Bracket Loaded Webs with Low Slenderness Ratios

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Welded bracket connections to the web of rolled sections are common connections. However, the AISC *Manual of Steel Construction* has no guidelines for estimating their capacity. A typical connection of this type is shown in Fig. 1. Generally, the maximum load corresponds to the development of a two-dimensional yield mechanism in the web; but for sections whose webs have significant thickness relative to the *T*-distance, the maximum load may occur in the flanges.

Abolitz and Warner[1] conducted a theoretical investigation into bending of seated connections. By applying work methods to several yield line patterns, they developed an expression for the lowest value of the ultimate load.

The expression:

$$P_u = (k m_e L)/e \tag{1}$$

is based on the two-dimensional yield line pattern shown in Fig. 1. Since Abolitz and Warner were unsure of the interaction between the flanges and the web, they developed expressions for plates with fixed edges or simply supported edges. Abolitz and Warner placed no restrictions on the slenderness ratio, μ, defined as the smaller of T/t_w or L/t_w.

In 1979 Joseph Hoptay[2,3] tested wide flange sections of large web slenderness ratios* (μ) between 34 and 48. He compared his experimental data with the solution of the expression for the ultimate load in Eq. 1 to determine if the expression is valid. Hoptay concluded that the expression gives reasonable results if the shape factor for simply supported edges:

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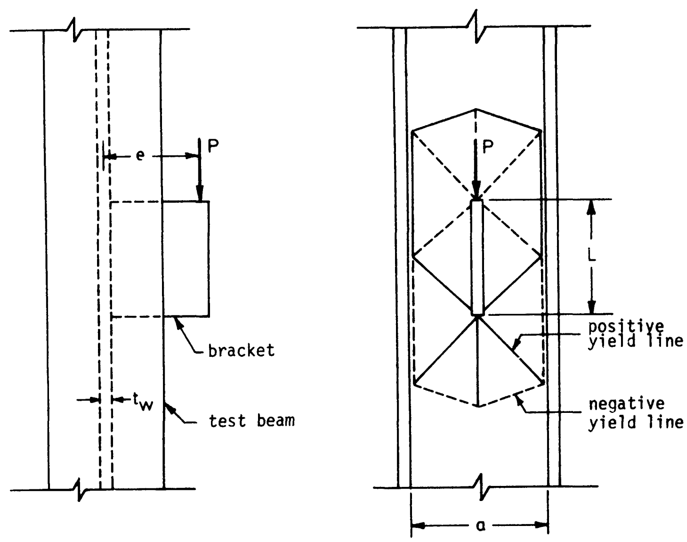

Fig. 1. Typical bracket connection and its yield line pattern at failure (Ref. 1).

$$k = 2a/L + 2L/a + 2\sqrt{7}$$

is used and the plastic moment capacity ($m_p = \sigma_y\, t_w^2\, /4$) is substituted for the value of the elastic moment capacity ($m_e = \sigma_y\, t_w^2\, /6$). Hoptay also concluded that membrane action is responsible for the load capacity beyond that predicted by Eq. 1. He speculated that the membrane effects would become less significant for sections with lower web slenderness ratios.

To establish the validity of Eq. 1 for sections with low web slenderness ratios, Walsh[4] tested wide flange sections with web slenderness ratios of 8 to 25. He compared his data to the solution of Eq. 1 and reached the same conclusions as Hoptay for sections with web slenderness ratios of 15 to 25. Walsh compared his data with Hoptay's and noted that membrane action is less signif-

**The web slenderness ratios (ratio of a section's T-distance to web thickness) for all the rolled sections listed in the Eighth Edition of the AISC Manual of Steel Construction range between 3 and 55.*

icant for sections with low web slenderness ratios. Walsh also noted that sections with web slenderness ratios less than 15 had a load capacity significantly lower than was predicted by Eq. 1. He attributed this to the development of a different failure mechanism which he was unable to define.

This paper presents the results of a research project which investigated bracket to web connections for sections with web slenderness ratios less than 15. The objectives of this project were to define the geometry of the new failure mechanism, establish the critical web slenderness ratio at which it becomes significant and to verify the validity of Eq. 1 for sections with web slenderness ratios less than 15.

TEST PROCEDURE

Five sections with web slenderness ratios less than 15 were chosen. Their properties are listed in Table 1. Two specimens with different bracket lengths were prepared from each section size. The bracket lengths were 8 and 12 in. The thickness of the brackets was chosen to be comparable to the web thicknesses, and the width was 2 in. more than the desired eccentricity of the load. Each bracket was welded, centered between the flanges with a fillet weld about the entire perimeter of the bracket. Stiffeners were welded to each side of the bracket to prevent buckling of the bracket before the web had failed. Each specimen was then mounted in the load frame shown in Fig. 2.

Strain gages were placed on the web where each of the idealized yield lines were expected to develop. The strain gage locations are shown in Fig. 3. Strain rosettes were used where the principal strain directions could not be anticipated. Each strain gage location was sanded to remove the rust and mill scale before the gages were bonded. Additional gages were located on the extreme edges of the flanges on the S4 × 9.5 section with the 12-in. bracket to verify the geometry of the new failure mechanism.

