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Cold-formed Steel Web Elements under Combined Bending and Shear

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COLD-FORMED STEEL WEB ELEMENTS UNDER COMBINED BENDING AND SHEAR

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INTRODUCTION

At the interior supports of continuous beams and the supports of cantilever beams, the web elements are subjected to a combination of maximum bending and high shear. This is a well-known fact and has been studied by numerous investigators (1-4,10) for plate girders with transverse stiffeners.

In order to determine the structural behavior of cold-formed steel beam webs without transverse stiffeners subjected to a combination of bending and shear, an experimental investigation was carried out at the University of Missouri-Rolla (UMR) under the sponsorship of the American Iron and Steel Institute (AISI). Prior to this phase of the investigation, studies were made to determine the ultimate strength of beam webs subjected to either bending or shear stress. The research findings were presented in Refs. 6, 7 and 9. This paper summarizes the test results and formulas developed from the study of cold-formed steel web elements subjected to combined bending and shear.

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EXPERIMENTAL INVESTIGATION

Twenty-five beam specimens were tested in this study. These specimens consisted of 8 built-up beam members (Fig. 1) and 17 modified beam members (Fig. 2). The beam specimens were fabricated from two channel sections connected by 3/4 x 3/4 x 1/8 in. (19.05 x 19.05 x 3.23 mm) angles at the compression flange, and by 1/8 x 3/4 in. (3.23 x 19.05 mm) rectangular bars at the tension flange. Self-tapping screws were used for connectors. Complete fabrication details are given in Ref. 6-8. The cross-section dimensions and pertinent parameters for each specimen are given in Tables 1 and 2.

Instead of using conventional bearing plates, the loads and reactions were introduced directly into the beam web to prevent a bearing failure. This was accomplished by using the loading assembly shown in Figs. 3, 4, and 5. By using this assembly the loads and reactions were applied through bearing plates to hot-rolled channels or 3/4 in. (19.05 mm) plates which transferred the loads and reactions to the beam webs by 1/2 in. (12.7 mm) diameter threaded rods. Spacers, 1/2 in. (12.7 mm) long, were used to avoid direct contact between the loading assembly and the test specimen.

Each specimen was tested as a simply supported beam subjected to a concentrated load at mid-span (Fig. 6). A detailed description of the test procedure is presented in Ref. 8. All beam specimens were tested to failure and these failure loads, $(P_u)_{test}$ are given in Table 3.

For each test specimen, the experimentally determined bending capacity, $(M_u)_{test}$, (Fig. 7) and the maximum tested compressive stress in bending, f_{bw}, computed on the basis of $(P_u)_{test}$ are also given in Table 3.

The ultimate shear at failure, $(V_u)_{test}$, is presented in Table 3 along with the average shear stress at failure, f_v, calculated on the basis of $(V_u)_{test}$.

EVALUATION OF TEST RESULTS

The objective of this study was to investigate the structural behavior of an unreinforced web element subjected to combined bending and shear. However, during the evaluation of the test data, it was realized that for the 17 modified specimens, which had steel sheets fastened to the top and bottom flanges, additional edge restraint was provided for the web element. This increased restraint improved the postbuckling strengths of the beam webs. Also, the loading system used in the tests enabled the formation of diagonal tension field action. For these reasons, due consideration must be given to the postbuckling strengths of the web for both bending and shear in order to obtain a good correlation between the tested and predicted ultimate strength.

For beam webs subjected to pure bending, the postbuckling strength factors were computed by using Eq. 1 (6,9). They are given in Table 4 for all test specimens.

$$\phi_b = \alpha_1\alpha_2\alpha_3\alpha_4 \tag{1}$$

in which $\alpha_1 = 0.017\ (h/t) - 0.790$

$\alpha_2 = 0.462\ |f_c/f_t| + 0.538$

$$\alpha_3 = 1.16 - 0.16\ [(w/t)/(w/t)_{lim}],\ \text{when}\ (w/t)/(w/t)_{lim} \leq 2.25$$

$$= 0.80,\ \text{when}\ (w/t)/(w/t)_{lim} > 2.25$$

$$\alpha_4 = 0.561\ (F_y/33) + 0.10$$

$(w/t)_{lim} = 171/\sqrt{f}$ according to Section 2.3.1.1 of the AISI Specification (11).

In the preceding equations, h = clear distance between flanges measured along the plane of the web, t = thickness of base steel, f_c = compressive bending stress in the web, f_t = tensile bending stress in the web, w = flat width of compression flange, f = actual stress in compression flange, and F_y = yield point of steel. The unit for all stresses is kips per square inch.

Consequently, the maximum compressive stress in bending is computed by Eq. 2 and given in Table 4.

$$(f_b)_{max} = \Phi_b\ (f_{cr})_{bw} \leq F_y \tag{2}$$

In Eq. 2, $(f_{cr})_{bw}$ = the critical web buckling stress due to bending determined by Eq. 3 (Table 4).

$$(f_{cr})_{bw} = \frac{k\pi^2 E}{12(1-\mu^2)} \left(\frac{t}{h}\right)^2 \tag{3}$$

in which E = modulus of elasticity, μ = Poisson's ratio, and k = buckling coefficient computed by using Eq. 4 (12)

$$k = 4 + 2(1+\beta)^3 + 2(1+\beta) \tag{4}$$

in which $\beta = |f_t/f_c|$.

For welded plate girders subjected to shear stress, the ultimate stress, τ_u, was evaluated by Gaylord, Fujii, and Selberg using the following formula (5):

$$\tau_u = (f_{cr})_v + F_y[1.0 - \frac{(f_{cr})_v}{\tau_y}][\frac{\sin\theta}{2+\cos\theta}] \tag{5}$$

in which θ = the angle of the web panel diagonal with the flange, τ_y = shear yield stress, and $(f_{cr})_v$ = the critical shear buckling stress given by Eq. 6.

$$(f_{cr})_v = \frac{k\pi^2 E}{12(1-\mu^2)} (\frac{t}{h})^2 \tag{6}$$

in which $k = 4.00 + \frac{5.34}{\alpha^2}$, when $\alpha \leq 1.0$

$= 5.34 + \frac{4.00}{\alpha^2}$, when $\alpha > 1.0$

α = aspect ratio = a/h

By substituting for $\sin\theta$ and $\cos\theta$ in terms of the aspect ratio α, the following expression for the postbuckling strength factor for shear, Φ_v, was obtained from Eqs. 5 and 6.

$$\Phi_v = \frac{\tau_u}{(f_{cr})_v} = 1.0 + \frac{\sqrt{3}\ [\frac{\tau_y}{(f_{cr})_v} - 1.0]}{2\sqrt{1+\alpha^2} + \alpha} \tag{7}$$

Table 4 contains the computed values of Φ_v for all test specimens.

