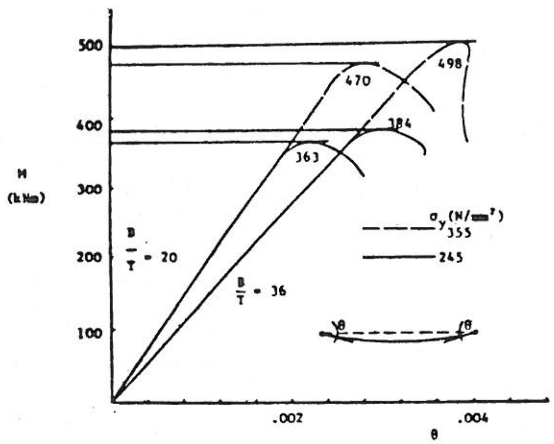


Fig. 13

AN INVESTIGATION INTO THE INTERACTION OF FLANGES AND WEBS IN WIDE FLANGE SHAPES

Donald L. Johnson
Senior Research Engineer
Butler Manufacturing Company

During the mid 1970's several full scale tests were run on experimental rigid frames at the Butler Research Center and the results were not satisfactory when compared to predicted values using the AISC Specifications. It appeared that premature flange buckling was occurring in a portion of the roof beam with high depth to web thickness ratios combined with high width to flange thickness ratios. To investigate this problem a series of tests were performed on beams with h/t ratios ranging from 73 to 176, dimensions of the test beams are tabulated in Table 1. All beams were 15'-8" long and supported on rollers at each end. Third point loading was applied by hydraulic cylinders attached to gusset plates welded to the bottom flange at 60 in. from each support. The ten beams were all the same depth with the web thickness of the middle third section varied to obtain the range of h/t values. To prevent lateral buckling the top (compression) flange was braced with a horizontal member free to move vertically but restrained in the horizontal direction. The outer web sections were designed to prevent shear failure. Results of the test are listed in Table 2.

The failure mode observed in the beam tests was the same as in the rigid frame tests. The flange and web rotated about a longitudinal axis coinciding with the flange and web intersection. The flange and web retained their 90° relationship. A check of AISC specifications again resulted in unconservative predictions. Two sections of the Specifications address this area: "1.10.6 Reduction in Flange Stress" and "Section C-2 Stress Reduction Factor - Unstiffened compression Elements". Using both resulted in the comparison with tested values shown in Table 3.

In a review of the literature, Ref. #1 explains that in developing the buckling equations in these two sections buckling coefficients (k) were arbitrarily selected to fall about midway between simply supported and fixed along the supported edge. This results in the assumption that the two elements, flange and web, are to some degree supporting each other. But suppose one of the elements has insufficient stiffness to provide even minimal support to the other element, obviously the critical buckling stress will be lower. With this assumption it would seem reasonable to develop design equations with k as a variable, and since beam failure cannot occur until the flange fails the critical buckling stress will be calculated for the flange. Beginning with the basic expression for elastic buckling stress:

$$F_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(b/t)^2} \qquad \text{(E-1)}$$

Since this only applies to the elastic buckling range a transition curve must be provided for the region between the Euler curve and the point where strain hardening commences. The transition curve may be taken as:

$$\frac{F_{cr}}{F_y} = 1 - \left(1 - \frac{F_p}{F_y}\right)\left(\frac{\lambda-\lambda_o}{\lambda_p-\lambda_o}\right)^n \qquad \text{(E-2)}$$

where λ_p = slenderness function at F_p = proportional limit.

Let $\lambda = \sqrt{\frac{F_y}{F_{cr}}}$

then $$\frac{F_{cr}}{F_y} = \frac{1}{\lambda^2} \qquad \text{(E-3)}$$

and $\lambda = \frac{b}{t}\sqrt{\frac{F_y\ (12)(1-\mu^2)}{\pi^2\ Ek}}$

In computing the transition curve, Ref. 3 suggests values of .46 for λ_o

and $\sqrt{2}$ for λ_p.

n can be taken as

$$n = \frac{2(\lambda_p - \lambda_o)}{(\lambda_p^2 - 1)} = 1.92 \qquad \text{(E-4)}$$

although values of 1.36 to 2 have been suggested.