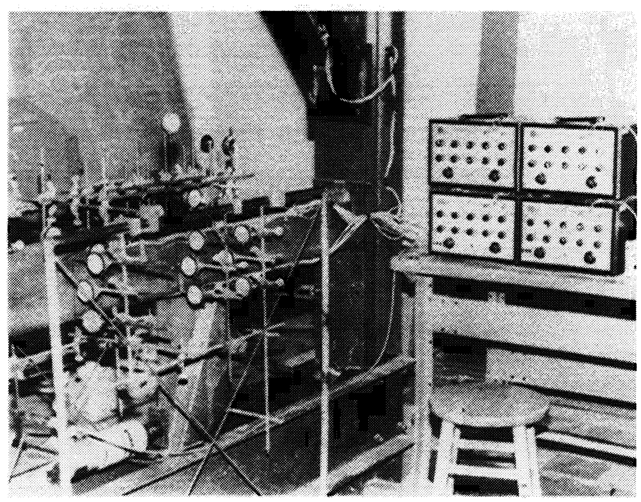

Fig. 2. Test set-up

Dial gages were used to measure the deflections of the web and flanges in each test. These gages were mounted on an independent test stand shown in Fig. 2 to avoid any interaction with the load frame. Deflections were measured at several points on the web in the region of the bracket and at several points along the longitudinal axis up to 15 in. from the corner of the bracket.

The deflections of the flanges were also measured at three cross sections: at the centerline of the bracket, at the corner of the bracket and at a point 15 in. out from the corner of the bracket. Each group of gages on the flanges consisted of three gages: one at the top, middle and bottom of the flange as shown in Fig. 4.

Table 1. Properties and Capacities of Test Sections

Section	Web Slenderness Ratio	Bracket Length (in.)	Eccentricity (in.)	Yield Stress (ksi)	*T* Distance (ksi)	Web Thickness (in.)	Ultimate[a] Load of Web ÷ 1.7 (kips)	Design Load of Flanges (kips)
W4 × 13	9.82	8	10.5	47.4	2.75	0.280	4.91	11.69
		12	18.9				5.03	7.91
W5 × 16	14.58	8	10.8	42.5	3.50	0.240	2.88	16.11
		12	18.9				2.91	11.07
M5 × 18.9	10.28	8	10.8	41.5	3.25	0.316	4.97	16.67
		12	18.9				5.12	11.40
W6 × 25	14.84	8	14.25		4.75	0.320	3.71	24.63
		12	18.9	44.6			4.75	20.91
S4 × 9.5[b]	7.67	8	10.5	44.2	2.50	0.326	6.40	4.05
		12	18.9				6.72	2.61

[a]The ultimate load is based on simply supported edges.

[b]Tests on these sections were terminated before the ultimate load was reached due to flange yielding.

