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Abstract: The structural behavior of high-strength cold-formed steel beams subjected primarily to web crippling is investigated analytically and experimentally. Hat sections and I-beams, formed from five different types of high-strength sheet steels, are investigated under various loading conditions. In addition to the test data gathered in this phase of study, the experimental results obtained from the beam tests conducted at various companies are carefully evaluated. The evaluation indicates that the present available design criteria are not suitable for high-strength materials with yield strengths exceeding 80 ksi (552 MPa). New design formulas are derived for different types of loading conditions on the basis of available test data with material yield strengths ranging from 30 to 165 ksi (207 to 1,138 MPa). These empirical equations distinguish between web crippling failure caused by overstressing (bearing failure) and web buckling. Interaction formulas are derived for sections under combined web crippling and bending moment. The newly proposed design equations are verified by test results.

Introduction

In recent years, high-strength steels have been widely used in automotive structural components to achieve weight reduction while complying with federal safety standards. Since 1981, the American Iron and Steel Institute (AISI) has published the *Guide for Preliminary Design of Sheet Steel Automotive Structural Components* (1981) and the *Automotive Steel Design Manual* (1986) for the effective use of sheet steels.

Because high-strength steels with yield strengths from 80 to 190 ksi (552 to 1,310 MPa) are now used for automotive structural components ("Steel" 1983; Errera 1982; Levy 1983) (Santaputra et al. 1986; Vecchio 1983; Yu et al. 1983), and because many of the existing design expressions have not been verified for very high yield strength materials, a comprehensive design guide is highly desirable. Recently, a research project entitled "Design of Automotive Structural Components Using High Strength Sheet Steels" has been conducted at the University of Missouri-Rolla under the sponsorship of AISI. The main purpose of the project has been to develop additional design criteria for the use of a broader range of high-strength sheet steels. Web crippling and a combination of web crippling and bending moment is one area that has been studied as a part of the research project.

This paper presents the results of tests carried out to determine the web-crippling strength of webs of cold-formed steel beams fabricated from high-strength sheet steels commonly used in the automobile industry. Various loading arrangements were investigated in the experimental study. New de-

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J. Struct. Eng., 1989, 115(10): 2511-2527

sign criteria to prevent web crippling and a combination of web crippling and bending moment are proposed.

Experimental Investigation

The theoretical analysis concerning web crippling of cold-formed steel beams is extremely complicated. There exists no exact or closed-form solution to the problem. Because of mathematical difficulties encountered, the present AISI design provisions for web crippling are based on extensive experimental investigations conducted at Cornell University, the University of Missouri-Rolla (UMR), other institutions and individual steel companies (Yu 1985). However, all of the mentioned web-crippling research work has been for materials with yield strengths lower than 60 ksi (414 MPa). In this paper, web-crippling tests of materials with yield strengths of 58.2 to 165.1 ksi (401 to 1,138 MPa) are discussed.

In the present study, web-crippling tests were carried out for the following four basic loading conditions.

1. Interior one-flange loading (IOF).
2. End one-flange loading (EOF).
3. Interior two-flange loading (ITF).
4. End two-flange loading (ETF).

To avoid the problem of discontinuity between the web-crippling equations for the aforementioned basic loading conditions, additional tests were performed for the transition ranges.

Two types of sections were tested. Hat sections, as shown in Fig. 1(*a*), were tested in order to study single unreinforced webs. For sections that provide a high degree of restraint against rotation of the webs, I-beams [Fig. 1(*b*)] were tested. The experimental investigations are summarized as follows.

Test Specimens

Hat sections and I-beams were fabricated from five different types of high-strength sheet steels. Tables 1 and 2 shows three different profiles for each type of cross section used in this study. The materials used to form these specimens included hot-rolled and cold-rolled sheet steels having yield strengths ranging from 58.2 to 165.1 ksi (401 to 1,138 MPa). Table 3 gives the me-

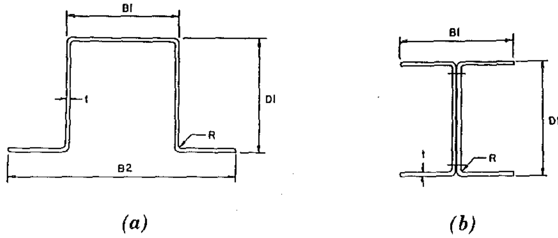

FIG. 1. Hat Sections and I-Beams Used in this Study

J. Struct. Eng., 1989, 115(10): 2511-2527

TABLE 1. Nominal Dimensions of Hat Sections Designed for Experimental Study

Profile number (1)	$B1$ (in.) (2)	$B2$ (in.) (3)	$D1$ (in.) (4)	R (in.) (5)
1	3.0	6.0	3.0	0.25
2	4.0	8.0	4.0	0.25
3	5.0	10.0	5.0	0.25

Note: See Fig. 1(a) for definitions of symbols. 1 in. = 25.4 mm.

TABLE 2. Nominal Dimensions of I-Beams Designed for Experimental Study

Profile number (1)	$B1$ (in.) (2)	$D1$ (in.) (3)	R (in.) (4)
1	3.0	3.0	0.25
2	4.0	4.0	0.25
3	5.0	5.0	0.25

Note: See Fig. 1(b) for definitions of symbols. 1 in. = 25.4 mm.

chanical properties and thicknesses of these sheet steels. Because all specimens were formed by a press-braked operation, there was little or no cold working effect, except in the corners.

