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Abstract

The results of an on-going research program to develop a design procedure for perforated industrial storage rack columns are presented. The design procedure includes the effects of distortional buckling. The types of columns studied include those commonly used in Europe and some that are used in the US. The design procedure is being developed based on finite element studies verified by physical testing. The design procedure involves the use of the current US rack column design approach with an extension to distortional buckling with Finite Strip Method (FSM) solutions and Direct Strength Method (DSM) approaches. The approach originally developed for individual columns is being studied for the behavior of such columns in a rack frame.

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Introduction

The objective of this paper is to summarize the results of the ongoing research on the design of columns of industrial cold-formed steel rack columns. The main focus of this research is the effect of perforations on the distortional buckling such columns.

The research is being conducted as an international joint effort in Spain and Turkey as well as the United States. Though the focus of this paper is the type of sections used in the United States, the research includes the types of sections commonly used in Europe and Australia.

This paper covers a possible simplified approach accounting for the effect of perforations using the finite strip approach (FSM) and the direct strength method (DSM). The finite strip approach was extended to perforated columns by determining a reduced thickness for the strips containing perforations.

The approach formulated for columns with idealized end conditions is being investigated for application for columns in a frame. The results to date are encouraging. The approach developed was confirmed by extensive physical testing and finite element (FEM) studies on European type columns. The approach developed was confirmed for sections used in the U. S. by finite element studies. Physical testing for such sections as columns with idealized end conditions and frames containing such columns are in progress.

Direct Strength Method

Direct strength method (DSM) described in References [9 and 10] is a part of the North American Specification [11]. It is used to determine the ultimate strength of columns and beams that are approved by the Specification. The scope covers local, distortional and global buckling loads and their interaction. The strength of a column is taken as the minimum of the global, local and distortional buckling strengths.

Buckling loads for these modes of buckling can be determined by the finite strip method analysis using the CUFSM3 and CUFSM4 programs described in [3] and available at http://www.ce.jhu.edu/bschafer/cufsm/index.htm. A reduced thickness approach was used for modeling the effect of perforations in the strips containing perforations. Such an approach was also used by Moen and Schafer [4] for large circular perforations in lipped channel cold-formed steel wall studs and joists. However, rack columns with closely spaced multiple perforations of variety of shapes necessitated a different approach. The details of the studies to reach a satisfactory expression for the reduced thickness are discussed in

Casafont, M., et al [5, 6]. These studies involved the testing and finite element modeling of the sections shown in Fig. 1 for various lengths at E.T.S. d'Enginyeria Industrial de Barcelona, Universitat Politècnica de Catalunya, Barcelona. Physical testing of Section type C8 which is a typical column section used in the U. S. will be carried out in the near future.

Global Buckling

The types of sections investigated in this research may in general be subject to torsional flexural buckling. On the basis of thousands of finite element studies and dozens of physical tests it was concluded and reported in Casafont, M., et al [5, 6] that for the calculation of torsional flexural buckling load by a reduced thickness of the strips containing web perforations may be taken as

$$t_{rG} = 0.7t\left(\frac{L_{np}}{L}\right) \qquad \text{Eq. 1}$$

where t is the wall thickness, the terms L_{np} and L are shown in Fig. 2. The flexural and torsional flexural buckling loads can be determined by CUTWP written originally by Andrew Sarawit as a part of his PhD research at Cornell University reported in http://ceeserver.cee.cornell.edu/tp26/TWResearchGroup/research%20reports.htm can be obtained from http://www.ce.jhu.edu/bschafer/cutwp/index.htm. This program does account for varying wall thickness around the perimeter of the cross-section. According to the North American Specification [11], once the elastic buckling load P_{cre} is determined, the nominal axial strength P_{ne} can be calculated as follows:

$$P_{ne} = \left(0.658^{\lambda_c^2}\right)P_y \ \text{ if } \lambda_c \le 1.5 \qquad \text{Eq. 2}$$

$$P_{ne} = \left(\frac{0.877}{\lambda_c^{\ 2}}\right)P_y \ \text{ if } \lambda_c > 1.5 \qquad \text{Eq. 3}$$

where $\lambda_c = \sqrt{\dfrac{P_y}{P_{cre}}}$ Eq. 4

P_{cre} is the minimum of the flexural, torsional or torsional- flexural buckling load

$$P_y = A_g F_y \qquad \text{Eq. 5}$$

In for perforated columns A_g may be substituted by A_{netmin} that is the minimum cross-sectional area obtained by passing a plane through the column normal to the axis of the column. F_y is the yield stress of the column material.