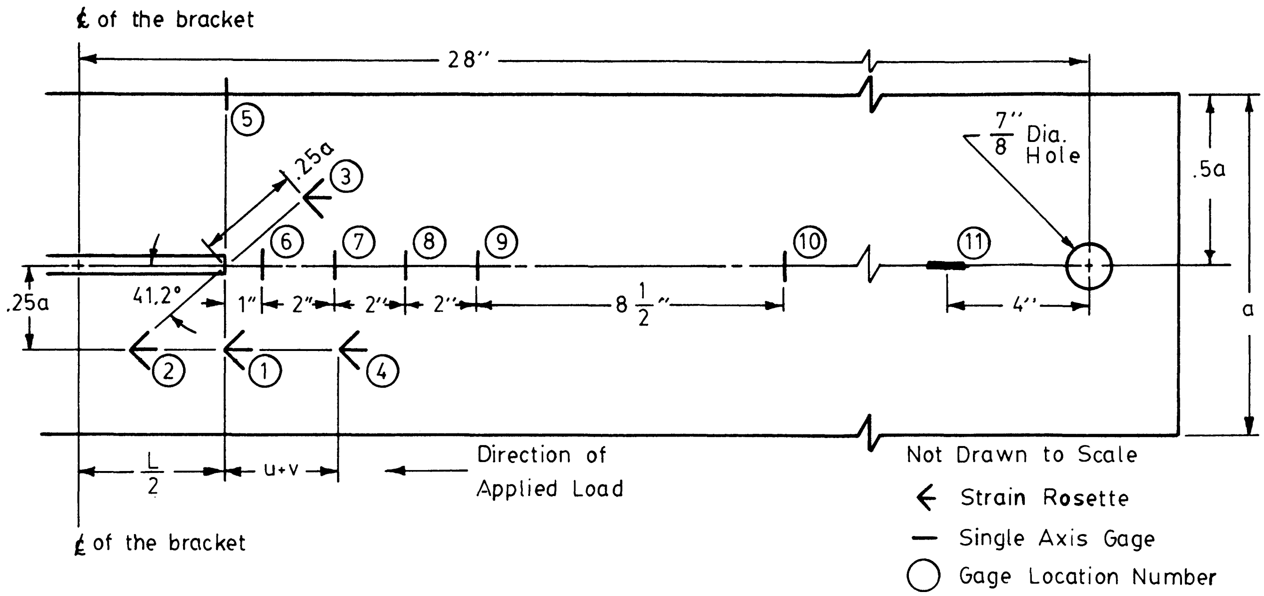

Fig. 3. Strain gage locations

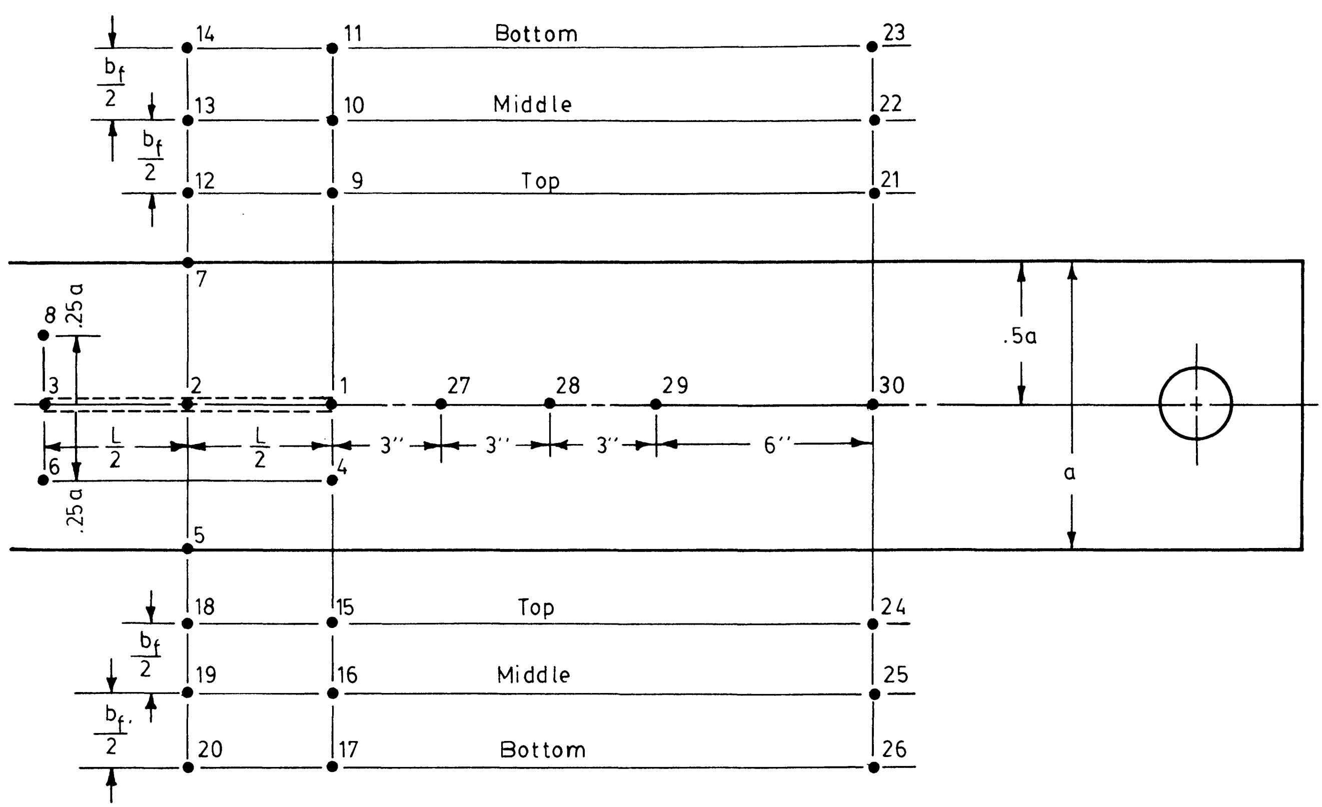

Fig. 4. Dial gage locations

Each specimen was loaded with a hydraulic system through a ball and socket joint used to maintain a constant load eccentricity. The loading apparatus can be seen in Fig. 2. The load was applied in increments with the strain and deflection data being taken while the load was held at a constant level. The first increment from no load was 1,000 lbs., all other increments being 500 lbs. Loading was continued until further load increases could not be sustained without excessive deflections.

Two tension coupons were cut from the web of each of the sections to determine the average yield stress. Each coupon was prepared and tested according to ASTM Standard A370-77. The average yield stress of each section is recorded in Table 1.

RESULTS

The principal strain directions were used to verify the two-dimensional yield line pattern developed by Abolitz and Warner. Figure 5 is a typical plot of the principal strain directions at each strain rosette location superimposed on the idealized pattern. In this figure, the bracket is loaded in the negative x direction. Gage locations 1 and 2 are not located on the yield line, but are inside a theoretically rigid plate segment. The principal strain directions of these gages do not correspond to any particular yield line, as can be expected. The strains at these locations are composed of strain components from each of the yield lines which border the plate segment.

Gages 3 and 4 are located directly over the idealized yield lines. The principal strain directions of these gages seem to correspond very well to the directions of the yield lines. This information indicates that the two-dimensional pattern of yield lines exists for rolled sections with a web slenderness ratio less than 15.

