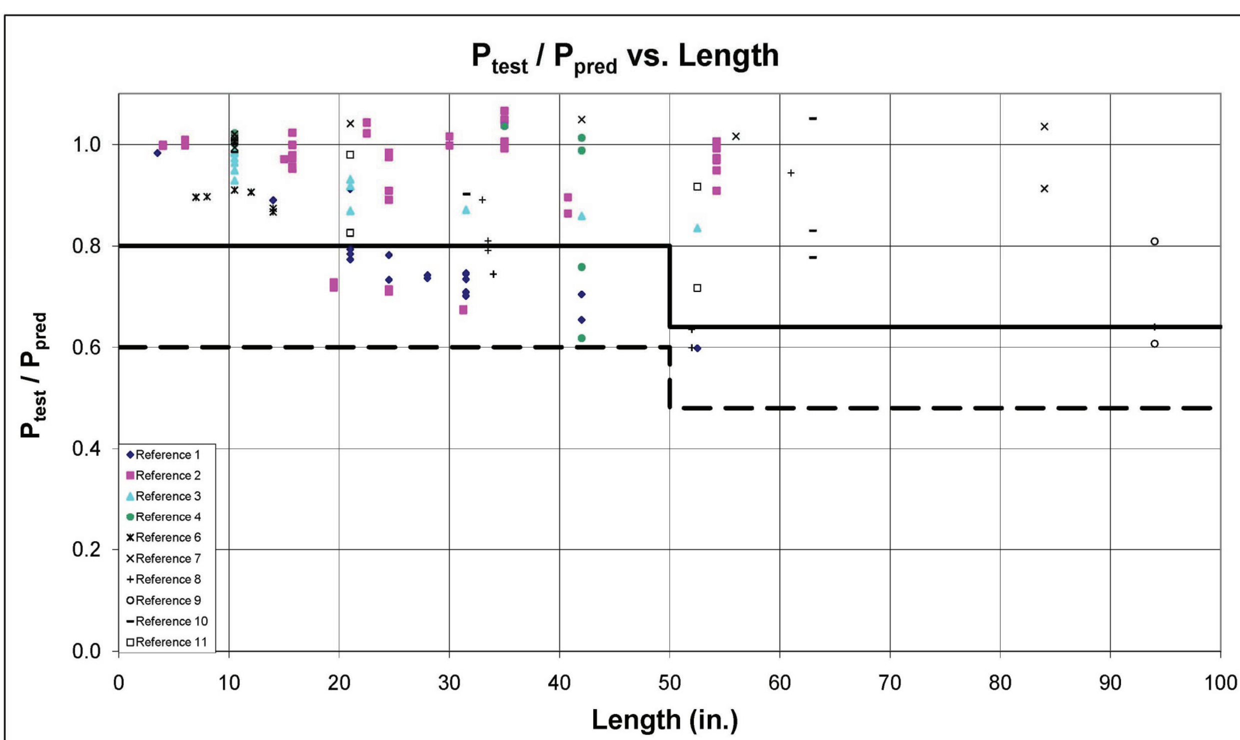

Fig. 2. Results of 119 connection tests with current design criteria superimposed.

DESIGN CRITERIA

The test data that are plotted above the $0.90P_{TEST}/P_{PRED}$ level, in Figure 3, indicate that under certain conditions there is no reduction in connection capacity regardless of the connection length. As previously noted, early research showed that as the ratio A_n/A_s increased, connection capacity also increased. A review of the test data indicated that there was a better correlation when the following ratios were compared to the P_{TEST}/P_{PRED} ratio:

$$\frac{A_n F_{up}}{A_s F_v} = N_1 \tag{3}$$

and

$$\frac{A_g F_{yp}}{A_s F_v} = N_2 \tag{4}$$

where

A_g = gross area of connection material, in.^{2}
A_n = net area of connection material, in.^{2}
A_s = total bolt area in shear plane, in.^{2}
F_{up} = nominal tensile strength of connection material, ksi
F_{yp} = yield stress of connection material, ksi
F_v = ultimate shear strength of the bolt, ksi
N_1, N_2 = target ratios, selected considering test data or specification criteria

Equation 3 represents, in non-dimensional form, the net section of a connection, and Equation 4 represents, in non-dimensional form, the gross section of the connection. Equation 3 can be considered to represent a strength relationship. Similarly, Equation 4 can be considered to represent a quasi-stiffness concept, because as the ratio increases, the plates essentially remain elastic as the ultimate shear strength of the bolt is reached. These are not unfamiliar concepts, because checking the net and gross sections of a connection has been part of AISC specifications for years. The numerical values of N_1 and N_2 must be chosen to satisfy both the test data and Chapter D of the AISC *Specification*.

For design purposes, it is more appropriate to rearrange Equations 3 and 4, substituting nominal values for ultimate values, e.g., replacing F_v with R_1F_u, taking R_1 as 0.625. The procedure is shown in Appendix A, which evaluates the net and gross section requirements of Chapter D of the AISC *Specification*, taking the length reduction factor, R_2, as 0.90. The computed values for N_1 and N_2 are determined to be 0.56 and 0.47, respectively. Coincidentally, the 0.56 value for N_1 is equivalent to a P_{TEST}/P_{PRED} ratio of 0.90 (shown by the horizontal line in Figure 3).