The buckling stress is

$$F_n = \frac{P_{ne}}{A_{net\,\min}} \qquad \text{Eq. 6}$$

Local Buckling and Interaction with Global Buckling

Determination of the local buckling load and the interaction of local and global buckling using finite strip approach using a reduced thickness approach was not as satisfactory due to the variety of perforation and section geometries and cold forming effects [5, 6]. The assessment of local buckling behavior is best done on the basis of stub column tests. Procedures for stub column tests are described in the RMI Specification [1] and its Commentary [2].

The interaction of local and global buckling was found to be best determined by the approach derived by Peköz, T [7, 8] and adopted in the RMI Specification [1]. The approach involves determining the effective area A_e by the expression

$$A_e = \left[1 - (1 - Q)\left(\frac{F_n}{F_y}\right)^Q\right] A_{net\,\min} \qquad \text{Eq. 7}$$

where Q is determined on the basis of stub column tests, F_n and F_y are the global buckling stress determined as described above and the yield stress, respectively. Thus column nominal axial strength is

$$P_{ne} = F_n \left[1 - (1 - Q) \left(\frac{F_n}{F_y} \right)^Q \right] A_{net\min} \qquad \text{Eq. 8}$$

Distortional Buckling

On the basis of thousands of finite element studies and dozens of physical tests it was concluded and reported in Casafont, M., et al [5, 6] that for the calculation of distortional buckling load by finite strip approach, the reduced thickness of the strips containing web perforations may be taken as

$$t_{rD} = 0.9t \left(\frac{L_{np}}{L} \right)^{1/3} \qquad \text{Eq. 9}$$

where t is the wall thickness, the terms L_{np} and L are shown in Fig. 2. This expression for reduced thickness was shown in Casafont, M. et al [5, 6] to be satisfactory within the limits of the geometric parameters in Table 1. Currently the extension to larger flange perforations is being studied.

The analyses using CUFSM3 and CUFSM4 are to be carried out applying no stress on the strips containing perforations in order to avoid local buckling modes that are not relevant. The validity of this assumption is verified in the studies of Casafont, M. et al [5, 6].

For typical European sections comparison of the distortional buckling loads using finite element calculations and finite strip calculations using reduced thickness is given in Table 2. It is seen that the finite strip approach is conservative and satisfactory.

According to the AISI Specification distortional buckling strength P_{nd} is determined as follows:

For $\lambda_d \le 0.561$ $P_{nd} = P_y$ Eq. 10

For $\lambda_d > 0.561$ $$P_{nd} = \left(1 - 0.25 \left(\frac{P_{crd}}{P_y} \right)^{0.6} \right) \left(\frac{P_{crd}}{P_y} \right)^{0.6} P_y \qquad \text{Eq. 11}$$

where $\lambda_d = \sqrt{\dfrac{P_y}{P_{crd}}}$ Eq. 12

and P_{crd} is the distortional buckling load.

The ultimate strength of a column is assumed to be the lower of the global buckling strength determined for the global buckling strength P_n and the distortional buckling strength P_{nd} . Different alternative versions of Equations 9, 10 and 11 were tried by substituting different terms for the term P_y .

Parametric Studies

The results of an extensive physical test program and accompanying finite element studies were carried out on European types sections at Universitat Politècnica de Catalunya, Barcelona. Sections shown in figure 1 except Section C8 are the European type sections. The results are reported by Casafont et al [5, 6].

Section C8 is a typical 3”x3” section having wall thicknesses of 0.067” and 0.1” with tear drop shaped perforations in the web used in the U.S. Extensive finite element studies were carried out in Fatih University both on individual column segments as well as upright frames. Finite element models of 24” and 42” are shown in Figure 5.

It is planned to test several Section C8 column specimens in Universitat Politècnica de Catalunya, Barcelona as well as several upright frames in the U. S. in the near future.

The finite element studies were carried out assuming different degrees of global, local and distortional geometric imperfections according to the magnitudes and procedure described by Schafer and Pekoz [12, 13]. The imperfections are designated, for example as $P(\Delta < d)25\%$ indicates the probability that a randomly selected imperfection value Δ is less than a discrete deterministic imperfection *d* is expected to have in 75% of the time. Some of the results of imperfection studies are shown in Figure 4.