The directions of the yield lines are verified by the principal directions of the measured strains. However, if a yield line is to exist, the magnitude of the strain measured at the surface of the web over a yield line must be greatly in excess of the value of the yield strain (ϵ_y). In theory, all yielding occurs at the yield line. In reality, yielding must exist over an area which is centered about the yield line. This is the reason Gages 1 and 2 were able to measure strain components from several yield lines. As the distance increases from the yield line, the value of the measured strain would decrease. This would

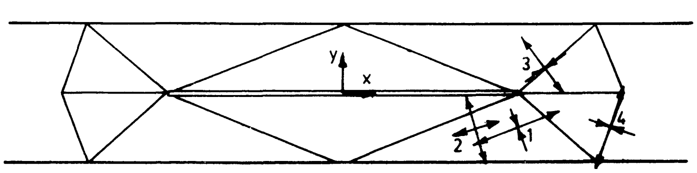

Fig. 5. Typical yield line pattern with principal directions of measures strain superimposed

Table 2. Magnitudes of maximum principal bending strains at ultimate load

Section	Bracket Length (in.)	Gage Location Number (Fig. 3)			
		1	2	3	4
W4 × 13	8	2870	1680	3360	—
	12	3360	2120	4380	1390
W5 × 16	8	1590	1550	3490	—
	12	2280	3430	4460	1140
M5 × 18.9	8	5390	3090	3420	860
	12	2180	2630	4640	1290
W6 × 25	8	2360	1780	3600	700
	12	2430	1250	4050	970

indicate the magnitude of the strains measured at Gages 1 and 2 should be greater than ϵ_y but less than those strains measured at gage locations 3 and 4 which are directly over the yield lines. Table 2 shows that the strains measured at locations 1 and 2 are indeed over yield and less than the strains measured at location 3.

The strains (Table 2) measured at gage location 4, which were expected to greatly exceed the value of ϵ_y, did not do so for any test. This clearly indicates that although the strain directions correspond to those of the idealized yield line, the yield line does not exist.

The lack of yielding is a result of the interaction between the flanges and the web. Because the webs of these sections are relatively thick when compared to their width ($\mu < 15$), the rotation of the plate segments of the collapse mechanism in the web forces the flanges to rotate as shown in Fig. 6 Sect. A–A. Outside the region

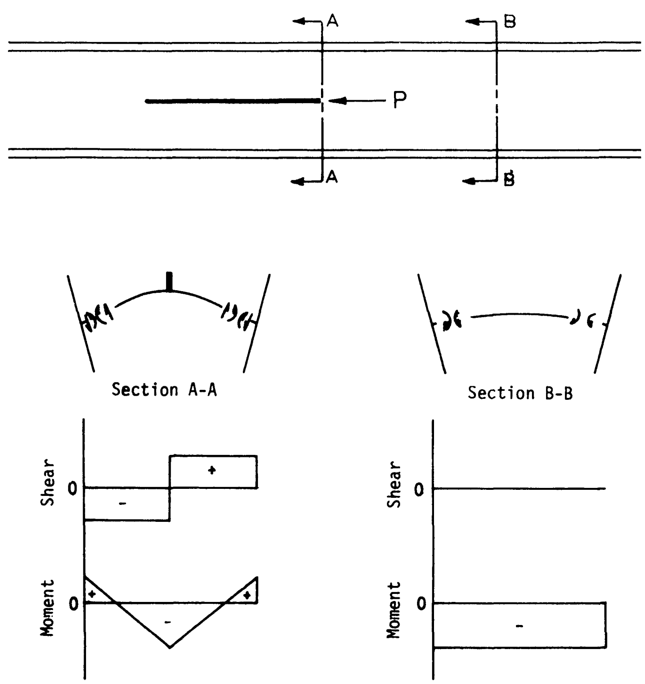

Fig. 6. Typical cross sections showing internal forces at failure

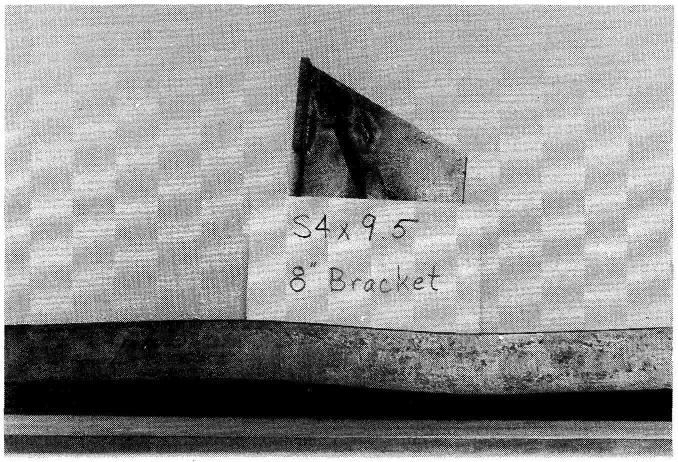

Fig. 7. Flange failure of the S4 × 9.5 sections

of the collapse mechanism, the flanges impose bending moments upon the edges of the web shown in Fig. 6 Sect. B–B. The direction of these bending moments directly oppose the bending moments developed in the web at the yield line so the yield line does not exist. From this we can see that the two-dimensional yield line pattern from which Abolitz and Warner derived Eq. 1 does not fully develop in sections with $\mu<15$.

