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High Strength Bolted Connections with Applications to Plastic Design

Assemblages par boulons à haute résistance et applications au calcul en plasticité

HV-Verbindungen und ihre Anwendung für plastische Bemessung

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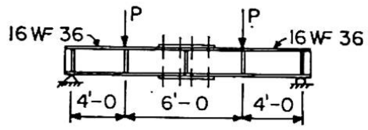
The present status of high strength bolted moment connections in plastic design is summarized in the American Society of Civil Engineers' Commentary on Plastic Design in Steel: "Although accurate procedures leading to the most economical safe design are not yet available, the results of research show that safe bolted joints can be designed to develop the plastic moment of the members with reasonable economy" [1]. Supporting this statement is the research of Johnson, Cannon and Spooner in England and Schutz in America. With these pilot studies lending background and some assurance of success, the authors undertook the investigation described herein. This project, conducted at Cornell University, covered the common types of T-stub and end plate moment connections. The study of end plate connections will not be reported here because of lack of space and because it is treated elsewhere [2]. Most of the questions to be discussed are of interest in working stress design as well as in plastic design, but attention is focused here on matters which have particular relevance to plastically designed beams and frames.

T-stub Web-to-Beam Flange Connections

To simulate the connection between T-stub webs and beam flanges, eight spliced beams were tested (Table 1). All of the beams and splices were ASTM A 7 steel, having a specified minimum yield point of 33,000 psi. Bolts were ASTM A 325, the high strength bolt commonly used in America. Bolts were designed for nominal shear stresses on the full area of 15,000, 22,000, or 30,000 psi at the working load on the beam. They were tightened to the specified proof load (approximately the yield strength). Lateral bracing on the beam was closely spaced, giving an effective slenderness ratio in the weak direction (l/r_y) of 25.

Typical non-dimensionalized load-deflection diagrams are shown in Fig. 1. The load causing a bending moment equal to the theoretical plastic resisting moment of the gross section (the full plastic moment) is plotted for reference. Also shown are the load required for full plastification of the net section (about 25% less than the other) and the conventional working load (P_w) causing a

Table 1. Summary of Beam Tests



Test	Description					Test Results	
	Splice Plates	Total Bolts (no.-diam.)	σ_y (ksi)	Z (in^3)	M_p (in-kips)	M_s(1) (in-kips)	M_{ult} (in-kips)
B 4 A	None-plain beam		40.3	65.5	2,640	—	2,690
B 8.1	None - $^{15}/_{16}''$ holes		40.0	65.9	2,640	—	2,690
B 9.1	$7'' \times {}^{11}/_{16}''$	48 $^3/_4$	38.5	66.4	2,560	None	2,760
B 10.1	$7'' \times {}^{11}/_{16}''$	32 $^3/_4$	38.5	66.4	2,560	1,730	2,590
B 10.2	$7'' \times {}^{11}/_{16}''$	32 $^3/_4$	38.5	66.9	2,580	2,130	2,690
B 10.3	$7'' \times {}^{11}/_{16}''$	32 $^3/_4$	39.6	66.9	2,650	2,190	2,690
B 11.1	$7'' \times {}^{11}/_{16}''$	24 $^3/_4$	40.0	65.9	2,640	1,720	2,710
B 13.1	$7'' \times {}^{3}/_{4}''$	24 $^7/_8$	38.2	65.8	2,510	1,770	2,670
B 13.2	$7'' \times {}^{3}/_{4}''$	24 $^7/_8$	38.2	65.7	2,510	1,960	2,730
B 13.3	$7'' \times {}^{3}/_{4}''$	24 $^7/_8$	39.6	65.7	2,640	2,090	2,800

(1) Bending moment at first major slip

maximum stress on the gross section of 0.6 times the yield point. The test of B 4 A, a plain unspliced beam, followed the upper theoretical curve quite closely until failure. In all beams tested, the full plastic moment was obtained. In most, bolt slip started at loads from 30 to 50% greater than the working load and progressed erratically. B 9.1, having bolts designed for a nominal working shear stress of 15,000 psi, experienced no major slip throughout test. Typically, ultimate failure occurred by gradual inelastic lateral bowing accompanied by progressive local flange buckling on the concave side of a lateral

