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DESIGN OF COLD-FORMED STEEL COLUMNS

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1. INTRODUCTION

It is well known that imperfections, residual stresses and basic material properties influence the behavior of columns. The imperfections and the residual stresses and the degree to which they influence the performance of columns depends on the methods and details of manufacture as well as the cross-sectional geometry. Cold-formed steel columns have a great variety of shapes and the details of manufacture vary widely. Two general studies on the flexural buckling of locally stable columns have been carried out recently at Cornell and reported in Dat and Pekoz [1980] and Weng and Pekoz [1987]. Several papers on the latter reference will be published in the near future. These general studies show that a simple formulation covering all types of columns is not possible.

This paper presents some of the results of a study to develop a design approach for a specific class of columns, namely, typical lipped channel columns with and without perforations used in industrial rack structures. Since very little test evidence existed, 30 stub columns and 33 columns were tested within this project. This study was sponsored by the Rack Manufacturers Institute. The results of these tests were evaluated along with the results of 42 unperforated column tests carried out in an American Iron and Steel Institute sponsored project reported in Weng and Pekoz [1987]. All the columns considered have either fully effective cross-sections or would have fully effective cross-sections if unperforated.

In the evaluation of the test results, 23 different approaches were used. The approach of the European Convention for Constructional Steelwork [1987], "European Recommendations for the Design of Light Gauge Steel Members" and its possible extensions to perforated columns were among those tried. These approaches will be referred to as the ECCS Approaches. Only five out of the 23 approaches will be focused upon in this paper. Four of these approaches are related to the ECCS Approach. An approach based on the basic AISI Column design equation will also be discussed.

More detailed information on the specimens tested and the other design approaches evaluated can be found in Pekoz [1987].

2. REVIEW OF DESIGN APPROACHES

2.1 THE ECCS APPROACH FOR UNPERFORATED COLUMNS

The ECCS Approach is intended for unperforated columns that may or may not be locally stable. The effect of local buckling on the overall buckling is accounted for through the use of the Q factor defined below. The approach involves the determination of the design strength N_d as follows (the notation has been changed slightly from that given in the ECCS Recommendations [1987]):

$$N_d = A_g F_n \qquad \text{Eq. 1}$$

where

$$F_n = F_y \{F - [F^2 - (Q / \lambda^2)]^{1/2}\} \qquad \text{Eq. 2}$$

where

$$F = [Q + (1 + \eta(\lambda - 0.2))/\lambda^2] / 2 \qquad \text{Eq. 3}$$

and

$$\lambda = (F_y / F_e)^{1/2} \qquad \text{Eq. 4}$$

F_e and F_y are the elastic buckling and yield stresses, respectively.

For flexural buckling this parameter becomes

$$\lambda = (L / r) (1 / \pi) (F_y / E)^{1/2} \qquad \text{Eq. 5}$$

Other terms of the above equations are

Q	=	A_{eu} / A_g	Eq. 6
A_{eu}	=	Effective area at uniform compression stress equal to yield stress	
A_g	=	Full cross-sectional area	
r	=	Radius of gyration of the full section	
L	=	Effective length	
η	=	$\alpha(4 - 3Q) \leq 0.76$	Eq. 7
α	=	0.34 for the sections tested	

The design strength N_d is to be not less than the axial force caused by the design loads times load factors. The load factor for live load is 1.5.

2.2 THE AISI APPROACH FOR UNPERFORATED COLUMNS

The AISI Approach is also intended for unperforated columns that may or may not be locally stable. An exception is discussed in the next section. In this approach first the nominal column buckling stress F_n is determined as follows:

For $F_e < F_y/2$ $\quad F_n = F_e$ $\quad$ Eq. 8
For $F_e > F_y/2$ $\quad F_n = F_y\ (1 - F_y\ /\ 4F_e)$ $\quad$ Eq. 9

F_e is the elastic column buckling stress, and in case of flexural buckling

$F_e = \pi^2 E\ /(KL/r)^2$ $\quad$ Eq. 10

In case of torsional flexural buckling F_e is the elastic torsional-flexural buckling stress.

