Design Aid: Anchor Bolt Interaction of Shear and Tension Loads

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A common practice among structural engineers involved in industrial building design is to specify either ASTM A36 threaded rod bolts or A307 headed bolts for use as cast-in-place anchors to carry combined shear and tension forces to the foundation. It is also fairly common to specify ASTM A325 or equivalent strength material for anchor bolts which must carry higher forces than can be accommodated by ordinary carbon steel bolts.

Several articles have been written in the past ten years which have addressed the problem of anchor bolt design for combined loadings. By consolidating and summarizing the available data, the problem can be simplified for most situations encountered in normal practice.

A conservative design approach is warranted, as suggested by Marsh and Burdette[1] since test data is limited and consequences of bolt failures are quite unacceptable for steel structures which must carry expensive industrial equipment.

DESIGN AID DEVELOPMENT

Shipp and Haninger (1983)[2] proposed that interaction curves based on working stress design (WSD) allowables be generated for combined shear/tension load cases. Since AISC Allowable Stress Design (ASD) is familiar to most practicing engineers and has been used in the past to define anchor bolt strengths, the author felt that a design aid should be developed utilizing ASD allowables. A straight-line relationship between shear and tension was used as recommended by Marsh and Burdette[1] and also Shipp and Haninger.[2] This is conservative when compared to the AISC[5] equations for bolt interaction as illustrated in Figs. 3 and 4.

The *C*-factor, a shear coefficient which accounts for the effect of various types of shear failure planes was used in Ref. 2 and originally in ACI 349 Appendix B.[3] The *C*-Factor has been defined in Ref. 2 as the inverse of the friction coefficient (μ) of ACI 349 Appendix B:

Therefore:

$$C = 1 / 0.9 = 1.11$$

steel plate embedded with top surface flush with concrete surface

$$C = 1 / 0.7 = 1.43$$

steel plate against concrete or grout surface (plate exposed)

$$C = 1 / 0.55 = 1.82$$

steel plate on grout pad on top of concrete surface

The authors of Ref. 1 recommended a numerically equivalent value (ϕ) be used, which is unrelated to friction and corresponds to ACI 349 Appendix B as follows:

$$\begin{aligned} \phi &= \text{ACI 349 } \phi\text{-factor} \times \mu \text{ (friction coeff.)} \\ &= 0.85 \times 0.9 = 0.765 \\ &= 0.85 \times 0.7 = 0.595 \\ &= 0.85 \times 0.55 = 0.468 \end{aligned}$$

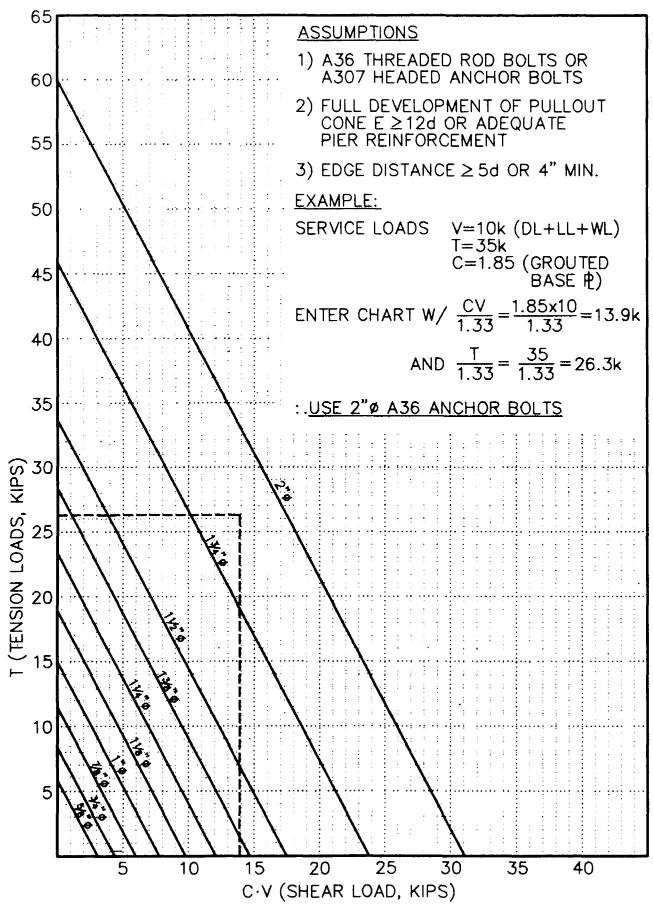

Fig. 1. Anchor bolt design—carbon steel bolts.