If we assume $F_p = 0.5\ F_y$ then

$$\frac{F_{cr}}{F_y} = 1-(1-.5)\left(\frac{\lambda - .46}{1.41 - .46}\right)^{1.92} \qquad \text{(E-5)}$$

$$= 1 - .55\ (\lambda - .46)^{1.92} \qquad \text{(E-6)}$$

We now have two equations for predicting buckling of the flange (E-3 and E-6). For a given F_y the only variables are b/t of flange and k. Considerable research has been conducted on the value of k for all types of plates with varying edge support conditions. The cases of interest here are: (1) one edge free - opposite edge fixed where k = 1.277 and (2) one edge free - opposite edge pinned where k = .425. In selecting a k value for flange buckling, AISC took what would appear to be a conservative value of 0.7 (midway between the fixed and pinned edges. See Ref. 1 p. 301 and Ref. 4 p. 534). However use of any value of k greater than 0.425 assumes restraint from the web and it is normally assumed that k values less than 0.425 would not be possible. But if the h/t ratio of the web is allowed to increase to the point that the web adjacent to the compression flange becomes unstable, it is not unreasonable to assume that torsional buckling of the flange could be induced with a resulting effective k value of less than 0.425. Ref. 2 describes work performed by Stowell and Lundquist in this area and the resulting charts project k values of less than 0.425 although the sections studied were all column sections.

By solving Equations E-3 and E-6 for k given the experimental values for F_{cr}/F_y from the test data, a range of k values can be computed. These are shown in Table 4. An expression for k was then derived as a function of h/t as shown in Fig. 1. The expression

$$k = \frac{4.44}{\sqrt{h/t}} \qquad \text{(E-7)}$$

seemed to fit the data.

The preceding approach to flange design has been used by our designers with good success, however, it has been suggested that the range of parameters used in the test program was too narrow for the conclusions to be applied to beams with h/t values in excess of 200. Therefore in early 1985 a second test program was undertaken to extend the h/t range and confirm or modify the previous results. Figure 2 shows the parameter distribution for the two series of tests. In order to achieve the required h/t values thirty foot long beams were fabricated with depths of 20 and 24" and nominal 0.10 web thickness. See Table 5. The beams were loaded with four concentrated loads through gussets attached to the bottom flange.

The results of the tests were similar to that of the first series except that the sections with very low b/t values (7.1) showed no indication of local buckling and failure was by lateral buckling. A summary of the results is given in Table 6. After including these additional tests in the data base and reanalyzing for k the following expression for k was derived, see Table 7 and Fig. 3.

$$k = \frac{4.05}{(h/t)^{.46}} \qquad \text{(E-8)}$$

In order to put the above results in the same form as the AISC Appendix C, set E-1 equal to $Q\,F_y$ this results in:

$$Q = \frac{\pi^2 Ek}{12(1-\nu^2)(b/t)^2 F_y} \qquad \text{(E-9)}$$

Substituting numerical values for the constants results in:

$$Q = \frac{3.14^2 \times 29000 \times k}{12(1-0.3^2)(b/t)^2 F_y} = \frac{26200\ k}{(b/t)^2 F_y} \qquad \text{(E-10)}$$

Similarly E-6 can be reduced to

$$Q = 1.0 - 0.55\ (.006164\sqrt{\frac{F_y}{k}}\ \frac{b}{t} - .46)^{1.92} \qquad \text{(E-11)}$$

In the interest of simplicity (as in AISC C2-3) this can be replaced with a straight line form

$$Q = 1.293 - .00309\ \frac{b}{t}\sqrt{\frac{F_y}{k}} \qquad \text{(E-12)}$$

An examination of the curves indicates that $Q = 1.0$ when

$$\frac{b}{t}\sqrt{\frac{F_y}{k}} = 95 \qquad \text{(E-13)}$$

and the two curves intersect at

$$\frac{b}{t}\sqrt{\frac{F_y}{k}} = 195 \qquad \text{(E-14)}$$

We now have a set of equations that could be directly substituted for current AISC specifications:

$$k = \frac{4.05}{(h/t)^{.46}}$$

When $95.0/\sqrt{\frac{F_y}{k}} < b/t < 195/\sqrt{\frac{F_y}{k}}$

$$Q_s = 1.293 - .00309\ \frac{b}{t}\sqrt{\frac{F_y}{k}}$$

when $b/t \geq 195/\sqrt{\frac{F_y}{k}}$

$$Q_s = 26200\ k/\left[F_y(b/t)^2\right]$$

A comparison of the test results against current AISC specifications and the proposed specification is shown in Table 8.