The column strength P_n is determined as

$P_n = A_e\ F_n$ $\quad$ Eq. 11

A_e is the effective area of the column at stress F_n. When A_e cannot be determined analytically, the value of A_e for a given F_n can be determined on the basis of stub column test results. The value of A_e can be determined either from the measured axial shortening or from the value of the effective area A_{eu} at ultimate load. These approaches were derived by Pekoz [1986]. The latter approach leads to the following expression:

$A_e = A - (A - A_{eu})\ (F_n\ /\ F_y)^{Aeu/A}$ $\quad$ Eq. 12

A_{eu} is the effective area at ultimate stub column load P_t determined as

$A_{eu} = (P_t)\ /\ F_y$ $\quad$ Eq. 13

The column strength P_n is divided by a factor of safety to find the allowable load. For sections thinner that .09 inches a constant factor of safety of 1.92 is used. For thicker sections the factor of safety varies from 1.67 for very short columns to 1.92 for long columns.

2.3 THE AISC APPROACH FOR UNPERFORATED COLUMNS

The American Institute of Steel Construction [1986], "Load and Resistance Factor Design Specification for Structural Steel Buildings" is for hot-rolled steel sections. The AISC design formulation that is also primarily for locally stable unperforated columns is given here for information purposes.

The column strength P_n is determined as

$P_n = A\ F_{cr}$ $\quad$ Eq. 14

for $\lambda \leq 1.5$ $\quad F_{cr} = (.658^{\lambda^2})\ F_y$ $\quad$ Eq. 15

for $\lambda > 1.5$ $\quad F_{cr} = (.877 / \lambda^2) F_y$ $\qquad$ Eq. 16

The design strength is ϕP_n where ϕ is the resistance factor equal to .85. The required strength is determined using load factors.

2.4 EXTENSIONS OF THE ECCS AND THE AISI APPROACHES TO PERFORATED COLUMNS

The ECCS Approach can be extended to perforated columns in several ways. The first that comes to mind is to use the value of Q determined by test on stub column specimens. The configuration of perforations in general does not allow the calculation of Q analytically. Another possible extension would involve combining the ECCS column design curve with the AISI Specification [1986] approach for handling the interaction of local and overall buckling. Information on the development of this approach can be found in Pekoz [1986].

The extension of the AISI Approach that gave the most satisfactory results will be discussed below. The AISI Specification contains design provisions for columns with circular perforations within certain limits. Typical rack columns do not fall within these limits of applicability.

3. TEST RESULTS

Two groups of test results were used in this study. The average dimensions of the sections are given in Table 1. The geometry of the sections and the cross-sectional notation are illustrated in Figs. 1 and 2. In Table 1 T is the wall thickness, r' is the average inside corner radius and Q_n is A_{eu} divided by the net minimum area.

3.1 SPECIMENS OF PEKOZ [1987]

The perforated and unperforated columns of Pekoz [1987] have the designations AU1, AU2, AP1, AP2, BU1, BU2, BP1, BP2). Two types of sections were taken from regular manufacturing lines of two different companies designated A and B.

The perforated and unperforated sections are designated as P and U. The letters U and P are followed by 1 or 2 which designate the thinner and the thicker walled-sections, respectively. The number following the thickness designation is the number of the test in the series. The last letter in the designation indicates the end conditions as follows:

F Fixed-ended column. In these tests base plates were welded and the columns were tested flat-ended. Hydrostone was placed at each end to assure uniform distribution of the end loads and the end fixity.

H Hinge-ended. Specially designed end fixtures were used to assure hinged condition about the centroidal axis perpendicular to the axis of symmetry. The difference in the location of the net and the full section centroidal axes were insignificant compared with the accuracy of the cross sectional dimensions and the accuracy with which the column could be aligned.

In this group of specimens, all the fixed ended columns were subject to torsional-flexural and all the hinge ended specimens were subject to flexural buckling. The modes of buckling predicted were confirmed by the appearances of buckling modes.

3.2 SPECIMENS OF WENG AND PEKOZ [1987]

The unperforated columns of Weng and Pekoz [1987] have the designations RFC11, RFC13, RFC14, PBC13, PBC14, P11, P16, R13, R14. RFC and PBC as well as R and P indicate roll-formed and press-braked C-sections, respectively. The numbers that follow indicate the gage of the material.