The general forms of the design equations in Appendix A follow as Equations 5 and 6. Because of the uncertainties associated with bolt installation (pretensioned versus snug-tight), second-order effects, and the dictated resistance

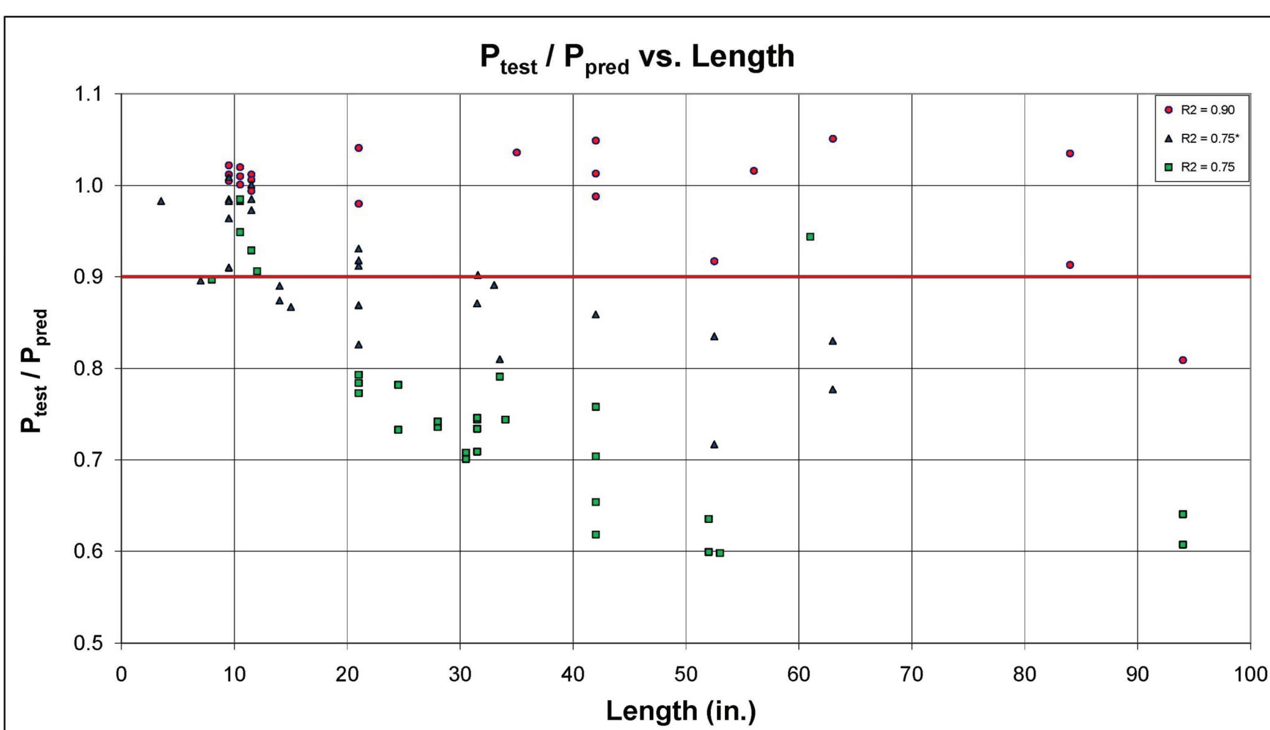

Fig. 3. Test results (79) identified by strength and quasi-stiffness criteria.

factors, N_2 was set equal to N_1 at a common value of 0.56. Solving for the net and gross areas so that the predicted P_{TEST}/P_{PRED} ratio will exceed 0.90 yields:

$$A_n \geq 0.56 A_s F_u / F_{up} \tag{5}$$

and

$$A_g \geq 0.56 A_s F_u / F_{yp} \tag{6}$$

where F_u is the nominal tensile strength of the bolt in ksi. The 0.56 factor results from the product of 0.625 and 0.90 ($R_1 \times R_2$) rounded to two significant figures.

The values for A_n and A_g, as well as the computed values for Equations 5 and 6, are shown in Table 1 for the test data. Because the shear strength of the bolts was established in the research reports, there was no need to convert the bolt tensile strength to the bolt shear strength. When the data were tabulated, three conditions were identified. The first condition was when both net and gross area, A_n and A_g, exceeded the respective inequalities shown in Equations 5 or 6, respectively. These P_{TEST}/P_{PRED} data are shown as circles in Figure 3. The second condition was when only one of the two inequalities was exceeded. Typically it was the net area, A_n, and these are shown with triangles in Figure 3. The third condition was when neither inequality was satisfied, and these are shown as squares in Figure 3. The letters C, T and S are used in to R_2 column and footnote of Table 1 to identify the shape of the data point in Figure 3.

With the exception of the one test at 94 in., all of the test results out of the 21 shown as circles satisfied both inequalities and had P_{TEST}/P_{PRED} ratios greater than 0.90. This one test, No. 66, with a P_{TEST}/P_{PRED} ratio of 0.809, was for an ASTM A502 Grade 1 rivet with ASTM A572 connection plates. The other 20 tests had P_{TEST}/P_{PRED} ratios that varied from 0.913 to 1.051.

The next group of bolts consists of 28 test results where only one inequality was satisfied, shown as triangles. Only 26 test data show up in Figure 3 because in two cases, both at a connection length of 10.5 in., the test results were essentially identical. Twenty-three of these data had connection lengths less than 38 in. and 14 were less than 14 in. For the short connection lengths the test data fall above or very close to the $0.90 P_{TEST}/P_{PRED}$ line.