Although the postbuckling strength factor for shear was derived from research findings conducted on welded plate girders, it appears to be equally applicable to the cold-formed steel sections used in this test program. This is demonstrated by the values of Φ_v'/Φ_v given in

Table 5 which are approximately equal to unity, for the specimens that failed by combined bending and shear. In the above expression, Φ_v' is defined as $(V_u)_{test}/(f_{cr})_v A_w$, where A_w is the cross-sectional area of the beam webs.

Consequently, the maximum shear stresses for all test specimens were determined by using Eq. 8 and are listed in Table 4.

$$(f_v)_{max} = \Phi_v (f_{cr})_v \leq \tau_y \tag{8}$$

Based on the maximum stresses for bending and shear (Eqs. 2 and 8), the relationship between $f_{bw}/(f_b)_{max}$ and $f_v/(f_v)_{max}$ is shown in Fig. 8 for all test results. This interaction is conveniently represented by the trilinear diagram ABCD, in which line BC is defined by the following formula:

$$0.8 \frac{f_{bw}}{(f_b)_{max}} + \frac{f_v}{(f_v)_{max}} = 1.4 \tag{9}$$

Figure 8 indicates that for values of $f_v/(f_v)_{max} < 0.6$, no reduction in the bending capacity occurs as the result of shear. Also, when $f_{bw} \leq 0.5\ (f_b)_{max}$, the shear capacity of the beam web is not significantly affected by bending.

The well known interaction curve represented by a quadrant of a unit circle, as defined by the following formula, is also presented in Fig. 8.

$$\left[\frac{f_{bw}}{(f_b)_{max}}\right]^2 + \left[\frac{f_v}{(f_v)_{max}}\right]^2 = 1.0 \tag{10}$$

From Fig. 8, it can be observed that the preceding equation provides a slightly conservative estimate of the true interaction behavior of the test specimens. This is possibly due to the fact that Eq. 10 was derived for the critical buckling stress in combined bending and shear (13), whereas the test data was based on the ultimate strength of web elements.

In addition to the study of interaction between bending and shear stresses, an evaluation was made of the relationship between bending moment and shear force. In this investigation, the predicted ultimate moments for the beam specimens, $(M_u)_{comp}$, were evaluated by using the effective web depth method as discussed in Refs. 6 and 9. The numerical values of $(M_u)_{comp}$ are given in Table 4.

The ultimate shear force is computed by the following formula:

$$(V_u)_{comp} = (f_v)_{max} A_w \tag{11}$$

The numerical values of $(V_u)_{comp}$ are also given in Table 4.

Figure 9 presents graphically the relationship between the quantities $(M_u)_{test}/(M_u)_{comp}$ and $(V_u)_{test}/(V_u)_{comp}$ as given in Table 5 for all test specimens. From a regression analysis of the test data, the interaction curve is represented by ABCD, where curve BC is represented by Eq. 12.

$$1.077\{[\frac{(M_u)_{test}}{(M_u)_{comp}}]^2 - 0.732\ \frac{(M_u)_{test}}{(M_u)_{comp}}\} + 1.100\ \frac{(V_u)_{test}}{(V_u)_{comp}} = 1.0 \tag{12}$$

A simplified interaction diagram was developed and is depicted by AB'C'D, which is also shown in Fig. 9. Line B'C' is defined by Eq. 13

$$\frac{(M_u)_{test}}{(M_u)_{comp}} + \frac{(V_u)_{test}}{(V_u)_{comp}} = 1.6 \tag{13}$$

Also presented in Fig. 9 is the interaction curve represented by a part of a unit circle and defined by Eq. 14;

$$[\frac{(M_u)_{test}}{(M_u)_{comp}}]^2 + [\frac{(V_u)_{test}}{(V_u)_{comp}}]^2 = 1.0 \qquad (14)$$

As shown in Fig. 9, when the shear force is less than approximately 65 percent of the computed shear capacity, the full bending capacity of the member was developed. Conversely, when the bending moment in the member is less than approximately 50 percent of the predicted maximum bending capacity, the moment has little or no effect on the shear capacity. It can also be observed from this figure that the interaction curve described by Eq. 14 offers a conservative estimate of the true interaction behavior of the test specimens.

SUMMARY

This investigation was initiated to study the structural behavior of cold-formed steel beam webs subjected to combined bending and shear. Formulas were derived and are presented for the interaction behavior of the test specimens used in this study. In developing these formulas, due consideration was given to the postbuckling strengths of web elements for both bending and shear.

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APPENDIX I.--REFERENCES

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APPENDIX II.--NOTATIONS

The following symbols are used in this paper:

A_w = cross-sectional area of beam web, in square inches;

a = length of web element, in inches;

E = modulus of elasticity, in kips per square inch;

F_y = tensile yield point, in kips per square inch;

f = actual stress in compression flange, in kips per square inch;

$(f_b)_{max}$ = maximum computed stress governing bending, in kips per square inch;

f_{bw} = bending stress at failure in the web, in kips per square inch;

f_c = compressive bending stress in web, in kips per square inch;

$(f_{cr})_{bw}$ = critical buckling stress for bending, in kips per square inch;

$(f_{cr})_v$ = critical shear buckling stress, in kips per square inch;

f_t = tensile bending stress in web, in kips per square inch;

f_v = average shear stress at failure, in kips per square inch;

$(f_v)_{max}$ = maximum computed stress governing shear for reinforced web, in kips per square inch;

h = clear distance between flanges measured along the plane of the web, in inches;

k = buckling coefficient;

$(M_u)_{test}$ = tested ultimate bending moment, in inch-kips;

$(M_u)_{comp}$ = computed ultimate bending moment, in inch-kips;

$(P_u)_{test}$ = failure load, in kips;

t = thickness of base steel, in inches;

$(V_u)_{test}$ = shear force at failure, in kips;

$(V_u)_{comp}$ = computed ultimate shear force, in kips;

w = flat width of compression flange, in inches;

α = aspect ratio of web;

α_1 = postbuckling factor for h/t;

α_2 = postbuckling factor for f_c/f_t;

α_3 = postbuckling factor for $w/t/(w/t)_{lim}$;

α_4 = postbuckling factor for F_y;

β = $|f_t/f_c|$;

θ = angle of the web panel diagonal with the flange;

μ = Poisson's ratio;

τ_u = computed ultimate shear stress, in kips per square inch;

τ_y = shear yield stress, in kips per square inch;

Φ_b = postbuckling strength factor for bending;

Φ_v = postbuckling strength factor for shear; and

Φ_v' = tested postbuckling strength factor for shear.