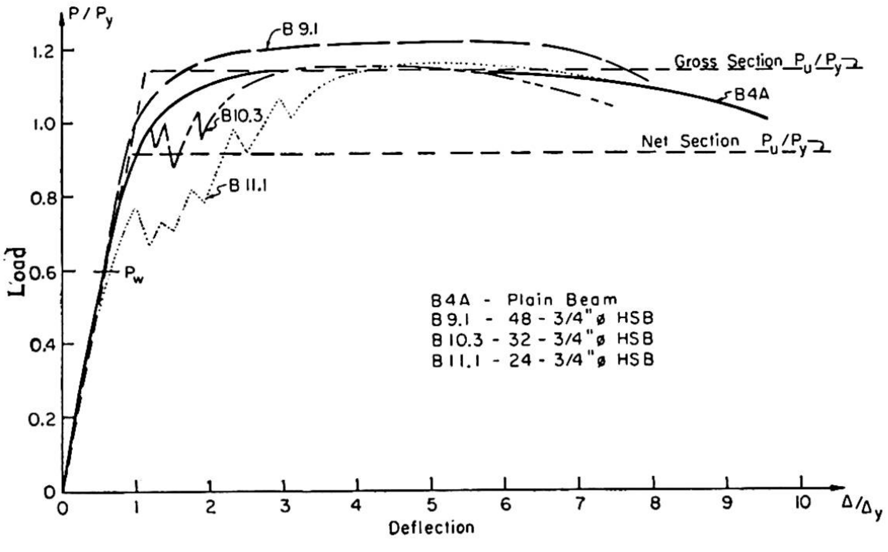


Fig. 1. Typical load-deflection curves.

buckle. Outside of the splice plates, yield lines traced on the whitewash coating extended nearly to mid-depth of the web, but web yielding stopped almost abruptly at the vertical plane through the end bolts. Flange yielding extended through one or more bolt rows.

It is likely that the attainment of the full plastic moment was largely the result of strain hardening of the flange material remaining around the holes. This conclusion is supported by B 8.1, an unspliced specimen with $^{15}/_{16}$ inch diameter open holes in each flange. In this test the full plastic moment was reached before the compression flange buckled locally. At least in beams with normal proportions and locations of flange holes, one may rely upon the development of the full plastic moment.

The moment-curvature relationship for each beam in Table 1 was also measured but the results are not reported because the curvatures, being averages for the spliced and plain portions of the constant moment region, have no direct application. They did, however, give an indication of the ability of the connection to rotate inelastically. The rotation capacities were at least as large as some of those cited as satisfactory in Reference 1.

T-stub Flange-to-Column Connections

Simulating the connection between a T-stub flange on the tension side of a beam and its supporting column, the specimens in Table 2 were tested. Again, the rolled sections were A 7 steel and the bolts were type A 325, tightened to the proof load.

The most important questions in either elastic or plastic design are the flexure of the flange plates and the tension in the bolts. As long as there is contact between the joined plates of a pretensioned connection, the bolt force is statically indeterminate. At low loads it is only slightly larger than the initial tension. At high loads it may be substantially greater than the applied load F (Fig. 2b) if there is prying by the portions of the flange outside of the bolts (causing the force Q). The AISC Specification prescribes that any additional rivet or bolt tension resulting from prying action be added to the stress calculated directly from the applied force in proportioning the fasteners. Unfortunately, there is, to the authors' knowledge, no sound analytical