REFERENCES

1. Salmon and Johnson, "Steel Structures Design and Behavior" 1971.
2. Bleich, "Buckling Strength of Metal Structures" 1952.
3. Johnston, B. G. "The Column Research Guide to Design Criteria for Metal Compression Members" Second Edition 1966.
4. Beedle, et al. "Structural Steel Design" 1954.
5. AISC "Specification for the Design Fabrication and Erection of Structural Steel for Buildings" 1978.
6. Dhalla and Errera "Unstiffened Compression Elements of High Strength Carbon Steel" 1968.
7. AISI "Specification for the Design of Cold-Formed Steel Structural Members" 1980.

No.	D Depth	B Width	t Flg	t Web	F_y Flg	h/t	B/2t
1A	12.00	5.23	0.183	0.066	55.10	176.3	14.3
2A	12.02	5.23	0.183	0.067	54.00	174.0	14.3
3A	12.09	5.23	0.183	0.076	54.40	154.2	14.3
4A	12.05	5.22	0.183	0.076	54.70	153.7	14.3
5A	12.10	5.23	0.185	0.106	54.30	110.7	14.5
6A	12.07	5.22	0.184	0.107	55.30	109.4	14.5
7A	12.07	5.24	0.183	0.159	55.10	73.6	14.2
8A	12.00	5.23	0.183	0.159	55.90	73.2	14.1
9A	12.10	6.42	0.278	0.076	62.70	151.9	11.5
10A	12.05	6.42	0.280	0.077	61.60	149.2	12.0

Table 1

No.	D Depth	B Width	h/t Web	B/2t Flg	P Ult.	F_c Ult.
1A	12.00	5.23	176.3	14.3	8.80	41.57
2A	12.02	5.23	174.0	14.3	8.70	41.10
3A	12.09	5.23	154.2	14.3	9.10	42.00
4A	12.05	5.22	153.7	14.3	8.90	41.39
5A	12.10	5.23	110.7	14.5	10.31	44.82
6A	12.07	5.22	109.4	14.5	10.25	44.56
7A	12.07	5.24	73.6	14.2	11.90	47.90
8A	12.00	5.23	73.2	14.1	12.27	49.76
9A	12.10	6.42	151.9	11.5	17.29	46.31 *
10A	12.05	6.42	149.2	12.0	19.50	52.23

Table 2

*Fixture Failure

No.	D Depth	B Width	h/t	B/2t	F_c Ult.	AISC 1.10.6	AISC C-2	F_c Calc	[illegible]
1A	12.00	5.23	176.3	14.3	41.57	0.982	0.951	51.46	[illegible]
2A	12.02	5.23	174.0	14.3	41.10	0.983	0.956	50.76	[illegible]
3A	12.09	5.23	154.2	14.3	42.00	0.990	0.954	51.39	[illegible]
4A	12.05	5.22	153.7	14.3	41.39	0.990	0.953	51.61	[illegible]
5A	12.10	5.23	110.7	14.5	44.02	1.000	0.948	51.48	[illegible]
6A	12.07	5.22	109.4	14.5	44.56	1.000	0.944	52.19	[illegible]
7A	12.07	5.24	73.6	14.2	47.90	1.000	0.954	52.59	[illegible]
8A	12.00	5.23	73.2	14.1	49.76	1.000	0.954	53.35	[illegible]
9A	12.10	6.42	151.9	11.5	46.31	0.993	1.000	62.27	[illegible] *
10A	12.05	6.42	149.2	12.0	52.23	0.994	1.000	61.25	[illegible]

