An Experimental Look at Bracket-Loaded Webs

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Brackets attached rigidly to the web of a rolled member require a check of the bending stresses in the web in order to avoid overstressing or tearing of the web. Figure 1 describes a typical bracket attachment.

No guidance for sizing the web is found in the AISC Specification, but Abolitz and Warner[1] have presented a theoretical yield line analysis for bending under seated connections that estimates the capacity of the web. Their method of analysis yielded a very simple and elegant upper bound solution. This paper compares some experimental data with the theoretical solution proposed by Abolitz and Warner,[1] in order to assess the applicability and safety of an upper bound solution.

Abolitz and Warner used yield line analysis and the method of work to find a two-dimensional failure pattern which corresponded to the smallest failure load. They examined a number of yield patterns and found that the collapse pattern shown in Fig. 1 gave the lowest collapse load. A further reduction in the collapse load was found to occur if the edges along the flanges were assumed to be simply supported. They expressed the ultimate moment capacity of the web of the column as

$$M_a = k m_e L \tag{1}$$

where k is the shape factor of the collapse mechanism, m_e is the elastic moment capacity per unit length, and L is the length of the bracket. The shape factor for a simply supported web segment is defined as

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

and a is the T-distance of the column.

TEST PROCEDURE

Eight tests were performed on wide flange sections of two different sizes, W10x15 and W12x16.5. The type of column section tested and its physical dimensions and properties

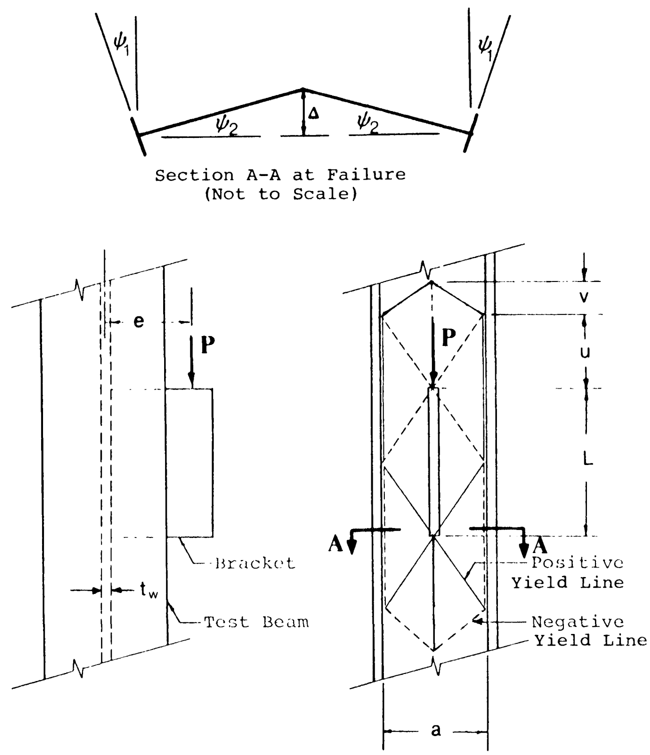

Fig. 1. Typical bracket and its yield pattern at failure (Ref. 1)

are tabulated in Table 1. Each test specimen was sanded to remove all mill scale, rust and grease, in order to apply the stress coat and strain gages, and to assure that the dial gages would be able to function properly. Each test specimen was mounted in the test frame shown in Fig. 2.

Bracket lengths of 8 in. and 12 in. were used in this test series. The thickness of the bracket, 0.25 in., was chosen to be approximately equal to the thickness of the web of the test beams. All sections were tested to a load of 10,000 lbs or until tearing of the web. Each bracket was securely welded to the web along the center line of the test section. The test section was then sprayed with Photolastic Tens-lac U-10-A Undercoating and allowed to dry for 30 min before the application of Magnaflux ST 60F/16C Stress Coating. At the same time and in the same manner as the test spec-

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imen was prepared, four stress coat calibration bars were prepared. The test specimen and the calibration bars were allowed to cure for 48 hrs.