All columns of this group were tested with hinges about the minor axis and all the specimens were subject to flexural buckling. The loads were aligned with the aid of strain gages to be concentric at about one fourth of the expected ultimate load.

4. EVALUATION OF TEST RESULTS

The approaches summarized in Table 2 involve the calculation of the nominal column strength P_n as follows:

$$P_n = \text{(AREA) (NOMINAL STRESS)}$$

Depending on the approach different definitions of AREA and the NOMINAL STRESS were used.

AREA

The following possibilities for the calculation of the AREA were considered:

NET is the net minimum area of the section.

EFFECTIVE is the area determined at the nominal stress determined by one of the two approaches:

- Using the effective width equations of the AISI Specification. For the sections considered these equations are basically the same as those of the ECCS Recommendations.

- Based on test results using the Eq. 12 where A_e is the effective area of the column at NOMINAL STRESS, F_n. The area A was taken consistent with the cross section used in determining NOMINAL STRESS, namely it was the net area when the net section radius of gyration was used in calculating the NOMINAL STRESS and gross area was used when the gross section radius of gyration was used.

NOMINAL STRESS

In determining the NOMINAL STRESS, F_n, both the AISI and the ECCS column design equations were used. The radius of gyration r was taken either for the net section or for the gross section as indicated in Table 2. When net section was used

$$r = (I_m/A_c)^{1/2} \qquad \text{Eq. 17}$$

where I_m is the minimum net moment of inertia and A_c is the area of the section giving the minimum moment of inertia.

In case of torsional flexural buckling F_e is the elastic torsional-flexural buckling stress based on sharp corner gross section properties.

4. EVALUATION OF TEST RESULTS

The results of the evaluations are summarized in Tables 3a, 3b and 3c. Since the hinge-ended and the fixed-ended columns failed by flexural and torsional-flexural buckling, respectively, some conclusions can be drawn about the accuracy of the approaches for both types of buckling. Tables 3a and 3c are for columns where flexural buckling was the governing failure mode. All the columns shown in Table 3b failed by torsional-flexural buckling.

In Tables 3a, 3b and 3c, the ratios of the observed maximum test load P_t divided by the calculated load and their means, standard deviations and coefficient of variations are given. The subscript of the calculated load P refers to the procedures listed in Table 2. In Table 3c which is for locally stable unperforated columns, the procedures PE1, PE2, PE3 and PE4 give the same results and are referred to as PE. For perforated sections λ is calculated for the net section.

It is seen in Tables 3a and 3b that the approach PE2 which is the ECCS Approach modified by the AISI Approach for treating the interaction of local and overall buckling gives smallest standard coefficient of variation for flexural buckling of both the unperforated and perforated columns. This approach also gives the lowest coefficient of variation for torsional-flexural buckling of perforated columns as well. For torsional-flexural buckling of unperforated columns the coefficients of variation for all types of analyses, the coefficient of variations are nearly equal. It should be noted that for the modified AISI approach (PA) the mean quite a bit below 1.0. However, the ultimate load calculated by the column curve is not the only factor to be considered in assessing the design approach. As discussed below the load and resistance factors and the factors of safety involved also need to considered.

5. SOME OBSERVATIONS ON THE DESIGN APPROACHES

The test results of Weng and Pekoz [1987] on unperforated columns as well as the AISI, AISC and the ECCS column curves are plotted in Fig. 3. It is seen that the ECCS curve provides a lower limit and the AISI curve provides an upper limit to the observed results. It is of interest how the allowable loads would compare if one includes the effect of the load and resistance factors and the factor of safety in the calculations. The following parametric study was carried out for such a comparison.

The results of a parametric study are summarized in Tables 4a and 4b. In these tables the modified ECCS approach design load and the modified AISI approach allowable load are compared for perforated and unperforated type A and B columns for various lengths. These tables show the comparisons for the case of constant factor of safety as it is prescribed in the AISI Specification for thicknesses less than .09 inches and the variable factor of safety for the case of thicknesses larger than .09 inches. It is seen that for the case of constant factor of safety the modified AISI approach gives close but consistently lower loads than the modified ECCS approach design strength divided by a load factor equal to 1.5.