Several possible design procedures were investigated, but due to space restrictions only the following three most promising three are presented. Each alternative starts with the following:

1. Determine elastic global buckling load P_{cre} (possibly by program CUTWP) using reduced thickness by Eq. 1
2. Determine elastic distortional buckling load P_{crd} (possibly by CUFSM 3 or CUFSM 4) using the reduced thickness by Eq. 9

After the above calculations, the following are the alternatives:

- Alternative 1 - P_{rmi1} : Determine P_{nd} by Equations 10, 11 and 12 substituting P_y by P_{ne} determined P_{ne} based on P_{cre} and Equations 2 and 3.

- Alternative 2 - P_{rmi2} : Determine P_{nd} by Equations 10, 11 and 12 substituting P_y by P_{ne} determined P_{ne} based on P_{cre} and Eq. 8.

- Alternative 3 - P_{rmi3} : Determine P_{nd} by Equations 10, 11 and 12.

Column strengths P_{rmi1} , P_{rmi2} and P_{rmi3} are the smaller of P_{ne} determined by Eq. 8 based on P_{cre} and P_{nd} according to the alternatives 1, 2 and 3, respectively.

The correlation studies for these possible design procedures are shown in Tables 3 and 4. These should be reviewed with the trends observed about the effect of imperfections in Tables 5 and 6. It is seen in Table 3 that the ratio of the results for perforated columns having perfect geometry to results for imperfections of (D<d) 75%ranges from 1.06 to 1.14, the same ration in Table 4 varies from 1.00 to 1.11. Therefore an accuracy of the design procedures of the order of 10% appears to be acceptable. Thus, results in Tables 5 and 6 indicate that the approaches P_{rmi1} and P_{rmi2} would be acceptable. These approaches reflect the interaction between local, global and distortional buckling. This subject is of current research interest. Using just Eq. 8 gives the axial load

without considering distortional buckling effects is P_{rmi} and the ratio $\frac{P_{rmi}}{P_{TEST}}$ indicates the error in ignoring distortional buckling effects.

Column Behavior in a Frame

The design approach originally developed for individual columns is being studied for the behavior of such columns in a rack frame. Extensive FEM studies are being carried out and a testing program is also planned for the near future. The frame being studied and will be tested in near future is shown in Figure 6. The unbraced length of the column in the frame that appears buckled is 43” Preliminary results such as shown in Figure 6 show that the buckling load (39 kips) for the 42” individual column with pinned end compares reasonably well with the value obtained in the FEM analysis of the upright frame (43 kips). Further information will be forthcoming.

Conclusions and future work

A design approach for the design of perforated cold formed steel industrial rack columns is developed. A test program for the behavior and design of such columns in upright frames is planned for the near future.

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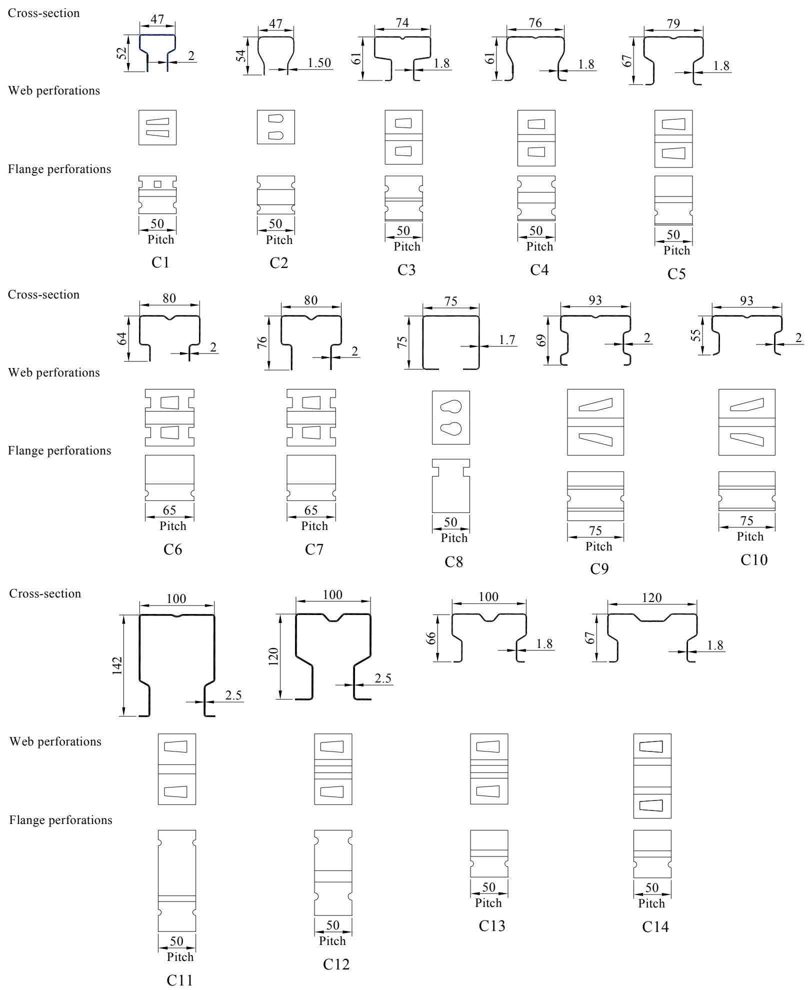