To record the deflection patterns, 28 mechanical dial gages were used to record the web and flange deflections. All the dial gages were supported by an independent support frame in order to eliminate any interaction between the loading frame and the dial gage support frame.

Prior to the start of the actual test, the strain gage calibration bars were tested to determine the strain at which the stress coating would crack. The overall average of the strain readings of the four calibration bars was used as the threshold value of the stress coat.

The load was applied by a hydraulic system capable of maintaining the applied load on the bracket constant while strain gage and dial gage readings were recorded. The load was applied in 250-lb and 500-lb increments to a ball and socket joint placed between the ram and the bracket to ensure that the load was applied at a constant eccentricity throughout the test. The first and second loading increments were 500 lbs each, after which the load on the bracket was increased by increments of 250 lbs until the load on the bracket was 6500 lbs. The loading increment was then increased to 500 lbs and was increased by that amount until the completion of the test. Strain gage and dial gage readings were recorded after each increment of the load. The maximum rate of deflection was observed to be approximately 0.004 in./min or less.

Photographs of the pattern of cracks in the stress coating were taken at selected values of the load on the bracket. The cracks in the stress coat were made visible through the use of an electrically charged powder that was sprayed on the crack pattern.

At the conclusion of the test, tension coupons were cut from each end of the test specimen, so that both longitudinal and transverse tension tests could be performed in order to obtain the yield strength of each specimen in accordance with ASTM A370-77. Finally, the thickness of the web was measured at five points across the depth of the beam in order to determine the effect sanding had on the thickness of the web. A more detailed description of the test procedure may be found in Ref. 3.

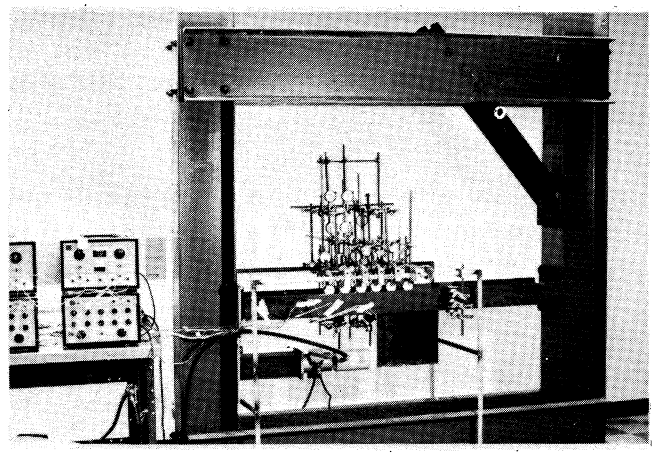

Fig. 2. Test stand

DISCUSSION OF RESULTS

The major difficulty with determining the ultimate capacity of any two-dimensional structure is deciding what constitutes the ultimate capacity. For example, Fig. 3 shows the load versus maximum deflection curve for Test No. 1. This load-deflection curve is a smooth, continuously increasing curve; the sharp bend near the ultimate load, which is typical for beam and frame structures, is missing. One might even question if the theoretical upper bound load can be regarded as the actual failure load or ultimate load-carrying capacity, since the web can support loads that are twice the upper bound value. Similar load deflection curves were obtained for all the tests conducted during this investigation.

Save and Massonnet[2] have compared experiments performed on circular steel plates by a number of investigators, and found that the sharp bend in the load versus deflection diagram tends to disappear as the slenderness ratio, μ, approaches 40. (The slenderness ratios of the two columns tested varied from 34 to 47.) Therefore, they suggest an alternative deflection criterion for establishing the collapse load of the plates. This criterion states that ultimate failure occurs when the maximum deflection is 2% of the diameter.