All I-beam specimens were fabricated from two identical channels connected back-to-back by self-tapping screws (14 × 3/4 Tek screws) located at a distance of 1/2 in. (12.7 mm) from top and bottom flanges. The self-tapping screws were spaced along the beam length at a constant distance of 2 in. (50.8 mm) from center to center. To minimize the initial deformations, the screws were driven from alternate sides of the webs.

The pilot tests of I-beam specimens indicated that there was premature failure caused by rotation of the flanges about the screw lines. The premature failure was caused by the large bend radii, which are required for the high-strength and low-ductility sheet steels. This type of failure is also a function of the distance between the flanges and screw lines. Fig. 2 shows the sketch of this type of failure. To prevent this premature failure before web crippling could be developed, bearing plates were bolted to flanges of I-beams.

TABLE 3. Material Properties[a] and Thicknesses of Sheet Steels

Material (1)	F_y (ksi)[c] (2)	F_u (ksi) (3)	Elongation[b] (%) (4)	t (in.)[d] (5)
80DK	58.2	87.6	25.7	0.048
80XF	77.1	89.1	20.4	0.088
100XF	113.1	113.1	8.1	0.062
140XF	141.2	141.2	4.4	0.047
140SK	165.1	176.2	4.3	0.046

[a]Material properties are based on average longitudinal tension tests.
[b]Elongation in 2-in. gage length.
[c]1 ksi = 6.894 MPa.
[d]1 in. = 25.4 mm.

J. Struct. Eng., 1989, 115(10): 2511-2527

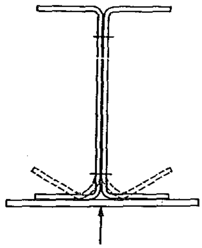

FIG. 2. Sketch Showing Bending Failure about Connection Line

For hat sections, the bottom unstiffened flanges were connected by 1/8 × 3/4 in. (3.175 × 19.05 mm) rectangular bars at appropriate locations to maintain the shape of the cross sections during the test.

Test Procedure

All tests were performed in a 120,000-lb (533,770-N) Tinius Olsen universal testing machine. All specimens were loaded to failure. During the test, loads were applied slowly at an increment of approximately 15% of the predicted ultimate loads and kept constant at each load level for 5 min.

Vertical movement of the bearing plate by which the load was applied was recorded at each step to detect the load and time at which the bearing plate started to penetrate into the web.

Lateral deformation of the webs of hat sections at the location of expected failure was measured. The lateral movement was measured at 1/2-in. (12.7-mm)spacing along the centerline of the bearing plate by linear variable differential transformer (LVDT). Readings were taken at each load interval. For the I-beam specimens, there was no lateral movement of the webs until the ultimate loads were reached.

Vertical strain distribution in the web beneath the bearing plate was also investigated by attaching strain gages to some of the hat sections. For each of the specimens studied, three pairs of strain gages were attached back-to-back vertically along a horizontal line at a distance of 1/4 in. (6.35 mm) from the center of the gages to the web-flange junction. Details of the test arrangement for each loading condition are as follows.

Interior One-Flange Loading (IOF)

All specimens were tested as simply supported flexural members subjected to a concentrated load as shown in Fig. 3(*a*). A total of 36 hat sections and 24 I-beams were loaded at mid-span with the clear distance between opposite bearing plates (e_1 and e_2) of 1.5h, where h is the clear distance between flanges measured along the plane of the web. An additional 36 hat sections were tested under unsymmetric loading, in which e_1 varied from 0.75h to 1.25h and e_2 varied accordingly from 2.25h to 1.75h.

A 2-in. (50.8-mm) bearing plate was placed under the applied concentrated load, while 4-in. (101.6-mm) bearing plates were used at both ends. To prevent end failure from occurring before the expected interior failure developed, wood blocks were inserted at both ends of the specimens.

J. Struct. Eng., 1989, 115(10): 2511-2527

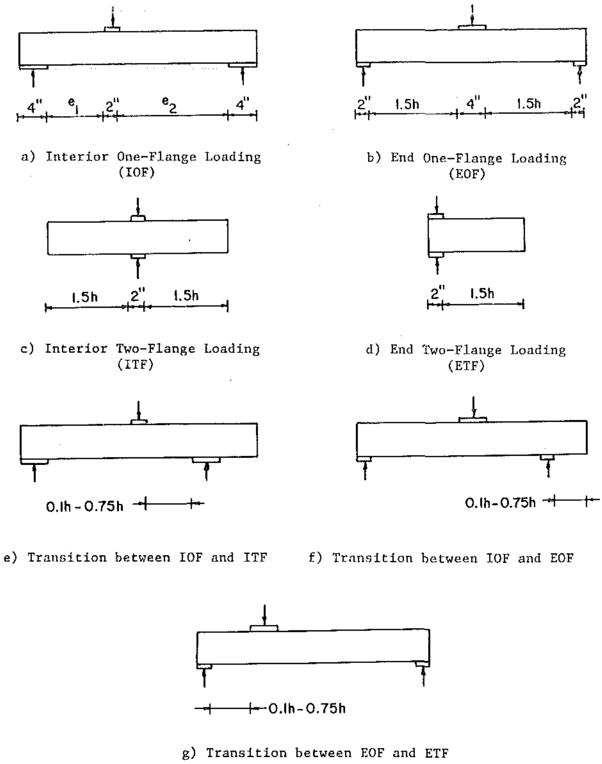

FIG. 3. Test Arrangements

End One-Flange Loading (EOF)

For this loading condition, 30 hat sections and 24 I-beams were tested as simply supported flexural members subjected to a concentrated load at midspan [Fig. 3(*b*)]. A 4-in. (101.6-mm) bearing plate was under the concentrated load while 2-in. (50.8-mm) bearing plates were used at both ends. The clear distance between the opposite bearing plates was set at $1.5h$.