Table 3

*Premature Fixture Failure

No.	D Depth	h/t	b/2t	k Calc
1A	12.00	176.3	14.3	0.341
2A	12.02	174.0	14.3	0.337
3A	12.09	154.2	14.3	0.352
4A	12.05	153.7	14.3	0.340
5A	12.10	110.7	14.5	0.418
6A	12.07	109.4	14.5	0.403
7A	12.07	73.6	14.2	0.474
8A	12.00	73.2	14.1	0.515
9A	12.10	151.9	11.5	0.241*
10A	12.05	149.2	12.0	0.351

Table 4

*Premature Fixture Failure

No.	D Depth	B Width	t Flg	t Web	Fy Flg	h/t	B/2t
1	23.97	5.36	0.377	0.217	59.38	107.0	7.1
2	23.68	6.01	0.194	0.095	62.12	245.2	15.5
3	23.91	5.56	0.375	0.097	54.56	238.8	7.4
4	23.52	5.96	0.194	0.098	59.04	236.1	15.4
5	19.91	6.03	0.194	0.096	62.00	203.3	15.5
6	19.87	6.02	0.194	0.097	63.41	200.8	15.5
7	19.91	5.54	0.379	0.098	57.86	195.4	7.3
8	19.92	5.33	0.377	0.097	53.68	197.5	7.1
9	23.95	5.53	0.365	0.098	43.20	236.9	7.6

Table 5

No.	D Depth	h/t	B/2t	k
1	23.97	107.0	7.1	0.000
2	23.68	245.2	15.5	0.363
3	23.91	238.8	7.4	0.000
4	23.52	236.1	15.4	0.346
5	19.91	203.3	15.5	0.369
6	19.87	200.8	15.5	0.374
7	19.91	195.4	7.3	0.000
8	19.92	197.5	7.1	0.000
9	23.95	236.9	7.6	0.000
1A	12.00	176.3	14.3	0.362
2A	12.02	174.0	14.3	0.365
3A	12.09	154.2	14.3	0.381
4A	12.05	153.7	14.3	0.364
5A	12.10	110.7	14.5	0.486
6A	12.07	109.4	14.5	0.455
7A	12.07	73.6	14.2	0.568
8A	12.00	73.2	14.1	0.630
9A	12.10	151.9	11.5	0.000
10A	12.05	149.2	12.0	0.412

Table 7

No.	D Depth	B Width	h/t Web	B/2t Flg	P Ult.	F_c Ult.
1	23.97	5.36	107.0	7.1	11.08	44.99*
2	23.68	6.01	245.2	15.5	5.21	39.63
3	23.91	5.56	238.8	7.4	10.58	50.01*
4	23.52	5.96	236.1	15.4	5.02	38.49
5	19.91	6.03	203.3	15.5	4.25	40.06
6	19.87	6.02	200.8	15.5	4.31	40.67
7	19.91	5.54	195.4	7.3	8.38	48.98*
8	19.92	5.33	197.5	7.1	8.99	54.29
9	23.95	5.53	236.9	7.6	8.07	39.20*

Table 6

*Failure by lateral buckling

No.	h/t	B/2t	F_c Ult.	AISC 1.10.6	AISC C-2	F_c Calc	$\frac{F_c \text{ Calc}}{F_c \text{ Ult}}$	η Prop.	[illegible]
1	107.0	7.1	44.99	1.000	1.000	59.38	1.32	1.000	[illegible]
2	245.2	15.5	39.63	0.886	0.881	48.47	1.22	0.566	[illegible]
3	238.8	7.4	50.01	0.943	1.000	51.44	1.03	0.997	[illegible]
4	236.1	15.4	38.49	0.894	0.899	47.45	1.23	0.617	[illegible]
5	203.3	15.5	40.06	0.937	0.880	51.14	1.28	0.615	[illegible]
6	200.8	15.5	40.67	0.937	0.875	51.98	1.28	0.605	[illegible]
7	195.4	7.3	48.98	0.970	1.000	56.14	1.15	1.000	[illegible]
8	197.5	7.1	54.29	0.971	1.000	52.11	0.96	1.000	[illegible]
9	236.9	7.6	39.20	0.951	1.000	41.07	1.05	1.000	[illegible]
1A	176.3	14.3	41.57	0.982	0.951	51.48	1.24	0.757	[illegible]
2A	174.0	14.3	41.10	0.983	0.956	50.76	1.24	0.764	[illegible]
3A	154.2	14.3	42.00	0.990	0.954	51.39	1.22	0.777	[illegible]
4A	153.7	14.3	41.39	0.990	0.953	51.61	1.25	0.776	[illegible]
5A	110.7	14.5	44.82	1.000	0.948	51.48	1.15	0.809	[illegible]
6A	109.4	14.5	44.56	1.000	0.944	52.19	1.17	0.806	[illegible]
7A	73.6	14.2	47.90	1.000	0.954	52.59	1.10	0.858	[illegible]
8A	73.2	14.1	49.76	1.000	0.954	53.35	1.07	0.858	[illegible]
9A	151.9	11.5	46.31	0.993	1.000	62.27	1.34	0.849	[illegible]
10A	149.2	12.0	52.23	0.994	1.000	61.25	1.17	0.836	[illegible]