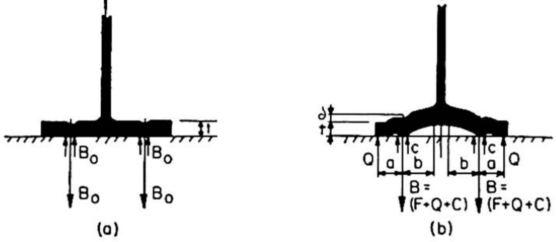
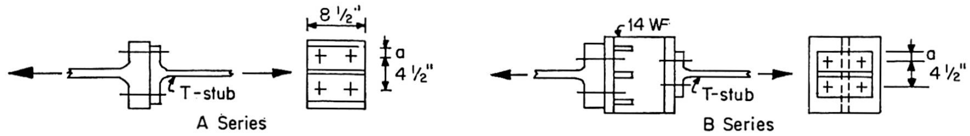


Fig. 2. Analytical model of T-stub flange.

Table 2. Summary of T-Stub Tests



Test	T-stub	Base	Bolt Diam. (in)	Edge Dist. a (in)	Comp. Bolt Strength (kips)	Comp. Fail. Load (kips)	Actual Fail. Load (kips)	Bolt Effi-ciency	Comp. Fail. Mode	Actual Fail. Mode
A 1	18 WF 70	Rigid	7/8	1.50	224	155	176	0.78	Bolt Fract.	Bolt Fract.
A 3	36 WF 300	Rigid	7/8	1.50	248	248	256	1	Bolt Fract.	Bolt Fract.
A 4	2"Flange/1"Web	Rigid	7/8	1.50	237	237	219	0.93	Bolt Fract.	Nut Strip.
A 5	18 WF 70	Rigid	1 1/8	1.50	408	214	224	—	Web	Flange
A 7	36 WF 300	Rigid	1 1/8	1.50	408	401	392	0.96	Bolt Fract.	Bolt Fract.
A 8	2 1/2" Fl./1" Web	Rigid	1 1/8	1.50	423	423	(2)	—	Bolt Fract.	(2)
A 9	18 WF 70	Rigid	7/8	1,75	224	162	177	0.79	Bolt Fract.	Bolt Fract.
A 10	24 I 105.9	Rigid	7/8	1.66	244	200	240	0.98	Bolt Fract.	Bolt Fract.
A 11	36 WF 300	Rigid	7/8	1.75	247	247	256	1	Bolt Fract.	Bolt Fract.
A 12	2" Fl./1" Web	Rigid	7/8	1.75	239	239	245	1	Bolt Fract.	Bolt Fract.
A 13	18 WF 70	Rigid	1 1/8	1.75	404	217	228	—	Web	Web
A 14	24 I 105.9	Rigid	1 1/8	1.66	388	300	286	—	Bolt Fract.	Web (3)
A 15	36 WF 300	Rigid	1 1/8	1.75	400	400	404	1	Bolt Fract.	Bolt Fract.
A 16	2 1/2" Fl./1" Web	Rigid	1 1/8	1.75	426	426	(2)	—	Bolt Fract.	(2)
B 1	18 WF 70	14 WF 150	7/8	1.50	256	171	202	0.79	Bolt Fract.	Bolt Fract.
B 3	24 I 105.9	14 WF 150	7/8	1.66	248	204	230	0.93	Bolt Fract.	Bolt Fract.
B 4	24 I 105.9	14 WF 287	7/8	1.66	240	200	228	0.95	Bolt Fract.	Bolt Fract.
B 5	24 I 105.9	14 WF 426	7/8	1.66	240	200	230	0.96	Bolt Fract.	Bolt Fract.
B 6	36 WF 300	14 WF 150	7/8	1.50	240	228	254	1	Bolt Fract.	Bolt Fract.
B 7	36 WF 300	14 WF 287	7/8	1.50	222	222	233	1	Bolt Fract.	Bolt Fract.
B 9	36 WF 300	14 WF 150	1 1/8	1.50	388	316	348	0.90	Bolt Fract.	Bolt Fract.
B 10	36 WF 300	14 WF 287	1 1/8	1.50	396	396	403	1	Bolt Fract.	Bolt Fract.
B 12	2 1/2" Fl./1" Web	14 WF 150	1 1/8	1.50	400	318	378	0.94	Bolt Fract.	Bolt Fract.
B 13	2 1/2" Fl./1" Web	14 WF 287	1 1/8	1.50	398	398	(2)	—	Bolt Fract.	(2)

(2) Exceeded machine capacity of 404 kips. (3) Imperfection in material

method for doing this and there is little empirical data for guidance. In the absence of a better approach, it has long been the custom to use clearly approximate methods for estimating prying action.