There are 30 test results shown as a square in Figure 3, where neither inequality was satisfied. Only 29 squares are evident because there is a duplicate at 31.5 in. The one test data at 61 in. for an enclosed shingle connection with a P_{TEST}/P_{PRED} ratio of 0.944 is an anomaly. A review of the original research data did not identify any obvious inconsistency. The square and triangular data indicate that between approximately 21 in. and 42 in. there is a transition in connection behavior depending on the material properties and plate area (A_g, A_n) proportions relative to the total bolt shear area, A_s.

DESIGN EQUATIONS

A practical approach must be chosen to satisfy the needs of design office, detailer and fabrication requirements. For shear connections with lengths less than approximately 15.5 in., a basic reduction factor R_2 of 0.90 is recommended to account for variability in connection behavior. This is an increase from the current basic reduction factor value of 0.80, resulting in a 12.5% increase in bolt capacity from current methods for "short" connections. The resistance factor, ϕ, of 0.75 is still appropriate.

The increase in R_2 is considered appropriate because all of the tests were uni-axial, whereas actual connections typically have a nominal bi-axial contribution. Finite element studies have demonstrated this effect for shear and bending at the end of both simply-supported and fixed-end beams. Similarly, the connections at the ends of diagonals in long span trusses, although designed with pin ends, actually have some bending due to transverse differential displacements at their ends as the truss deflects under load.

For connection lengths greater than 15.5 in. but less than 28.8 in., R_2 could be taken as a function of connection length as follows:

$$R_2 = 1.075 - 0.0113L \tag{7}$$

R_2 is limited to a minimum of 0.75. Beyond approximately 28.8 in. there is a constant strength reduction, R_2, of 0.75. With the application of $\phi = 0.75$, the overall bolt design value is less than all of the test data considered, whether bolt, rivet or Huck connector. Connection lengths greater than 28.8 in. result in a nominal bolt strength increase of 17.2% compared to current practice, because the length reduction factor would increase from 0.64 to 0.75.

With the foregoing design criteria in mind, and observing the distribution of the test data in Figures 2 and 3, a simplified design criterion was chosen. An initial straight line with a constant R_2 of 0.90 extending to 38 in. was chosen. Next is a step function that drops to 0.75. Thereafter, R_2 remains constant at 0.75 for connection lengths greater than 38 in. These design equations are shown graphically in Figure 4, where they are superimposed on the data of Figure 2. In both cases ϕ remains at 0.75. The proposed design criteria are compatible with the theoretical results shown in Figure 5.18 of the *Guide* (Kulak et al., 1987).

The 38-in. length was arbitrarily chosen because it was not a multiple of any of the standard bolt gage spacings and was less than 42 in. The 42-in. length represents the beginning of the lower plateau for test results. Once again the resulting final design criterion is less than the least of the test data. Using the proposed criteria, a revision to the bolt shear portion of AISC *Specification* Table J3.2, is presented in Table 3. Similar revisions to RCSC Table 5.1 for ASTM A325 and ASTM A490 bolts are appropriate.

Table 3. Proposed Bolt Shear Revisions to AISC Table J3.2		
Nominal Shear Stress in Bearing-Type Connection, F_{nv} (ksi)		
Bolt Type	**Less than 38 in.**	**38 in. and greater**
A307	27	23
A325 threads included	54	45
A325 threads excluded	68	56
A490 threads included	68	56
A490 threads excluded	84	70
Threaded rods threads included	$0.45F_u$	$0.375F_u$
Threaded rods threads excluded	$0.563F_u$	$0.469F_u$

The historical tests were performed on fully tightened high-strength bolts with hardened washers using the turn-of-nut method. A high degree of slip resistance (friction) was achieved. The effect of pretensioned bolts is demonstrated by examining Figure 3. The eight test data identified by a circle with connection length greater than 38 in. and above the 0.90 horizontal line indicate that the shear strength of all bolts was reached. In Table 4 the ratio of connection net area divided by Equation 5 is greater than 1.0 with an average value of 1.43 and a standard deviation of 0.24. Similar results for the less critical gross area represented by Equation 6 yield an average value of 1.32 and a standard deviation of 0.26. With a connection frictional component of approximately 30%, these relationships would indicate that all the bolt shear strength would be fully engaged at the connection's ultimate load.

With the use of snug-tight bolts there is effectively no frictional component to the connection capacity. Regardless, the

