Eccentrically Loaded Steel Single Angle Struts

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SUMMARY

Results of experimental data on eccentrically loaded steel single angle struts are compared with AISC LRFD (1986) and AISC ASD (1989) specifications. It is shown that the current design practice is conservative and needs reevaluation with respect to axial load-bending moment interaction formulas. Various simplified alternative approaches for the design of steel single angle struts are discussed and results presented.

INTRODUCTION

Steel single angles are extensively used in several kinds of structures. An important application of these angles in latticed towers, trusses, etc. is to connect them by one leg to carry compressive loads. This loads the member in axial compression with end moments due to the eccentric connection. The resulting problem is too complex to be analyzed precisely, because of the eccentricity of load with respect to both principal axes and the uncertain nature of the end restraints which would render the problem of finding an effective length factor difficult. Traditionally, various national design practices on eccentrically loaded single angle struts differed from each other very widely. The slenderness ratios of eccentrically loaded single angle struts are modified in ASCE Manual No. 52 (Am. Soc. of Civil Engrs., 1988) to make use of the formulas applicable to concentrically loaded struts. Canadian tower design practice CSA-S37 (Canadian Standards Association, 1986) and British practice (British Standards Institution, 1985) is to ignore the eccentricity and limit the strength of eccentrically loaded single angle struts to a certain percentage of the strength of corresponding concentric axially loaded struts. Such methods simplify the design of these members but make the design of single angle struts less rational than would sometimes be desirable. However, similar design simplifications are not allowed in AISC LRFD (1986), AISC ASD (1989) and CSA-S16.1 (Canadian Standards Association, 1989) specifications. The AISC LRFD provisions are as given below:

Case 1: For $\frac{P_u}{\phi_c P_n} \geq 0.2$,

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad (1)$$

Case 2: For $\frac{P_u}{\phi_c P_n} < 0.2$,

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad (2)$$

where

P_u = required compressive strength
P_n = nominal compressive strength for concentric axial load
M_u = required flexural strength
M_n = nominal flexural strength
ϕ_c = resistance factor for compression = 0.85
ϕ_b = resistance factor for flexure = 0.90

These formulas take into consideration the load-moment interaction effect on the section and the amplification of the end moments due to P–Δ effect. The values for moment interaction factors are obtained from experimental investigation on 82 sidesway frames made of doubly symmetric wide flange shapes (AISC LRFD, 1986). The stability and strength checks for the member are essentially combined in the above formulas. The basis for LRFD provisions in general is the first order probabilistic design procedure (Ravindra and Galambos, 1978). The basic form of the above formulas is derived from beam-column theory and is adjusted by using experimental observations to achieve an *acceptable* level of complexity. The load factors and resistance factors are arrived at by using a uniform "probability of failure" level for the full range of basic variables for all member types. In the case of beam-columns (as in the case of several stuctural components), the accepted level of safety corresponds to $\beta = 3.0$, where β is the safety index. This value is used to estimate the load and resistance factors for the member for a chosen set of design formulas.

Interaction formulas similar to those in LRFD are also provided in AISC ASD (1989) Specification. For single angle members, the ASD Specification gives special provisions for calculating the allowable bending stress about principal axes and geometric axes. Such special treatment is not given for single angle members in LRFD Specification. As a guide, the LRFD Manual gives an example for the design of a single angle strut under eccentric loading in which the critical bend-

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ing stress for the section is taken as F_y for bending about either of the principal axes.

The design provisions as per ASD are as follows:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1.0 \quad (3)$$

$$\frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \quad (4)$$

where

F_a = the allowable axial compression stress
F_b = the permissible bending compression stress
F'_c = the Euler stress divided by 23/12
f_a = the computed axial compression stress
f_b = the computed bending compression stress
C_m = the coefficient as specified in AISC ASD specification