Figure 1 Cross-sections and perforations of the columns used to verify the model of reduced thickness (all dimensions in mm).

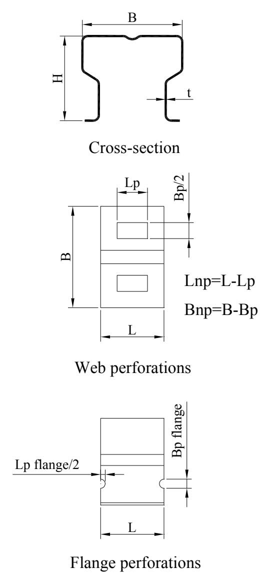

Figure 2 Main geometric parameters of the column

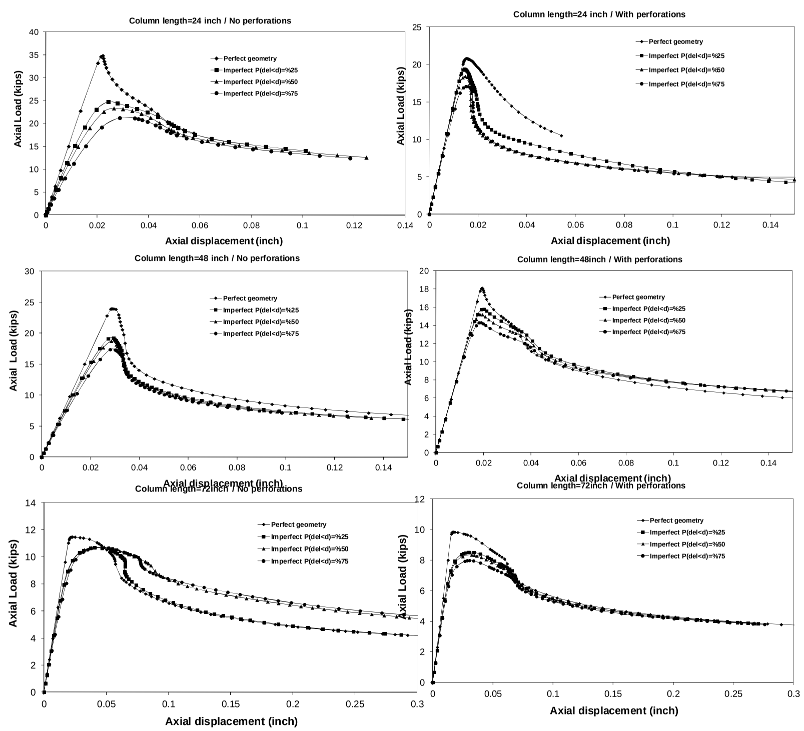

Figure 4 FEM imperfection sensitivity study results on Column C8 t=0.067”

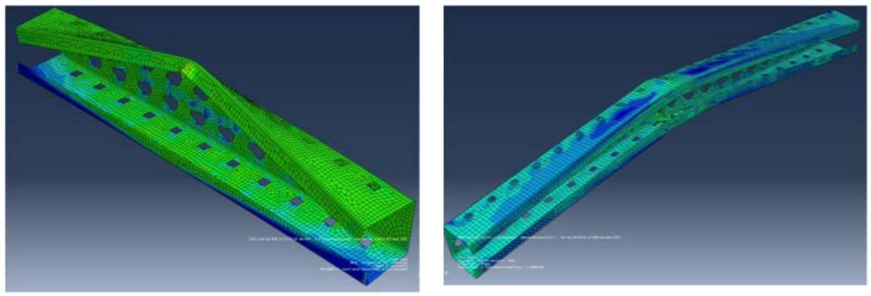

Figure 5 FEM Model of C8 L = 24” and 42”