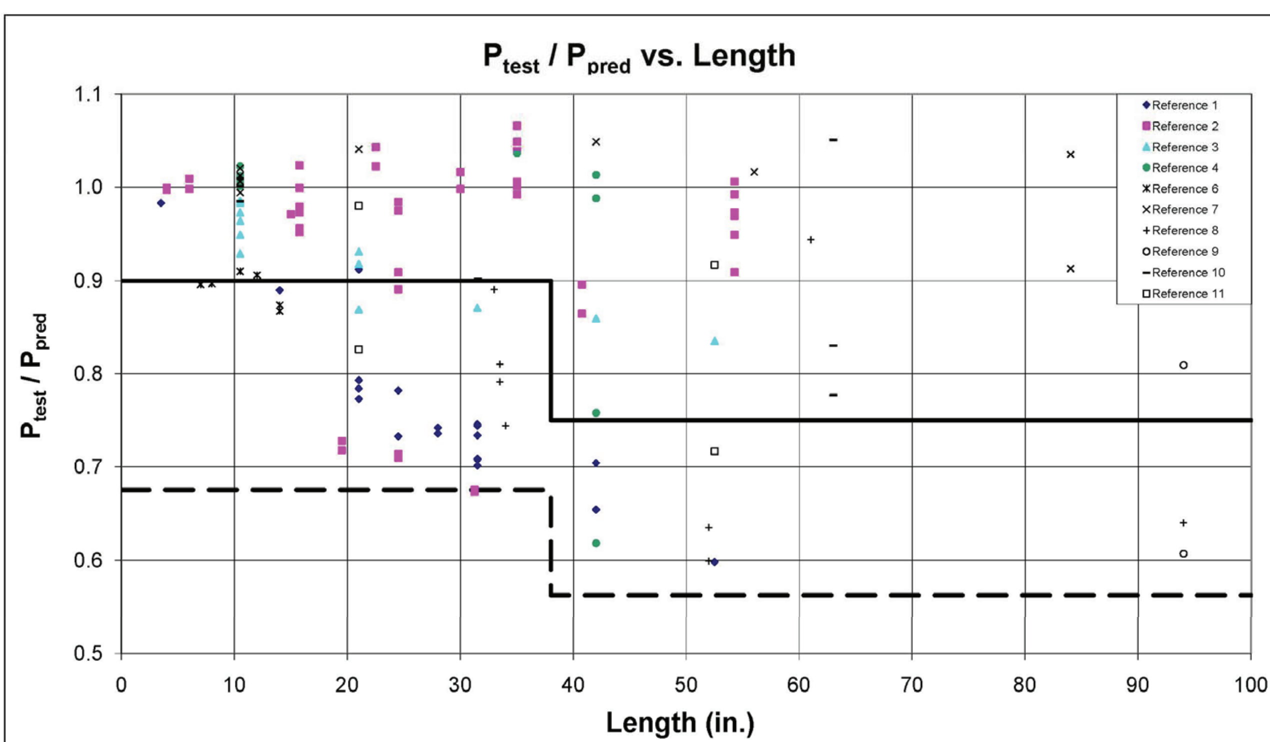

Fig. 4. Proposed design criteria superimposed on 119 connection test results.

Table 4. Ratio of A_g and A_n to Proposed Design Criteria								
Test No.	Length (in.)	$\frac{P_{TEST}}{P_{PRED}}$	A_g (in.)	Eq. 6 (in.^{2})	$\frac{A_g}{\text{Eq. 6}}$	A_n (in.^{2})	Eq. 5 (in.^{2})	$\frac{A_n}{\text{Eq. 5}}$
9	42	1.013	33.6	30.5	1.10	29.8	17.6	1.69
10	42	0.988	29.8	25.7	1.16	26.1	14.8	1.76
16	42	1.049	28.6	19.7	1.45	23.7	18.5	1.28
79	52.5	0.917	107.4	60.4	1.78	46.2	36.0	1.28
17	56	0.989	20.4	16.1	1.27	18.5	15.1	1.23
51	63	1.051	33.8	33.0	1.02	30.0	18.7	1.60
18	84	0.913	28.4	23.7	1.20	24.6	22.2	1.11
19	84	1.035	37.6	23.7	1.59	33.7	22.2	1.52
Average					1.32			1.43
Standard Deviation					0.26			0.24

test results can still be used for snug-tight bolted connections because the frictional component is offset by using a reduced ϕ of 0.75 from 0.85 (Moore et al., 2008), a reduction of 13%; and by limiting the connection length reduction factor, R_2, to 0.90 from 1.0 (the single-bolt connection case), a reduction of 11%. In addition, the coefficient N_2 in Equation 6 was increased from 0.47 to 0.56 (19%), reducing the stress on the plates and more uniformly distributing the force to the bolts. As reported in the literature (Fisher and Kulak, 1968), the bolts were ordered and supplied near the low end of the applicable ASTM standard. In comparison, it is likely that the average production bolt will have a slightly higher ultimate strength. All of these factors would justify not having a length reduction factor less than 0.90 for connections exceeding 38 in. The coefficients could be fine tuned by performing a limited number of tests. However, the proposed step function is conservative.

The proposed design criteria do not require any appreciable difference in design methodology from current methods. The only new item is that the total bolt shear area, A_s, has to be computed. Because A_s reflects the number of bolts in the connection times the shear area (a function of the bolts being in single or double shear, including or excluding the threads), it is a number that the connection designer already has available. The design equation is a modification of Equation 1 as follows:

$$P_n = P_u A_b R_1 R_2 R_3 \tag{8}$$

The value of R_2 is either 0.90 or 0.75 depending on the connection's strength and quasi-stiffness as well as whether the connection has a length greater than 38 in.

Bolt Shear Design Sequence

1. Determine design load, P.
2. Initially assume maximum bolt capacity and $L \le 38$ in. Select ASTM A325 or A490 bolts, bolt diameter, thread condition (included or excluded), and single or double shear to obtain V_n.
3. Determine number of bolts by dividing P by V_n.
4. Calculate A_s, considering thread condition (included or excluded) and single or double shear.
5. Choose a bolt pattern and determine the connection length, L.
6. If $L \le 38$ in., the design is complete for bolt shear. If $L > 38$ in., continue.
7. Compute A_g and A_n.
8. Check Equations 5 and 6 ($L > 38$ in.):
   Equation 5: $A_n \ge 0.56 A_s F_u / F_{up}$
   Equation 6: $A_g \ge 0.56 A_s F_u / F_{yp}$
9. If A_g and A_n criteria are not satisfied, revise bolt capacity for $L > 38$ in. criteria and recompute the number of bolts.
10. Size splice plates to satisfy main member requirements.

