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ABSTRACT

This paper gives an overview of the design procedures formulated for plain channels. These formulations are based on experimental and finite element studies. The scope of the work covers laterally braced beams, columns and beam-columns of cross-sections in the range of practical applications by the industry. The recommendations treat members that are made up of elements that may be in the post-buckling or post yielding range.

KEY WORDS: cold-formed steel; plain channels; design; effective width

INTRODUCTION

Cold-formed steel plain channels shown in Figure1 are used in several applications such as bracing members in racks and tracks in steel framed housing. This paper gives an overview of the design procedures developed for laterally braced beams, columns and beam-columns of plain channels. Current design procedures were found to be inaccurate.

The design procedures developed are applicable to cross-sections in the range of practical sections used in the industry, namely $b_2 / b_1 \leq 1$. The design procedures developed are consistent with AISI Specification for calculating the overall capacity of plain channels.

ELASTIC BUCKLING

The determination of the ultimate strength when the plate elements are in the post buckling range is based on the effective width procedure. The effective width procedure necessitates the use of the plate buckling stress or the plate buckling coefficient k. Simple equations for plate buckling coefficient k considering the interaction between plate elements were developed for minor and major axis bending as well as for columns. These equations were obtained by using a computer program CUFSM developed by Schafer (1997) at Cornell University. The dimensions of plain channels are in the ranges of practical applications by the industry. Below is a list of these equations.

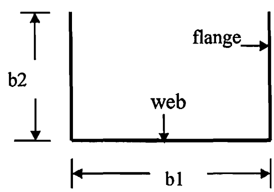

Figure 1 Plain Channel

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Minor Axis Bending with Stiffened Element in Tension (Figure 2)

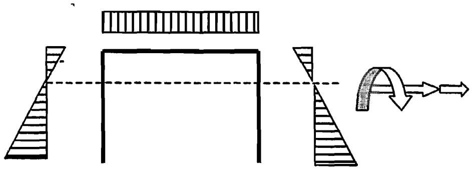

Figure 2

$$k_f = 0.1451(\frac{b_2}{b_1}) + 1.2555$$

Minor Axis Bending with Stiffened Element in Compression (Figure 3)

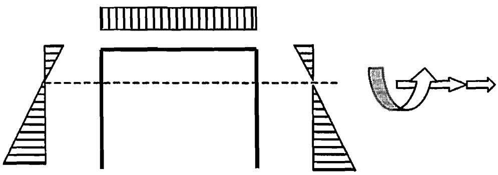

Figure 3

$$k_f = 4.5119(\frac{b_2}{b_1})^2 + 6.5345(\frac{b_2}{b_1}) - 0.2064$$

$$k_w = (b_1 / b_2)^2 k_f$$

2.2 Major Axis Bending with Unstiffened Element in Uniform Compression

Type a (Figure 4)

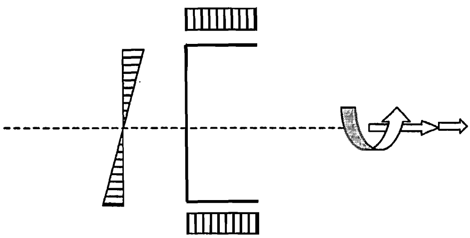

Figure 4

$$k_f = 0.3561(\frac{b_2}{b_1}) + 0.7452, \text{ when } \frac{b_2}{b_1} > 0.2264$$

$$k_f = 4.2346(\frac{b_2}{b_1}) - 0.1329, \text{ when } \frac{b_2}{b_1} \leq 0.2264$$

$$k_w = (b_1 / b_2)^2 k_f$$

Type b (Figure 5)

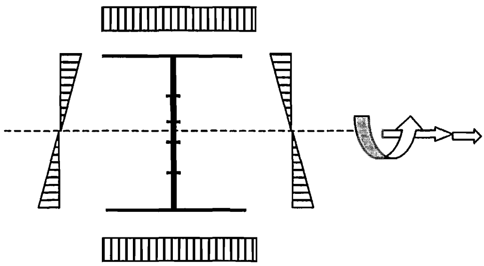

Figure 5

$$k_f = 0.0348(\frac{b_2}{b_1}) + 1.1246$$

$$k_w = (b_1 / b_2)^2 k_f$$

2.5 Columns (Figure 6)

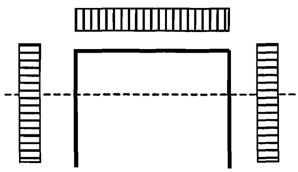

Figure 6

$$k_f = 1.2851(\frac{b_2}{b_1}) - 0.0237, \text{ when } \frac{b_2}{b_1} \leq 0.7201$$

$$k_f = 0.0556(\frac{b_2}{b_1}) + 0.8617, \text{ when } \frac{b_2}{b_1} > 0.7201$$