Interior Two-Flange Loading (ITF)

A total of 24 hat sections and 24 I-beams were tested as shown in Fig. 3(*c*). Two 2-in. (50.8-mm) bearing plates were used in the middle of the specimens for both top and bottom flanges. The designed clear distance between the edge of bearing plates to the end of the specimens was $1.5h$.

End Two-Flange Loading (ETF)

The numbers of specimens are the same as the ITF case. It can be seen from Fig. 3(*d*) that two 2-in. (50.8-mm) bearing plates were used at one end of the specimens. At the unloaded end of the specimens, an elastic support was used to keep the specimens in a horizontal position throughout the test.

J. Struct. Eng., 1989, 115(10): 2511-2527

Transition between Interior One-Flange Loading and Interior Two-Flange Loading

For the transition ranges between interior one-flange loading and interior two-flange loading, six hat sections were tested. Fig. 3(*e*) shows the test arrangement, which was the same as that of the IOF case, except that one end-bearing plate was moved to vary the clear distance between the opposite bearing plates from $0.1h$ to $0.75h$. The expected failure was under the applied concentrated load.

Transition between Interior One-Flange Loading and End One-Flange Loading

The test setup was the same as that of the EOF case, except that the end-bearing plates were moved closer to the bearing plate under the applied concentrated load [Fig. 3(*f*)]. For this transition range also, six hat sections were tested. Failure was expected at the end support reaction.

Transition between End One-Flange Loading and End Two-Flange Loading

As in the previous case, the test arrangement was the same as that of the EOF case, but the bearing plate under the applied concentrated load was moved closer to one end of the specimen [Fig. 3(*g*)]. The expected failure was at the end-bearing plate closer to the applied load. Similarly to the other two transition ranges, six hat sections were tested for this case.

Test Results

The ultimate web-crippling loads were recorded for all tests. Lateral deformations of the webs and vertical deflections of the beams were recorded, as discussed earlier. Because the specimens were unstable at the ultimate loads, all deflection and deformation measurements could not be obtained at this load level. The nature of failure was carefully inspected throughout the tests. The following two types of web crippling failure were observed.

Overstressing (Bearing) Failure

Failure of this type occurred just under the bearing plates, with relatively small lateral deformations of the webs of the hat sections. For I-beam specimens, no lateral movement of the webs was observed before the ultimate loads were reached. The applied load increased steadily up to the ultimate load and remained at that level for a long period of time while the bearing plate gradually penetrated into the web. It was believed that overstressing of the web underneath the bearing plate caused this type of failure.

Buckling Failure

For this type of failure, the load increased steadily up to the ultimate load. At the ultimate load, the web became unstable and the load dropped suddenly. There were relatively large lateral deformations of the webs of the hat sections even before failure. Paired strain gages that were attached to the webs of some of the hat sections indicated that the strains caused by bending of the web were much more pronounced than the strain caused by the vertical compressive stress. However, few, if any, lateral deformations occurred in the webs of the I-beams prior to buckling. Figs. 4 and 5 show the typical failure for each case of the four basic loading conditions for hat sections and I-beams, respectively.

J. Struct. Eng., 1989, 115(10): 2511-2527

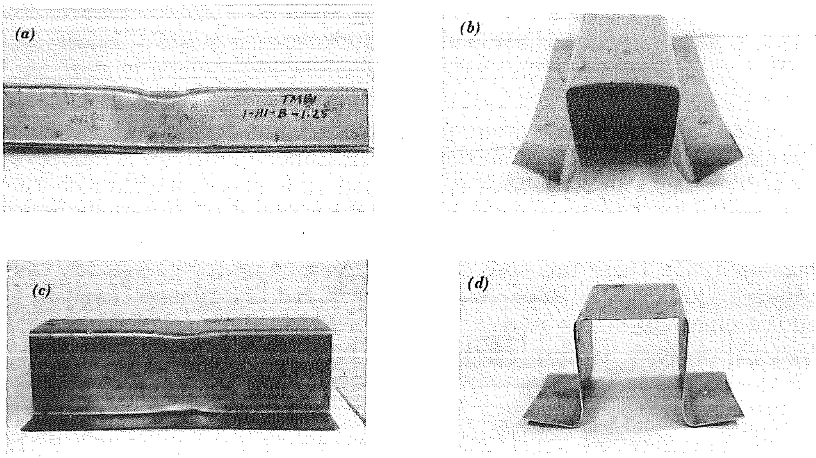

FIG. 4. Typical Failure Modes of Hat Sections: (*a*) Interior One-Flange Loading; (*b*) End One-Flange Loading; (*c*) Interior Two-Flange Loading; (*d*) End Two-Flange Loading

Evaluation of Experimental Data

The results of tests obtained from this phase of the investigation were evaluated by comparing the tested failure loads to the predicted ultimate web-crippling loads based on the AISI *Guide* (1981) and the *Specification* (1986). These comparisons are discussed for each type of section. It should be noted that the empirical expressions for these predictions were derived