Because a precise analysis is virtually impossible, one of the commonly used approximate methods were developed into a complete theory of flange and bolt behavior to aid in interpreting the Cornell tests [3]. The analytical model is shown in Fig. 2. Under zero applied load the tension in each bolt is B_0, the initial value. The equilibrating compression is assumed localized around the bolts with the remainder of the plates in contact but exerting no pressure on each other. The bolt pressure on the flange causes its upper surface to dish locally. When the load $2F$ is applied, the flange is assumed to flex as shown in Fig. 2b. Should the outer portions remain in contact with the support, prying forces develop. These are assumed to act as line loads at the ends of the spans a. The residual contact force at the bolt is C. While the flange remains elastic, its upward deflection at the bolt is, from ordinary flexure theory:

$$\delta = \frac{a b^2}{E w t^3/12}\left\{\frac{F}{2} - \frac{a}{b}\left[\frac{1}{3}\left(\frac{a}{b}\right)+1\right] Q\right\}, \tag{1}$$

where w is the width of the flange normal to the figure and E is Young's modulus. The corresponding bolt stretch is:

$$\delta = [(F+Q)-(B_0-C)]\frac{l_b}{A_b E_b}, \tag{2}$$

where l_b, A_b, and E_b are, respectively, the effective length, area and modulus of the bolt. Until C becomes zero and the initial thickness of the flange is restored, the plate's local expansion is

$$\delta = (B_0-C)\frac{l_p}{A_p E_p}, \tag{3}$$

where l_p, A_p, and E_p are, respectively, the effective thickness, area and modulus of the compressed portion of the plate. Eliminating δ and (B_0-C):

$$Q = \left[\frac{\dfrac{1}{2} - \dfrac{E w t^3/12 a b^2}{r_b + r_p}}{\dfrac{a}{b}\left(\dfrac{a}{3b}+1\right) + \dfrac{E w t^3/12 a b^2}{r_b+r_p}}\right] F, \tag{4}$$

where r_b and r_p are the bolt and plate stiffnesses $l_b/A_b E_b$ and $l_p/A_p E_p$. The application of Eq. (4) depends upon the establishment of reasonable values of the parameters. Also, it applies only while there is bolt line contact and the flange remains elastic. Nevertheless, the same concepts may be used to derive formulas for Q as a function of F following separation at the bolt line and including plastic flow and strain hardening of the flange. Strain hardening, which is disregarded in the simple plastic analysis of frames because of the difficulty of preventing local buckling in the strain hardened range, may be considered here because of the absence of local buckling. In fact, its inclusion is necessary to explain the performance of thin flanges. In addition to the rigid support in Fig. 2, flexible supports of the type provided by most column flanges were treated. The several equations for Q will not be reproduced because they are unwieldy and have no direct design utility.

In applying the theory to the test specimens, the distance a was taken to the edge of the flange and b to the center of the web fillet. In the inelastic range, moment-curvature relationships for a rectangular section of structural steel were used. Bolt stiffness was determined from calibration tests of bolts of the same size. Bolt tensions were measured by calibrated elongation readings. The effective compressed area of the plate was taken as a cylinder or truncated cone (when a washer was used on one side only) having an outer diameter defined by the distance across flats of the nuts and by the washer diameter (when present). While not to be thought of as general guides, these quantities, when used in combination to supplement the theory, gave results that were in good accord with the tests. The true distribution of prying forces is unknown but the four tested specimens in Fig. 3 show that, at least at high