Equation 4 above does not govern the design of eccentrically loaded single angle struts and hence need not be accounted for. As mentioned above, these formulas are derived primarily for doubly symmetric wide flange sections. In the case of singly symmetric sections (such as equal leg single angles, tees, and channels) and unsymmetric sections, the formulas lead to conservative results. This is due to the fact that the moment ratios in the interaction formulas are evaluated for the case of maximum stresses about each of the principal axes independent of the other. This practice does not result in any problem for the design of doubly symmetric sections since one of the four corners is critical for moments about both principal axes simultaneously. But for singly symmetric sections and unsymmetric sections, the points of maximum stress for both the principal axes moments do not usually coincide. If the interaction equation is applied at each of the possible critical points by evaluating the bending stress due to the two moments simultaneously and the sum is taken to evaluate the load capacity, the resulting loads are higher than the load capacity computed using the present practice. For example, a 3×3×¼-in. angle of length 36 in. shows an increase of up to 10 percent in the calculated load capacity if the load is placed in the first or fourth quadrants (Fig. 1) with respect to the W-Z (principal) axes. This difference is mainly due to the difference in principal axes coordinates of the two points at the toes of legs (points 2 and 5 or points 4 and 6 of Fig. 1). If the load is placed in second or third quadrants, the increase can be more than 50 percent. The effect is even more pronounced for unequal leg angles. Hence, it is necessary to study the effects of the eccentricity of load on single angle struts using the experimental results available in literature.

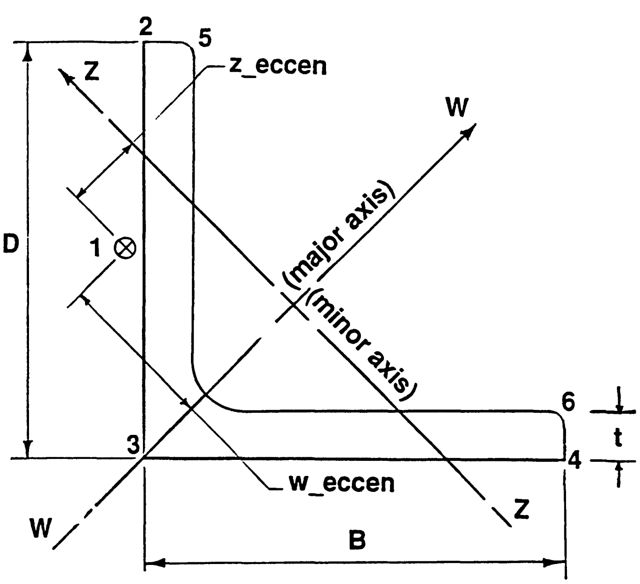

Fig. 1. Cross-section of an angle showing possible critical points 2 to 6 (load is applied at point 1).

In the present study, results of experimental investigation on eccentrically loaded single angle struts from three sources are compared with the compressive strength computed according to AISC LRFD and AISC ASD specifications. A total of 71 test results from experimental studies are included in the analysis. The test results of hundreds of eccentrically loaded single angle struts available in the literature are examined in detail and only 71 specimens are considered for the present study as these are the only specimens with clearly known end restraints and effective lengths about both principal axes and clearly defined load application points. The relevant information about the experimental investigations included in the study is as follows:

1. Wakabayashi and Nonaka (1965): Tests were conducted on five series of mild steel 90×90×7 mm (3.54×3.54×0.276 in.) angle specimens with slenderness ratios of 20, 40, 60, 70, 80, 90, 100, 110, 130, and 150, in each series. Two of the series had loading on the major principal axis of the cross-section at an eccentricity with respect to minor axis equal to the minimum radius of gyration of 17.7 mm (0.697 in.). (In one series, the load was away from the shear center, while the other series had load towards the shear center.) The third series had eccentric load on the minor principal axis at an eccentricity with respect to major axis of 17.4 mm (0.685 in.) equal to one-half of the maximum radius of gyration. The next series had eccentricity about both principal axes (eccentricities equal to 17.7 mm and 17.4 mm with respect to minor and major principal axes respectively). The remaining specimens were concentric axially loaded struts. The material used in the investigation had a guaranteed minimum yield stress of 225 MPa (32.6 ksi). The end supports were designed to eliminate constraints against rotation and twisting at the ends of the speci-

Table 1a.
Results of Experimental Study on Eccentrically Loaded Single Angles
Wakabayashi & Nonaka (1965)