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The desired *C*-factor can be applied similarly to a load factor in ultimate strength design and the required bolt size read directly from Figs. 1 and 2.

Based on the yield strength of the material, the safety factor for tension in A36 bolts is $F_y = 36$ ksi / $F_t = 19.1$ ksi = 1.89 in ASD.

The safety factor for shear is $F_y = 36$ ksi / $F_v = 9.9$ ksi = 3.64 for A36 threaded rod. The safety factor proposed by Shipp and Haninger[2] is approximately equal to $F_y = 36$ ksi / 0.55×36 ksi = 1.82. If tensile stress area is used to calculate allowable forces, the effective safety factor is increased slightly to approximately 1.82 / 0.75 = 2.42 with 0.75 being the approximate ratio of tensile stress area to gross (nominal) area per AISC.[5]

The tensile stress area was used in earlier versions of the AISC Specification[6] and ACI Appendix B[3] to determine allowable tension load. However, the AISC Ninth Edition[5] uses the nominal gross area to compute allowable loads in A36 threaded rod and A307 headed bolts. Table 1 was therefore produced from the Ninth Edition values for allowable shear and tension to define the limits of the straight-line charts.

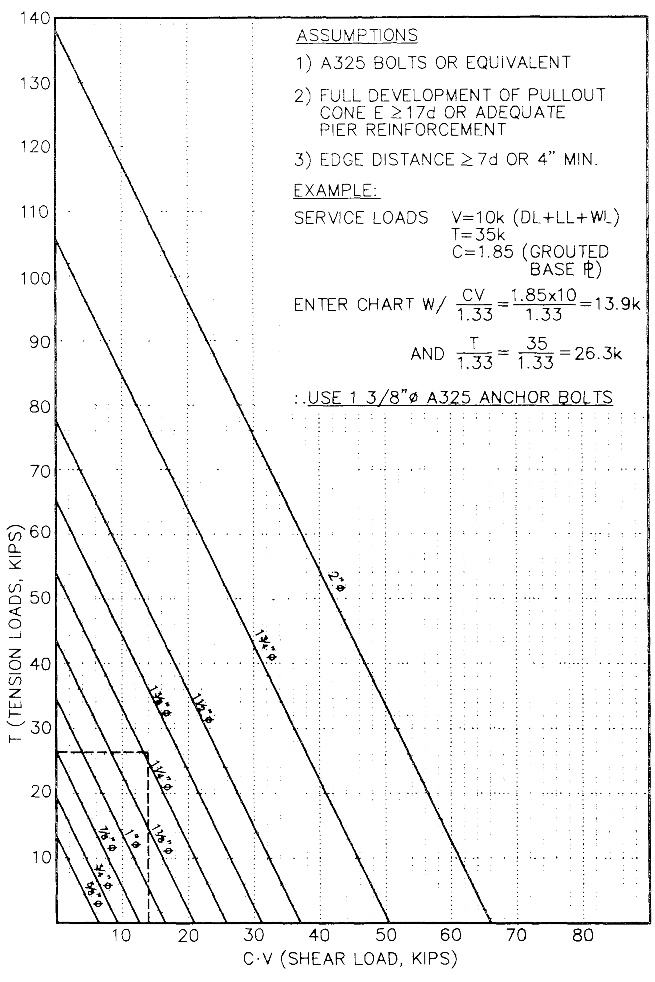

Fig. 2. Anchor bolt design—high strength bolts.