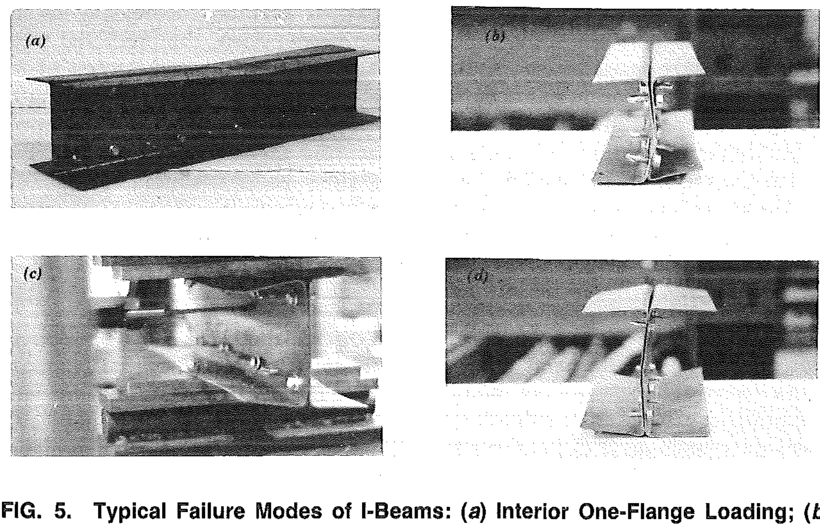

FIG. 5. Typical Failure Modes of I-Beams: (*a*) Interior One-Flange Loading; (*b*) End One-Flange Loading; (*c*) Interior Two-Flange Loading; (*d*) End Two-Flange Loading

from test data with yield strengths ranging from 30 to 57 ksi (207 to 393 MPa), which are applicable to building products.

Hat Sections

For hat sections formed from very high-strength sheet steels, the tested web-crippling loads are underestimated. This underestimation is caused by the yield strength functions used in the equations to predict the ultimate web-crippling loads. The functions that reflect the effect of yield strength on the ultimate web crippling load are the same in both the *Guide* (1981) and *Specification* 1986. These functions for the yield strength for interior and end-loading conditions can be expressed as Eq. 1 and Eq. 2, respectively

$$f_1(F_y) = \left[1.22 - 0.22\left(\frac{F_y}{33}\right)\right]\left(\frac{F_y}{33}\right) \qquad (1)$$

$$f_2(F_y) = \left[1.33 - 0.33\left(\frac{F_y}{33}\right)\right]\left(\frac{F_y}{33}\right) \qquad (2)$$

According to these two equations, the ultimate web-crippling load for a given section increases as the yield strength, F_y, increases up to a certain value, beyond which the ultimate web-crippling load decreases as the yield strength increases. The functions $f_1(F_y)$ and $f_2(F_y)$ reach their maximum values when F_y is 91.5 and 66.5 ksi (630.8 and 458.5 MPa), respectively. In Eqs. 1 and 2, when the actual yield strength was greater than this limit, the value of 91.5 or 66.5 ksi (630.8 or 458.5 MPa) was used in lieu of the actual value of F_y. Even though these modifications of the yield strength functions were used, the inaccuracy of the predicted web crippling loads still persisted.

I-Beams

The predicted ultimate web-crippling loads based on the 1981 *Guide* overestimate the failure loads with the degree of accuracy decreasing as the yield strength increases for all four loading cases. Also, the predicted web-crippling loads based on the 1986 *Specification* have the same trend of inaccuracy. This overestimation resulted mainly from using the yield strength, F_y, as a parameter in the prediction equations, even though most of the specimens have a buckling-type failure.

For the cases of ITF and ETF, the equations based on the 1986 *Specification* are independent of F_y, which is consistent with the observations made in this phase of study. The inaccuracy in predictions for these cases may result from using inappropriate parameters for the buckling-type failure.

Development of New Equations

As discussed earlier, even though the present AISI design criteria for web crippling can be used for the design of buildings, they are not suitable for application with sections fabricated from materials with yield strengths higher than approximately 80 ksi. To cover a wider range of yield strengths, new prediction equations were developed. The new equations distinguish between overstressing (bearing) and buckling failure. These equations were developed on the basis of the available data obtained from the following sources.

J. Struct. Eng., 1989, 115(10): 2511-2527

1. Previous Cornell and UMR tests reported in Hetrakul and Yu (1978).
2. Recent UMR tests conducted by Lin and reported in Lin (1984).
3. Tests of high-strength sections conducted in this study.

The parameters used in the derivation of the new equations are nondimensional terms. A nonlinear least-squares iteration technique was used to determine the empirical constants. The form for each type of equation is discussed as follows.