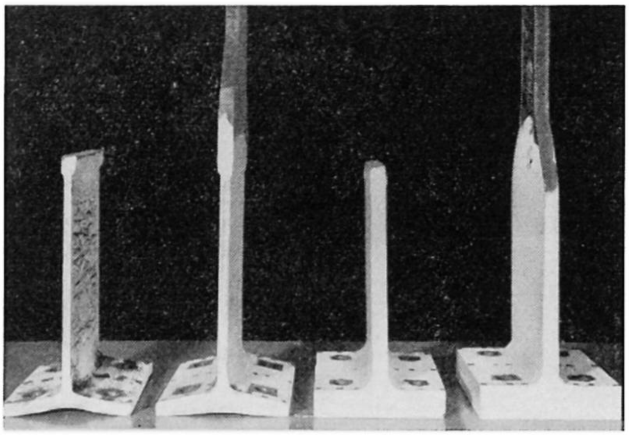


Fig. 3. T-stub flanges after testing.

loads and in light flanges most subject to prying, the only contact is very close to the edge. Local compression of the flanges around the bolt, also highly indeterminate, is a factor only so long as contact remains in that area.

In Table 2 the computed bolt strengths are based on the bolt calibration tests. The computed failure loads were calculated from the theory. Values less than the bolt strength mean that prying was computed to persist until failure.

The behavior of one specimen, A 1, is shown in Fig. 4. Characteristically, prying decreased at high loads. At an applied load of 26 kips, the bolt tension was 42 kips. Since bolt line separation had occurred, the prying force was 16 kips, or 61% of the applied load. At an applied load of 39 kips it was 38% of that load and at the failure load of 44 kips, 27%. The theory predicted most of this, but no attempt was made to account for the final reduction in prying which took place as the bolts elongated under almost constant load just prior to rupturing. In some other specimens this was sufficient to cause complete flange separation and to make the bolts 100% efficient at rupture even though, at loads not greatly less than the ultimate, prying was still significant. For heavy flanges prying action was unimportant at all loads.

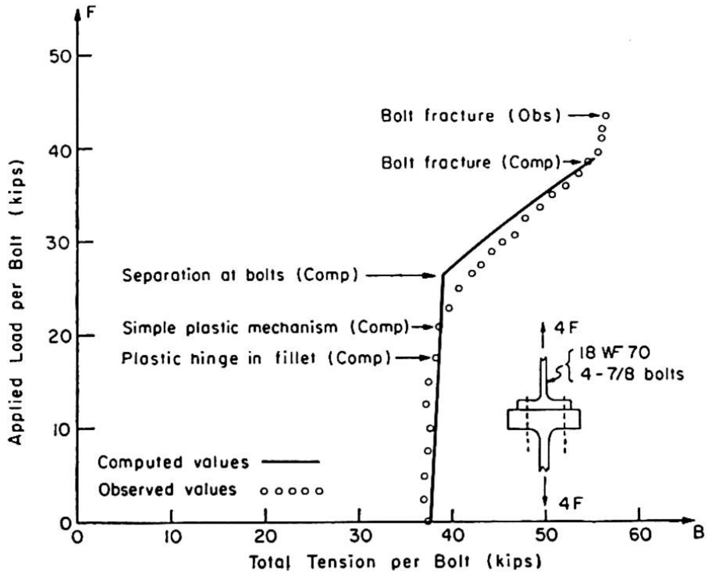


Fig. 4. Bolt tension, test A 1.

Assembled Connections

Finally, the assembled T-stub moment connections shown in Table 3 were tested. The T-stub and column combinations in these tests were the same as some of those in Table 2. The T-stub web-to-beam flange bolts were proportioned for a nominal shear stress of 15,000 psi when the moment on each beam at the inner row of bolts was equal to its working value. Prior to failure or reaching the capacity of the equipment, the full plastic moment of each beam was attained. The moment-rotation characteristics of the connections are believed satisfactory for plastically designed structures. From both the theory and tests it appears possible to use bolted T-stub connections in plastically designed frames without loss of section efficiency, even when the beam flange connection is bolted.