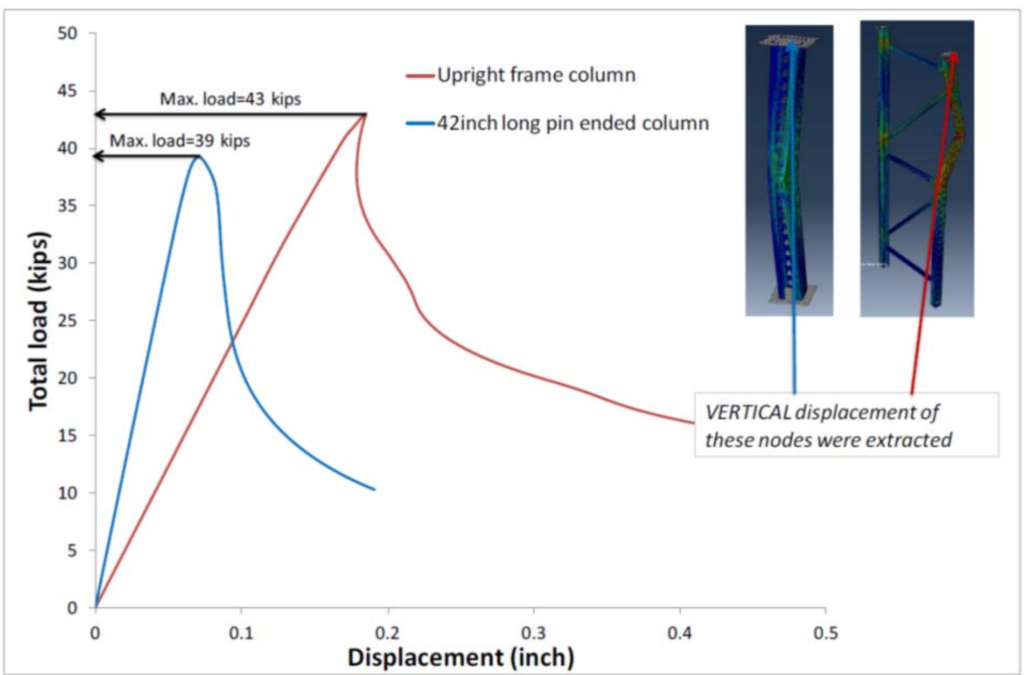

Figure 6 Upright frame with C8 t= 0.1” columns loaded axially

Table 1 Limits of the application of the Reduced Thickness for Web Perforations

Parameters	Limits
B/t	24 to 88
H/t	26 to 83
H/B	0.48 to 1.87
L	50 mm to 75 mm
B_p/L_p	≤1.6
L_{np}/L	0.33 to 0.62
B_{np}/B	0.51 to 0.90
$B_{pflange}/H$	< 0.33
$L_{p\ flange}/L$	< 0.35
$(B_{pflange}\ L_{p\ flange})\ /(B \cdot H)$	<0.058

Table 2 Comparison of the distortional Buckling Loads determined by Finite Element Method and Finite Strip Method using Reduced Thickness Approach

Column	t (mm)	L (mm)	FEM (N)	CUFSM (N)	$\frac{P_{crCUFSM}}{P_{crFEM}}$
C1	2.01	250	139111	136672	0.98
C2	1.56	250	147030	136672	0.92
C3	1.83	250	48479	44828	0.92
C4	1.88	250	198888	193212	0.97
C5	1.78	250	143734	137090	0.95
C5	1.83	400	143415	134710	0.94
C5	2	250	132695	120717	0.91
C5	2.54	250	122875	111619	0.91
C11	2.53	450	119930	121696	1.01
C12	2.53	400	129515	121696	0.93
C13	1.76	350	168190	156875	0.93
C13	2.01	350	191013	181449	0.95
C13	2.55	350	151733	148839	0.98
C14	1.76	400	323191	310328	0.96
C14	2.02	400	154076	145948	0.95
C14	2.53	400	167414	157931	0.94
Avg.					0.95
St. Dev.					0.028
C. O. V.					0.030