ULTIMATE STRENGTH

The formulations developed involve the use of effective widths for the component plate elements that are in the post-buckling. Using these effective widths effective section properties and hence the ultimate load carrying capacities are determined. The approach is thus in agreement with the frame work of the unified approach of Pekoz (1987) used in the AISI Specification (1996).

For members that exhibit inelastic reserve capacity, post yield strain reserve capacity expressed in terms of a ratio, C_y that is the ratio ultimate strain divided by the yield strain. The ultimate moment of a flexural member is determined by statics based on the ultimate strain capacity as is done in the AISI Specification (1996). The details of the equations developed are given below.

MINOR AXIS BENDING WITH STIFFENED ELEMENT IN TENSION

Effective Width Model for Flanges

$\lambda = 1.052(b_2/t)\sqrt{f_y/Ek_f}$ or $\lambda = \sqrt{f_y/f_{cr}}$

$k_f = 0.1451(b_2/b_1) + 1.2555$

if $\lambda > 0.859$

$$\rho = 0.925\left(\frac{f_{cr}}{f_y}\right)^{1/3.9}$$

if $\lambda \leq 0.859$

$\rho = 1$

$b_e = \rho b_2$

$M_{ns} = f_y S_e$

Post-yield Strain Reserve Capacity Model

$C_y = 3.0$ for $\lambda \leq 0.535$

$C_y = \frac{0.5877}{(\lambda - 0.0924)^2}$ for $0.535 < \lambda < 0.859$

$C_y = 1$ for $\lambda \geq 0.859$

The nominal moment capacity is determined as described in AISI Specification (1996) Section C3.1.1 b.

MINOR AXIS BENDING WITH STIFFENED ELEMENT IN COMPRESSION

Effective Width Model

For stiffened elements in uniform compression:

The effective width, b, is to be determined using AISI Specification Section B2.1 $f = F_y$, $k = k_w$. The value of k_w is to be determined by the equations given above.

For unstiffened elements under a stress gradient:

For the post-buckling behavior of unstiffened elements a consistent effective width shown in Figure 7 as suggested by Schafer (1997) is used.

When $\psi = \frac{f_2}{f_1}$,

$$b_{1o} = \frac{b\omega}{(1-\psi)}$$

$$b_{2o} = \left(\frac{b}{(1-\psi)}\right)\sqrt{\omega^2 - 2\omega + \rho}$$

where

$0 \leq \rho < 0.77 \quad \omega = 0.30\rho$

$0.77 \le \rho < 0.95 \quad \omega = 0.23$

$0.95 \le \rho \le 1.00 \quad \omega = -4.6\rho + 4.6$

in which,

$\rho = 1$ when $\lambda \le 0.673$

$\rho = \frac{(1 - 0.22/\lambda)}{\lambda}$ when $\lambda > 0.673$

$\lambda = \sqrt{\frac{f_1}{f_{cr}}}$

$M_{ns} = F_y S_e$

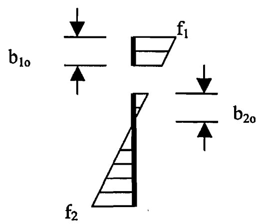

Figure 7

When unstiffened elements does not undergo local buckling, the nominal moment capacity is determined based on initiation of yielding or its ultimate moment. The ultimate is determined based on the ultimate (post-yield) strain capacity.

Post-yield Strain Capacity Model

$C_y = 3$ for $\lambda \le 0.46$

$C_y = 3 - 2 * \frac{(\lambda - 0.46)}{(0.673 - 0.46)}$ for $0.46 < \lambda < 0.673$

$C_y = 1$ for $\lambda \ge 0.673$

The nominal moment capacity is determined as described in AISI Specification (1996) Section C3.1.1 b.

MAJOR AXIS BENDING

For unstiffened element in uniform compression, the effective widths are determined as described in AISI Specification Section B3.2 with $f = F_y$, and using the plate buckling coefficient as given above, namely $k = k_f$

For stiffened element under a stress gradient, the consistent effective width described above is used and $M_{ns} = F_y S_e$.