LIST OF CAPTIONS

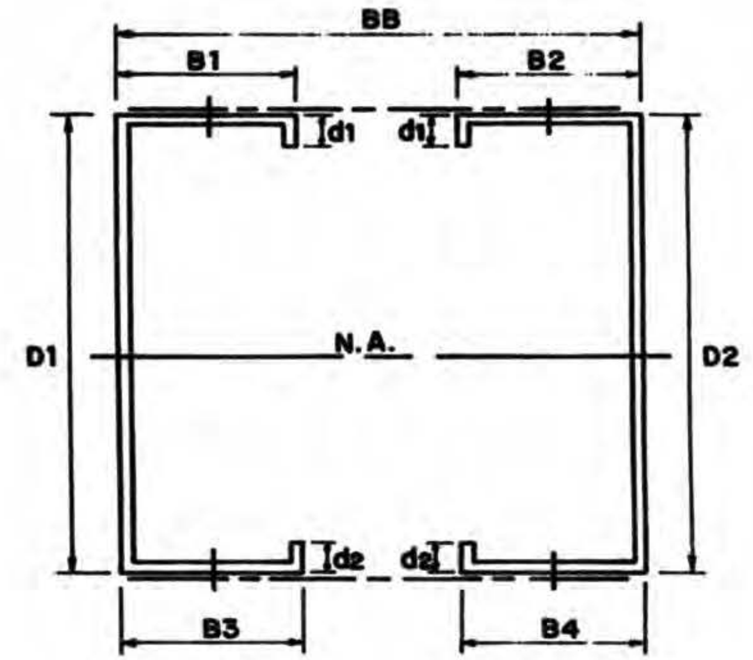
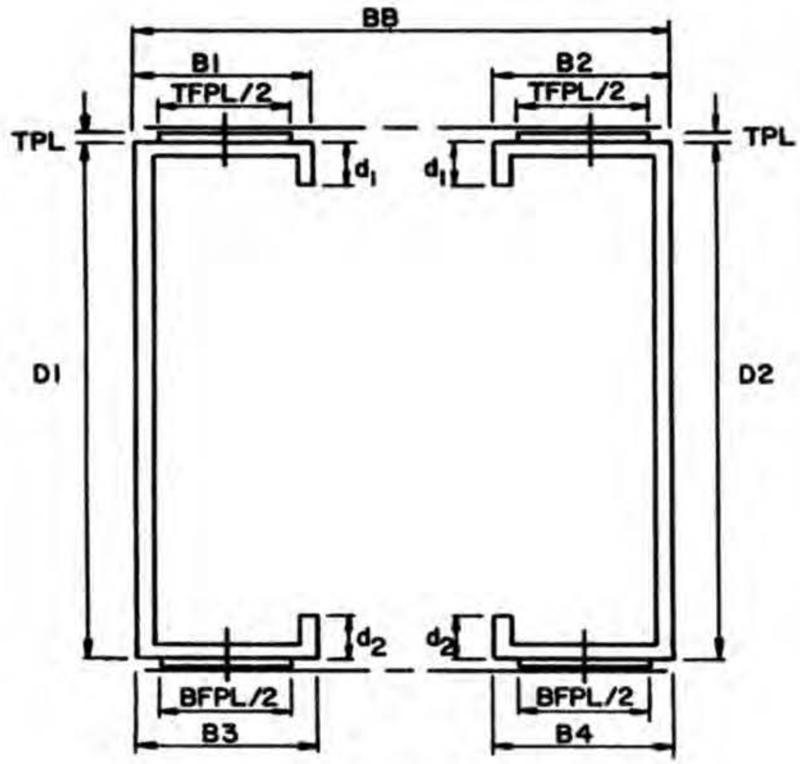


Fig. 1



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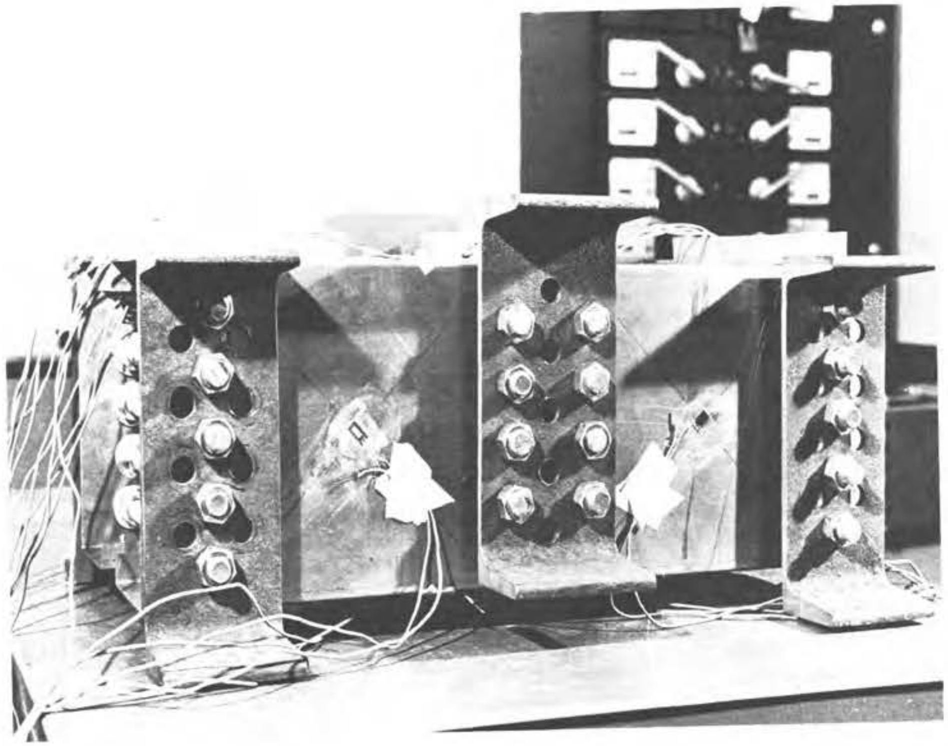

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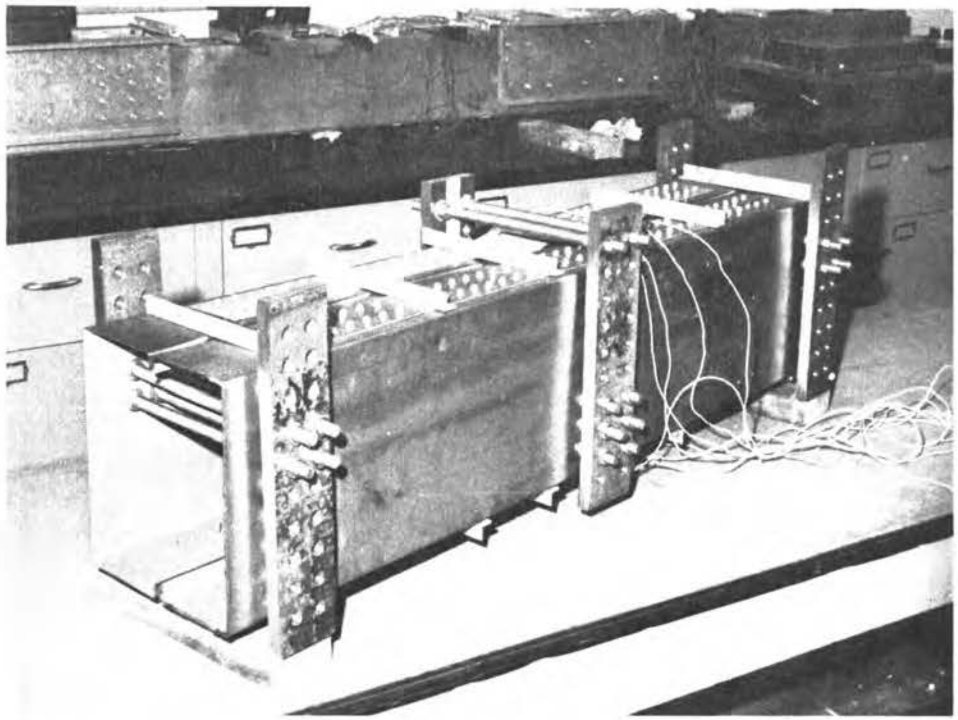