For unperforated locally stable columns subject to flexural buckling, a comparison of the ECCS Recommendations, the AISI and the AISC Specifications is shown graphically in Fig. 3. In this figure the curves marked ECCS, AISI and AISC are for nominal strengths according to the respective documents. The curve marked $ECCS_a$ is the ECCS design strength divided by a live load factor of 1.5. The curves marked $AISI_a$ and $AISI_b$ are for the nominal strength divided by a factor of safety of 1.92 and by the varying factor of safety stipulated in the AISI Specification for thicknesses greater than .09 inches. The curve marked $AISC_a$ is for the nominal strength multiplied by a resistance factor of .85 and divided by a live load factor of 1.6. Since the columns considered have very high live to dead load ratios only live load factors are considered.

Since only locally stable unperforated columns can be shown in Fig. 3, the conclusions from this figure are strictly correct for such columns. However, the results for perforated columns should follow similar relative trends. It is seen that for the most part the AISI approaches with constant and variable factors of safety give more conservative results than the ECCS Approach.

It should be noted that stub column results are not plotted in this figure. If they were plotted they would all fall above the strength curves. The behavior of very short columns can be predicted quite conservatively. For this reason a variable factor of safety which is lower for shorter columns appears justified.

Dat and Pekoz [1980] and Weng and Pekoz [1987] show that the lipped channel sections with component elements just at the limit of becoming partially effective, the predictions of the AISI column curve give upper bounds to test results. Dat and Pekoz [1980] also show that members with component elements that do not have slendernesses in this range, the AISI curve gives satisfactory results. Most of the columns in the present study had component elements that were at the limit of being partially effective. It is expected that the procedures discussed above would give more conservative results for other cold-formed steel compression members having component elements not in the range of limiting slenderness between the fully effective and partially effective.

5. SUMMARY AND CONCLUSIONS

Several design provisions for perforated and unperforated lipped channel columns were studied. The formulation that gave the best results for flexural buckling involved obtaining a nominal failure stress using the ECCS Column curve with Q = 1 and with net section properties. The nominal strength or the design strength is found by multiplying the nominal failure stress by the effective area determined by stub column tests. For torsional-flexural buckling the nominal failure stress is found on the basis of the full section assuming sharp corners.

It was seen that when the entire design approaches including the column curves, factors of safety, resistance and load factors are considered, the ECCS, AISI and AISC documents lead to closer agreement than just a comparison of the column curves would indicate.

It is hoped that the observations of this paper will aid the specification writing committees in their deliberations.

7. FUTURE WORK

All the columns considered in this study have either fully effective cross-sections or would have had fully effective cross-sections if unperforated. Using the approach developed in this study, namely combining the ECCS column curve with AISI approaches for the interaction of local and overall buckling for the case when the sections are not fully effective, appears promising. This topic will be studied in the near future.

Columns are usually subjected to axial loads in combination with moments. This case is usually treated in design specifications by interaction equations. The load carrying capacity for concentric loading which is the case studied in the present project is one of the parameters that are used in these interaction equations. Therefore the case of combined axial loading and bending will also be studied in the near future.

8. ACKNOWLEDGEMENTS

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TABLE 1
CROSS-SECTIONAL DIMENSIONS

SECTION	T (in)	A' (in)	B' (in)	C' (in)	r' (in)	F_y (ksi)	Q_n
AU1	.073	3.00	1.63	.80	.17	54.68	.94
AU2	.106	3.03	1.66	.74	.19	57.71	.98
AP1	.069	3.00	1.63	.80	.17	64.00	.77
AP2	.102	3.03	1.66	.74	.19	62.75	.80
BU1	.086	3.00	1.63	.62	.25	51.55	1.00
BU2	.098	3.03	1.63	.61	.24	49.17	1.00
BP1	.086	3.00	1.63	.62	.25	44.64	1.00
BP2	.098	3.03	1.63	.61	.24	45.04	1.00
RFC11	.119	3.16	1.64	.71	.16	40.38	1.00
RFC13	.096	3.07	1.63	.70	.16	51.85	1.00
RFC14	.075	2.99	1.76	.69	.22	55.09	1.00
PBC13	.087	3.02	1.62	.61	.16	38.40	1.00
PBC14	.071	3.00	1.62	.61	.16	36.30	1.00
P11	.118	5.03	2.48	.88	.13	30.59	1.00
P16	.064	2.65	1.37	.63	.09	33.45	1.00
R13	.086	3.01	1.63	.61	.22	50.15	1.00
R14	.075	3.02	1.64	.61	.22	49.73	1.00