FLAT-ENDED AND PIN-ENDED COLUMNS

Flat-ended columns: as the shift of the line of action of the internal forces caused by local buckling deformations does not induce overall bending in flat-ended columns, column equation can be used to design flat-ended columns.

Pin-ended columns: as the shift of neutral axis caused by local buckling is significant in overall bending of pin-ended columns, beam-column equation can be used to design pin-ended columns. Two thirds of the maximum eccentricity is selected for the beam-column equation because the eccentricity varies along the length of the column.

BEAM-COLUMNS

Strength of plain channel beam columns can be determined by the interaction equations (AISI Specification Section C5.2.2) with the improved plate buckling coefficient k described above.

The parameters for the column part of the beam-column equations, flat-ended columns are to be treated as concentrically loaded columns; while pin-ended columns are treated as beam-columns. The eccentricity of the load should be determined on the basis of the location of the load and the average deflections of the beam column instead of the maximum deflections. The parameters for the beam part of the beam-column equations, the formulations developed above are to be used.

COMPARISON STUDIES

The experimental results of several studies were first evaluated by finite element approaches as to their validity. Some of the test results were not reliable due to some deficiencies in the tests. The finite element studies indicated which test results should be excluded from further evaluation.

Minor Axis Bending with Stiffened Element in Tension

Experimental result of El Mahi and Rhodes (1985), Enjiky (1985), Jayabalan (1989), and the tests carried out at Cornell University in 1999 by Fang Yiu and Teoman Pekoz were used to formulate the provisions for the case of minor axis bending with stiffened element in tension.

The mean value of M_{ns} over M_{test} ratio (excluding the results for plain channels where the flanges are not at right angles to the web) is 0.993; the sample standard deviation is 0.114; resistance factor ϕ is 0.718 in probability model. For specimens with post-yield reserve capacity, that is, Cy>1, ϕ is 0.690; When Cy=1, ϕ is 0.740. The comparison results are shown in Figure 8.

Minor Axis Bending with Stiffened Element in Compression

Test results of Enjiky (1985) are used for minor axis bending with stiffened element in compression. Comparison in Figure 8 shows that the mean value of M_{ns} over M_{test} ratio is 1.038; the sample standard deviation is 0.087; resistance factor $\phi = 0.769$ in probability model. The comparison results are shown in Figure 9.

Major Axis Bending with Unstiffened Elements in Uniform Compression

Test results of Reck reported by Kalyanaraman (1976) and Talja (1992) provided the basis for the design procedure. For the relevant test data from these references the mean value of M_{ns} over M_{test} ratio is 0.956 and the sample standard deviation is 0.080. The comparison results are shown in Figure 10.

Flat-ended Columns

Data from Talja (1990), Young (1997), Mulligan & Pekoz (1983) provided the basis for the procedures for flat-ended columns. The mean value of M_{ns} over M_{test} ratio for the data is 0.950; the sample standard deviation is 0.126; resistance factor ϕ is 0.670 in probability model for fixed-ended columns. The results are illustrated in Figure 11.

Pin-ended Columns

Test results from Young (1997) for two series of pin-ended column test Series P36 and P48 were used in the development of the design procedures. The mean value of M_{ns} over M_{test} ratio is 0.920; the sample standard deviation is 0.078; resistance factor ϕ is 0.675 in probability model. The results are illustrated in Figure 12.

Beam – Columns

Jayabalan (1989) and Srinivasa (1998) provide results of beam-column experiments with eccentricity of the load in the plane of symmetry. Fang Yiu and Pekoz in 2000 tested beam-column with eccentricity of the load in the plane of asymmetry. Only the data corresponding to practical cross-sections are evaluated and plotted in Figure 13.

The correlation of the test results of C, channel, and hat section beam-columns with the use of interaction equations was plotted in Figure 7.3-1 of Pekoz (1987). This figure presented the results of all the tests with loads with uniaxial or biaxial eccentricities. Rp, Rx and Ry represent the first, second and the third terms of the AISI interaction equation. Ro equals 0.707(Rx+Ry). The projections of test points on the Rp-Ro plane was plotted. The results that fell outside of the solid line in the Figure 12 on the right indicated that the interaction equation is conservative for those cases. Results from Jayabalan (1989) and Srinivasa (1998) and Fang Yiu and Pekoz are added to the Pekoz (1987) figure and given in Figure 13. It is seen that the interaction equation is also conservative for plain channel section.