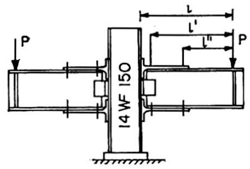
Tentative Design Method

To develop design techniques using the assumption that T-stub flanges behave as in Fig. 2, the theory may be modified empirically to obtain useful formulas for prying force. Formulas giving reasonable results when compared with the Cornell data are:

For $(F+Q) \leqq$ the initial tension B_0 (to be used in elastic design):

$$Q = \left[\frac{\frac{1}{2} - \frac{w\,t^4}{30\,a\,b^2 A_b}}{\frac{3\,a}{4\,b}\left(\frac{a}{4\,b}+1\right) + \frac{w\,t^4}{30\,a\,b^2 A_b}}\right] F = p_1 F. \tag{5}$$

Table 3. Summary of Assembled Connection Tests



Test	Description										Test Results	
	Beams	T-stub	Bolts (diam)	l (in)	l' (in)	l'' (in)	σ_y (k_{si})	Z (in^3)	M_p (in-kips)	$P_p = \frac{M_p}{l}$ (kips)	P_{max} (kips)	Failure Mode
D_1	14 WF 34	18 WF 70	$7/8$	30	27	15.8	37.2	56.7	2,120	70.5	93.0	Bolts
D_2	16 WF 40	24 I 105.9	$7/8$	36	32	20.8	34.2	74.4	2,550	71.0	108.0	(4)
D_3	21 WF 62	36 WF 300	$1\,1/8$	66	61	46.5	35.2	140.6	4,950	75.0	108.0	(4)

(4) Exceeded jack capacity of 108 kips

For $(F+Q) > B_0$ (to be used in plastic design):

$$Q = \left[\frac{\frac{1}{2} - \frac{w\,t^4}{30\,a\,b^2 A_b}}{\frac{a}{b}\left(\frac{a}{3\,b}+1\right) + \frac{w\,t^4}{6\,a\,b^2 A_b}}\right] F = p_2 F\,, \tag{6}$$

where A_b is the full area of the bolt and the other terms are as defined earlier. If the second term in the numerator is equal to or greater than one-half, no prying develops since Q cannot be negative. The prying forces decrease as the flexural stiffness of the inner part of the flange — measured by w, t and $1/b$ — increases. Prying increases with A_b. It generally decreases with an increase in a, but, because of the limited range of edge distances tested, complete reliance upon this is not advisable. It is suggested that, in using Eqs. (5) and (6), a be taken as $1.25\,b$ when its actual dimension is greater than this.

Assuming that in a plastically designed connection there has been bolt line separation by the time the plastic moment of the supported beam is reached, the bolt force will be $B = (F+Q)$ where, in this case, F is the beam's plastic moment divided by the effective depth of the connection and number of bolts. From the test data, limiting the value of B so computed to not more than 1.33 times the specified proof load of the bolt (presumed to be the pretension also) prevents premature bolt fracture. Eq. (6) gives prying forces generally somewhat greater than those corresponding to the ultimate efficiencies in Table II, providing a margin of safety attributable to disregarding partially the final bolt elongation just prior to rupture.

In plastic design one must be concerned with conditions at working load as well as at failure. In the present instance the most important precaution is the prevention of bolt line separation at working loads with the consequent danger of inelastic stretching and loosening of bolts. It appears that this may control the total force permitted on the bolts in most cases. For example, the ultimate uniformly distributed load on a fixed end beam is $16\,M_p/l^2$. The end moment at working load is $q\,l^2/12$ or, assuming a load factor of 1.85, 72% of the plastic moment. If a total bolt force of 1.33 times the initial pretension is permitted at the plastic moment, then the tests in which there was prying action show that there might be separation at the working load. From a study of the data and practical cases, it is believed possible to prevent premature separation in connections where prying occurs by limiting the total bolt force to 1.15 times the initial tension when the ultimate load acts on the frame. A double standard is therefore indicated: 1. for connections at the last plastic hinge to form and those in which Eq. (6) shows no prying, use 1.33 times the initial tension; 2. for others use 1.15 times the initial tension. Because of the restrictions on the higher value, it is probable that $1.15\,B_0$ would normally be used.