TABLE 2
EVALUATION OF LONG COLUMN TEST RESULTS

PROCEDURE	AREA (1)	RADIUS OF GYRATION (1)	DESIGN EQUATION
PE1	GROSS	GROSS	ECCS
PE2	EFFECTIVE (2)	NET (4)	ECCS(3)
PE3	EFFECTIVE	NET (4)	ECCS
PE4	EFFECTIVE (2)	GROSS	ECCS
PA	EFFECTIVE (2)	NET (4)	AISI

(1) please refer to the text for description
(2) determined from stub column test
(3) determined taking Q=1
(4) net section radius of gyration is used for flexural buckling and gross properties are used for torsional flexural buckling. In both cases the net minimum area is used in calculating the effective area from stub column test results.

TABLE 3a
EVALUATION OF TEST RESULTS
(Pekoz [1987])

TEST	λ	Pt/PE1	Pt/PE2	Pt/PE3	Pt/PE4	Pt/PA
Unperforated Columns						
AU1-1H	0.885	1.296	1.238	1.352	1.296	1.041
AU2-1H	0.954	0.949	0.934	0.961	0.949	0.759
BU1-1H	0.694	1.065	1.065	1.065	1.065	0.953
BU1-2H	0.902	0.813	0.813	0.813	0.813	0.673
BU2-1H	0.706	1.031	1.031	1.031	1.031	0.919
BU2-2H	0.900	0.796	0.796	0.796	0.796	0.660
MEAN		0.992	0.979	1.003	0.991	0.834
STDEV		0.185	0.168	0.203	0.185	0.159
COV		0.187	0.171	0.203	0.187	0.190
Perforated Columns						
AP1-1H	0.987	1.001	1.009	1.398	1.016	0.837
AP1-2H	0.677	0.679	0.705	0.992	0.708	0.647
AP2-1H	0.709	1.052	0.905	1.488	1.052	0.821
AP2-2H	1.062	1.188	1.014	1.607	1.187	0.814
AP2-3H	1.018	1.083	0.921	1.474	1.083	0.751
AP2-4H	1.337	1.018	0.903	1.336	1.018	0.689
BP1-1H	0.612	1.084	1.082	1.324	1.084	0.992
BP1-2H	0.864	1.102	1.093	1.346	1.102	0.919
BP2-1H	0.628	0.920	0.904	1.125	0.920	0.825
BP2-2H	0.876	1.000	0.976	1.222	0.999	0.818
BP2-3H	0.876	0.937	0.915	1.145	0.937	0.766
MEAN		1.006	0.948	1.314	1.010	0.807
STDEV		0.132	0.107	0.181	0.125	0.096
COV		0.132	0.113	0.138	0.124	0.119

TABLE 3b
EVALUATION OF GROUP 1 TEST RESULTS
(Pekoz [1987])

TEST	λ	Pt/PE1	Pt/PE2	Pt/PE3	Pt/PE4	Pt/PA
Unperforated Columns						
AU1-1F	0.725	0.958	0.921	1.006	0.967	0.820
AU1-2F	1.001	1.151	1.100	1.195	1.151	0.883
AU2-1F	0.755	1.091	1.076	1.108	1.093	0.945
AU2-2F	1.021	1.016	1.001	1.029	1.016	0.793
BU1-1F	0.751	1.011	1.011	1.011	1.011	0.888
BU1-2F	1.033	1.091	1.091	1.091	1.091	0.857
BU2-1F	0.724	1.025	1.025	1.025	1.025	0.908
BU2-2F	1.027	1.094	1.094	1.094	1.094	0.862
MEAN		1.055	1.040	1.070	1.056	0.870
STDEV		0.062	0.062	0.065	0.060	0.048
COV		0.059	0.060	0.061	0.057	0.056
Perforated Columns						
AP1-1F	0.802	1.076	1.086	1.544	1.110	0.961
AP1-2F	1.105	1.089	1.088	1.494	1.090	0.866
AP2-1F	0.803	1.287	1.087	1.802	1.287	0.959
AP2-2F	1.058	1.100	0.931	1.489	1.100	0.749
BP1-1F	0.720	1.019	1.011	1.245	1.019	0.897
BP1-2F	0.964	1.070	1.068	1.307	1.069	0.863
BP2-1F	0.717	1.092	1.067	1.335	1.092	0.948
BP2-2F	0.987	1.055	1.041	1.290	1.055	0.833
MEAN		1.099	1.047	1.438	1.103	0.884
STDEV		0.080	0.054	0.184	0.080	0.073
COV		0.073	0.052	0.128	0.073	0.083