Fig. 4

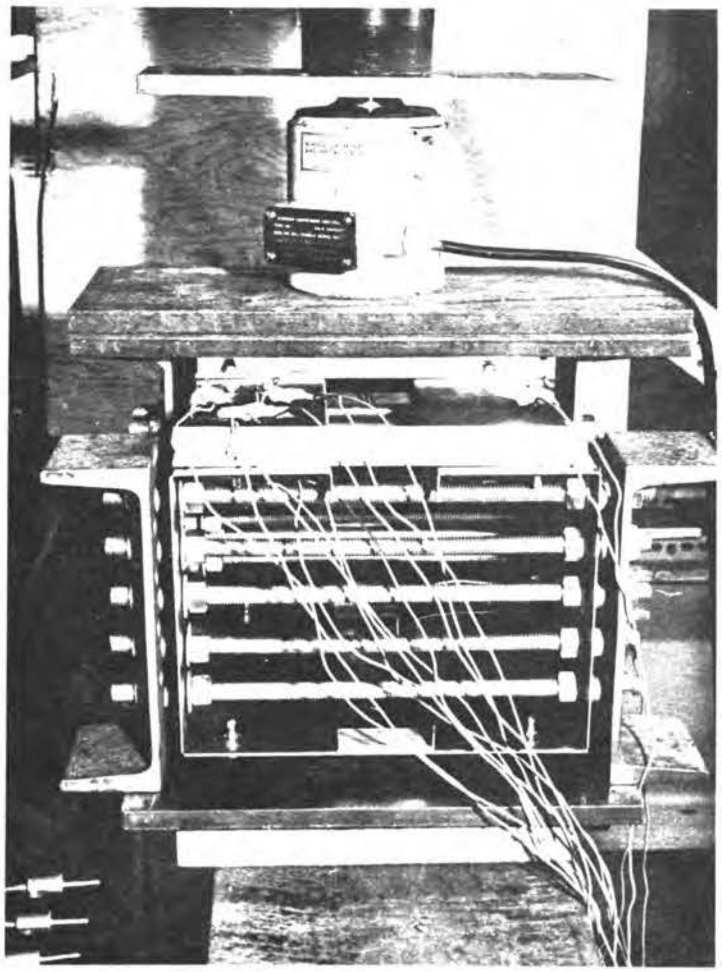


Fig. 5

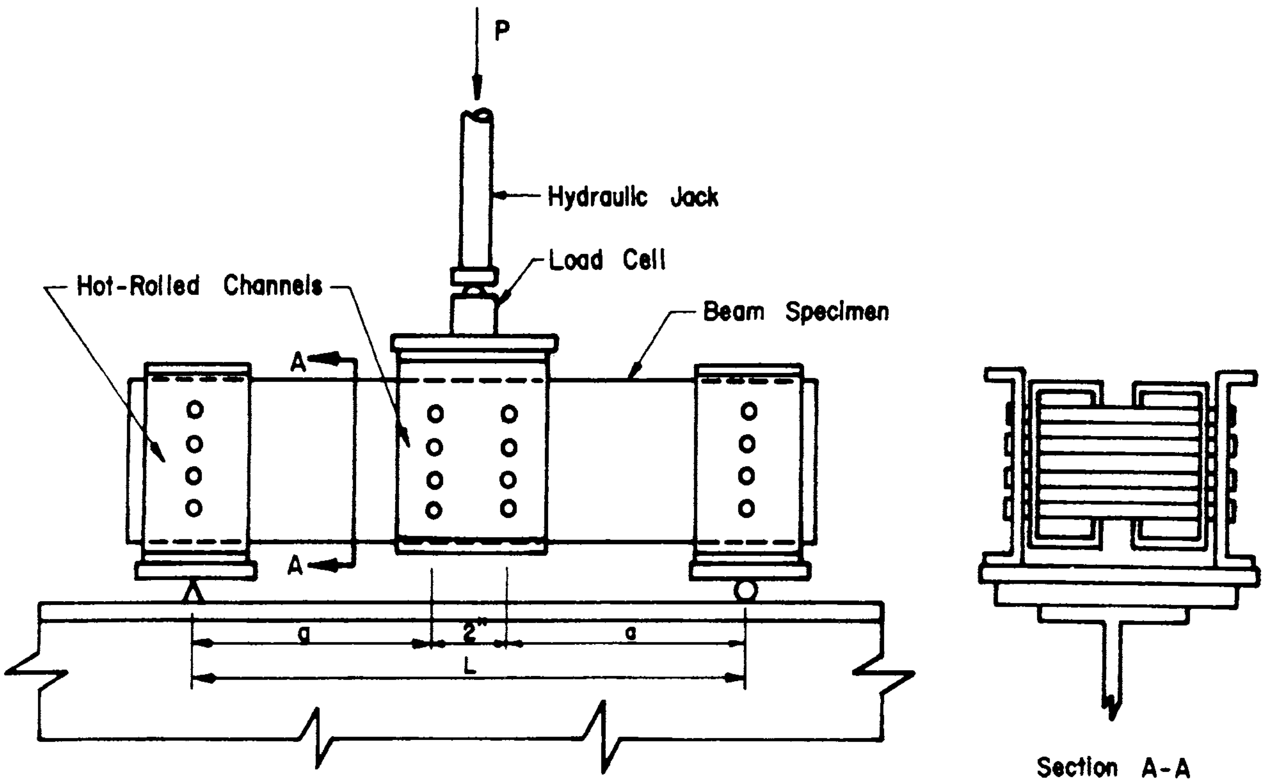
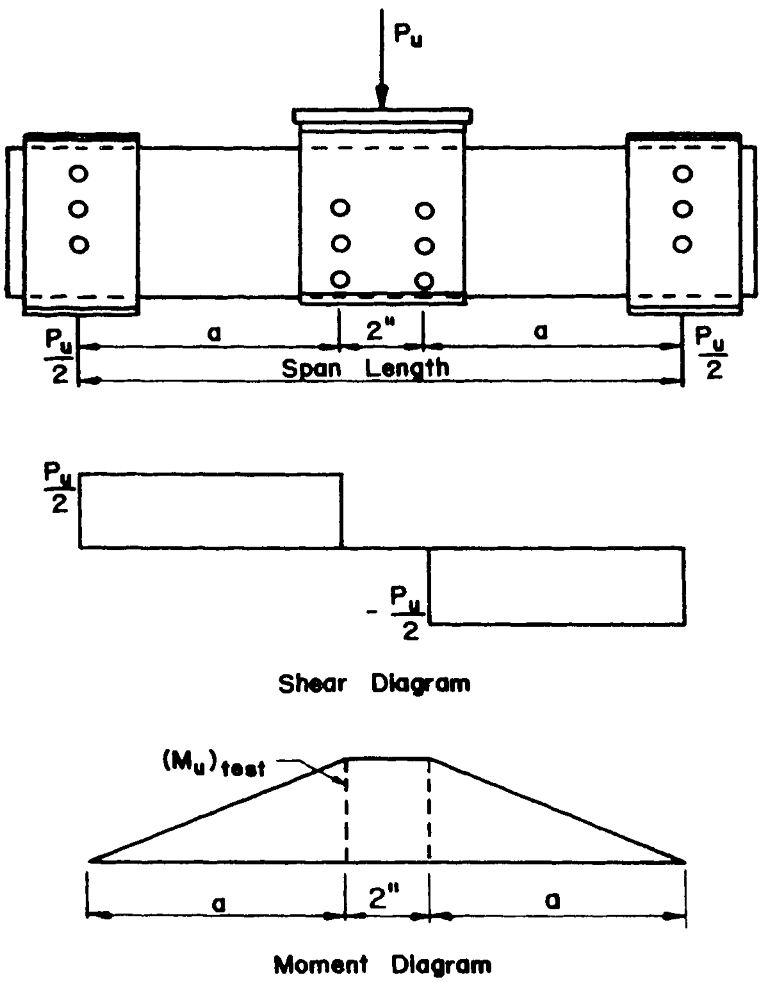
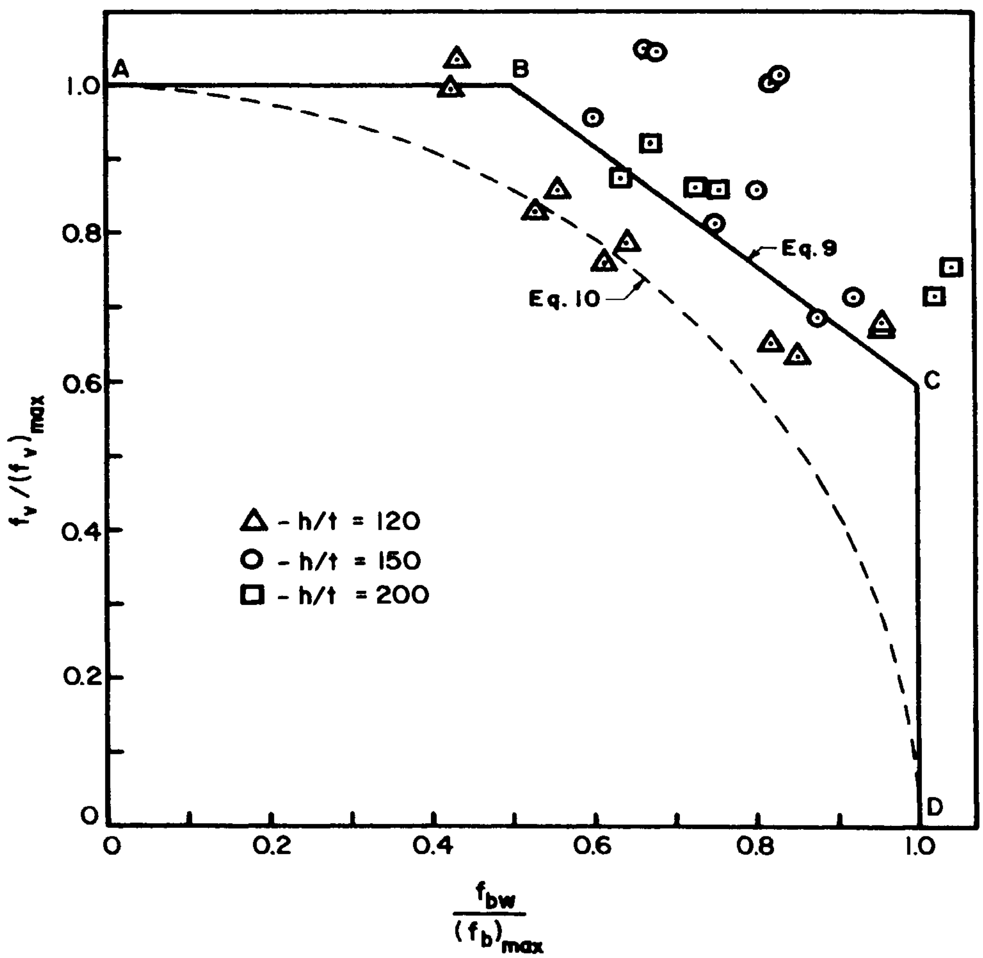

P
Hydraulic Jack
Load Cell
Hot-Rolled Channels
Beam Specimen
A
A
a
2"
a
L
Section A-A

Fig. 6



P_u
$\frac{P_u}{2}$
a
2"
a
$\frac{P_u}{2}$
Span Length
$\frac{P_u}{2}$
$-\frac{P_u}{2}$
Shear Diagram
$(M_u)_{test}$
a
2"
a
Moment Diagram

Fig. 7



A
B
C
D
Eq. 9
Eq. 10
1.0
0.8
0.6
0.4
0.2
0
0.2
0.4
0.6
0.8
1.0
$f_v/(f_v)_{max}$
△ - h/t = 120
○ - h/t = 150
□ - h/t = 200
$\frac{f_{bw}}{(f_b)_{max}}$