These ideas and the remainder of the Cornell studies suggest a procedure for proportioning T-stub moment connections at plastic hinge locations in plastically designed structures. It assumes that the bolts are pretensioned to the specified proof load. At a plastic hinge the force acting on the T-stub web (the force F — as defined above — times the number of bolts) will be designated as F'. An outline of the full procedure follows:

T-stub Web-to-Beam Flange Connections

1. Make the net area of the T-stub web times its yield point equal to or greater than F'.
2. Provide sufficient bolts in shear so that F' divided by the total bolt area is not greater than 22,000 psi (for A 325 bolts).

T-stub Flange-to-Column Flange Connections

1. Select trial sizes and dimensions.
2. Compute p_2, the ratio of prying to applied force, from Eq. (6).
3. Check to see that
   a) at the last plastic hinge to form and at other connections where $p_2 = 0$: $(1 + p_2)\, F \leqq 1.33 \times$ proof load;
   b) at all other connections: $(1 + p_2)\, F \leqq 1.15 \times$ proof load.
4. Compute the bending moment in the T-stub flange at the bolt line and at the outer edge of the fillet:

$$M_{bolt\,line} = p_2 F a,$$

$$M_{fillet} = -\left[p_2 a - \left(b - \frac{r}{2}\right)\right] F,$$

   where r is the fillet radius.
5. Check to see that the moment on either section does not exceed $w t^2 \sigma_y / 4$ where σ_y is the yield point of the T-stub flange.
6. Use the following modifications in special cases:
   a) When $a \geqq 1.25\, b$, use $1.25\, b$.
   b) When the column flange is thinner than the T-stub flange and is unstiffened, use the average of the two thicknesses in computing prying forces and stresses. All other dimensions are to be taken as those of the T-stub.

The design procedure tends to underestimate the capabilities of light T-stub flanges because it does not account directly for the reserve of strength attributable to strain hardening. In most cases this tendency is not objectionable because heavier flanges would be used to reduce prying action and to use the bolts to better advantage, that is, the bolts rather than the flange thickness usually control the design.

Observed separation and failure loads and values computed by the suggested procedure are shown in Table 4 for all of the Cornell direct tension