Table 8

$\bar{x} = 1.184$

$s = 0.090$

*Excluded from $\bar{x}$ calcula[illegible]

K VALUES FROM TEST

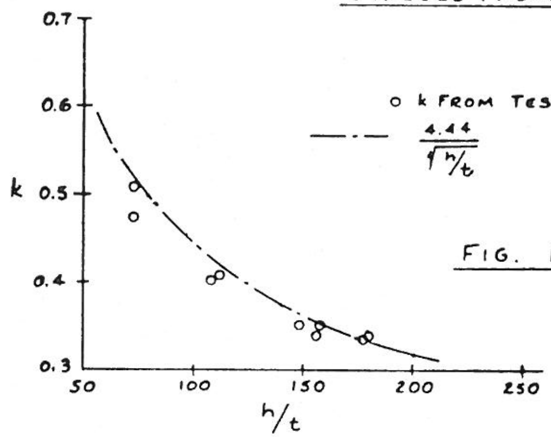
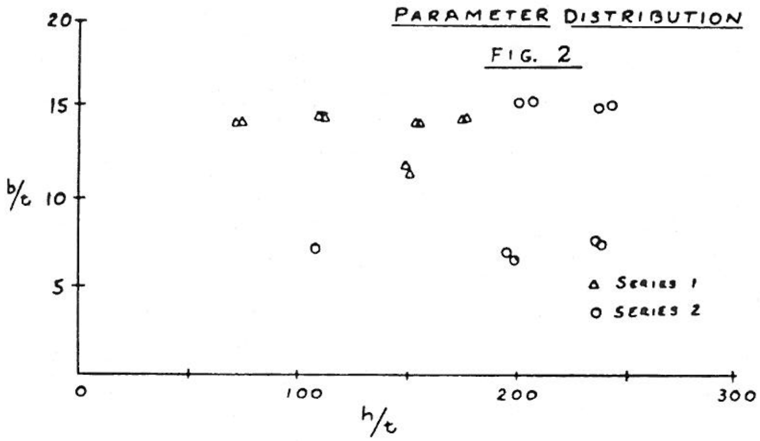


FIG. 1

PARAMETER DISTRIBUTION

FIG. 2



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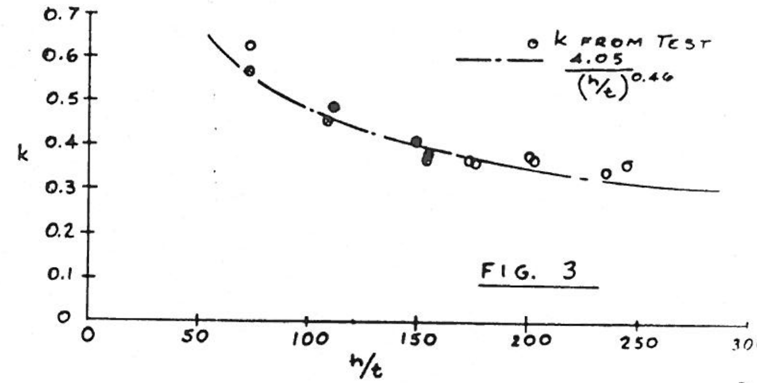

FIG. 3