Fig. 8

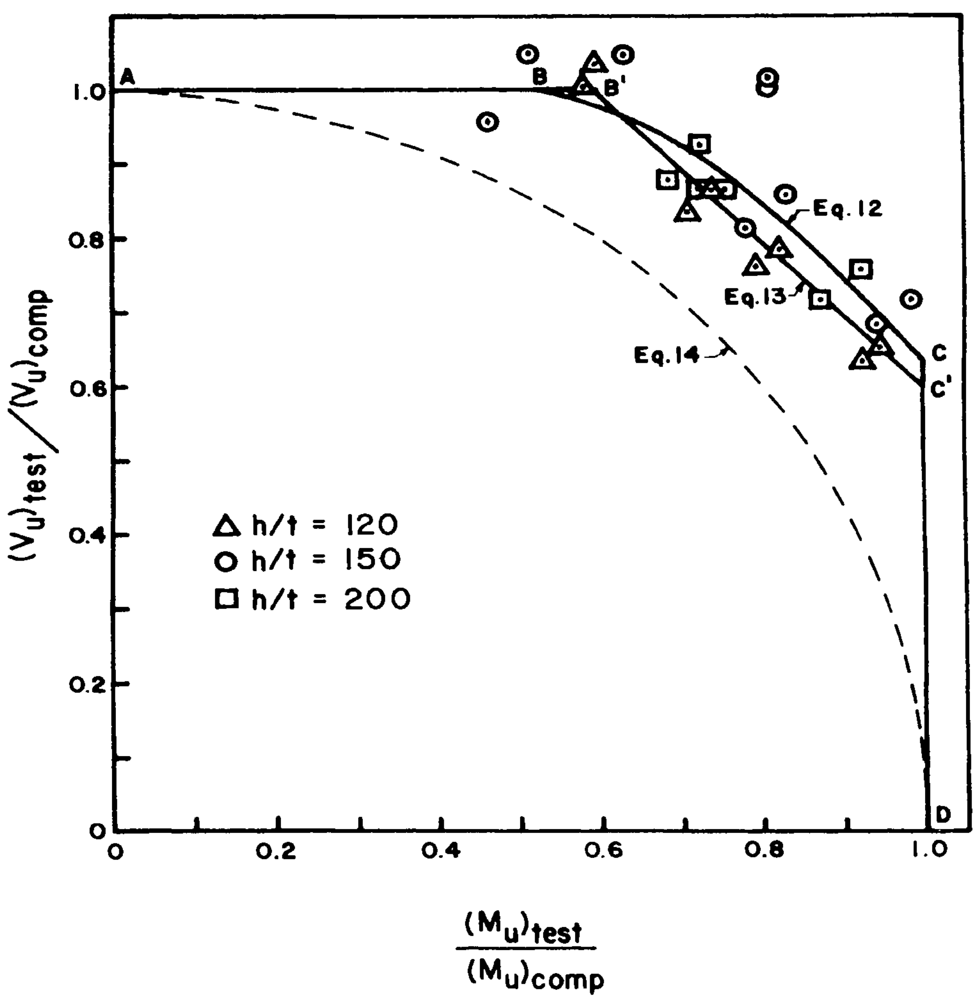

A
B
B'
C
C'
D
Eq. 12
Eq. 13
Eq. 14
△ h/t = 120
○ h/t = 150
□ h/t = 200
1.0
0.8
0.6
0.4
0.2
0
0.2
0.4
0.6
0.8
1.0
$(V_u)_{test} / (V_u)_{comp}$
$\frac{(M_u)_{test}}{(M_u)_{comp}}$

Fig. 9

TABLE 1

CROSS-SECTION DIMENSIONS OF TEST SPECIMENS SUBJECTED TO COMBINED BENDING AND SHEAR

Beam specimen	Cross-Section Dimensions, in inches												
No. (1)	Thick. (2)	B1 (3)	B2 (4)	B3 (5)	B4 (6)	d1 (7)	d2 (8)	D1 (9)	D2 (10)	TFPL (11)	BFPL (12)	TPL (13)	BB (14)
BS-2-1a	0.0500	1.950	1.974	1.955	1.957	0.610	0.625	6.163	6.130	--	--	--	6
BS-2-1b	0.0500	1.968	1.975	1.964	1.973	0.610	0.613	6.150	6.145	--	--	--	6
BS-8-1a	0.0509	2.979	2.996	3.002	2.965	0.595	0.610	6.121	6.152	--	--	--	9
BS-8-1b	0.0509	2.979	2.995	2.999	2.995	0.606	0.604	6.145	6.132	--	--	--	9
BS-8-2a	0.0500	2.988	2.983	2.984	2.998	0.600	0.610	6.117	6.135	3.613	3.544	0.0492	9
BS-8-2b	0.0500	2.982	2.985	2.967	2.979	0.627	0.613	6.122	6.113	3.613	3.544	0.0492	9
BS-8-3a	0.0504	2.997	2.984	2.974	3.002	0.596	0.602	6.153	6.135	6.921	6.918	0.0470	9
BS-8-3b	0.0504	2.994	2.973	2.975	2.991	0.609	0.616	6.152	6.144	6.918	6.921	0.0470	9
BS-8-4a	0.0502	2.989	2.997	2.977	2.980	0.613	0.614	6.184	6.196	7.135	7.173	0.0940	9
BS-8-4b	0.0502	2.998	2.992	2.999	2.992	0.615	0.594	6.186	6.183	7.135	7.173	0.0940	9
BS-9-1a	0.0510	3.512	3.474	3.489	3.505	0.609	0.626	7.281	7.303	--	--	--	9
BS-9-1b	0.0509	3.475	3.482	3.482	3.497	0.602	0.632	7.328	7.321	--	--	--	9
BS-9-2a	0.0510	3.506	3.510	3.497	3.505	0.614	0.610	7.295	7.283	1.810	1.772	0.0492	9
BS-9-2b	0.0509	3.513	3.481	3.502	3.512	0.607	0.616	7.313	7.281	1.810	1.772	0.0492	9
BS-9-3a	0.0511	3.511	3.501	3.506	3.481	0.607	0.604	7.303	7.325	3.491	3.398	0.0470	9
BS-9-3b	0.0509	3.510	3.503	3.508	3.484	0.612	0.614	7.314	7.313	3.469	3.439	0.0470	9
BS-9-4	0.0511	3.476	3.508	3.524	3.495	0.606	0.607	7.266	7.269	3.449	3.460	0.0940	9
BS-9-5	0.0511	3.495	3.487	3.481	3.511	0.622	0.609	7.301	7.278	3.449	3.460	0.1310	9
BS-9-6	0.0511	3.501	3.507	3.463	3.507	0.605	0.601	7.316	7.318	3.449	3.460	0.1310	9
BS-10-1a	0.0504	4.507	4.508	4.508	4.486	0.683	0.690	9.852	9.890	--	--	--	9.125
BS-10-1b	0.0504	4.493	4.517	4.498	4.507	0.680	0.694	9.836	9.892	--	--	--	9.125

TABLE 1

CROSS-SECTION DIMENSIONS OF TEST SPECIMENS SUBJECTED TO COMBINED BENDING AND SHEAR
(Continued)

Beam specimen No. (1)	Cross-Section Dimensions, in inches												
	Thick. (2)	B1 (3)	B2 (4)	B3 (5)	B4 (6)	d1 (7)	d2 (8)	D1 (9)	D2 (10)	TFPL (11)	BFPL (12)	TPL (13)	BB (14)
BS-10-2a	0.0496	4.498	4.501	4.524	4.484	0.670	0.680	9.896	9.900	3.971	3.959	0.0496	9.125
BS-10-2b	0.0497	4.517	4.500	4.504	4.505	0.680	0.676	9.844	10.036	3.971	3.959	0.0496	9.125
BS-10-3a	0.0497	4.519	4.505	4.502	4.506	0.666	0.642	9.911	9.833	5.962	5.942	0.0500	9.125
BS-10-3b	0.0496	4.484	4.497	4.475	4.500	0.702	0.679	9.960	9.895	5.962	5.942	0.0500	9.125

Notes:
1. 1 in. = 25.4 mm.
2. See Figs. 1 and 2 for symbols used for dimensions.
3. Inside bend radius is assumed to be equal to the thickness.
4. Combined bending and shear specimens are designated as follows:

BS	-	10	-	1a
Bending and Shear Beam Section		Channel No.		Test No.