Overstressing (Bearing) Failure

The basic nondimensional parameters used in the derivations of the equations are $P/(t^2F_y)$, N/t, and R/t. For this type of failure, the ratio h/t has little or no effect on the ultimate web-crippling load. All data used for evaluating the interior one-flange loading case were selected in such a way that the ratio M/M_u was less than 0.3. Previous research work (Hetrakul and Yu 1978) indicated that when the moment ratio is less than this limit there is little or no interaction between web crippling and bending moment. Eq. 3 deals with this type of failure

$$\frac{P_{cy}}{t^2F_y} = f\left(\frac{N}{t}, \frac{R}{t}\right) \qquad (3)$$

Buckling Failure

The design equation for buckling failure was developed by using the same form of the equation as used for buckling of a rectangular flat plate subjected to partial edge loading (Khan and Walker 1972; Khan et al. 1977), except that the length-to-depth ratio (L/h) was replaced by the clear distance between the opposite bearing plate-to-depth ratio (e/h). By using this parameter, the equations can be applied to either symmetric or unsymmetric loading. This type of equation can be written as

$$\frac{P_{cb}}{Et^2} = f\left(\frac{N}{h}, \frac{h}{t}, \frac{e}{h}\right) \qquad (4)$$

Combined-Bending and Web Crippling

To be consistent with the current AISI *Specification* (1986), the interaction equations were determined in terms of the moment ratio (M/M_u) and the load ratio (P_{mc}/P_{cy}) as given here

$$\left(\frac{M}{M_u}\right) + A\left(\frac{P_{mc}}{P_{cy}}\right) \leq C \qquad (5)$$

In Eq. 5 A and C are constants to be determined by regression analysis. The predicted combined bending and web-crippling load, P_{mc} determined from Eq. 5 must be checked against P_{cb} of the interior one-flange loading case. The smaller value between these predicted loads governs the design.

All of the newly developed equations are included in the design recommendations that are proposed in this paper. The predicted ultimate web-crippling loads based on these newly developed equations give good agreement with the tested failure loads for all cases with yield strengths from 30 to 165 ksi (207 to 1,138 MPa). Fig. 6 shows the effect of the material yield strength,

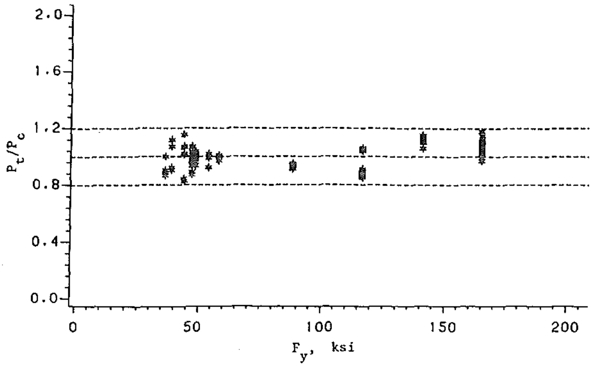

FIG. 6. Load Ratio, P_t/P_c, versus Yield Strength, F_y, for Hat Sections Subjected to Interior One-Flange Loading

F_y, on the ratio of the tested failure loads to the predicted ultimate web-crippling loads, P_t/P_c, for hat sections subjected to interior one-flange loading. For other cases, see Santaputra and Yu (1986) and Santaputra (1986).

Proposed Design Recommendations

Based on the research findings, design methods intended to cover all possible failure modes of web crippling and a combination of web crippling and bending moment were developed. It should be noted that the following design equations predict the ultimate strength without any factor of safety.

Concentrated Loads and Reactions

The ultimate strength of an unreinforced beam web subjected to concentrated loads or reactions can be estimated by the equations given in design recommendation A for beams having single webs and in design recommendation B for I-beams with flanges connected to bearing plates. The equations apply to beams when $F_y \leq 190$ ksi (1,310 MPa), $h/t \leq 200$, $N/t \leq 100$, $N/h \leq 2.5$, and $R/t \leq 10$.

The design methods for web crippling were categorized into nine cases depending on the values of e and Z (Figs. 7–10). The equations used for Cases 1, 2, 4, and 5 were derived from the test data obtained from the four basic loading conditions classified as end one-flange loading, interior one-flange loading, end two-flange loading, and interior two-flange loading, respectively. Cases 3, 6, 7, 8, and 9 represent the transitions of the four basic loading conditions and can be determined by using simple interpolation.

Design Recommendation A

Ultimate concentrated loads and reactions for shapes having single unreinforced webs—$Z = 0$: $(P_c)_1$ is the smaller of P_{cy} or P_{cb} where

$$P_{cy} = 9.9t^2F_yc_{11}c_{21}(\sin\theta) \qquad e \geq 0.5h \quad (6)$$

$$P_{cb} = 0.047Et^2c_{41}c_{51}(\sin\theta) \qquad e \geq 0.5h \quad (7)$$

$Z \geq 0.5h$: $(P_c)_2$ is the smaller of P_{cy} or P_{cb} where

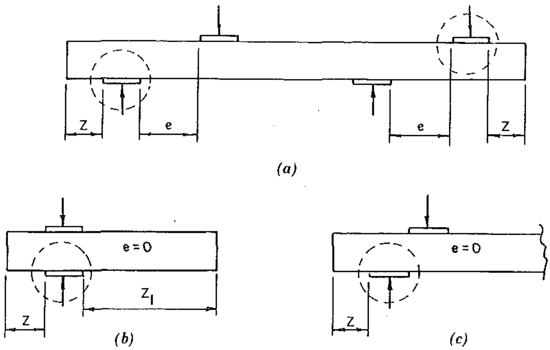

FIG. 7. Definitions of e and Z for Reactions and Concentrated Loads on Cantilevers

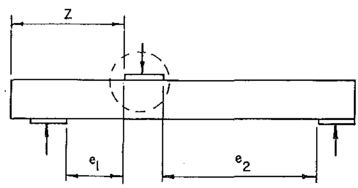

FIG. 8. Definitions of e and Z for Interior Concentrated Loads

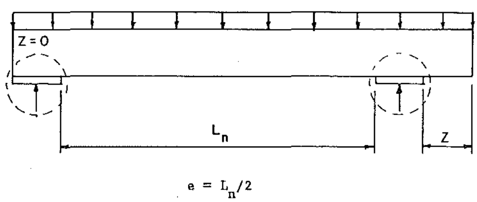

FIG. 9. Definitions of e and Z for End Reactions of Beams Supporting Uniformly Distributed Loads