SPECIMEN #		D/in. × B/in. × t/in.	KL/r	E_y/ksi	Z-eccen. in.	W-eccen. in.	P-test kips
Ser.	2- 20	3.54×3.54×0.276	19.4	46.9	0.697	0.000	43.7
Ser.	2- 40	3.54×3.54×0.276	39.8	46.9	0.697	0.000	38.6
Ser.	2- 60	3.54×3.54×0.276	59.7	46.9	0.697	0.000	32.5
Ser.	2- 70	3.54×3.54×0.276	69.6	46.9	0.697	0.000	29.9
Ser.	2- 80	3.54×3.54×0.276	79.5	46.9	0.697	0.000	28.2
Ser.	2- 90	3.54×3.54×0.276	89.5	46.9	0.697	0.000	25.8
Ser.	2-100	3.54×3.54×0.276	99.4	46.9	0.697	0.000	23.6
Ser.	2-110	3.54×3.54×0.276	109.4	46.9	0.697	0.000	21.6
Ser.	2-130	3.54×3.54×0.276	129.3	46.9	0.697	0.000	17.8
Ser.	2-150	3.54×3.54×0.276	149.2	46.9	0.697	0.000	15.5
Ser.	3- 20	3.54×3.54×0.276	19.4	46.9	−0.697	0.000	51.6
Ser.	3- 40	3.54×3.54×0.276	39.8	46.9	−0.697	0.000	41.7
Ser.	3- 60	3.54×3.54×0.276	59.7	46.9	−0.697	0.000	35.8
Ser.	3- 70	3.54×3.54×0.276	69.6	46.9	−0.697	0.000	33.4
Ser.	3- 80	3.54×3.54×0.276	79.5	46.9	−0.697	0.000	31.0
Ser.	3- 90	3.54×3.54×0.276	89.5	46.9	−0.697	0.000	27.8
Ser.	3-100	3.54×3.54×0.276	99.4	46.9	−0.697	0.000	26.1
Ser.	3-110	3.54×3.54×0.276	109.4	46.9	−0.697	0.000	21.6
Ser.	3-130	3.54×3.54×0.276	129.3	46.9	−0.697	0.000	17.8
Ser.	3-150	3.54×3.54×0.276	149.2	46.9	−0.697	0.000	15.5
Ser.	4- 20	3.54×3.54×0.276	19.4	45.5	0.000	0.685	50.9
Ser.	4- 40	3.54×3.54×0.276	39.8	45.5	0.000	0.685	48.9
Ser.	4- 60	3.54×3.54×0.276	59.7	45.5	0.000	0.685	46.5
Ser.	4- 70	3.54×3.54×0.276	69.6	45.5	0.000	0.685	43.7
Ser.	4- 80	3.54×3.54×0.276	79.5	45.5	0.000	0.685	45.3
Ser.	4- 90	3.54×3.54×0.276	89.5	45.5	0.000	0.685	46.2
Ser.	4-100	3.54×3.54×0.276	99.4	45.5	0.000	0.685	42.5
Ser.	4-110	3.54×3.54×0.276	109.4	45.5	0.000	0.685	43.3
Ser.	4-130	3.54×3.54×0.276	129.3	45.5	0.000	0.685	34.2
Ser.	4-150	3.54×3.54×0.276	149.2	45.5	0.000	0.685	25.8
Ser.	5- 20	3.54×3.54×0.276	19.4	42.7	0.697	0.685	37.8
Ser.	5- 40	3.54×3.54×0.276	39.8	42.7	0.697	0.685	31.6
Ser.	5- 60	3.54×3.54×0.276	59.7	42.7	0.697	0.685	27.9
Ser.	5- 70	3.54×3.54×0.276	69.6	42.7	0.697	0.685	25.4
Ser.	5- 80	3.54×3.54×0.276	79.5	42.7	0.697	0.685	23.8
Ser.	5- 90	3.54×3.54×0.276	89.5	42.7	0.697	0.685	22.1
Ser.	5-100	3.54×3.54×0.276	99.4	42.7	0.697	0.685	21.7
Ser.	5-110	3.54×3.54×0.276	109.4	42.7	0.697	0.685	20.7
Ser.	5-130	3.54×3.54×0.276	129.3	42.7	0.697	0.685	18.2
Ser.	5-150	3.54×3.54×0.276	149.2	42.7	0.697	0.685	14.7
Ser.	1- 20	3.54×3.54×0.276	19.4	44.0	000.0	0.000	79.0
Ser.	1- 40	3.54×3.54×0.276	39.8	44.0	000.0	0.000	79.0
Ser.	1- 60	3.54×3.54×0.276	59.7	44.0	000.0	0.000	76.7
Ser.	1- 70	3.54×3.54×0.276	69.6	44.0	000.0	0.000	78.0
Ser.	1- 80	3.54×3.54×0.276	79.5	44.0	000.0	0.000	70.6
Ser.	1- 90	3.54×3.54×0.276	89.5	44.0	000.0	0.000	62.4
Ser.	1-100	3.54×3.54×0.276	99.4	44.0	000.0	0.000	59.0
Ser.	1-110	3.45×3.54×0.276	109.4	44.0	000.0	0.000	51.1

mens. Complete description of the specimens is provided in Table 1(a).