TABLE 2

PERTINENT PARAMETERS OF TEST SPECIMENS SUBJECTED TO COMBINED BENDING AND SHEAR

Beam specimen No. (1)	F_y, in kips per square inch (2)	τ_y, in kips per square inch (3)	w/t (4)	$(w/t)_{lim}$ (5)	h/t (6)	a/h (7)	Span length, in inches (8)
BS-2-1a	47.12	27.20	35.48	32.16	121.26	2.02	27.75
BS-2-1b	47.12	27.20	35.50	32.16	121.00	2.02	27.75
BS-8-1a	47.12	27.20	54.86	32.16	118.86	2.02	27.75
BS-8-1b	47.12	27.20	54.84	32.16	118.73	2.03	27.75
BS-8-2a	47.12	27.20	55.76	32.16	120.70	2.03	27.75
BS-8-2b	47.12	27.20	55.70	32.16	120.44	2.03	27.75
BS-8-3a	47.12	27.20	54.46	32.16	120.08	2.02	27.75
BS-8-3b	47.12	27.20	55.40	32.16	120.06	2.02	27.75
BS-8-4a	47.12	27.20	55.48	32.16	121.43	2.01	27.75
BS-8-4b	47.12	27.20	55.72	32.16	121.23	2.01	27.75
BS-9-1a	47.12	27.20	64.86	32.16	141.20	2.08	27.75
BS-9-1b	47.12	27.20	64.41	32.16	141.97	2.08	27.75
BS-9-2a	47.12	27.20	64.82	32.16	141.04	2.09	27.75
BS-9-2b	47.12	27.20	65.02	32.16	141.67	2.08	27.75
BS-9-3a	47.12	27.20	64.71	32.16	141.35	2.08	27.75
BS-9-3b	47.12	27.20	64.96	32.16	141.69	2.08	27.75
BS-9-4	47.12	27.20	64.65	32.16	142.25	2.06	27.75
BS-9-5	47.12	27.20	64.40	32.16	140.88	2.08	27.75
BS-9-6	47.12	27.20	64.63	32.16	141.21	2.08	27.75
BS-10-1a	36.88	21.29	85.44	36.35	194.23	1.99	21.29
BS-10-1b	36.88	21.29	85.62	36.35	194.27	1.99	21.29

TABLE 2

PERTINENT PARAMETERS OF TEST SPECIMENS SUBJECTED TO COMBINED BENDING AND SHEAR (Continued)

Beam specimen No. (1)	F_y, in kips per square inch (2)	τ_y, in kips per square inch (3)	w/t (4)	$(w/t)_{lim}$ (5)	h/t (6)	a/h (7)	Span length, in inches (8)
BS-10-2a	36.88	21.29	86.75	36.35	197.60	1.99	21.29
BS-10-2b	36.88	21.29	86.89	36.35	199.93	1.96	21.29
BS-10-3a	36.88	21.29	86.93	36.35	197.42	1.99	21.29
BS-10-3b	36.88	21.29	86.67	36.35	198.81	1.98	21.29

Note: 1 in. = 25.4 mm; 1 ksi = 6.9 MN/m^2.

TABLE 3

EXPERIMENTAL DATA FOR TEST SPECIMENS SUBJECTED TO COMBINED BENDING AND SHEAR

Beam specimen No. (1)	$(P_u)_{test}$, in kips (2)	$(M_u)_{test}$, in inch-kips (3)	$(V_u)_{test}$, in kips (4)	f_{bw}, in kips per square inch (5)	f_v, in kips per square inch (6)	Failure Mode (7)
BS-2-1a	12.91	83.11	6.46	45.01	10.68	B
BS-2-1b	12.98	83.56	6.49	45.20	10.73	B
BS-8-1a	12.63	81.31	6.32	37.39	10.26	B
BS-8-1b	13.01	83.75	6.51	38.63	10.58	B
BS-8-2a	15.00	96.56	7.50	30.27	12.43	B
BS-8-2b	14.53	93.54	7.27	29.17	12.07	B
BS-8-3a	16.67	107.38	8.34	26.07	13.67	BS
BS-8-3b	16.01	103.00	8.01	24.89	13.13	BS
BS-8-4a	19.80	123.60	9.60	19.94	15.69	BS
BS-8-4b	19.74	127.07	9.87	20.55	16.15	BS
BS-9-1a	13.50	105.47	6.75	36.29	9.19	B
BS-9-1b	14.07	109.92	7.04	38.07	9.57	B
BS-9-2a	15.98	124.84	7.99	35.55	10.89	B
BS-9-2b	16.80	131.28	8.40	37.77	11.44	B
BS-9-3a	20.00	156.25	10.00	39.24	13.55	BS
BS-9-3b	19.67	153.67	9.84	38.69	13.40	BS
BS-9-4	21.00	164.06	10.50	32.01	14.13	S
BS-9-5	18.93	147.89	9.47	28.35	12.87	BS
BS-9-6	20.74	162.03	10.37	31.29	14.06	S
BS-10-1a	12.62	123.05	6.31	26.57	6.39	B
BS-10-1b	12.82	125.00	6.41	27.12	6.49	b

TABLE 3

EXPERIMENTAL DATA FOR TEST SPECIMENS SUBJECTED TO COMBINED BENDING AND SHEAR (Continued)

Beam specimen No. (1)	$(P_u)_{test}$, in kips (2)	$(M_u)_{test}$, in inch-kips (3)	$(V_u)_{test}$, in kips (4)	f_{bw}, in kips per square inch (5)	f_v, in kips per square inch (6)	Failure Mode (7)
BS-10-2a	15.03	146.64	7.52	23.05	7.73	BS
BS-10-2b	15.15	147.81	7.58	23.52	7.67	BS
BS-10-3a	16.02	156.20	8.01	21.44	8.21	BS
BS-10-3b	15.18	148.00	7.59	20.00	7.76	BS

Note: 1. 1 kip = 4.45 kN; 1 ksi = 6.9 MN/m^2; 1 in.-kip = 113 N·m.
2. Failure modes are designated as follows:
B-bending, S-shear, BS-combined bending and shear.