TABLE 3c
EVALUATION OF GROUP 2 TEST RESULTS
(Weng and Pekoz [1987])

TEST	λ	Pt/PE	Pt/PA
RFC11-1	0.52	1.093	1.027
RFC11-2	0.75	1.198	1.051
RFC11-3	0.97	1.375	1.106
RFC11-4	1.21	1.241	0.925
RFC13-1	0.58	1.025	0.947
RFC13-2	0.84	1.194	1.014
RFC13-3	1.11	1.286	0.984
RFC13-4	1.37	1.235	0.918
RFC14-1	0.55	0.996	0.927
RFC14-2	0.80	1.038	0.896
RFC14-3	1.05	0.981	0.766
RFC14-4	1.30	1.000	0.740
RFC14-5	1.56	1.033	0.808
PBC13-1	0.51	0.903	0.850
PBC13-2	0.74	1.020	0.900
PBC13-3	0.97	1.145	0.922
PBC14-1	0.49	1.023	0.967
PBC14-2	0.71	1.137	1.012
PBC14-3	0.92	1.135	0.933
PBC14-4	1.14	1.225	0.928
PBC14-5	1.36	1.366	1.013
P11-1	0.60	1.034	0.950
P11-2	0.82	1.079	0.923
P11-3	1.04	1.087	0.853
P11-4	1.26	1.124	0.832
P16-1	0.62	1.026	0.938
P16-2	0.82	1.103	0.944
P16-3	1.03	1.089	0.859
P16-4	1.22	1.160	0.864
P16-5	1.35	1.219	0.904
R13-1	0.58	1.071	0.989
R13-2	0.85	1.186	1.005
R13-3	1.10	1.148	0.881
R13-4	1.37	1.166	0.868
R13-5	1.59	1.124	0.884
R14-1	0.57	1.077	0.997
R14-2	0.82	1.070	0.916
R14-3	1.08	1.101	0.852
R14-4	1.33	1.109	0.821
R14-5	1.57	1.056	0.827
MEAN		1.117	0.919
STDEV		0.100	0.078
COV		0.090	0.085

Table 4a
PARAMETRIC STUDIES
Hinge Ended Columns - Flexural Buckling

SECTION	L	λ	R1	R2	R3
Unperforated Columns					
AU1	40	0.86	1.63	1.09	0.94
AU1	80	1.71	1.55	1.04	1.04
AU1	120	2.57	1.69	1.12	1.12
BU1	40	0.88	1.61	1.07	0.93
BU1	80	1.75	1.56	1.04	1.04
BU1	120	2.63	1.69	1.13	1.13
AU2	40	0.89	1.60	1.07	0.93
AU2	80	1.79	1.57	1.04	1.04
AU2	120	2.68	1.69	1.13	1.13
BU2	40	0.86	1.62	1.08	0.93
BU2	80	1.73	1.55	1.03	1.03
BU2	120	2.59	1.69	1.12	1.12
Perforated Columns					
AP1	40	0.94	1.62	1.08	0.94
AP1	80	1.87	1.60	1.07	1.07
AP1	120	2.81	1.71	1.14	1.14
BP1	40	0.82	1.65	1.10	0.95
BP1	80	1.63	1.52	1.02	1.02
BP1	120	2.45	1.67	1.11	1.11
AP2	40	0.94	1.61	1.07	0.93
AP2	80	1.88	1.60	1.07	1.07
AP2	120	2.82	1.71	1.14	1.14
BP2	40	0.83	1.64	1.09	0.95
BP2	80	1.65	1.53	1.02	1.02
BP2	120	2.48	1.67	1.12	1.12