Table 4. Comparison of Design Procedure and Tests

Specimen	T-stub σ_y (ksi)	B_0 (Bolt Pretension) (kips)	p_2 (Eq. 6)	Comp. Values of F (kips)			Obs. Values of F (kips)	
				$\frac{1.15\,B_0}{(1+p_2)}$	$\frac{1.33\,B_0}{(1+p_2)}$	$\frac{w\,t^2\,\sigma_y}{4\left(p_2 a-b-\frac{r}{2}\right)}$	At Bolt Separation	At Failure
A 1	34.5	37.1	0.42	30.1	34.8	22.8	22	44.0
A 3	26.0	33.2	—	38.2	44.2	89.6	34	64.0
A 4	31.1	34.5	—	39.7	45.9	100.0	35	55.0
A 5	33.3	52.5	0.44	42.1	48.6	22.7	27	55.8
A 7	27.0	51.2	0.01	58.1	67.2	100.4	45	98.0
A 8	31.0	54.6	—	62.9	72.6	156.0	60	(6)
A 9	34.5	36.0	0.35	30.6	35.3	22.7	24	44.3
A 10	31.1	34.5	0.16	34.2	39.6	46.5	28	60.0
A 11	26.0	34.5	—	39.7	45.9	89.6	(5)	64.0
A 12	31.1	34.5	—	39.7	45.9	100.0	35	61.5
A 13	33.3	53.0	0.37	44.6	51.6	22.3	27	57.0
A 14	29.5	53.6	0.23	49.9	57.7	48.1	35	71.5
A 15	27.0	54.1	0.03	60.4	69.8	104.6	(5)	101.0
A 16	31.0	54.7	—	63.0	72.8	156.0	60	(6)
B 1	34.5	33.0	0.42	26.8	30.9	22.8	25	50.5
B 3	31.1	31.5	0.23	29.6	34.2	40.0	20	57.5
B 4	31.1	33.5	0.16	33.2	38.4	45.4	24	57.0
B 5	31.1	37.5	0.16	37.1	42.9	45.4	30	57.5
B 6	26.0	32.5	0.04	35.9	41.5	71.1	25	63.5
B 7	26.0	35.5	—	40.8	47.2	89.6	(5)	58.2
B 9	27.0	50.5	0.11	52.5	60.8	84.5	35	87.0
B 10	27.0	53.0	0.01	50.1	69.5	100.4	55	101.0
B 12	31.0	53.0	0.03	59.2	68.4	77.2	40	94.5
B 13	31.0	53.5	—	61.5	71.1	156.0	54	(6)

(5) No abrupt increase in tension. Separation point not clear.
(6) Exceeded machine capacity of 404 kips.

specimens. The measured initial tension rather than the specified proof load is used in these computations. For all cases in which 1.15 B_0 controls, the observed separation load is at least two-thirds of the computed value of F. This is believed to be an appropriate lower limit for practical conditions. For all cases in which 1.33 B_0 controls there is a margin of safety against failure which may be attributed to the factors mentioned previously. Although no design procedure may be considered proved until it has withstood the tests of time and experience, from the data available this one appears reasonable.

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3. R. T. DOUTY: "Ultimate Characteristics of High Strength Bolted Connections." Ph. D. thesis, Cornell University, 1963.

Summary

Tests and analytical studies were made of the components and complete assemblies of T-stub moment connections having high strength bolts (ASTM-A 325). Of particular interest was the question of their applicability in plastic design. In the tests of beams and connections having holes in the beam flanges, the full plastic moment of the beam was developed. In the tests of T-stub flanges the development of prying forces and response of the flange was observed. Semi-empirical formulas for estimating prying are presented and incorporated in a tentative design procedure for the use of connections of this type in plastic design.

Résumé

Les auteurs ont étudié par le calcul et par des essais les éléments des assemblages fléchis comportant des T de liaison fixés par boulons HR (ASTM-A 325) ainsi que les assemblages entiers. Ils ont tout particulièrement considéré leur emploi dans les constructions calculées en plasticité. Dans les essais auxquels ont été soumis des poutres et des assemblages comportant des trous dans les ailes, on a réalisé le moment plastique théorique. Dans les essais sur les T de liaison, on a observé un effet de levier et les sollicitations qui en résultent dans les ailes. Les auteurs établissent des formules semi-empiriques pour l'estimation de l'effet de levier, et ces formules trouvent leur place dans une méthode proposée pour l'étude d'assemblages de ce type dans le calcul en plasticité.

Zusammenfassung

Versuche und analytische Studien an Elementen und an vollständigen T-Stück-Verbindungen mit HV-Schrauben werden beschrieben. Die Anwendbarkeit dieser Verbindungen bei Bemessung nach der Plastizitätstheorie wurden besonders untersucht. Trägerstöße erreichten dabei trotz der Lochschwächung das volle plastische Moment ungestoßener Träger. An T-Stück-Elementen wurde der Einfluß der Reaktionskräfte der Flanschen infolge der Flanschenverformung auf die Größe der Schraubenkräfte untersucht. Empirische Formeln werden angegeben für die Bestimmung dieser Reaktionskräfte sowie ein Verfahren für die plastische Bemessung von T-Stück-Verbindungen.