Table 5. Reliability (β) and Resistance (ϕ) Values for Current AISC/RCSC Design Criteria*

Connection Length (in.)	R_2	No. of Tests	Mean Value	Std. Dev.	Reliability (β) Live/Dead Load Ratio (L_n/D_n)				Resistance (ϕ) Live/Dead Load Ratio (L_n/D_n)			
					2	3	4	5	2	3	4	5
10.5	0.80	21	0.988	0.0293	5.5	5.1	5.0	4.8	0.933	0.949	0.958	0.963
21.0	0.80	9	0.893	0.0889	4.4	4.2	4.1	4.1	0.807	0.818	0.825	0.829
31.5	0.80	7	0.772	0.0804	3.7	3.6	3.5	3.4	0.731	0.738	0.743	0.746
42.0	0.80	7	0.848	0.176	3.2	3.1	3.1	3.1	0.714	0.718	0.721	0.723
52.3	0.64	7	0.756	0.159	3.5	3.5	3.4	3.4	0.757	0.762	0.766	0.768
62.0	0.64	4	0.901	0.122	5.1	4.9	4.8	4.7	0.897	0.912	0.921	0.928
94.0	0.64	2	0.624	0.0233	4.2	4.0	3.9	3.8	0.781	0.790	0.795	0.798

Notes:
* Rivet tests not included in these values
$\phi_{average} = 0.814$

RELIABILITY

With the bolt shear strength design criteria established, it is now possible to evaluate the results in terms of LRFD concepts. The reliability, β, is determined using the equation:

$$\beta = \frac{\ln\left(\dfrac{\bar{R}}{\bar{Q}}\right)}{\sqrt{V_R^2 + V_Q^2}} \tag{9}$$

and the corresponding resistance factor, ϕ, is determined using the equation:

$$\phi = \left(\frac{R_m}{R_n}\right)e^{-0.55\beta V_R} \tag{10}$$

In Equation 10, ϕ is dependent on knowing β. Similarly, when the step by-step-procedures are followed in Equation 9, ϕ is required to solve for β. This dilemma is resolved by using the current ϕ = 0.75 from the AISC *Specification*. A step-by-step solution for these equations and explanation of terms is given in Appendix B using criteria established by Fisher et al. (1978).

There are two procedures that can be followed to determine β. One approach would be to establish a least-square determination of the P_{TEST}/P_{PRED} relationships relative to the overall connection length and solve for one β for the total database. The second approach would determine β at discrete connection lengths that have adequate test results. The first procedure has the advantage of using more test data in one computation; however, large amounts of data at one length can disproportionately mask other issues. The second procedure was used in this study to try to identify significant variables from the multitude that were identified in the testing programs.

The critical issue was the importance of connection strength and quasi-stiffness as the connection became longer. Once the connection strength and quasi-stiffness exceeded a predetermined amount, length was no longer a variable in the performance of the connection. As a result, the test data was examined for both cases: first, by examining the data from the test results above the $0.90P_{TEST}/P_{PRED}$ ratio, and then by examining the data excluding the test results above the $0.90P_{TEST}/P_{PRED}$ ratio.

The current design criteria were examined for all the applicable bolt test data for nominal live load (L_n) to dead load (D_n) ratios of 2.0 to 5.0. The results for β and ϕ are given in Table 5. Because of the scatter in test data, β is quite variable and ranges from 3.1 to 5.5. It is not surprising that short connections have a high β value because connection length is not really a variable, although a 0.75 reduction factor is mandated. A similar spread in ϕ was also obtained, ranging from 0.714 to 0.963, with a high average value of 0.814. This would suggest that the bolt shear design criteria could be increased.

A review of reliability, β, for the CSA S16 (CSA, 2001, 2005) and Eurocode EN 1933 (CEN, 2003) criteria shown in Table 6 will reinforce the understanding that increasing bolt strength with bolt diameter is not justified. For 1-in.-diameter bolts, the β values drop to approximately 2.0 for a live to dead load ratio of 5.0. When the β values are computed for 1½-in.-diameter bolts, the value drops to 1.9, an unacceptably low value. Although 1½-in.-diameter bolts were not tested, the computed β values are sufficiently accurate because the test data have been converted to a non-dimensional format. The ⅞-in., 1-in. and 1⅛-in. bolt data indicate that the non-dimensional concept appears reasonable. The CSA S16 document uses a ϕ of 0.80.