Notes: L is the effective length in inches. The end conditions are taken to be hinged about the minor axis and for twisting and fixed about major axis.

$$R1 = \frac{\text{(ECCS design strength)}}{\text{(modified AISI allowable with FS = 1.92)}}$$

$$R2 = \frac{\text{(ECCS design strength) / (load factor = 1.5)}}{\text{(modif. AISI allow. with FS = 1.92)}}$$

$$R3 = \frac{\text{(ECCS design strength) / (load factor = 1.5)}}{\text{(modif. AISI allow. with varying FS)}}$$

Table 3c
PARAMETRIC STUDIES
Fixed Ended Columns - Torsional-Flexural Buckling

SECTION	L	λ	R1	R2	R3
Unperforated Columns					
AU1	40	0.54	1.80	1.20	1.04
AU1	80	1.05	1.51	1.01	0.88
AU1	120	1.51	1.49	1.00	1.00
BU1	40	0.57	1.78	1.19	1.03
BU1	80	1.08	1.48	0.99	0.86
BU1	120	1.50	1.48	0.99	0.99
AU2	40	0.57	1.78	1.19	1.03
AU2	80	1.07	1.50	1.00	0.87
AU2	120	1.47	1.47	0.98	0.98
BU2	40	0.55	1.79	1.19	1.03
BU2	80	1.04	1.51	1.01	0.87
BU2	120	1.42	1.44	0.96	0.96
Perforated Columns					
AP1	40	0.59	1.80	1.20	1.04
AP1	80	1.14	1.51	1.01	0.87
AP1	120	1.65	1.56	1.04	1.04
BP1	40	0.53	1.80	1.20	1.04
BP1	80	1.00	1.53	1.02	0.88
BP1	120	1.40	1.44	0.96	0.83
AP2	40	0.59	1.80	1.20	1.04
AP2	80	1.12	1.51	1.01	0.88
AP2	120	1.54	1.52	1.02	1.02
BP2	40	0.53	1.80	1.20	1.04
BP2	80	0.99	1.53	1.02	0.89
BP2	120	1.35	1.42	0.95	0.82

Notes: L is the effective length in inches. The end conditions are taken to be hinged about the minor axis and for twisting and fixed about major axis.

$$R1 = \frac{\text{(ECCS design strength)}}{\text{(modified AISI allowable with FS = 1.92)}}$$

$$R2 = \frac{\text{(ECCS design strength) / (load factor = 1.5)}}{\text{(modif. AISI allow. with FS = 1.92)}}$$

$$R3 = \frac{\text{(ECCS design strength) / (load factor = 1.5)}}{\text{(modif. AISI allow. with varying FS)}}$$