Table 3 Test and FEM results for different imperfections for a Typical U. S. Section

L (in)		P_{FEM} (kips) PERFECT GEOM.	P_{FEM} (kips) (D<d) 25%	P_{FEM} (kips) (D<d) 50%	P_{FEM} (kips) (D<d) 75%
24	Unperforated	34.80	24.74	23.30	21.21
24	Perforated	20.81	19.39	18.41	17.07
48	Unperforated	23.94	19.18	18.70	17.36
48	Perforated	18.10	15.77	15.19	14.29
72	Unperforated	11.47	10.68	10.71	10.68
72	Perforated	9.84	8.52	8.34	7.98

Table 4 Test and FEM results for different imperfections for Typical European Sections

Column	L (mm)	P_{FEM} PERFECT GEOM. (N)	P_{FEM} (kips) (D<d) 25% (N)	P_{FEM} (kips) (D<d) 75% (N)
C1	750	67210	68950	62175
C1	875	63110	68280	61832
C1	1000	55600	63369	60263
C3	900	150070	142270	130460
C3	1300	134300	137130	126493
C3	1650	118010	119143	115457
C4	900	90860	101860	91590
C4	1100	87730	102673	91687
C5F	900	115990	131780	119287
C5F	1200	112140	127250	117393
C5F	1500	100930	115653	110664
C5F	1800	85340	105627	107176

Table 5 Comparison of Test results with Possible Design Procedures

for Typical European Sections (Fig. 1)

Column	L (mm)	P_{TEST} (N.)	P_{rmi1}/P_{TEST}	P_{rmi2}/P_{TEST}	P_{rmi3}/P_{TEST}	P_{rmi}/P_{TEST}
C1	750.0	67210	1.018	0.977	1.092	1.092
C1	875.0	63110	1.051	1.010	1.115	1.115
C1	1000.0	55600	1.151	1.107	1.207	1.207
C3	900.0	150100	0.839	0.797	0.889	0.952
C3	1300.0	134300	0.883	0.842	0.978	0.978
C3	1650.0	118000	0.937	0.896	1.009	1.009
C4	900.0	90860	1.000	0.914	1.073	1.073
C4	1100.0	87730	0.995	0.912	1.053	1.053
C5	900.0	116000	0.924	0.864	0.991	1.114
C5	1200.0	112100	0.933	0.871	1.059	1.105
C5	1500.0	100900	0.968	0.908	1.109	1.109
C5	1800.0	85340	1.050	0.990	1.159	1.159
Avg.			0.979	0.924	1.061	1.081
St. Dev.			0.084	0.085	0.086	0.073
C. O. V.			0.086	0.091	0.081	0.068

Notes: Simply supported ends

P_{rmi} by Eq. 8

Table 6 Comparison of FEM results with Possible Design Procedures

for a typical US Section (Section C8 in Fig. 1)

Column	L (in)	P_{FEM} (k.)	P_{rmi1}/P_{FEM}	P_{rmi2}/P_{FEM}	P_{rmi3}/P_{FEM}	P_{rmi}/P_{FEM}
C8 t=0.1"	24	34.010	1.081	1.081	1.185	1.249
C8 t=0.1"	42	32.240	0.915	0.915	0.972	0.972
C8 t=0.1"	48	27.180	0.980	0.980	1.014	1.014
C8 t=0.1"	72	15.400	0.932	0.932	0.965	0.965
C8U t=0.067"	24	20.810	1.038	0.879	1.038	1.038
C8U t=0.067"	48	18.100	0.834	0.761	0.834	0.834
C8U t=0.067"	72	9.840	0.851	0.851	0.851	0.851
C8L t=0.067"	24	17.710	1.219	1.033	1.219	1.265
C8L t=0.067"	48	14.290	1.056	0.963	1.056	1.056
C8L t=0.067"	72	7.980	1.049	1.049	1.049	1.049
Avg.			0.996	0.944	1.018	1.029
St. Dev.			0.117	0.098	0.124	0.142
C. O. V.			0.118	0.104	0.122	0.138

Notes: Simply supported ends,

P_{rmi} by Eq. 8

C8 calculated for no imperfections

C8U calculated for no imperfections

C8U calculated for (D<d) 75%

t is the wall thickness