The S4 × 9.5 sections ($\mu=7.67$) collapsed in a manner very dissimilar to the other four section sizes which were tested. The new failure mechanism can be seen in Fig. 7. These sections had an insufficient section modulus about the $Y-Y$ axis to resist the bending imposed by the bracket loads on the web. Plastic hinges formed in the flanges at the corners of the bracket, the points of maximum moment.

By using the simplified loading shown in Fig. 8, it is possible to determine at which level of loading first yield should occur in the flanges. The stresses which must be considered are those due to bending and the axial load. First yield of the flanges of the S4 × 9.5 section with the 12-in. bracket was estimated to be at 3,400 lbs. The strain data collected from the flanges indicates that yielding actually occurred at approximately 3,350 lbs. The onset of failure at about 3,000 lbs. can be seen in the load-curvature plot in Fig. 9. This plot was created from the strain data collected in the experiment. The strains used to create the plot were measured at the edges of the flanges at a cross section even with the corner of the bracket to which the load was applied. This value of load which caused yielding in the flanges is well below the theoretical ultimate load for failure of the web.

The existence of the different failure mechanisms can clearly be seen by comparing Figs. 10 and 11. Figure 10 is the deflection along the centerline at the ultimate load calculated from Eq. 1 for the W5 × 16 section which exhibited a web failure. This plot is a representative plot of centerline deflections of all the sections exhibiting

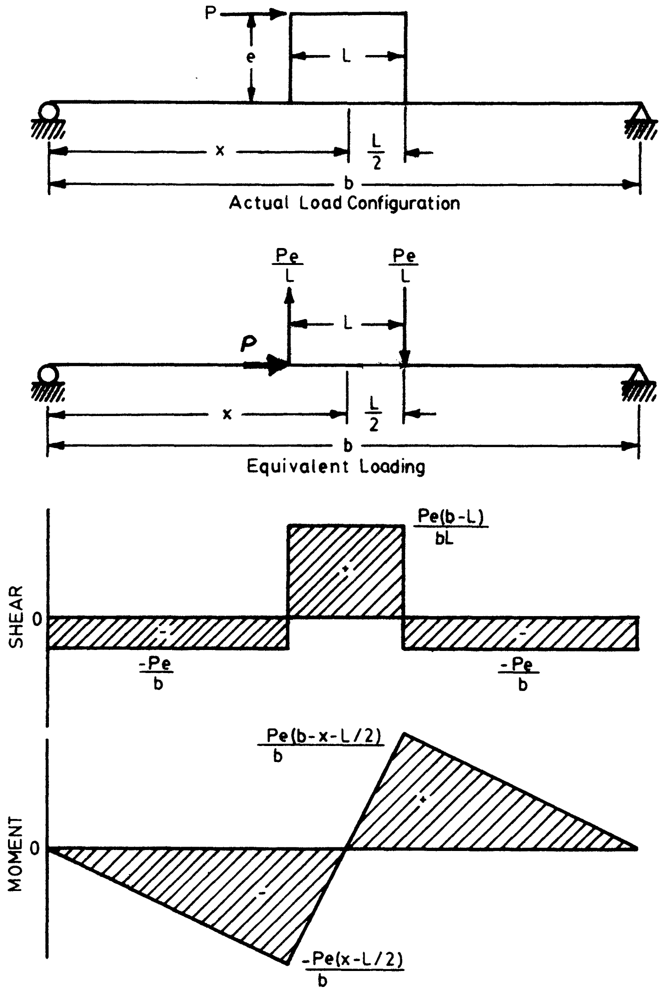

Fig. 8. Simplified equivalent loading

web failures. Figure 11 is the plot of the centerline deflections of the S4 × 9.5 sections at 3,000 lbs. In each of these plots the distance along the centerline is measured from the center of the bracket to the tension support. The deflection of the support was assumed to be zero which is the reason for the sharp break in the curves at approximately 20 in. Comparing Figs. 9 and 10 shows that the sharp break at approximately 8 in. in Fig. 10 is missing in Fig. 11. This break is the end of the web collapse mechanism. Figure 11 shows a break at the corner of the bracket where the plastic hinge formed and a smooth curve from that point out.