2. Mueller and Erzurumlu (1983): Test results of fourteen 3×3×¼-in. and two 5×3×¼-in. single angle specimens are included in the present study. The specimens had ball joints at the ends to permit free rotation in any direction. The slenderness ratios were approximately 60, 120, and 200. The details of the specimens selected for the present study are given in Table 1(b).
3. Ishida (1968): Tests were conducted by Ishida on seven high strength steel angles of size 75×75×6 mm (3×3×¼-in.). Load was applied on the major principal axis to-

Table 1b. Results of Experimental Study on Eccentrically Loaded Single Angles Mueller & Erzurumlu (1983)						
SPECIMEN #	D/in. B/in. t/in.	KL/r	F_y/ksi	Z-eccen. in.	W-eccen. in.	P-test kips
S3 BB 36-1	3.00×3.00×0.250	60.8	54.8	–0.200	0.000	33.0
S3 BB 36-2	3.00×3.00×0.250	60.8	54.8	0.200	0.000	37.5
S1 BB 36-2	3.00×3.00×0.250	111.4	54.8	0.200	0.000	22.0
S2 BB 36-3	3.00×3.00×0.250	192.4	50.6	0.200	0.000	9.9
T2 BB 50-1	3.00×3.00×0.250	192.4	53.8	–0.086	–1.105	8.6
T2 BB 36-1	3.00×3.00×0.250	192.4	54.8	–0.086	–1.105	10.0
T1 BB 50-1	3.00×3.00×0.250	111.4	53.8	–0.086	–1.105	22.0
T3 BB 50-1	3.00×3.00×0.250	60.8	53.8	–0.086	–1.105	35.8
T3 BB 36-1	3.00×3.00×0.250	60.8	54.8	–0.086	–1.105	30.2
S3 BB 50-1	3.00×3.00×0.250	60.8	56.4	0.200	1.390	26.2
SR3 BB 36-1	3.00×3.00×0.250	60.8	54.8	–0.200	0.000	46.0
TR3 BB 5011	3.00×3.00×0.250	60.8	61.3	–0.086	–1.105	44.0
TR3 BB 50-1	3.00×3.00×0.250	60.8	61.3	–0.086	–1.105	41.0
TR2 BB 50-1	3.00×3.00×0.250	192.0	51.0	–0.086	–1.105	10.6
T4 BB 36-1	3.00×5.00×0.250	77.4	48.1	0.200	0.000	12.3
T4 BB 36-2	3.00×5.00×0.250	77.4	48.1	0.200	0.000	11.8

wards the shear center at an eccentricity with respect to minor principal axis equal to the minimum radius of gyration. Slenderness ratios of specimens varied from 20 to 100. Complete description of the selected specimens is provided in Table 1(c).

All the eccentricities given above are with respect to the principal axes as shown in Fig. 1.

The main aim of the present paper is to bring to the attention of the designers the degree of conservatism involved in the design of eccentrically loaded steel single angle struts. The