Table 1. Description of Test Sections

Test No.	Section	Plate Length (in.)	Web Thickness[1] (in.)	T-distance (in.)	Yield Stress[2] (ksi)	Slenderness Ratio, μ
1	W10x15	12	0.205	8.375	41.3	40.8
2	W10x15	12	0.208	8.375	39.3	40.3
3	W10x15	12	0.220	8.375	39.7	38.0
4	W12x16.5	12	0.220	10.375	40.3	47.1
5	W12x16.5	8	0.230	10.375	37.5	34.8
6	W12x16.5	8	0.234	10.375	39.5	34.1
7	W10x15	8	0.234	8.375	40.0	34.1
8	W12x16.5	12	0.235	10.375	37.6	44.1

[1] Average thickness after sanding, measured at five points across the web.
[2] Average yield stress obtained from transverse and longitudinal tension tests.

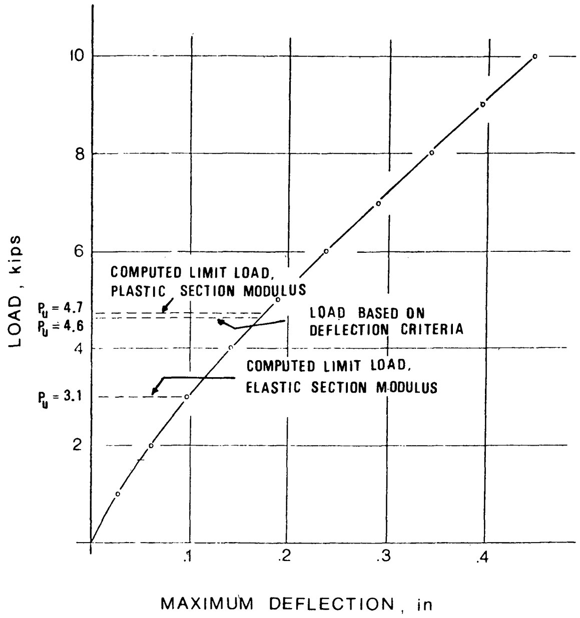

Fig. 3. Load vs. Maximum Deflection for Test No. 1

Table 2. Comparison of Flange Rotation and Web Rotation at Ultimate Load

Test No.	Ultimate Load (kips)	Angle of Flange Rotation ψ_1 (radians)	Angle of Web Rotation ψ_2 (radians)
1	4.6	0.0270	0.0305
2	4.2	0.0240	0.0358
3	4.8	0.0248	0.0331
4	4.7	—	0.0358
5	3.2	0.0158	0.0304
6	4.0	0.0241	0.0301
7	4.0	0.0239	0.0376
8	5.5	0.0209	0.0342

Therefore, the data obtained from these tests will also be correlated with this criteria. For the bracket-to-web connection, this criteria implies that the ultimate capacity is reached when the maximum deflection becomes 2% of the shortest span. For this application the shortest span is defined as twice the distance from the corner of the bracket to the nearest support. For connections whose bracket length is less than the depth of the column, the span is established by the half-length of the bracket. This is due to the fact that deflections measured along a line perpendicular to the flange at the middle of the bracket show that the deflections are at least an order of magnitude less than the maximum deflection. Therefore, this line may be considered to be a line of simple support. However, in those cases where the bracket length is larger than the distance between the flanges, the distance between the flanges will be the shortest span. The only question remaining is whether to use the clear distance or the T-distance for determining the span. As proposed by Abolitz and Warner, the T-distance of the column is used.

In using the shape factor k, it had been assumed that the flanges provide a simple support for the web of the column. This assumption is confirmed by the deflection data shown in Table 2. The local rotation of the flanges was visually evident and indicated that the flanges offer little or no fixity to the web. The deflections of the web near the flanges are an order of magnitude less than the maximum deflection along the center of the web (Table 3). This further confirms the assumption that the flanges provide a line of support.

In Eq. (1) the allowable moment capacity of the web is based on the elastic section modulus, $m_e = \sigma_y t_w^2/6$. It does not seem to make sense to use elastic section properties for a plastic failure theory. Therefore, the ultimate moment capacity of the web will also be computed by utilizing the plastic section modulus, $m = \sigma_y t_w^2/4$. The value of the plastic failure load based on the elastic and plastic section properties will be compared with the failure load based on the 2% deflection criteria. The results of this comparison are tabulated in Table 4.