SUGGESTED DESIGN PROCEDURE

When both of the failure mechanisms are understood, design becomes straightforward. The capacity of welded bracket connections to the webs of sections with $\mu<15$ can be limited by either the capacity of the web or the capacity of the flanges. The ultimate load capacity of the web derived from Eq. 1 divided by a load factor of

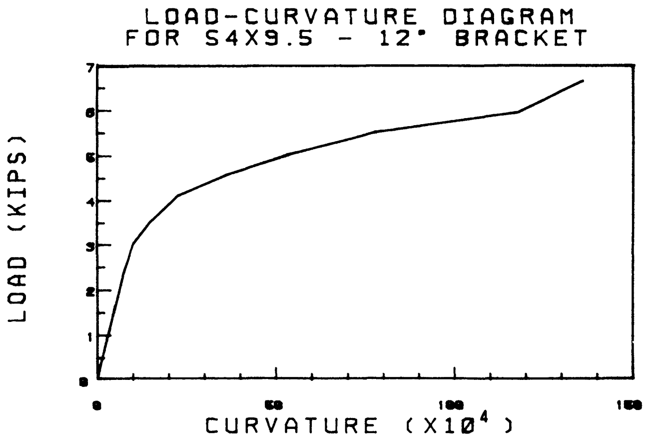

Fig. 9. Load-curvature diagram for S4 × 9.5 section with 12-in. bracket

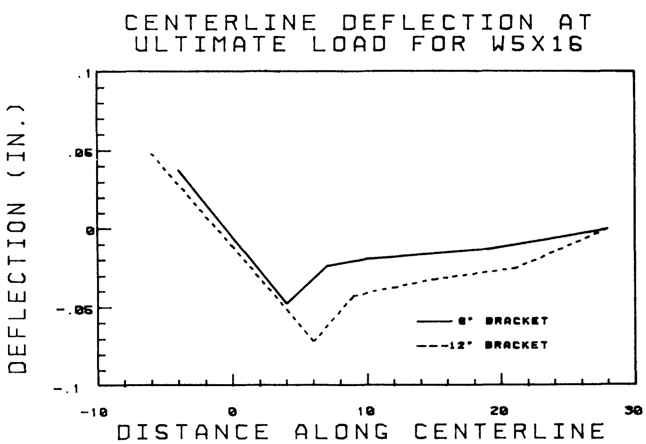

Fig. 10. Deflections along centerline of web at ultimate load for section exhibiting web failure

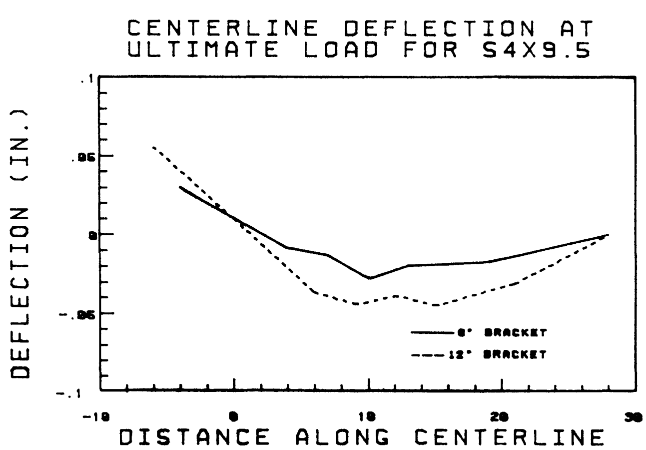

Fig. 11. Deflections along centerline of the web of section exhibiting flange failure

1.7 is compared to the load capacity of the flanges. The smaller of these two values represents the design capacity of the section.

The allowable load capacity of the flanges may be estimated using the following equations, which are based on the loading configuration shown in Fig. 8.

For $x \leq b/2$

$$P = (0.75F_y\,Sb)\;/\;\{\,e[b - x - L/2 + Sb/(RAe)\,]\,\} \tag{2}$$

For $x \geq b/2$

$$P = (0.75F_y\,Sb)\;/\;\{\,e[x - L/2 + Sb/(RAe)\,]\,\} \tag{3}$$

where:

$$R = \frac{1 - (Kl/r)^2/(2C_c^2)}{\frac{5}{3} + \frac{3}{8}\left(\frac{Kl/r}{C_c}\right) - \frac{1}{8}\left(\frac{Kl/r}{C_c}\right)^3}$$

Equations 2 and 3 were developed from Eq. 1.6-2 in the AISC *Specification for the Design, Fabrication and Erection of Steel Buildings* assuming $f_a/F_a < 0.15$. A more detailed derivation can be found in Ref. 5. The value obtained from either Eq. 2 or 3 must be checked to insure the assumption is valid. If the ratio $f_a/F_a > 0.15$ the value of P must be reduced by approximately 25% until the requirements of Eqs. 1.6-1a and 1.6-1b of Section 1.6 are met. This new value of P is the design capacity of the flanges.