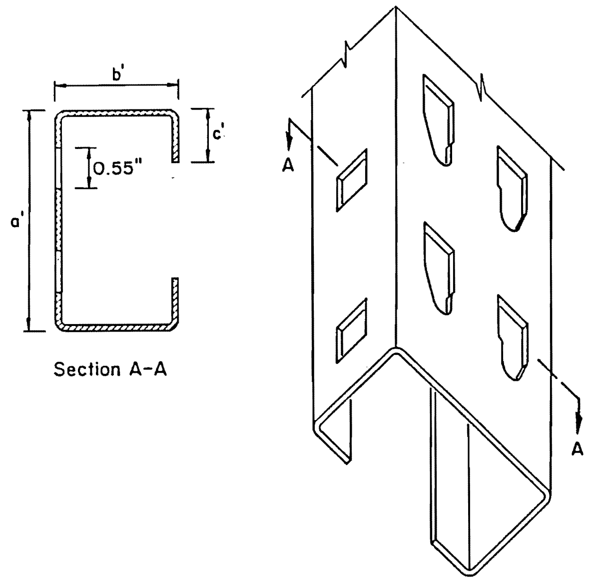

Fig. 1 Type A Specimen

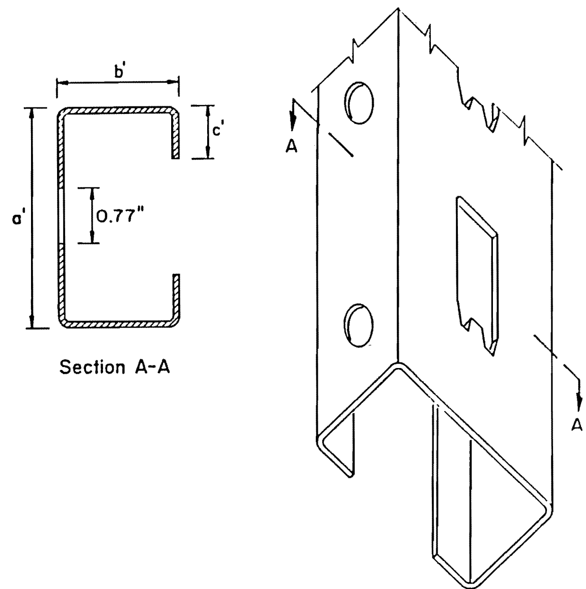

Fig. 2 Type B Specimen

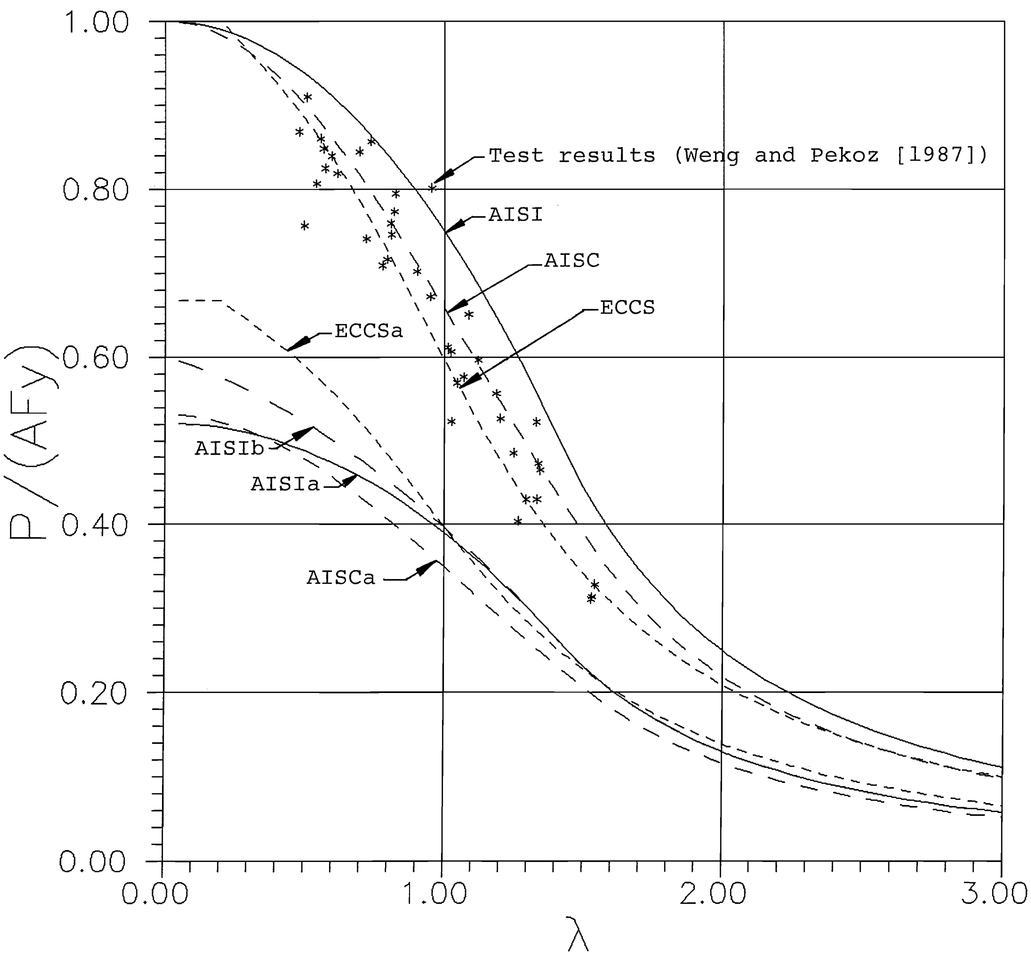

Fig. 3
Unperforated column
test results and
design equations