From Table 4 it can be seen that if the plastic section properties are used in Eq. (1), they provide a collapse load that is very near the collapse load established by the 2% deflection criterion. If this result is assumed to be the actual

Table 3. Comparison of Maximum Deflection and Deflection Along the Flange at Ultimate Load

Test No.	Ultimate Load[1] (kips)	Maximum Deflection (in.)	Deflection Along the Flange (in.) Distance from Corner of Bracket 0 in.	3 in.	6 in.	9 in.
1	4.6	0.164	—	—	—	—
2	4.2	0.1681	0.0151	—	0.0229	—
3	4.8	0.1654	0.0173	—	0.0286	—
4	4.7	0.2102	0.0244	0.0355	0.0352	0.034
5	3.2	0.1677	0.0096	0.0144	0.0168	0.0165
6	4.0	0.1618	−0.0054	0.0053	0.0092	0.0105
7	4.0	0.1579	−0.0004	0.0099	0.0116	0.0125
8	5.5	0.2083	—	0.0309	0.0314	0.0303

[1] Based on plastic section properties.

Table 4. Comparison of Computed Solutions and Test Results

Test No.	Computed Ultimate Load, kips		Test Load Corresponding to 2% Deflection Criteria, kips
	Elastic Section Modulus	Plastic Section Modulus	
1	3.1	4.7	4.6
2	3.1	4.6	4.2
3	3.5	5.2	4.8
4	3.5	5.2	4.7
5	2.4	3.6	3.2
6	2.5	3.8	4.0
7	2.6	3.9	4.0
8	3.7	5.5	5.5

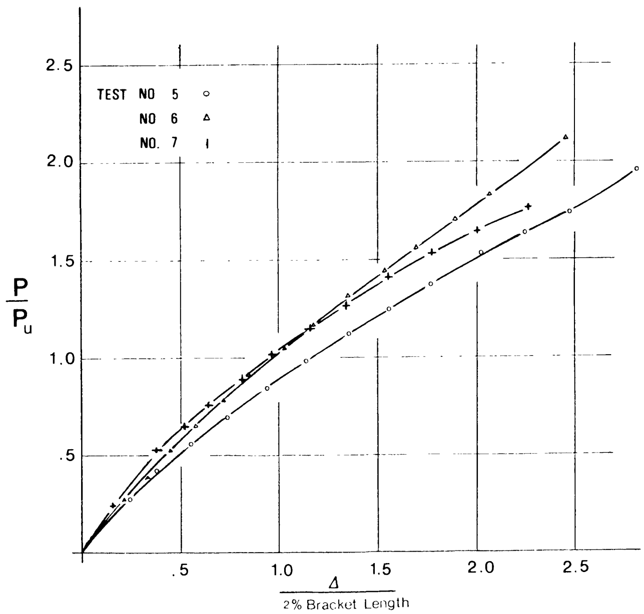

Fig. 5. Normalized Load vs. Normalized Maximum Deflection. Tests No. 5, 6, 7

collapse load, and all the load versus maximum deflection curves are normalized with respect to this value, Figs. 4 to 6 will result. These show that, based on the deflection criterion, the collapse load varies from $0.89P_u$ to $1.05P_u$. Therefore, the plastic moment capacity, m, will be recommended as the value to be used in Eq. (1).