Table 6. CSA-S16 and Eurocode Reliability (β)*						
Bolt Dia. (in.)	Connection Length (in.)	Live/Dead Load Ratio				ϕ Avg.
		2	3	4	5	
1	10.5	3.8	3.6	3.4	3.4	0.82
	21.0	3.1	2.9	2.9	2.8	0.74
	31.5	2.6	2.5	2.5	2.4	0.70
	42.0	2.6	2.5	2.5	2.5	0.72
	52.3	2.3	2.3	2.3	2.3	0.70
	62.0	3.9	3.7	3.6	3.6	0.85
	94.0	2.9	2.7	2.6	2.6	0.72
1½	10.5	3.8	3.6	3.4	3.4	0.82
	21.0	2.9	2.8	2.7	2.7	0.73
	31.5	2.4	2.3	2.2	2.2	0.67
	42.0	2.3	2.2	2.2	2.2	0.69
	52.3	2.0	2.0	2.0	1.9	0.66
	62.0	3.3	3.2	3.1	3.1	0.78
	94.0	2.8	2.6	2.6	2.5	0.71

* ϕ = 0.80 in CSA-S16

Table 7. Reliability (β) and Resistance (ϕ) Values for Proposed AISC/RCSC Design Criteria*												
Connection Length (in.)	R_2	No. of Tests	Mean Value	Std. Dev.	Reliability (β)				Resistance (ϕ)			
					Live/Dead Load Ratio (L_n/D_n)				Live/Dead Load Ratio (L_n/D_n)			
					2	3	4	5	2	3	4	5
10.5	0.90	21	0.988	0.0293	4.8	4.6	4.4	4.3	0.856	0.868	0.875	0.879
21.0	0.90	9	0.893	0.0889	3.9	3.7	3.6	3.6	0.747	0.755	0.760	0.763
31.5	0.90	7	0.772	0.0804	3.2	3.1	3.0	3.0	0.677	0.682	0.685	0.687
42.0	0.75	7	0.848	0.176	3.4	3.4	3.3	3.3	0.740	0.745	0.748	0.751
52.3	0.75	7	0.756	0.159	3.0	2.9	2.9	2.9	0.693	0.696	0.698	0.699
62.0	0.75	4	0.901	0.122	4.4	4.2	4.1	4.1	0.813	0.824	0.831	0.835
94.0	0.75	2	0.624	0.0233	3.4	3.2	3.1	3.1	0.696	0.701	0.704	0.707

Notes:
* Rivet tests not included in these values
$\phi_{average}$ = 0.754

The proposed AISC/RCSC design criteria, all the reported bolt test data, and the computed β and ϕ values are given in Table 7. The range in β values has been reduced to 2.9 to 4.8. The test data ratios of P_{TEST}/P_{PRED} that are still above 0.90 result in a large coefficient of variation resulting in the low values for the 52.3 in. connection length. As previously mentioned, high-strength bolts are a manufactured product, which suggests that a β of approximately 3.0 would be acceptable. The resistance factor, ϕ, has a similar variation in value because of the test results. The magnitude of the values is centered, average 0.754, on the starting value of 0.75. This indicates that appropriate adjustments have been made to the current design criteria.

The final set of computations included only the test data that exhibited a change in performance with connection length. The test data with a P_{TEST}/P_{PRED} ratio above 0.90 were excluded. These β and ϕ results are shown in Table 8. Once the high P_{TEST}/P_{PRED} data are removed from the calculations, the

Table 8. Reliability (β) and Resistance (ϕ) Values with Limited P_{TEST}/P_{PRED} Data (< 0.90)*												
Connection Length (in.)	R_2	No. of Tests	Mean Value	Std. Dev.	Reliability (β) Live/Dead Load Ratio (L_n/D_n)				Resistance (ϕ) Live/Dead Load Ratio (L_n/D_n)			
					2	3	4	5	2	3	4	5
10.5	0.90	0	—	—	—	—	—	—	—	—	—	—
21.0	0.90	4	0.813	0.0438	3.7	3.6	3.5	3.4	0.731	0.738	0.742	0.745
31.5	0.90	6	0.751	0.0618	3.2	3.1	3.0	2.9	0.676	0.681	0.684	0.686
42.0	0.75	4	0.722	0.108	3.3	3.2	3.2	3.1	0.701	0.706	0.710	0.712
52.3	0.75	5	0.677	0.101	3.0	3.0	2.9	2.9	0.675	0.679	0.682	0.684
62.0	0.75	2	0.804	0.0385	4.6	4.4	4.3	4.2	0.830	0.841	0.848	0.852
94.0	0.75	2	0.624	0.0233	3.4	3.2	3.1	3.1	0.696	0.701	0.704	0.707

Notes:
* Rivet tests not included in these values
$\phi_{average} = 0.726$

β values ranged from 2.9 to 4.6. The β of 2.9 does not change because at 52.3 in. there was no data above 0.90. As previously explained, for a manufactured product, a β of 2.9 is acceptably close to the target value of 3.0. This change is reflected in the resistance factor, ϕ, that on average (0.726) is below the starting value of 0.75. The difference is not significant.

SUMMARY AND CONCLUSIONS

A review of the historic research test data was made to determine bolt shear strength in terms of LRFD principles. A total of 119 connection tests were identified. Of these, 40 tests were with rivets associated with the design and construction of the San Francisco–Oakland Bay Bridge. Unfortunately, insufficient information was reported to allow full utilization of the test data. Of the remaining 79 connection tests, the connector distribution was 54 with ASTM A325 bolts, 18 with ASTM A490 bolts, 5 with rivets, and 2 with Huck bolts. The statistical analysis was performed using the ASTM A325 and A490 bolts. Subsequently, it was possible to show that the rivet and Huck bolt test data were compatible with the recommended design criteria.

