View metadata, citation and similar papers at core.ac.uk

brought to you by CORE

provided by Missouri University of Science and Technology (Missouri S&T): Scholars' Mine

Missouri University of Science and Technology

Scholars' Mine

International Specialty Conference on Cold-Formed Steel Structures

(2004) - 17th International Specialty Conference on Cold-Formed Steel Structures

Oct 26th, 12:00 AM

Design of Cold-formed Web Members with Non-uniform Cross Sections

Serge Parent

Joseph J. Pote

Kenneth W. Neale

Follow this and additional works at: https://scholarsmine.mst.edu/isccss

Part of the Structural Engineering Commons

Recommended Citation

Parent, Serge; Pote, Joseph J.; and Neale, Kenneth W., "Design of Cold-formed Web Members with Non-uniform Cross Sections" (2006). *International Specialty Conference on Cold-Formed Steel Structures*. 3. https://scholarsmine.mst.edu/isccss/17iccfss/17iccfss-session3/3

This Article - Conference proceedings is brought to you for free and open access by Scholars' Mine. It has been accepted for inclusion in International Specialty Conference on Cold-Formed Steel Structures by an authorized administrator of Scholars' Mine. This work is protected by U. S. Copyright Law. Unauthorized use including reproduction for redistribution requires the permission of the copyright holder. For more information, please contact scholarsmine@mst.edu.

Seventeenth International Specialty Conference on Cold-Formed Steel Structures
Orlando, Florida, U.S.A, November 4-5, 2004

Design of Cold-Formed Web Members with Non-Uniform Cross Sections

Serge Parent[1], PE, Joseph J. Pote[2], PE, Kenneth W. Neale[3], Ph.D., Eng.

Abstract

In this paper, design algorithms are proposed for the design of cold-formed webs used in joists and girders built with conventional hot rolled chords. Two types of web members with non-uniform cross section are investigated: single channels periodically closed and back-to-back channels with batten plates. The design method for periodically closed sections is based on the representation of the cross sectional properties using Fourier series introduced in an energy balance for the determination of the buckling loads about each of the three member axes. The back-to-back channels case is solved by the adaptation of the classical Engesser solution and by the critical shear ratio approach. In all cases, the proposed algorithms are integrated in the actual frame of the AISI (1996) standard design curves with appropriate effective lengths coefficients.

[1] Ph.D. Candidate, SMI Joist Iowa 1120 Commercial St. P.O. Box 340 Iowa Falls, IA 50126, USA
[2] Divisional Director of Engineering and Head of Research and Development, SMI Joist Arkansas 3665 Highway 32 North P.O. Box 2000 Hope, AR 71801 USA
[3] Eng. Canada Research Chair in Advanced Engineered Material Systems Department of Civil Engineering Université de Sherbrooke Sherbrooke, Quebec, Canada J1K 2R1

Introduction

The costly use of square tube sections in joist applications justifies the development of non-uniform or partially closed cold-formed shapes. A second example of a better material usage investigated in this research is the double web member made of back-to-back channels with batten plates. In this study, two types of single non-uniform web members have been experimentally and numerically tested: the partially closed channel with plates and the stitched closed channel. Both types are presented in the first part of this paper. The second part covers the case of double web members made with two back-to-back channels used in girder applications. In this assembly, an increased axial strength is obtained when compared with the conventional hot rolled double angles of same weight. For both types of web members, the proposed design algorithms, a description of the experimental program and the results obtained are presented. A brief discussion completes both parts, followed by the list of references and symbols.

Singly Non-Uniform Cold-Formed Web Members

Figure 1 shows the back-to-back channels with battens and a periodically closed channel. In all cases, the single cold-formed channel webs are welded to conventional hot rolled angle chords. The design of this type of non uniform single member is based on the representation of the periodicity of the geometry by Fourier series. The Fourier series approach is required as opposed to simply weighing the various geometric properties because it accounts for the position of the transition in the cross section (a channel can be closed on its fourth face using different patterns and still give the same averages properties as opposed to an exact description using periodic series). Timoshenko quotients (Bažant, Cedolin, 1991) are then adapted to calculate the buckling loads with respect to all axes. These values are used to obtain the nominal buckling stress F_n as defined in AISI (1996) Standard. The effective area is calculated and weighted between the closed and open section using a boxing ratio defined by Φ = (number of closures · length of closure)/(length of web member between the chords). The nominal buckling strength of the web member is finally defined by $P_n = A_e \cdot F_n$ as per AISI (1996) Standard.