TABLE 4

THEORETICAL DATA FOR TEST SPECIMENS SUBJECTED TO COMBINED BENDING AND SHEAR

Beam specimen No. (1)	$(M_u)_{comp}$, in inch-kips (2)	$(f_{cr})_{bw}$, in kips per square inch (3)	$(f_{cr})_v$, in kips per square inch (4)	ϕ_b (5)	ϕ_v (6)	$(V_u)_{comp}$, in kips (7)	$(f_v)_{max}$, in kips per square inch (8)	$(f_b)_{max}$, in kips per square inch (9)
BS-2-1a	84.49	43.34	11.46	1.13	1.36	9.43	15.59	47.12
BS-2-1b	84.57	43.52	11.51	1.13	1.36	9.47	15.65	47.12
BS-8-1a	88.09	45.11	11.93	1.04	1.34	9.85	15.99	46.91
BS-8-1b	88.43	45.20	11.94	1.04	1.34	9.84	16.00	47.01
BS-8-2a	117.45	43.74	11.55	1.19	1.36	9.48	15.71	47.12
BS-8-2b	117.73	43.93	11.60	1.19	1.36	9.50	15.78	47.12
BS-8-3a	145.08	44.19	11.69	1.17	1.35	9.63	15.78	47.12
BS-8-3b	145.30	44.21	11.69	1.18	1.35	9.63	15.78	47.12
BS-8-4a	213.04	43.22	11.45	1.35	1.37	9.60	15.69	47.12
BS-8-4b	212.94	43.36	11.48	1.19	1.36	9.54	15.61	47.12
BS-9-1a	112.09	31.96	8.38	1.29	1.58	9.73	13.24	41.23
BS-9-1b	112.10	31.62	8.29	1.31	1.59	9.70	13.18	41.42
BS-9-2a	158.75	32.03	8.38	1.53	1.58	9.71	13.24	47.12
BS-9-2b	157.65	31.75	8.32	1.54	1.59	9.71	13.23	47.12
BS-9-3a	191.00	31.89	8.36	1.53	1.58	9.75	13.21	47.12
BS-9-3b	189.68	31.74	8.32	1.54	1.59	9.71	13.23	47.12
BS-9-4	252.79	31.49	8.27	1.50	1.60	9.83	13.23	47.12
BS-9-5	316.34	32.11	8.42	1.51	1.58	9.79	13.30	47.12
BS-9-6	317.31	31.96	8.38	1.52	1.58	9.76	13.24	47.12
BS-10-1a	140.82	16.89	4.49	1.54	2.01	8.91	9.02	26.01
BS-10-1b	134.72	16.88	4.49	1.54	2.01	8.91	9.02	26.00

TABLE 4

THEORETICAL DATA FOR TEST SPECIMENS SUBJECTED TO COMBINED BENDING AND SHEAR
(Continued)

Beam specimen No. (1)	$(M_u)_{comp}$, in inch-kips (2)	$(f_{cr})_{bw}$, in kips per square inch (3)	$(f_{cr})_v$, in kips per square inch (4)	ϕ_b (5)	ϕ_v (6)	$(V_u)_{comp}$, in kips (7)	$(f_v)_{max}$, in kips per square inch (8)	$(f_b)_{max}$, in kips per square inch (9)
BS-10-2a	202.97	16.32	4.34	1.96	2.05	8.65	8.90	31.99
BS-10-2b	195.01	15.94	4.26	1.96	2.09	8.79	8.90	31.24
BS-10-3a	214.52	16.35	4.34	1.94	2.05	8.68	8.90	31.72
BS-10-3b	215.72	16.12	4.29	1.95	2.07	8.69	8.88	31.43

Note: 1 kip = 4.45 kN; 1 ksi = 6.9 MN/m^2; 1 in.-kip = 113 N·m.

TABLE 5

COMPARISONS OF EXPERIMENTAL AND THEORETICAL DATA FOR TEST SPECIMENS SUBJECTED TO COMBINED BENDING AND SHEAR

Beam specimen No. (1)	$\frac{(M_u)_{test}}{(M_u)_{comp}}$ (2)	$\frac{(V_u)_{test}}{(V_u)_{comp}}$ (3)	$\frac{f_{bw}}{(f_b)_{max}}$ (4)	$\frac{f_v}{(f_v)_{max}}$ (5)	$\phi'_v = \frac{(V_u)_{test}}{(V_{cr})_{theo}}$ (6)	$\frac{\phi'_v}{\phi_v}$ (7)
BS-2-1a	0.984	0.685	0.955	0.685	--	--
BS-2-1b	0.988	0.685	0.959	0.685	--	--
BS-8-1a	0.923	0.642	0.797	0.642	--	--
BS-8-1b	0.947	0.662	0.822	0.661	--	--
BS-8-2a	0.822	0.791	0.642	0.791	--	--
BS-8-2b	0.795	0.765	0.619	0.765	--	--
BS-8-3a	0.740	0.867	0.553	0.866	1.17	0.867
BS-8-3b	0.709	0.832	0.528	0.832	1.12	0.830
BS-8-4a	0.580	1.000	0.423	1.000	1.37	1.000
BS-8-4b	0.597	1.035	0.436	1.035	1.41	1.037
BS-9-1a	0.941	0.694	0.880	0.694	--	--
BS-9-1b	0.981	0.726	0.919	0.726	--	--
BS-9-2a	0.786	0.823	0.754	0.823	--	--
BS-9-2b	0.833	0.865	0.802	0.865	--	--
BS-9-3a	0.818	1.026	0.833	1.026	1.60	1.012
BS-9-3b	0.810	1.013	0.821	1.013	1.60	1.006
BS-9-4	0.649	1.068	0.679	1.068	1.68	1.050
BS-9-5	0.468	0.967	0.602	0.968	1.51	0.956
BS-9-6	0.511	1.063	0.664	1.062	1.66	1.050
BS-10-1a	0.874	0.708	1.022	0.708	--	--
BS-10-1b	0.928	0.719	1.043	0.720	--	--

TABLE 5

COMPARISONS OF EXPERIMENTAL AND THEORETICAL DATA FOR TEST SPECIMENS SUBJECTED TO COMBINED BENDING AND SHEAR
(Continued)

Beam specimen No. (1)	$\frac{(M_u)_{test}}{(M_u)_{comp}}$ (2)	$\frac{(V_u)_{test}}{(V_u)_{comp}}$ (3)	$\frac{f_{bw}}{(f_b)_{max}}$ (4)	$\frac{f_v}{(f_v)_{max}}$ (5)	$\phi'_v = \frac{(V_u)_{test}}{(V_{cr})_{theo}}$ (6)	$\frac{\phi'_v}{\phi_v}$ (7)
BS-10-2a	0.722	0.869	0.721	0.869	1.77	0.864
BS-10-2b	0.758	0.862	0.753	0.862	1.80	0.861
BS-10-3a	0.728	0.923	0.676	0.922	1.89	0.922
BS-10-3b	0.686	0.873	0.636	0.874	1.81	0.874