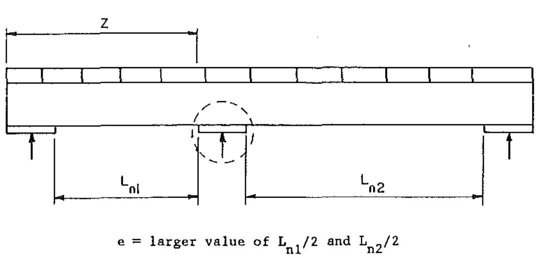

FIG. 10. Definitions of e and Z for Interior Reactions of Continuous Beams Supporting Uniformly Distributed Loads

$$P_{cy} = 7.80t^2F_y c_{12}c_{22}(\sin\theta) \qquad e \geq 0.5h \qquad (8)$$

$$P_{cb} = 0.028Et^2c_{32}c_{42}c_{52}(\sin\theta) \qquad e \geq 0.5h \qquad (9)$$

$0 < Z < 0.5h$:

$$(P_c)_3 = (P_c)_1 + [(P_c)_2 - (P_c)_1]\left(\frac{Z}{0.5h}\right) \qquad e \geq 0.5h \qquad (10)$$

$Z = 0$: $(P_c)_4 = P_{cb}$ where

$$P_{cb} = 0.011Et^2c_{33}c_{43}c_{73}(\sin\theta) \qquad e = 0 \qquad (11)$$

$Z \geq 0.5h$: $(P_c)_5$ is the smaller of P_{cy} or P_{cb} where

$$P_{cy} = 7.8t^2F_y c_{12}c_{22}(\sin\theta) \qquad e = 0 \qquad (12)$$

$$P_{cb} = 0.0041Et^2c_{34}c_{44}c_{64}(\sin\theta) \qquad e = 0 \qquad (13)$$

$0 < Z < 0.5h$:

$$(P_c)_6 = (P_c)_4 + [(P_c)_5 - (P_c)_4]\left(\frac{Z}{0.5h}\right) \qquad e = 0 \qquad (14)$$

$Z = 0$:

$$(P_c)_7 = (P_c)_4 + [(P_c)_1 - (P_c)_4]\left(\frac{e}{0.5h}\right) \qquad 0 < e < 0.5h \qquad (15)$$

$Z \geq 0.5h$:

$$(P_c)_8 = (P_c)_5 + [(P_c)_2 - (P_c)_5]\left(\frac{e}{0.5h}\right) \qquad 0 < e < 0.5h \qquad (16)$$

$0 < Z < 0.5h$:

$$(P_c)_9 = (P_c)_6 + [(P_c)_3 - (P_c)_6]\left(\frac{e}{0.5h}\right) \qquad 0 < e < 0.5h \qquad (17)$$

Design Recommendation B

Ultimate concentrated loads and reactions for I-Beams with Unreinforced webs—$Z = 0$: $(P_c)_1 = P_{cb}$ where

$$P_{cb} = 0.063Et^2c_{45}c_{55} \qquad e \geq 0.5h \qquad (18)$$

$Z \geq 0.5h$: $(P_c)_2$ is the smaller of P_{cy} or P_{cb} where

$$P_{cy} = 15t^2F_y c_{12} \qquad e \geq 0.5h \qquad (19)$$

$$P_{cb} = 0.032Et^2c_{36}c_{46} \qquad e \geq 0.5h \qquad (20)$$

$0 < Z < 0.5h$:

$$(P_c)_3 = (P_c)_1 + [(P_c)_2 - (P_c)_1]\left(\frac{Z}{0.5h}\right) \qquad e \geq 0.5h \qquad (21)$$

$Z = 0$: $(P_c)_4 = P_{cb}$ where

J. Struct. Eng., 1989, 115(10): 2511-2527

$$P_{cb} = 0.015Et^2c_{37}c_{47} \qquad e = 0 \quad (22)$$

$Z \geq 0.5h$: $(P_c)_5$ is the smaller of P_{cy} or P_{cb} where

$$P_{cy} = 15t^2F_yc_{12} \qquad e = 0 \quad (23)$$

$$P_{cb} = 0.051Et^2c_{38}c_{48}c_{68} \qquad e = 0 \quad (24)$$

$0 < Z < 0.5h$:

$$(P_c)_6 = (P_c)_4 + [(P_c)_5 - (P_c)_4]\left(\frac{Z}{0.5h}\right) \qquad e = 0 \quad (25)$$

$Z = 0$:

$$(P_c)_7 = (P_c)_4 + [(P_c)_1 - (P_c)_4]\left(\frac{e}{0.5h}\right) \qquad 0 < e < 0.5h \quad (26)$$

$Z \geq 0.5h$:

$$(P_c)_8 = (P_c)_5 + [(P_c)_2 - (P_c)_5]\left(\frac{e}{0.5h}\right) \qquad 0 < e < 0.5h \quad (27)$$

$0 < Z < 0.5h$:

$$(P_c)_9 = (P_c)_6 + [(P_c)_3 - (P_c)_6]\left(\frac{e}{0.5h}\right) \qquad 0 < e < 0.5h \quad (28)$$