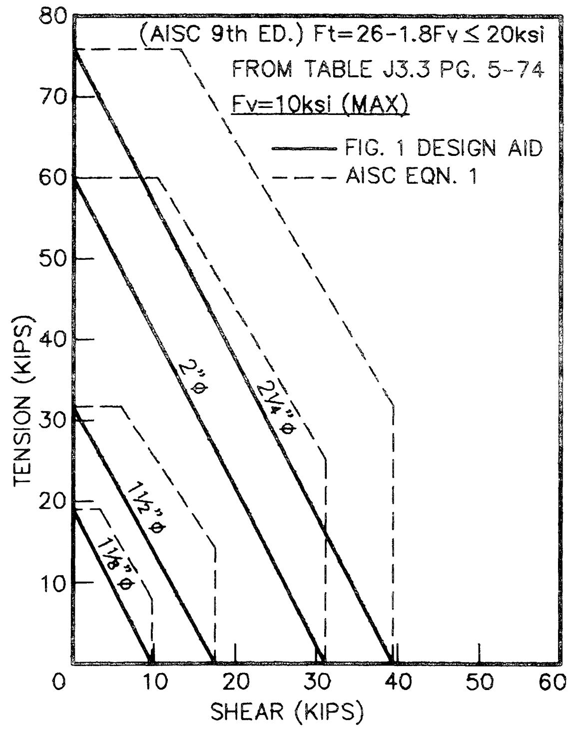

Fig. 3. Comparison of straight-line interaction curve to AISC equation for A36/A307 bolts.

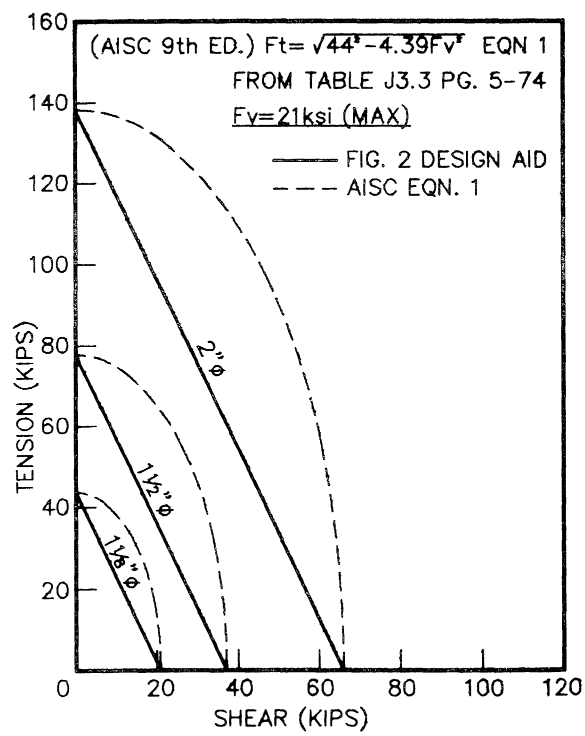

Fig. 4. Comparison of straight-line interaction curve to AISC equation for A325 bolts.

USE OF THE DESIGN AID

In most practical building design work, anchor bolts are grouped in either two-bolt or four-bolt (square or rectangular) patterns.

These bolts are confined by pier reinforcement such as vertical dowels and hoop ties which serve to transfer the tensile stresses and distribute the shear forces from the bolts to the concrete footing. Therefore, for most practical designs—if proper embedment length is provided—a minimum of 12*d* for A36 / A307 bolts or 17*d* for A325 bolts,[2,8] and the bolts are fairly close in plan to the vertical reinforcement (e.g., within three-bolt diameters), the tensile force is transferred to the vertical reinforcement and pullout is prevented.

A minimum edge distance is also practical, since it is difficult to place dowels, ties, and bolts in a pier and maintain ACI minimum cover requirements without an edge distance of at least 5*d* or four inches minimum for smaller bolts (¾-in. diameter or less). Therefore, this requirement from Ref. 2 was retained for good performance of the bolts against lateral bursting failure.

Shear failure of the concrete is prevented by the hoop tie reinforcement if the top tie is placed within about two to three inches of the pier top and additional ties are closely spaced to the top tie.

For the rare cases encountered in practice where unreinforced concrete is used or a mat of bars is below the anchor bolts, the performance of the bolts is dependent solely on the concrete in tension and the effective pullout area can be calculated according to Ref. 3. The effect of shear against a free edge can also be evaluated according to Ref. 3. The effect of overlapping pullout cones can be evaluated using Ref. 4.

It is felt that use of the design charts presented in Figs. 1 and 2 will result in conservative designs for most of the situations encountered in general building design work. Although not a requirement, a minimum compressive strength of 3,000 psi is generally assumed for the use of the charts.