Proposed Design Algorithm-Single Member

The complete algorithm includes the following 11 steps:

STEP 1: Determine the geometric properties of a single cold-formed channel (use section properties for cold-formed member).
STEP 2: Redo STEP 1 for a tube obtained by closing the channel on its fourth face.
STEP 3: Calculate the periodic cross-sectional properties by first defining the following geometric variables shown on Figure 1: PITCH = the regular interval at which the centerline of a closed area repeats itself, LENGTH = the length of the closure over the channel, L = the overall length of the member being analyzed. The period is then set equal to the PITCH value and the lower and upper limit for integration respectively as:

$$AA = p = 0.5\ PITCH \quad BB = -AA - (LENGTH - PITCH) \tag{1}$$

The appropriate numerical value of any geometric property is attributed to the variables CHANNEL and TUBE. All variables are unitless and are defined as the functions being integrated to calculate the series coefficient. For example, for the area in in^2, we define:

$$f_1(x) = CHANNEL = \frac{AREA_C}{in^2} \qquad f_2(x) = TUBE = \frac{AREA_T}{in^2} \tag{2}$$

The coefficients needed to define the complete series are expressed by (Zwillinger, 1996):

$$a_o = \frac{1}{2p}\int_{-AA}^{BB} f_1(x)dx + \frac{1}{2p}\int_{BB}^{AA} f_2(x)dx \tag{3}$$

$$a_n = \frac{1}{p}\int_{-AA}^{BB} f_1(x)\cos\left(\frac{n\pi x}{p}\right)dx + \frac{1}{p}\int_{BB}^{AA} f_2(x)\cos\left(\frac{n\pi x}{p}\right)dx \tag{4}$$

$$b_n = \frac{1}{p}\int_{-AA}^{BB} f_1(x)\sin\left(\frac{n\pi x}{p}\right)dx + \frac{1}{p}\int_{BB}^{AA} f_2(x)\sin\left(\frac{n\pi x}{p}\right)dx \tag{5}$$

The geometric property over the length of a member can finally be defined by the Fourier series:

$$AREA(x) = a_o + \sum_n a_n \cos\left(\frac{n\pi x}{p}\right) + \sum_n b_n \sin\left(\frac{n\pi x}{p}\right) \tag{6}$$

The same operations are performed for the inertia in x-x I_x, inertia in y-y I_y, warping constant C_w, St-Venant constant for torsion J and the polar radius of gyration about the shear center r_o. A summation over a value n = 30 gives a smooth curve that represents well the periodic variation of a geometric property.
STEP 4: Calculate the buckling loads for the combined section using the two Timoshenko quotients and the proposed expression for P_z defined as:

$$P_x = \frac{\int_0^L \left(\frac{d\,y(x)}{dx}\right)^2 dx}{\left(\int_0^L \frac{y(x)^2}{EI_x} dx\right) k_x^2} \quad P_y = \frac{\int_0^L \left(\frac{d\,y(x)}{dx}\right)^2 dx}{\left(\int_0^L \frac{y(x)^2}{EI_y} dx\right) k_y^2} \tag{7}$$

$$P_z = \frac{\int_0^{0.5L} GJ\left(\frac{d\beta(x)}{dx}\right)^2 dx + \int_0^{0.5L} \frac{EC_w}{k_z^2}\left(\frac{d\beta^2(x)}{dx^2}\right)^2 dx}{2\int_0^{0.5L}\left[\sqrt{1+\left(\frac{d\beta(x)}{dx}\right)^2 r_o^2} - 1\right] dx} \tag{8}$$

and setting $y(x) = \sum_{n=1}^{top} \sin\left(\frac{n\pi x}{k_x L}\right)$ and $\beta(x)\sum_{n=1}^{top} \frac{1}{n^{top}} \cos\left(\frac{n\pi x}{k_z L}\right)$ for the shape functions where L is the member's overall length, top =1 for y(x) and 10 for β(x)

STEP 5: Define the squash load P_{Fy} using the area of the channel section and F_y such as $P_{Fy} = A_{channel} \cdot F_y$.

STEP 6: Calculate the parameter $\beta = \Phi\beta_{tube} + (1-\Phi)\beta_{channel}$ with Φ the boxing ratio defined above and the property $\beta = 1-(x_o/r_o)^2$ with x_o the x-coordinate of the shear center.

STEP 7: Calculate F_n using

$$\begin{cases} (F_n)_I = \left(0.658^{\lambda_c^2}\right) F_y & \text{when} \quad \lambda_c \le 1.5 \\ (F_n)_e = \left(\frac{0.877}{\lambda_c^2}\right) F_y & \text{when} \quad \lambda_c > 1.5 \end{cases} \tag{9}$$

where $(F_n)_I$ is the nominal inelastic buckling stress, $(F_n)_e$ is the nominal elastic buckling stress, λ_c is the column slenderness parameter equal to $\sqrt{P_{Fy}/P_e}$, in which P_e is the theoretical elastic flexural buckling load of the column defined as the minimum of $\{P_x, P_y, P_{e2} = \frac{1}{2\beta}\left[(P_x + P_z) - \sqrt{(P_x+P_z)^2 - 4\beta P_x P_z}\right]\}$ (Yu, 2000), (Timoshenko, 1945).