The values obtained from the above procedure represent reasonable estimates of the design capacity of this type of connection for sections with $\mu < 15$. Figures 12–15 are plots of load vs. the maximum deflection of the web. The load capacity of the web from Eq. 1 divided by 1.7 and the design capacity of the flanges calculated from either Eq. 2 or 3 are superimposed on these plots. In each plot, the design capacity of the section, be it due to the web or flanges, intersects the load-deflection curve at the approximate end of the linear region. The estimate may be slightly conservative for the S4 × 9.5 section

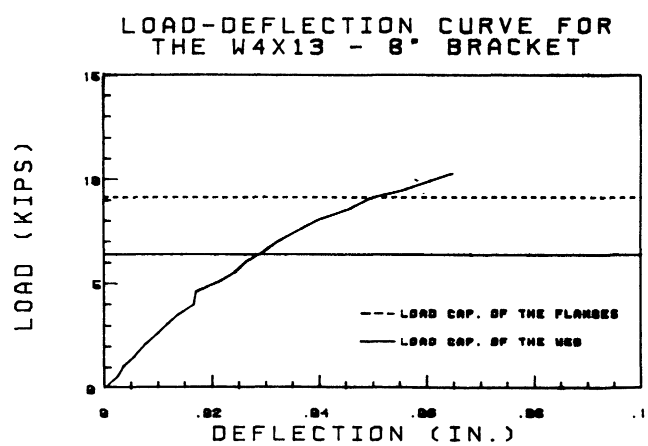

Fig. 12. Plot of load versus maximum deflection of web for W4 × 13 section with 8-in. bracket

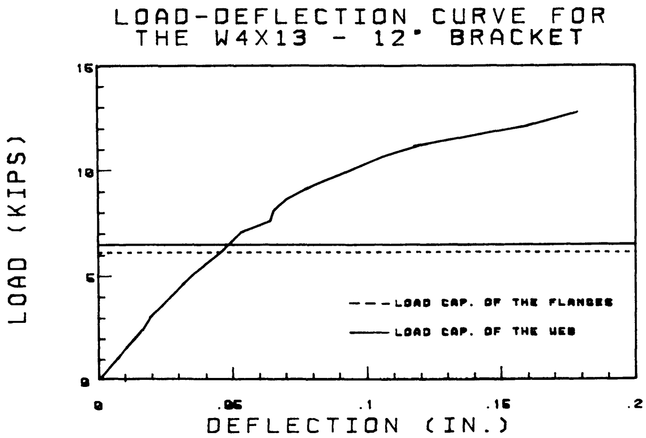

Fig. 13. Plot of load vs. maximum deflection of web for W4 × 13 section with 12-in. bracket

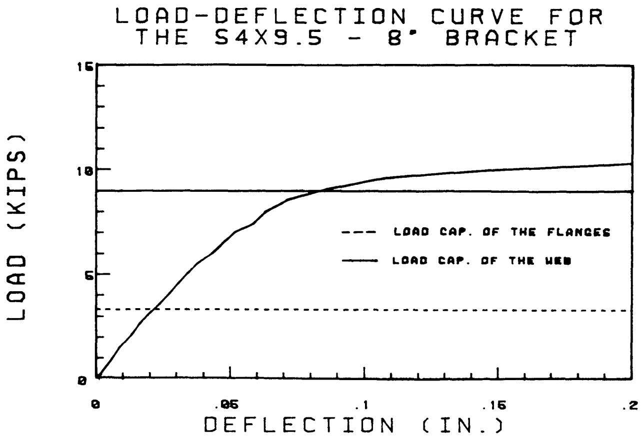

Fig. 14. Plot of load vs. maximum deflection of web for S4 × 9.5 section with 8-in. bracket

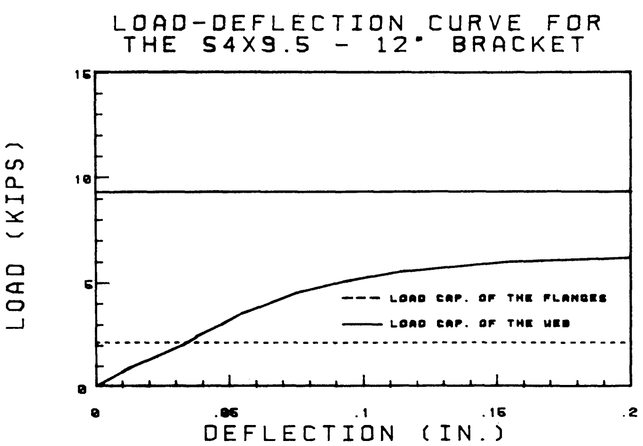

Fig. 15. Plot of load vs. maximum deflection of web for S4 × 9.5 section with 12-in. bracket

with the 8-in bracket. Figure 12 is typical of the load deflection curves for the W5 × 16, M5 × 18.9 and the W6 × 25 sections.

For the sections tested, the web capacity controls for all but the S4 × 9.5 section. The W4 × 13 section with a 12-in. bracket is close to being a flange failure. During the testing of the W4 × 13 with the 12-in bracket, the S shape characteristic of the flange failure could clearly be seen at a load of 1.4 times the ultimate load give by Eq. 1. Since the strains in the flanges of this section were not measured, it is impossible to determine if the flanges did yield.

CONCLUSIONS

1. The two-dimensional yield line pattern predicted by Abolitz and Warner does not fully develop for sections with web slenderness ratios less than 15. The interaction between the web and flanges transfers some of the bending to the web outside the collapse region. The bending outside the collapse region interferes with the development of the yield lines at the boundary between the collapse region and the remainder of the web. The expression which Abolitz and Warner developed and as modifed by Hoptay[2] represents a reasonable estimate of the design capacity for the sections tested.
2. A new maximum load mechanism develops in sections with very low web slenderness ratios. This mechanism is characterized by plastic hinges which form in the flanges of the section at the cross sections even with the corners of the brackets. This flange failure occurs as a result of low web slenderness ratios and low section moduli about the *Y-Y* axis. Such beam sections must be checked for both maximum load mechanisms, because either could occur.