In design recommendations A and B, E = modulus of elasticity of steel = 29,500 ksi (203,373 MPa); e = clear distance between edges of adjacent opposite bearing plates, in. (For reactions or concentrated loads on cantilevers, see Fig. 7. For interior concentrated load shown in Fig. 8, e is taken as the smaller value of e_1 and e_2); F_y = yield strength of the web, ksi; h = clear distance between flanges measured along plane of web, in.; N = actual length of bearing, in.; P_c = governing ultimate web-crippling load, per web, kips; P_{cb} = web-crippling load caused by buckling, per web, kips; P_{cy} = web-crippling load caused by bearing, per web, kips; R = inside bend radius, in.; t = web thickness, in.; Z = distance between edge of bearing plate to near end of beam, in. (see Figs. 7 and 8); Z_1 = distance between edge of bearing plate to far end of beam, in. (see Fig. 7); and θ = angle between plane of web and plane of bearing surface > 45° but no more than 90°.

$$c_{11} = 1 + 0.0122\left(\frac{N}{t}\right) \leq 2.22 \quad (29)$$

$$c_{12} = 1 + 0.217\left(\frac{N}{t}\right)^{0.5} \leq 3.17 \quad (30)$$

$$c_{21} = 1 - 0.247\left(\frac{R}{t}\right) \geq 0.32 \quad (31)$$

$$c_{22} = 1 - 0.0814\left(\frac{R}{t}\right) \geq 0.43 \quad (32)$$

$$c_{32} = 1 + 2.4\left(\frac{N}{h}\right) \leq 1.96 \quad (33)$$

J. Struct. Eng., 1989, 115(10): 2511-2527

$$c_{33} = 1 + 0.54\left(\frac{N}{h}\right) \leq 1.41 \qquad (34)$$

$$c_{34} = 1 + 0.729\left(\frac{N}{h}\right) \leq 1.30 \qquad (35)$$

$$c_{36} = 1 + 1.318\left(\frac{N}{h}\right) \leq 1.53 \qquad (36)$$

$$c_{37} = 1 + 1.262\left(\frac{N}{h}\right)^{1.5} \leq 1.82 \qquad (37)$$

$$c_{38} = 1 + 0.109\left(\frac{N}{h}\right)^{3} \leq 2.69 \qquad (38)$$

$$c_{41} = 1 - 0.00348\left(\frac{h}{t}\right) \geq 0.32 \qquad (39)$$

$$c_{42} = 1 - 0.00170\left(\frac{h}{t}\right) < 0.81 \qquad (40)$$

$$c_{43} = 1 - 0.00245\left(\frac{h}{t}\right) \geq 0.51 \qquad (41)$$

$$c_{44} = 1 - 0.0000141\left(\frac{h}{t}\right)^{2} \geq 0.44 \qquad (42)$$

$$c_{45} = 1 - 0.00118\left(\frac{h}{t}\right) \leq 0.82 \qquad (43)$$

$$c_{46} = 1 - 0.000471\left(\frac{h}{t}\right) \leq 0.95 \qquad (44)$$

$$c_{47} = 1 - 0.0017\left(\frac{h}{t}\right) \geq 0.66 \qquad (45)$$

$$c_{48} = 1 - 0.0060\left(\frac{h}{t}\right) \geq 0.46 \qquad (46)$$

$$c_{51} = 1 - 0.298\left(\frac{e}{h}\right) \geq 0.52 \qquad (47)$$

$$c_{52} = 1 - 0.120\left(\frac{e}{h}\right) \geq 0.40 \qquad (48)$$

$$c_{55} = 1 - 0.233\left(\frac{e}{h}\right) \geq 0.58 \qquad (49)$$

$$c_{64} = 1 + 4.547\left(\frac{Z}{h}\right) \leq 7.82 \qquad (50)$$

J. Struct. Eng., 1989, 115(10): 2511-2527

$$c_{68} = 1 + 0.109\left(\frac{Z}{h}\right) \leq 1.22 \quad (51)$$

$$c_{73} = 1 + 0.56\left(\frac{Z_1}{h}\right) \leq 1.98 \quad (52)$$

For uniform loading, the distance e may be considered as follows: (1) For end reactions, e is taken as half of the clear distance between the adjacent bearing plates—see Fig. 9 (i.e., $e = L_n/2$); (2) for interior reactions (Fig. 10), e is taken as the larger value of $L_{n1}/2$ and $L_{n2}/2$. Additional tests are required for refinement of these recommendations for uniform loading.

Combined Bending and Web Crippling

Unreinforced flat webs of shapes subjected to a combination of bending and reaction or concentrated load shall be designed to meet the following requirements.

Shapes Having Single Webs

$$\left(\frac{M}{M_u}\right) + 1.10\left(\frac{P_{mc}}{P_{cy}}\right) \leq 1.42 \quad (53)$$

where M = applied bending moment, at or immediately adjacent to the point of application of the concentrated load or reaction, P_{mc}, kip-in.; M_u = ultimate bending moment permitted if bending stress only exists, kip-in.; P_{mc} = concentrated load or reaction in the presence of bending moment, kips; and P_{cy} = concentrated load or reaction in the absence of bending moment determined from Eq. 8, kips.

The value of P_{mc} determined from Eq. 53 shall not be larger than P_{cb} calculated from Eq. 9.