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APPENDIX 1
COMPARISON OF DESIGN METHODS

Example:

Service loads:

$T = 15$ kips (ASTM A36 anchor bolts)

$V = 4$ kips (live + wind)

Per ACI 349-85 Appendix B; USD (Ultimate Strength) Approach:

Maximum steel stresses:

Tension $\phi f_y = 0.9 \times (36) = 32.4$ ksi
or $0.8 f_{ut} = 0.8 \times (58) = 46.4$ ksi
Shear $\phi f_y = 0.85 \times (36) = 30.6$ ksi

Factored loads:

$T_u = 0.75 \times 1.7 \times 15 = 19.13$ kips
$V_u = 0.75 \times 1.7 \times 4 = 5.1$ kips

Area required:

$T_u / U_t + V_u / \mu \times U_v$

where
$\mu = 0.55$ (grouted base)
$= (19.13 / 32.4) + (5.1 / 0.55 \times 30.6) = 0.893$ in.^{2}

Use 1¼-in. diameter A36 bolt

$A_t = 0.969$ in.^{2}

Per Fig. 1: ASD Approach:

Enter chart with

$C \times V / 1.33 = 1.85 \times 4 / 1.33 = 5.56$ kips
and $T / 1.33 = 15.0 / 1.33 = 11.28$ kips

Read 1¼-in. diameter bolt required.

REFERENCES

1. Marsh, M. Lee and Edwin G. Burdette, "Anchorage of Steel Building Components to Concrete," AISC *Engineering Journal,* First Quarter, 1985.
2. Chipp, John G. and Edward R. Haninger, "Design of Headed Anchor Bolts," AISC *Engineering Journal,* Second Quarter, 1983.
3. "Appendix B—Steel Embedments," *ACI 349-85, Code Requirements for Nuclear Safety Related Structures.*
4. Marsh, M. Lee and Edwin G. Burdette, "Multiple Bolt Anchorages: Method for Determining the Effective Projected Area of Overlapping Stress Cones," AISC *Engineering Journal,* First Quarter, 1985.
5. AISC, *Manual of Steel Construction,* Ninth Edition, 1989.
6. AISC *Manual of Steel Construction,* Seventh Edition, 1970.
7. Cannon, R. W., D. A. Godfrey and F. L. Moreadith, "Guide to the Design of Anchor Bolts and Other Steel Embedments," *Concrete International,* July 1981.
8. DeWolf, John T., *Column Base Plates,* AISC Design Guide Series, No. 1, September 1991.

Table 1.
ASD Allowable Loads per AISC[5]

A) Tension, kips

Material	F_y (ksi)	F_t (ksi)	Area (based on nominal diameter) in.^{2}											
			0.196	.3068	.4418	.6013	.7854	.9940	1.227	1.485	1.767	2.405	3.142	3.976
			Bolt diamter (in.)											
			½	⅝	¾	⅞	1	1⅛	1¼	1⅜	1½	1¾	2	2¼
A307	—	20.0	3.9	6.1	8.8	12.0	15.7	19.9	24.5	29.7	35.3	48.1	62.8	79.5
A36	36	19.1	3.8	5.9	8.4	11.5	15.0	19.0	23.4	28.4	33.7	45.9	60.0	75.9
A325*	92 81	44.0	8.6	13.5	19.4	26.5	34.6	43.7	54.0	65.3	77.7	105.8	138.2	174.9

B) Shear, kips

Material	F_y (ksi)	F_v (ksi)	Area (based on nominal diameter) in.^{2}											
			0.196	.3068	.4418	.6013	.7854	.9940	1.227	1.485	1.767	2.405	3.142	3.976
			Bolt diamter (in.)											
			½	⅝	¾	⅞	1	1⅛	1¼	1⅜	1½	1¾	2	2¼
A307	—	10.0	2.0	3.1	4.4	6.0	7.9	9.9	12.3	14.8	17.7	24.1	31.4	39.8
A36	36	9.9	1.9	3.0	4.4	6.0	7.8	9.8	12.1	14.7	17.5	23.8	31.1	39.4
A325*	92 81	21.0	4.1	6.4	9.3	12.6	16.5	20.9	25.8	31.2	37.1	50.5	66.0	83.5

*A325 spec. includes bolt diameters from ½-in. to 1½-in. for bolt diameter geater than 1½-in., equivalent strength material is available.