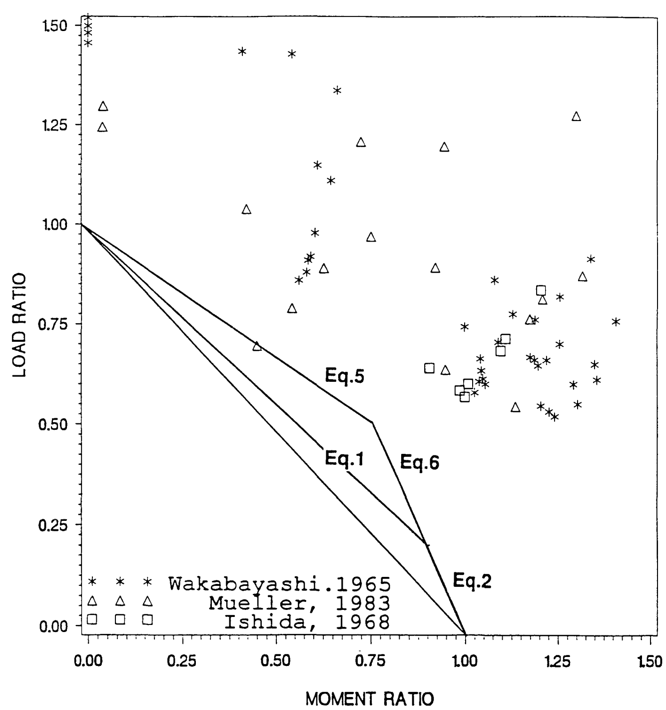

Fig. 2. Load ratio vs. moment ratio for test specimens as per AISC LRFD provisions with critical bending stress equal to yield stress.

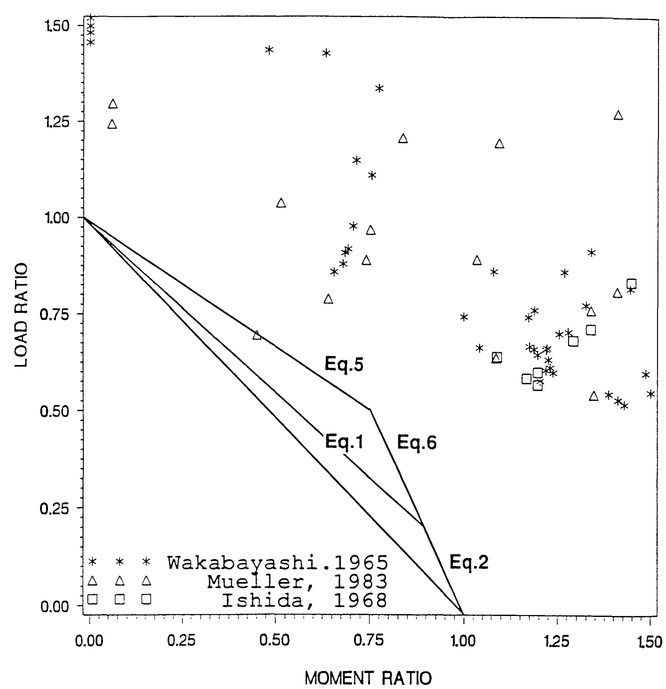

Fig. 3. Load ratio vs. moment ratio for test specimens as per AISC LRFD provisions with critical bending stress computed from formulas adapted from AISC ASD specification.

Table 1c. Results of Experimental Study on Eccentrically Loaded Single Angles Ishida (1968)						
SPECIMEN #	D × B × t (in. × in. × in.)	KL / r	F_y ksi	Z-eccen. in.	W-eccen. in.	P-test kips
SHY40-1E-1	2.97×2.97×0.255	19.8	63.9	0.586	0.000	47.4
SHY40-1E-2	2.95×2.99×0.255	39.5	63.9	0.586	0.000	39.1
SHY40-1E-3	2.96×2.96×0.256	59.6	63.9	0.583	0.000	30.5
SHY40-1E-4	2.94×2.94×0.260	79.9	63.9	0.580	0.000	25.6
SHY40-1E-5	2.94×2.94×0.253	99.7	63.9	0.581	0.000	19.4
SHY36-2E-1	2.95×2.95×0.255	39.7	58.8	0.583	0.000	35.6
SHY36-2E-2	2.95×2.95×0.252	79.5	58.8	0.583	0.000	23.4

study was originally undertaken in order to verify a growing feeling among some of the practising engineers that the LRFD formulas are conservative for the design of single angles even though the application of the formulas is quite involved in view of the necessity for solving cubic equations.