A study of the resulting displacement contours on the web of the column indicates that the difference between the actual yield line pattern and the pattern assumed by Abolitz and Warner is small. See Fig. 7. The difference is an intentional simplification for deriving the plastic moment capacity of the web. Since the actual yield line pattern is smaller than the idealized pattern, it appears that the results derived from Eq. (1) slightly over-estimate the actual web capacity. On the other hand, the yield line theory completely neglects the additional strength due to membrane action and work hardening. The load displacement curves, Figs. 4 to 6 show that there is significant membrane action present and the actual load-carrying capacity exceeds the

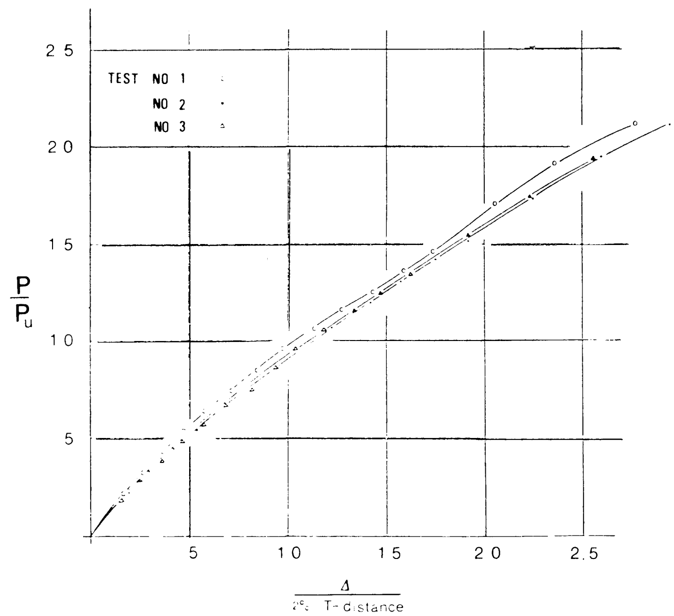

Fig. 4. Normalized Load vs. Normalized Maximum Deflection. Tests No. 1, 2, 3

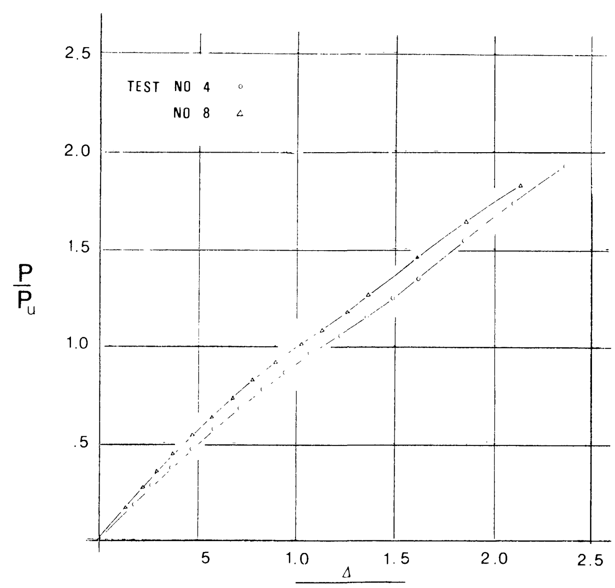

Fig. 6. Normalized Load vs. Normalized Maximum Deflection. Tests No. 4, 8

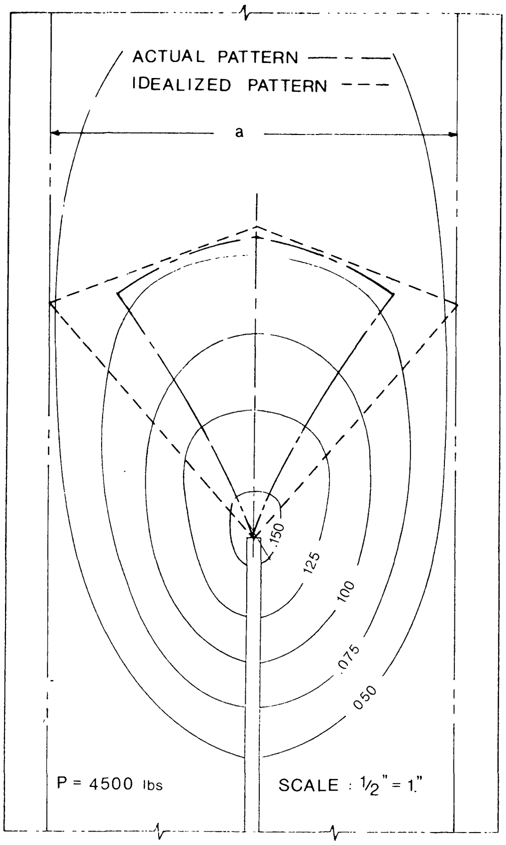

Fig. 7. Deflection pattern for Test No. 1 at P = 4500 lbs

values predicted by yield line theory. Therefore, it is felt that the error obtained from using the idealized pattern is more than compensated for by the membrane effects.