I-Beams

$$\left(\frac{M}{M_u}\right) + 1.07\left(\frac{P_{mc}}{P_{cy}}\right) \leq 1.28 \quad (54)$$

where P_{cy} shall be determined from Eq. 19. The value of P_{mc} determined from Eq. 54 shall not be larger than P_{cb} calculated from Eq. 20. M and M_u are as defined previously.

Conclusions

The structural behavior of thin-walled, cold-formed steel beam webs subjected to concentrated loads or reactions was discussed. Web-crippling tests of hat sections and I-beams fabricated from very high-strength sheet steels under various loading conditions were performed. The test results indicated that the 1981 AISI *Guide for Preliminary Design* underestimates the web-crippling strength of sections cold-formed from very high-strength sheet steels.

Empirical equations were derived to predict the ultimate web-crippling loads based on the test data with a large range of yield strengths of materials. The new equations distinguish between overstressing (bearing) failure and buckling failure. Interaction equations for combined bending and web crippling were also derived for both hat sections and I-beams.

Tests for the transition ranges between the four basic loading conditions were also performed. It was found that by using a simple interpolation between the four basic loading conditions, reasonable accuracy of prediction can be achieved. A new design method was introduced. This method covers all possible web-crippling failure modes and can be applied to a wider range of material strengths than the present design criteria.

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Appendix I. References

Automotive steel design manual. (1986). Amer. Iron and Steel Inst., Washington, D.C.

Guide for preliminary design of sheet steel automotive structural components. (1981). Amer. Iron and Steel Inst., Washington, D.C.

Errera, S. J. (1982). "Automotive structural design using the AISI guide." *SAE Tech. Paper Series 820021,* Soc. of Automotive Engrs., Warrendale, Pa.

Hetrakul, N., and Yu, W. W. (1978). "Structural behavior of beam webs subjected to web crippling and a combination of web crippling and bending." *Final Report,* Civ. Engrg. Study 78-4, Univ. of Missouri-Rolla, June, Rolla, Mo.

Khan, M. Z., and Walker, A. C. (1972). "Buckling of plates subjected to localized edge loading." *The Struct. Engr.,* 50(June).

Khan, M. Z., Johns, K. C., and Hayman, B. (1977). "Buckling of plates with partially loaded edges." *J. Struct. Div.,* ASCE, 103(3), 547–558.

Levy, B. S. (1983). "Advances in designing ultra high strength steel bumper reinforcement beams." *SAE Tech. Paper Series 830399,* Soc. of Automotive Engrs., Warrendale, Pa.

Lin, S. H. (1984). "Structural behavior of cold-formed steel beam webs subjected to partial edge loading." Thesis presented to the University of Missouri-Rolla, at Rolla, Mo., in partial fulfillment of the requirements for the degree of Master of Science.

Santaputra, C., Parks, M. B., and Yu, W. W. (1986). "Web crippling of cold-formed steel beams using high strength sheet steels." *SAE Tech. Paper Series 860823,* Soc. of Automotive Engrs., Warrendale, Pa.

Santaputra, C. (1986). "Web crippling of high strength cold-formed steel beams." Thesis presented to the University of Missouri-Rolla, at Rolla, Mo., in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Santaputra, C., and Yu, W. W. (1986). "Design of automotive structural components using high strength sheet steels: Web crippling of high strength cold-formed steel beams." *Eighth Progress Report,* Civ. Engrg. Study 86-1, Univ. of Missouri-Rolla, Rolla, Mo., Aug.

Specification for the design of cold-formed steel structural members. (1986). Amer. Iron and Steel Inst., Washington, D.C.

Steel meets the automotive engineering challenges. (1983). Distributed at the 1983 SAE Exposition, Amer. Iron and Steel Inst., Washington, D.C.

Vecchio, M. T. (1983). "Design analysis and behavior of a variety of as-formed mild and high strength sheet materials in large deflection bending." *SAE Tech. Paper Series 830398*, Soc. of Automotive Engrs., Warrendale, Pa.
Yu, W. W., Santaputra, C., and Parks, M. B. (1983). "Design of automotive structural components using high strength sheet steels." *First Progress Report*, Civil Engineering Study 83-1, Univ. of Missouri-Rolla, Rolla, Mo., Jan.
Yu, W. W. (1985). *Cold-formed steel design*. John Wiley & Sons, New York, N.Y.

Appendix II. Notation

The following symbols are used in this paper:

E = modulus of elasticity of steel = 29,500 ksi (203,373 MPa);
e = clear distance between edges of adjacent opposite bearing plates, in.;
F_y = yield strength, ksi;
h = clear distance between flanges measured along plane of web, in.;
L = total length of specimen, in.;
M = applied bending moment, at or immediately adjacent to the point of application of concentrated load or reaction, kip-in.;
M_u = ultimate bending moment if bending stress exists only, kip-in.;
N = actual length of bearing, in.;
P_c = governing ultimate web-crippling load, kips;
P_{cb} = ultimate web-crippling load due to web buckling, kips;
P_{cy} = ultimate web-crippling load due to overstressing under bearing plate, kips;
P_{mc} = computed load for combined bending moment and web crippling, kips;
R = inside bend radius, in.;
t = base steel thickness, in.;
Z = distance between edge of bearing plate to near end of beam, in.;
Z_1 = distance between edge of bearing plate to far end of beam, in.; and
θ = angle between plane of web and plane of bearing surface, degrees.

J. Struct. Eng., 1989, 115(10): 2511-2527