ANALYSIS OF TEST DATA

The data described above is used to evaluate the interaction formulas of LRFD and ASD specifications. Figures 2 through 6 show different possible ways of combining the effects of axial load and bending moments on the steel single angle struts for use with the current design practice. In all the figures, the Y-axis shows the axial load ratio (P_u / ϕP_n for LRFD—Figs. 2 to 4 and f_a / F_a for ASD—Figs. 5 and 6). The X-axis shows the corresponding moment ratios. The moment ratios in Fig. 2 are calculated using the present LRFD approach with a critical bending compresive stress equal to F_y (see example problem in AISC LRFD). Figure 3 calculates the critical bending stress using the formulas similar to those given in the current ASD Specification and uses the moment ratios as per LRFD. This approach was undertaken in order to see the effect of a more rational calculation of critical bending stress (as per ASD formulas) when compared to the presently used value of F_y. Figure 4 shows results obtained using the interaction formula at each of the possible critical points by considering the moment capacities due to the critical bending stress at that point (calculated using the formulas adopted from the ASD provisions) simultaneously with re-

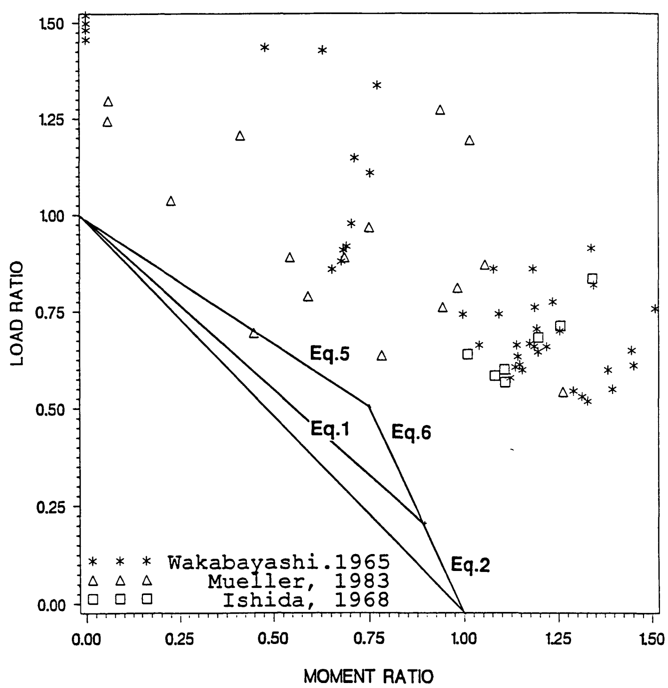

Fig. 4. Load ratio vs. moment ratio for test specimens using AISC LRFD provisions with critical bending stress computed from formulas adapted from AISC ASD specification and interaction applied at all possible critical points.

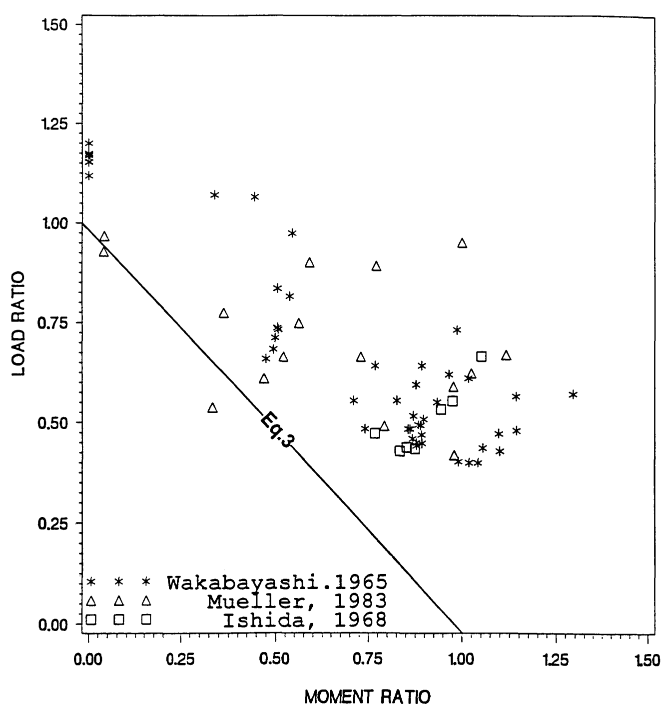

Fig. 5. Load ratio vs. moment ratio for test specimens as per AISC LRFD provisions.

spect to bending about both axes. The sign of the moment ratio is taken as positive if the bending moment about the corresponding principal axis produces compressive stress at the point under consideration and is taken as negative otherwise. The moment ratios due to the two bending moments are added algebraically and the largest sum is used for plotting the interaction diagram. Figure 5 shows the results plotted as per the existing ASD provisions. Figure 6 shows the results as per ASD provisions, plotted using an approach similar to that used for plotting Fig. 4. Table 2 gives the summary of the various comparisons.