Figures 8 through 11 show the typical progression of cracks in the brittle stress coat for test No. 7 at various load levels. The stress coat was applied to the web on the side

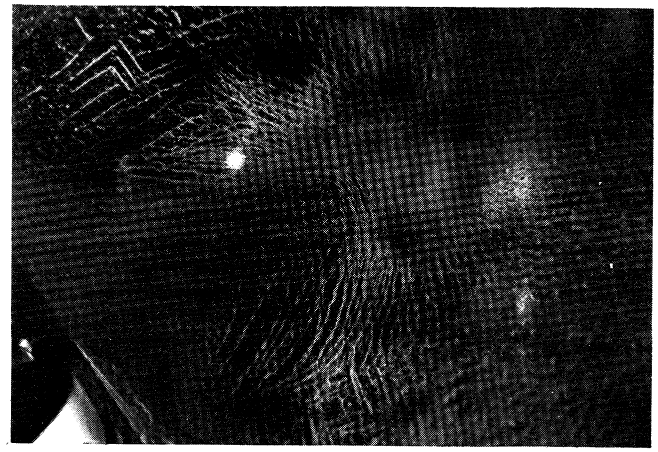

Fig. 9: Crack pattern at 3000 lbs applied load

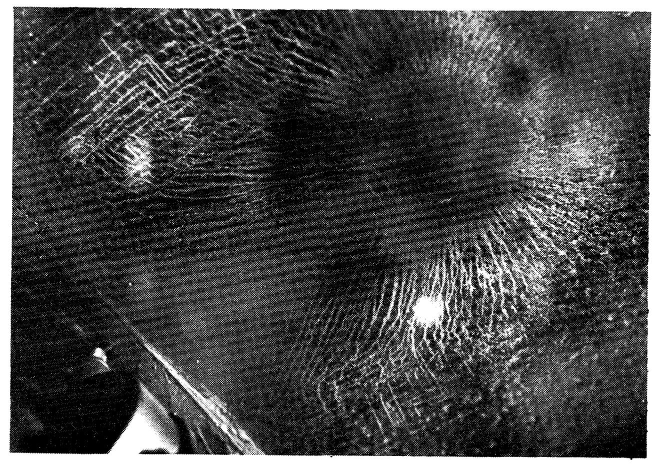

Fig. 10. Crack pattern at 4000 lbs applied load

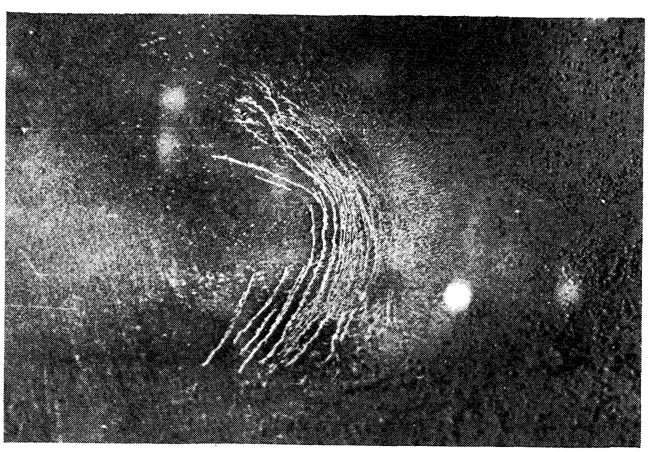

Fig. 8. Crack pattern at 1000 lbs applied load

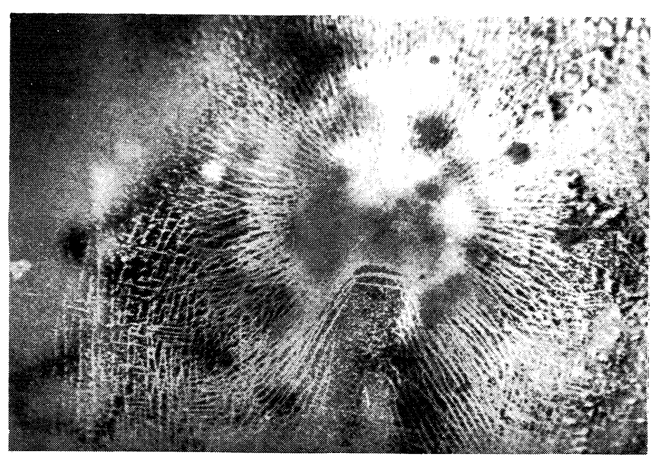

Fig. 11. Crack pattern at 5000 lbs applied load

opposite the bracket. The yield strains of the specimen and the threshold strain of the coating were as follows: $\epsilon_y = 1380\mu$ in./in., $\sigma_y = 40$ ksi, $\epsilon_{coating} = 1500\mu$ in./in. Thus, the coating for this particular test cracked slightly above the yield strain. In all the tests, significant cracks appeared in the stress coat at 1000 lbs load. The cracking indicates that, if the load distribution between the bracket and the web could be established and the elastic load capacity of the web could be computed, it would be less than 1000 lbs. Considering the actual load carrying ability of the web as demonstrated in this test sequence, it is obvious that the elastic capacity of the web is too conservative for practical use.

The stress coat photographs also show that only a small area under the corner of the bracket has yielded, while the remaining area under the bracket reveals no cracks in the coating and can therefore be assumed to remain elastic.

CONCLUSIONS

The results of the test program, although limited, verified the soundness of the collapse mechanism approach of plastic analysis as proposed by Abolitz and Warner,[1] for the design of bracket-to-web column connections. However, since Abolitz and Warner did not perform any tests, their recommended design procedure is still conservative. Reexamining their design suggestions in light of these test results reveals that:

1. The allowable bracket moment induced in the web of the column, M_a , should be based on the plastic section modulus, $t_w{}^2/4$, and not upon the elastic section modulus, $t_w{}^2/6$.
2. Based only on theoretical considerations, Abolitz and Warner were able to establish the lowest upper bound of the possible yield line patterns. However, they could not establish whether the flanges of the wide-flange section would offer any restraint. As a result, they established patterns for both cases: the flanges offering full restraint (fixed edges), and the flanges offering no restraint (free edges). To be safe, they recommended that the lowest value of the shape factor for all possible yield mechanisms be used. The smallest shape factor corresponded to the case with free edges. Based on these test results, this was a correct assumption.
3. Finally, Abolitz and Warner placed no restrictions on the slenderness ratio, μ, which is to be defined as the smaller of T/t_w or L/t_w. Save and Massonnet[2] point out that there must be an upper and lower limit on the web slenderness ratio. As the magnitude of the slenderness ratio increases to values of 95 and higher, membrane effects rather than bending effects govern from the beginning of the loading, and a yield line analysis based only on bending deformations is invalid. However, there is no practical necessity for placing an upper limit on the slenderness ratio, since the largest value of the slenderness ratio for standard wide-flange sections is 55, and most wide-flange sections have a smaller slenderness ratio that the tested sections (i.e., μ = 34 to 47). The lower limit on the slenderness ratio will, however, enter into the design of the heavier columns. Save and Massonnet[2] state that when the slenderness ratio is less than 10, yield line theory does not apply because of the influence of work hardening (and shear forces). However, the theoretical limit load for $\mu < 10$ does remain somewhat useful as a prediction of the conventional failure load corresponding to a relative deflection which will be less than 2% of either the T-distance or the length of the bracket.