Because of the many connection variables, the test data were reduced to a non-dimensional form to limit the significance of all the variables. As a result, the connection length remained as the desired and predominate independent variable. Recent tests sponsored by RCSC also indicated that the reliability, β, of the shear strength of bolts was similar to plates and shapes reported in earlier literature. Based on other anecdotal information there does not appear to be any justification to change the current resistance factor, ϕ.

In addition to the AISC/RCSC design criteria, the equivalent Canadian CSA and Eurocode provisions were examined. The CSA S16 provision was identical to and transferred from the Eurocode document. The two key issues in these provisions are variable and decreasing bolt shear strength with increasing connection length and increasing bolt shear strength with increasing diameter. The reviewed test data indicate that the first issue is justified, although the benefit gained by having a sliding scale is probably not justified relative to the complexity. The second issue is the increasing bolt shear strength with increasing diameter, which is not justified by the test data and at large bolt diameters results in unacceptably low reliability, β.

The current LRFD principles have a target reliability, β, of approximately 4.0 for connections, which include slip-critical connections and bolt-bearing connections. In comparison, the target β for main members—a manufactured product—typically have β of approximately 3.0, or slightly lower. Because the bolt itself is a manufactured product, there is some leeway as to what β is acceptable for bolts. As a practical consideration, it is reasonable to use a common resistance factor, ϕ, value of 0.75 for slip critical connections, bolt bearing connections, and for this study of bolt shear strength.

The current AISC/RCSC design criteria result in variable β from 3.1, and in some cases, to a conservatively high value of 5.5. In comparison, the proposed design criteria β range from 2.9 to 4.8. When the P_{TEST}/P_{PRED} test data above 0.90 are excluded, the range for β becomes 2.9 to 4.6. The short connection values for β are going to be high because the test results are for axially loaded specimens and do not include the secondary forces associated with biaxial beam end reactions or adjacent truss-panel-point relative displacement. The effect of pretensioned bolts versus snug tight bolts has not been directly evaluated.

The proposed criteria have a step function at 38 in. to permit an initial length reduction factor, R_2, increase from 0.80 to 0.90. This represents a 12.5% increase in bolt shear strength. Beyond 38 in., the length reduction factor is increased from 0.64 to 0.75, a 17.2% increase. As a result, the proposed design procedure is identical to the current requirements except with an increased bolt shear design value and a step function at 38 in. instead of 50 in.

An unexpected result of the study was the realization that under circumstances of sufficient connection strength, represented by the net area, A_n, and in conjunction with sufficient connection quasi-stiffness, represented by the connection gross area, A_g, in comparison to the total bolt shear area, A_s, there would be no need for a connection strength reduction less than 0.90 with increasing length. This condition exists when the inequalities expressed in Equations 5 and 6 are satisfied. Equation 6 is not exactly a stiffness criterion, but it indicates that the connection plates remain essentially elastic as the bolt ultimate shear strength is reached.

Because the historical tests were performed on fully tightened high strength bolts, the use of Equations 5 and 6 when snug-tight bolts are used has not been experimentally confirmed. However, the strength component attributed to friction has been offset by the reduced ϕ of 0.75 (13%), limiting the connection capacity ratio to 0.90 (11%) and increasing the gross area coefficient requirement in Equation 6 from 0.47 to 0.56 (19%). In addition, the statistical bolt strength will be somewhat higher than the research programs intentional use of low end bolt strength. Performing a few tests would quantify and refine the N_1 and N_2 coefficients.

The statistical study was based on ASTM A325 and A490 bolts; however, when the limited rivet and Huck bolt data were compared with the bolt results, no inconsistency was found. Similarly, the connection plate material varied from relatively low-strength ASTM A7 steel to high-strength ASTM A514 steel with intermediate-strength ASTM A440 and A572 steel in between, again with no inconsistencies. This would indicate that the data in a non-dimensional format did not have any apparent bias and indicates that the procedure is acceptable for all current grades of connectors, plates and shapes. In conclusion, the proposed design criteria are essentially identical to the current provisions except the bolt strengths are adjusted slightly upwards, resulting in a more uniform reliability, β, closer to the professionally accepted values. In addition, Equations 5 and 6 provide a means of proportioning a connection to gain optimum bolt shear strength.

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APPENDIX A

AISC SPECIFICATION SECTION D2

Plate Yielding

Use Equation D2-1 from the 2005 AISC *Specification*:

$$P_n = F_y A_g$$

The design tensile strength of the plate is $\phi_t P_n$ and $\phi_t = 0.90$. Thus,

$$\phi_t P_n = \phi_t F_y A_g$$

For bolts, the design shear strength is $\phi_b P_n$, with $\phi_b = 0.75$. Also, $P_n = P_u R_1 R_2$, assuming threads excluded from the shear plane. We also know that $P_u = F_u A_s$. Substituting, we have:

$$\phi_b P_n = \phi_b (P_u R_1 R_2) = \phi_b (F_u A_s) R_1 R_2$$

R_1 has been established at 0.625. Take R_2 as 0.90 by assuming $L < 38$ in., avoiding reducing R_2 from 0.90 to 0.75 for $L > 38$ in.