STEP 8: Calculate the properties of the channel effective section using $f = F_n$.

STEP 9: Calculate the properties of the tube effective section using $f = F_n$.

STEP 10: Calculate the effective area of the combined section with $A_e = \Phi A_{eff.\_tube} + (1-\Phi)A_{eff.\_channel}$.

STEP 11: Obtain the nominal axial strength $P_n = A_e F_n$.

Back-to-Back Cold-Formed Channels with Batten Plates

For this type of assembly, it is intended to develop design equations with due consideration for the presence of the batten plates at mid-height. Components and full scale girder tests (Parent, 2004) have shown that calculating the buckling strength as twice the single channel capacity with $L_x = L_y$ and a check on one single channel with $L_z = 0.5\ L_x$ gave lower than measured buckling loads. An adaptation of the classical equations for batten columns proposed by Engesser (Timoshenko, 1936) and Bleich (Bleich, 1952) is used. The critical shear ratio as defined by Galambos (1998) is also modified by the definition of appropriate design curves. As for the previous case, the structural analysis is introduced in the framework of the AISI (1996) Standard to calculate the axial capacity of the assembly.

Proposed Design Equations – Back-to-Back Members

A battened column is more flexible in shear than either a laced column or even a column with perforated cover plates. It acts as a Vierendeel truss. The effect of shear distortions can be significant and should be considered in calculating the compressive strength of the member (Galambos, 1998). The first theory analyzed for the problem of a back-to-back cold-formed channels with batten used for double web member is the one developed by Engesser (Engesser, 1889, 1895) and detailed by Timoshenko (Timoshenko, 1936). Figure 1 illustrates the geometry of the problem.

It has been demonstrated that the critical load of a batten compression member is always less than the one for a column that has the same cross section area and slenderness ratio. The buckling load is largely influenced by the spacing of the battens and chords. This decrease in axial capacity is mainly due to the great influence of shearing forces that increase the deflection during the buckling. The expression proposed by Timoshenko taking into consideration the shear in the batten is represented by:

$$P_{cr} = \frac{\pi^2 EI}{L^2} \frac{1}{1 + \frac{\pi^2 EI}{L^2}\left[\frac{ab}{12EI_b} + \frac{a^2}{24EI_c}\frac{1}{(1-\alpha)} + \frac{na}{bA_bG}\right]} \qquad (10)$$

$$\alpha = \frac{P_{cr}}{2\pi^2 EI_c / a^2}$$

In this expression, E = 29 500 ksi, G = 11 300 ksi, L is the overall length of the double web member, *a* is the panel length, *b* is the distance between the centroids of both channels, I_c the inertia of one channel (which should be substituted with the total inertia of the column), I_b the inertia of the batten plate (THICK·DEPTH^3/12), A_b the area of the batten plates (2·THICK·DEPTH), [π^2EI/L^2] an expression replaced with P_n as defined in AISI (1996) Standard which represents the column axial strength taken as a whole and *n* a numerical factor equal to 1.2 in the case of a rectangular cross section.

Since α contains P_{cr}, this equation can be solved only by a trial-and-error method. To account for the axial strength of the column taken as a whole, the expression π^2EI/L^2 is replaced with nominal axial capacity P_n as mentioned above. Furthermore, since only two panels are generated with a double batten at mid height, the last expression represents more adequately the test results if the total inertia of the column is used instead of I_c. In this sense, I_c is replaced with $I_{yt} = 2[I_c+(A_c)\cdot(b/2)^2]$ with A_c the area of one channel.

Bleich (1952) derived a procedure for the buckling load essentially as described in the preceding paragraph by using an energy formulation rather than an equilibrium approach. At the difference of Timoshenko's development, the effects of the distortion of the chord members and battens due to the shear stresses which occur during bending are disregarded. It will be shown later that the two formulations will lead to different critical loads for short members having only two panels. The approach leads to the formulation:

$$P_c = \frac{\pi^2 E_t I_0}{L^2} \frac{1}{1 + \frac{\pi^2}{24}\frac{I_0}{I_c}\left(\frac{a}{L}\right)^2 + \frac{\pi^2 E_t I_0}{L^2}\frac{ab}{12EI_b}} \qquad (11)$$

Reasoning in the same manner as in the preceding section, we may replace $I_0 = [A_c \cdot b^2/2]$ in the numerator by the moment of inertia $I = I_0 + 2I_c$ of the column. In this way we account for the flexural rigidity of the chords. Numerical and experimental results presented in this paper indicate that for short members, the ratio I_0/I_c contained in the second term in the denominator can be replaced with

I_o/I_{yt} to account for the neglected shear stress effects. In this ratio, $I_{yt}=2[I_c+A_c\cdot(b/2)^2]$ represents the inertia of the complete column instead of only twice the inertia of a simple chord. As with the previous formulation, the expression $[\pi^2 E_t I/L^2]$ is again replaced with P_n.