DISCUSSION

The current interaction formulas in AISC LRFD Specification were derived mainly for doubly symmetric sections that are normally used in frames. But as can be seen from Figs. 2 to 6 and Table 2, these interaction formulas are highly conservative when applied to eccentrically loaded single angle struts. Several alternatives which are not reported herein were studied to predict the strength of eccentrically loaded single angle struts. The alternatives included approaches similar to those adopted by different national specifications and other approaches which manipulate the interaction factors such as the moment amplification factors. All such methods are found to be either highly conservative or unacceptable. The simplest approach for modifying the present formulas is to retain the present interaction formulas in their entirety and imposing the

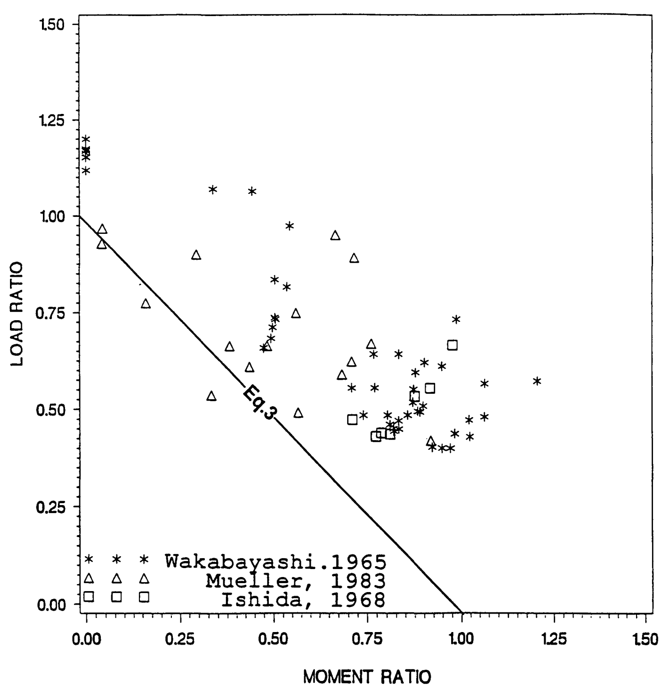

Fig. 6. Load ratio vs. moment ratio for test specimens using AISC LRFD provisions with interaction applied at all possible critical points.

application of interaction at all salient points on the cross-section. An additional benefit could be obtained simply by changing the values of the moment interaction factors resulting in equations given below:

Case 1: For $\frac{P_u}{\phi_c P_n} \geq 0.5$,

$$\frac{P_u}{\phi_c P_n} + \frac{2}{3}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad (5)$$

Case 2: For $\frac{P_u}{\phi_c P_n} < 0.5$,

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad (6)$$

This gives an increase of nine percent on an average in computed axial load capacity over simple interaction equation (without any modifying factors) for the test specimens under consideration while the existing provisions give a three percent higher axial load on an average over the load that is computed using a simple interaction equation. The same interaction factors are also applicable for the approaches shown in Figs. 3 and 4. Figure 3 does not show results that are drastically different from the results shown in Fig. 2 because for the majority of test results used in the investigation, the critical bending stress is only five percent to ten percent lower than the yield stress F_y although other practical cases can have a significantly different critical bending stress. Similarly, Fig. 4 is not very different from Fig. 3 for the data from Wakabayashi and Nonaka (1965) and Ishida (1968) because the load eccentricities of the test specimens are mostly either on the principal axes or in first and fourth quadrants with respect to the principal axes. The results for Mueller and Erzurumlu (1983) are different in Fig. 3 and Fig. 4 because the load point for most test cases falls narrowly into the third quadrant. Nevertheless, Figs. 3 and 4 serve to show that the corresponding approaches satisfy the test results. While the conservative results prove that the design is safe, they also show that the safety margin is not uniform for all the different types of structural members. The safety margin for LRFD is generally measured by the reliability index β. The value β effectively fixes the load and resistance factors. For a given value of β, the resistance factor could be computed as (Bjorhovde et al. 1978)

$$\phi = \frac{R_m}{R_n} e^{-\alpha\beta V_r} \qquad (7)$$

where R_m is the mean resistance, R_n is the nominal resistance, α is 0.55, and V_r is the coefficient of variation of the resistance of the member. Using the results shown in Table 2 and known values of other statistical properties and ϕ, reliability index β can be calculated. The calculations show a β value higher