SAMPLE DESIGN

Given:
Determine the allowable load capacity of an M5 × 18.9 section which is subjected to a bracket loaded web. The 12-in. bracket ($L = 12$ in.) is centered about a point 28 in. ($x = 28$ in.) from the tension support. The length of the section is 54 in. ($b = 54$ in.) and the yield stress of the steel is 36 ksi ($F_y = 36$ ksi). The load eccentricity e is 18 in. For the M5 × 18.9, $a = T = 3.25$ in., $S = 3.14$ in.^{3}, $r = 1.19$ in., $t_w = 0.316$ in., $A = 5.55$ in.^{2}

Solution:

The web slenderness ratio of the M5 × 18.9 section is:

$$T/t_w = 3.25/0.316 = 10.28$$

which is less than 15. Therefore, the load capacity of the web must be compared to the load capacity of the flanges. The smaller of the two values is the capacity of the section.

The ultimate load capacity of the web is determined from Eq. 1. The shape factor is:

$$\begin{aligned} k &= 2a/L + 2L/a + 2\sqrt{7} \\ &= 2(3.25)/12 + 2(12)/3.25 + 2\sqrt{7} \\ &= 13.22 \end{aligned}$$

The plastic moment capacity of the web is:

$$\begin{aligned} m_p &= F_y\, t_w^2 /4 \\ &= (36)(0.316)^2/4 \\ &= 0.90 \text{ kips} \end{aligned}$$

The load capacity of the web is:

$$\begin{aligned} P &= km_p\, L/e \\ &= (13.22)(.90)(12)/18 \\ &= 7.9 \text{ kips} \end{aligned}$$

(7.9 kips must be divided by the load factor of 1.7 for a design value of 4.65 kips for comparison with load capacity of the flanges.)

The load capacity of the flanges is determined from either Eq. 2 or 3. In this case, $x = 28$ and $b = 54$ so Eq. 3 is appropriate. The effective slenderness ratio (Kl/r) must be calculated first and it must be less than the column slenderness ratio (C_c). The effective length factor (K) is 1.0 for simply supported ends. The unbraced length $l = b = 54$ inches. The effective slenderness ratio is:

$$Kl/r = (1.0)(54)/(1.19) = 45.38$$

The column slenderness ratio is:

$$\begin{aligned} C_c &= \sqrt{2\pi^2 E/F_y} \\ &= \sqrt{(2)(3.14)^2(29{,}000)/(36)} \\ &= 126 \end{aligned}$$

The column slenderness ratio is greater than the effective slenderness ratio. The safety factor R is:

$$\begin{aligned} R &= \frac{1 - (Kl/r)^2/(2C_c^2)}{\frac{5}{3} + \frac{3}{8}\left(\frac{Kl/r}{C_c}\right) - \frac{1}{8}\left(\frac{Kl/r}{C_c}\right)^3} \\ &= \frac{1 - (45.38)^2/(2 \times 126^2)}{\frac{5}{3} + \frac{3}{8}\left(\frac{45.38}{126}\right) - \frac{1}{8}\left(\frac{45.38}{126}\right)^3} \\ &= 0.52 \end{aligned}$$

Substituting into Eq. 3:

$$\begin{aligned} P &= (0.75F_y\, Sb)/\{e[x - L/2 + Sb/(RAe)]\} \\ &= (0.75)(36)(3.14)(54)/\{(18)[28 - 12/2 + (3.14)(54)/(0.52)(5.55)(18)]\} \\ &= 10.0 \text{ kips} \end{aligned}$$

This value of P must meet the requirements of Specification Section 1.6. Equation 3 is based on the assumption that $f_a/F_a < 0.15$. In this case:

$$f_a/F_a = [(10.0)/(5.55)]/(0.52)(36) = .097$$

The assumption is correct. The load capacity of the section is the lower of the two values: 4.65 kips for the web and 10 kips for the flanges. Therefore, the design load capacity is 4.65 kips. The strength of the bracket, welds and the overall design of the section must be checked using established procedures.

ACKNOWLEDGMENTS

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NOMENCLATURE

a	=	T-distance (distance between the web fillets)
b	=	length of the section between simple supports (span)
e	=	load eccentricity
f_a	=	actual axial compressive stress
k	=	shape factor for the collapse mechanism
m_e	=	elastic moment capacity per unit length
m_p	=	plastic moment capacity per unit length
r	=	radius of gyration
t_w	=	web thickness
x	=	distance to the center of the bracket from the tension support
A	=	cross sectional area
F_a	=	allowable axial compressive stress
F_y	=	nominal yield stress
P	=	load applied to the bracket
P_u	=	ultimate load of the web
R	=	safety factor for determining the allowable axial compressive stress
S	=	section modulus
μ	=	web slenderness ratio
σ_y	=	yield stress of the material
ϵ_y	=	yield strain

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