DESIGN PROCEDURE

Given:
A column size of W14x99, a beam size of W18x55, the load on the bracket due to the dead load = 7.5 kips, and due to the live load = 15 kips, the yield stress of the steel, $\sigma_y = 36$ ksi. For the W14x99, $t_w = 0.485$ in., T-distance = 11.25 in.

Solution:
Since the allowable bracket moment is directly proportional to the length of the bracket, the bracket should be made as large as is practical. In this example the bracket length must be less than the T-distance of the W18x55, which is 15.50 in. Thus, a bracket length of 15 in. will be considered first. The slenderness ratio of the column web should be checked, and since the bracket length is larger than the T-distance of the W14x99, 11.25 in., the governing web slenderness ratio is:

$$T/t_w = 11.25/0.485 = 23.2$$

This satisfies the limits on μ and therefore this procedure is applicable.

The ultimate design load for this bracket is

$$1.7\ LL + 1.4\ DL = 36 \text{ kips}$$

The ultimate bracket moment, M_b , can be expressed in terms of the applied load, P, and the eccentricity, e:

$$M_b = Pe$$

The desired eccentricity is determined by the designer. In this example it will be assumed to be 6 in. Therefore, the bracket moment is:

$$M_b = Pe = 216 \text{ kip-in.}$$

The capacity of the W14x99 column with a 15-in. bracket must now be checked to ensure that the web can carry 216 kip-in.

The allowable bracket moment based on the physical parameters of the connection is:

$$M_a = km\,L$$

Since the flanges are free to rotate, the shape factor is

$$\begin{aligned} k &= (2a/L) + (2L/a) + 2\sqrt{7} \\ &= [2(11.25)/15] + [2(15)/11.25] + 2\sqrt{7} \\ &= 9.46 \end{aligned}$$

The ultimate internal moment capacity of a 1-in. strip of the web is

$$\begin{aligned} m &= \sigma_y (t_w^2 4) \\ &= 36\,(0.485)^2/4 \\ &= 2.12 \end{aligned}$$

Therefore, the allowable bracket moment is

$$\begin{aligned} M_a &= km\,L \\ &= 9.46\,(2.12)\,(15) \\ &= 301 \text{ kip-in.} > 216 \text{ kip-in.} \quad \textbf{o.k.} \end{aligned}$$

It may be desirable to have a shorter bracket or to increase the eccentricity of the applied load. Check if a 12-in. bracket will be adequate:

$$\begin{aligned} k &= 9.30 \\ M_a &= km\,L \\ &= 9.3(2.12)\,(12) \\ &= 237 \text{ kip-in.} > 216 \text{ kip-in.} \quad \textbf{o.k.} \end{aligned}$$

Or, using a 15-in. bracket, the eccentricity may be increased to 8.0 in.

$$\begin{aligned} M_b &= Pe \\ &= 36(8.0) = 288 \text{ kip-in.} < 301 \text{ kip-in.} \quad \textbf{o.k.} \end{aligned}$$

The strength of the bracket itself, the overall adequacy of the column due to other loads, and the design of the bracket weld must be checked, but this can be carried out in accordance with established procedures.

ACKNOWLEDGMENTS

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NOMENCLATURE

a = T-distance (distance between web fillets)
e = eccentricity of bracket load
k = shape factor for assumed collapse mechanism
m = ultimate moment capacity per unit length
m_e = elastic moment capacity per unit length
t_w = thickness of the web
u,v = distance along the x-coordinate
L = length of the bracket
M_a = allowable bracket moment
M_b = actual bracket moment
P = load applied to the bracket
P_u = ultimate bracket capacity, M_a/e
$\epsilon_{coating}$ = threshold strain of the brittel coating
ϵ_y = yield strain
μ = slenderness ratio, L/t_w or a/t_w
σ_y = yield stress
ψ_1 = angle of rotation of the flange
ψ_2 = angle of rotation of the web
Δ = maximum web deflection at bracket

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