Table 2. Comparison of Compressive Strengths Calculated According to AISC Interaction Formulas with Test Failure Loads (P_calc/P_test)								
	Wakabayashi...		Mueller...		Ishida		Total	
Method of Calculation	**Mean**	**Std. Dev.**	**Mean**	**Std. Dev.**	**Mean**	**Std. Dev.**	**Mean**	**Std. Dev.**
AISC LRFD existing provisions (Fig. 2)	0.65	0.08	0.69	0.10	0.68	0.08	0.66	0.08
Same as above but with interaction factor changed to ⅔	0.70	0.07	0.75	0.10	0.77	0.08	0.72	0.08
AISC LRFD with critical bending stress as per ASD provisions (Fig. 3)	0.62	0.06	0.66	0.11	0.62	0.08	0.63	0.08
Same as above but with interaction factor changed to ⅔	0.67	0.07	0.71	0.11	0.69	0.08	0.68	0.08
AISC LRFD with critical bending stress as per ASD and with interaction at all critical points (Fig. 4)	0.63	0.06	0.71	0.09	0.64	0.08	0.65	0.08
Same as above but with interaction factor changed to ⅔	0.68	0.07	0.76	0.09	0.72	0.08	0.71	0.08
AISC ASD existing provisions (Fig. 5)	0.77	0.08	0.83	0.14	0.76	0.10	0.78	0.10
AISC ASD with interaction at all critical points (Fig. 6)	0.79	0.07	0.90	0.12	0.79	0.10	0.81	0.10

than 6.0 for the cases listed and for several cases not presented herein. A consistently high value for the reliability index shows that the present alternatives relying on simple variation to the current LRFD formulas are unusually conservative when compared to the reliability index for the other types of members which is usually between two and four. Any further reduction in the degree of conservatism of the sections would warrant recourse to more complicated but exact formulas. One such set of formulas derived for beam columns uses an exponential format for the two moment ratios (Chen and Atsuta, 1976 and Chen and Lui, 1987). The calibration of these formulas involves extensive numerical integration and comparison with test results. This has been done for the case of I-sections in braced frames and the corresponding formulas are given in the appendix to AISC LRFD (1986). Such formulas tend to be very complicated for practical design applications and are not normally used. The alternative to such methods is simpler variations of the existing formulas in a more empirical sense. The present study shows the amount of improvement possible through such methods. Any further improvement in the formulas would most likely result in more complicated formulas.

The calculations also show that the critical bending stress calculated using the more rational formulas of ASD is less than the normally assumed value of F_y but not by a great margin for test specimens under consideration.

Figures 5 and 6 show the test results as per the ASD practice. Both the curves confirm that the ASD formulas are good for the test cases. However, in view of the discussion above, the approach of applying interaction at all possible critical points is highly desirable. It may be noted that for most practical cases, the load position is in the second and third quadrants where the difference in the two approaches is highly pronounced. It should also be noted that the critical point on the section is almost invariably the point of maximum compression. The determination of the point of maximum compression needs checking at two points if the load is placed in the first or fourth quadrants (points 2, 5 or 4, 6 respectively in Fig. 1) and at least three points if the load is placed in second or third quadrants (points 2, 3, 5 or 3, 4, 6 respectively in Fig. 1) for every angle. This would involve some extra computation when compared to current design practice. This however, would lead to a noticeable economy.

CONCLUSIONS

From the above discussion, the following conclusions can be arrived at:

1. Design interaction equations should combine the effects of biaxial moments at all critical points separately. This is especially so if the load is located in the second and third quadrants of principal axes (causing compression at heel).
2. The present AISC LRFD interaction equations are highly conservative for eccentrically loaded single angle struts.
3. The critical bending stress is less than the usually assumed value (equal to F_y) but is close to it for the test specimens studied.
4. For single angle struts, the moment interaction factors can be changed to ⅔ (from the present $\frac{8}{9}$) for the range of P_u / ϕP_n between 1.0 and 0.5.
5. The present AISC ASD specifications compare very favorably with the test results under consideration. However, it is highly desirable that the interaction is applied at all the possible critical points.

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