CONCLUSIONS

Design recommendations for calculating the overall capacity of plain channel sections in the range of practical applications by the industry are presented. Comparison studies indicate good agreement with experimental results.

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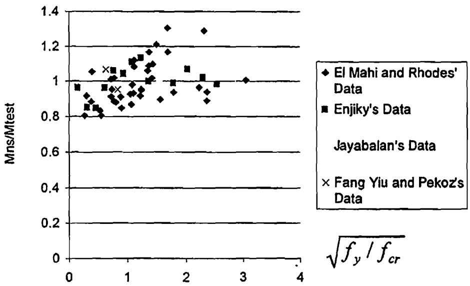

Figure 29 Comparative Study of Minor Axis Bending with Stiffened Element in Tension

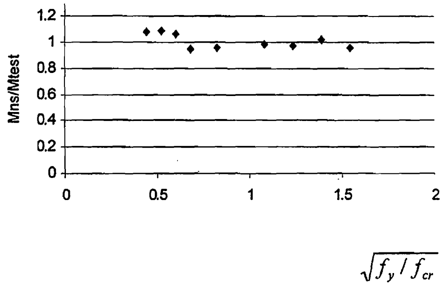

Figure 30 Comparative Study of Minor Axis Bending with Stiffened Element in Compression

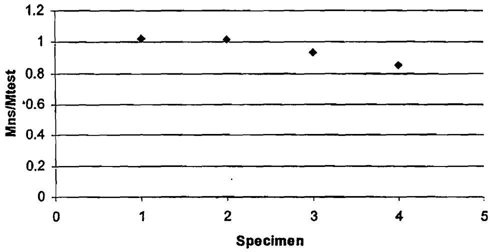

Figure 31 Comparative Study of Major Axis Bending

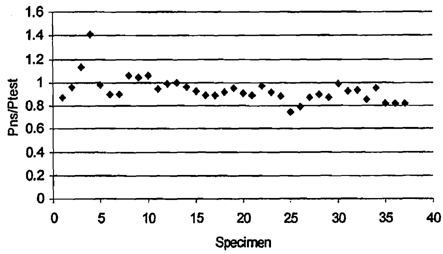

Figure 32 Comparative Study of Flat-ended Columns

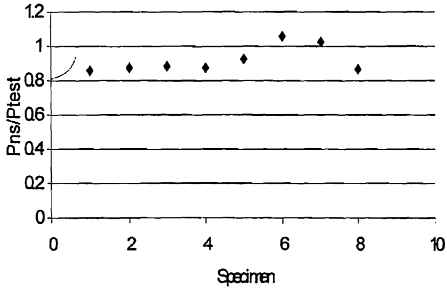

Figure 33 Comparative Study of Pin-ended Columns

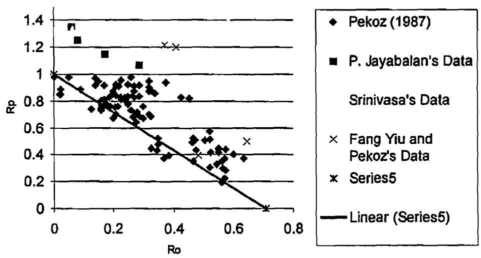

Figure 34 Comparative Study of Beam-Columns

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NOTATION:

E = Modulus of Elasticity

υ = Poisson's ratio

$G = \dfrac{E}{2(1+\upsilon)}$ = Shear Modulus

t = plate thickness

$D = \dfrac{Et^3}{12(1-\upsilon^2)}$ = plate rigidity

b_1 = Depth of web element
b_2 = Width of flange element
f_y = yield stress
f_{cr} = critical buckling stress of the cross-section
k_f = plate buckling coefficients considering interaction of plate elements in terms of flange width
k_w = plate buckling coefficients considering interaction of plate elements in terms of web depth
ρ = post-buckling reduction factor
λ = slenderness factor
M_{ns} = nominal moment capacity
S_e = elastic section modulus of the effective section
C_y = compression strain factor
f_1 = maximum compressive (+) stress on an element under a stress gradient
f_2 = tension (-) stress for an element under a stress gradient

$$\psi = \frac{f_2}{f_1}$$