Equating the bolt shear strength to the plate yield strength (with F_{yp} the yield stress of the plate):

$$\phi_b (F_u A_s) R_1 R_2 = \phi_t F_{yp} A_g$$

Solving for A_g,

$$A_g = \frac{\phi_b F_u A_s R_1 R_2}{\phi_t F_{yp}} = \frac{0.75(F_u A_s)(0.625)(0.90)}{0.90 F_{yp}} = \frac{0.469 F_u A_s}{F_{yp}}$$

Bolt shear will control as long as:

$$A_g \geq \frac{0.469 F_u A_s}{F_{yp}}$$

Plate Fracture

Use Equation D2-2 from the 2005 AISC *Specification*:

$$P_n = F_u A_n$$

The design rupture strength of the plate is $\phi_t P_n$ and $\phi_t = 0.75$. Thus,

$$\phi_t P_n = \phi_t F_u A_n$$

Equating the bolt shear strength to the plate rupture strength (with F_{up} the rupture stress of the plate):

$$\phi_b (F_u A_s) R_1 R_2 = \phi_t F_{up} A_n$$

Solving for A_n and substituting as before, we obtain:

$$A_n \geq \frac{0.563 F_u A_s}{F_{up}}$$

Bolt shear will control over tensile rupture as long as this inequality is satisfied.

Notes:

1. A_s and F_u are bolt properties.

2. For design purposes, use a coefficient of 0.56 for both calculations (i.e., for N_1 and N_2) until further research quantifies pretensioning and second-order effects.

APPENDIX B

SAMPLE β CALCULATION FOR BOLT SHEAR

$$\beta = \frac{\ln\left(\dfrac{\bar{R}}{\bar{Q}}\right)}{\sqrt{V_R^2 + V_Q^2}}$$

where
- $\bar{R}$ = mean resistance
- $\bar{Q}$ = mean load effect
- V_R = coefficient of variation
- V_Q = coefficient of variation

$$\bar{R} = f\{\bar{R}_1, F_{UN}, R_P, R_M, R_F\}$$

where
- F_{UN} = nominal tensile strength of bolt
- $\bar{R}_1$ = ratio of shear to tensile strength = 0.625, $V_{\bar{R}_1} = 0.05$
- R_P = mean test value, V_{RP} = ratio of standard deviation to mean test value
- R_M = fabricating factor = 1.20, $V_{RM} = 0.07$
- R_F = fabricating factor = 1.00, $V_{RF} = 0.02$

These data are from Fisher et al. (1978).

EXAMPLE

L = 42 in., 7 tests
$R_P = 0.848$, standard deviation = 0.1761

1. Mean Resistance

$\bar{R} = \bar{R}_1 R_P R_M R_F F_{UN}$
$\bar{R} = (0.625)(0.848)(1.20)(1.00)F_{UN} = 0.6360F_{UN}$
$V_{RP} = 0.1761/0.848 = 0.2077$

$$V_R = \sqrt{V_{\bar{R}_1}^2 + V_{R_P}^2 + V_{R_M}^2 + V_{R_F}^2}$$

$$V_R = \sqrt{0.05^2 + 0.2077^2 + 0.07^2 + 0.02^2} = 0.2257$$

2. Mean Load Effect

D_N, L_N = nominal dead and live loads, respectively
$\bar{D} = 1.05D_N$, $V_D = 0.10$
$\bar{L} = L_N$, $V_L = 0.25$

For this example, assume $L_N/D_N = 3.0$
Q_N = nominal load effect
$= 1.2D_N + 1.6L_N = D_N[1.2 + 1.6(L_N/D_N)] = 6.0D_N$
$\bar{Q}$ = mean load effect
$= \bar{D} + \bar{L}$
$= 1.05D_N + L_N = 1.05D_N + 3.0D_N = 4.05D_N$

$$V_Q = \frac{\sqrt{(\bar{D}V_D)^2 + (\bar{L}V_L)^2}}{Q}$$
$$= \frac{\sqrt{\left[(1.05D_N)(0.10)\right]^2 + \left[(3.00D_N)(0.25)\right]^2}}{4.05D_N}$$
$$= 0.1870$$

3. Nominal Design Strength

In non-dimensional form, $R_N = \bar{R}_1 R_2 F_{UN}$, where R_2 is a design level criteria. The connection length is 42 in., which is greater than 38 in. Therefore, R_2 is taken as 0.75.

$R_N = (0.625)(0.75)F_{UN}$
$= 0.4688F_{UN}$

Set ϕR_N equal to Q_N, with $\phi = 0.75$, and solve for D_N:

$0.75(0.4688F_{UN}) = 6.0D_N$
$D_N = 0.0586F_{UN}$

Now, calculate $\bar{Q}$:

$\bar{Q} = 4.05D_N$
$= 4.05(0.0586F_{UN})$
$= 0.2373F_{UN}$

The reliability, β, may now be calculated:

$$\beta = \frac{\ln\left(\dfrac{0.6360F_{UN}}{0.2373F_{UN}}\right)}{\sqrt{(0.2257)^2 + (0.1870)^2}}$$
$$= \frac{0.9859}{0.2931}$$
$$= 3.36$$

4. Resistance Factor, ϕ

$$\phi = \left(\frac{R_M}{R_N}\right)e^{-0.55\beta V_R}$$

R_M = mean test value (R_P)from β calculations
R_N = proposed design criteria, R_2
β = from previous calculations (Step 3)
V_R = coefficent of variation

$$\phi = \left(\frac{0.848}{0.75}\right)e^{-\left[(0.55)(3.36)(0.2257)\right]}$$
$$= (1.1307)e^{-0.417}$$
$$= (1.1307)(0.659)$$
$$= 0.745$$