Another solution to the problem is based on the use of shear shape factors and moments of inertia of the battens and chords (Galambos, 1998). The function $CRIT_{rat}(\mu) = \Psi e^{-(15\mu)^{\Omega}}$ with $[\Psi = 10, \Omega = 0.55]$ for $L/20 \cong 1$, $[\Psi = 7.5, \Omega = 0.65]$ for $L/20 \cong 2$ and $[\Psi = 5, \Omega = 0.75]$ for $L/20 \cong 3$ with L expressed in inch, is proposed to match the stub columns as well as the full scale girder tests (Parent, 2004). As for the previous two cases, the typical unit of a battened column has a length *a* center to center of battens and a width *b* between centroids of the chords.

In developing the shear flexibility effect for the highly redundant battened member, Lin et al. (1970) assumed points of inflection for symmetric members at the midpoints of the battens and midway between the battens for the chords. The analysis is conservative because the overall continuity of the longitudinal members is neglected. The shear flexibility parameter is then given by:

$$\mu = \left[\frac{1}{(L/r_c)^2} + \left(\frac{b}{2L}\right)^2\right]\left[\frac{A_c}{A_b}\left(\frac{ab}{6r_b^2} + 5.2\frac{a}{b}\eta_b\right) + 2.6\xi_a\eta_c + \frac{\xi_a^3}{12}\left(\frac{a}{r_c}\right)^2\right] \quad (12)$$

The nomenclature for μ is shown in Figure 1 and (additionally) as follows: ξ_a = (a – DEPTH)/a, r_c, r_b = radius of gyration of longitudinal and batten elements, respectively. η_c, η_b = shear shape factors for the longitudinal and batten elements, respectively, where the shear shape factor is the ratio of the total cross-sectional area to the shear area (Timoshenko and Gere, 1961). In this case $\eta_c = 1.5$ and $\eta_b = 1.2$.

To obtain the critical load using the shear factor curve, we first need to calculate μ using Equation (12). Then we use the pre-defined functions and to obtain $K = [1/\ CRIT_{rat}(\mu\ )]^{0.5}$. Knowing the equivalent length factor, we can complete the calculations using the AISI (1996) design equations for compression members. This way, only one slenderness coefficient is included in all the calculations by setting $k_x = k_y = k_y = K$ to obtain F_n introduced in $P_n = F_n A_e$.

Experimental Program

To validate the proposed design algorithms, stub columns and full scale joist and girder tests were performed. The stub columns tests were designed such that the webs were fixed inside small pieces of angle 6" long representing the chords. Blocks of steel were also inserted between the chords to maintain a constant gap. For the single partially closed member, specimens of 12", 18" and 24" were axially loaded up to failure by buckling using a Tinius Olsen press coupled to a CMH 496 controller unit. A loading rate of 2.4 ksi of cross-sectional area per minute between increments of 10% of the estimated ultimate test load obtained by finite element simulation was applied. For the back-to-back channel configuration, 24" specimens were tested using the same press. In all cases, four replicates per configuration were tested.

To obtain a better representation of the behavior of these non-uniform sections, full scale joists and girders were built and tested. All joists were loaded until the critical web member (third web member in from the end) failed in compression. The experimental program included a run of joists of 40' in length by 30" deep with channel sections of 1 13/16" web, 1 3/16" flanges by 0.187" thick. Two of the test joists were built using the same web channel closed with three plates of LENGTH = 5" by 1 ¾" in width at a PITCH = 8" value. During the same test, a 60" deep girder, 48' long built with back-to-back channels joined with two batten plates was loaded. The joist was evenly loaded at 17 panel points while the girder only had seven load points with all the same load. Increments of 1000 lbf were used to start the loading but were reduced to 500 and 250 lbf before the onset of buckling occurred.

Another run of tests included 18 joists 16' long with webs of square sections 1 ½" by 0.083" thick. Nine joists 18" deep and nine other of 32" were fabricated using three different types of web with conventional angle chords: the OPN, CLO and STI. OPN is an opened channel obtained by grooving one face of the square tube, CLO is the square tube and the STI is the OPN stitched welded at both ends and in the middle. The STI section has values LENGTH = 1½" and PITCH = 10" for the 18" joist and LENGTH = 1½" and PITCH = 20" for the 32" deep joist. Illustration of the press setup used is found in Dinehart and al., (2003). All the 16' joists were equally loaded at each panel point with bridging lines on top and bottom chord.

When not obtained from a square tube, the cold-formed sections were built by press braking using ASTM A1011-02, grade 50 ksi hot roll high strength low alloy approved HALAS-F steel. GMAW welding was utilized to weld the connections using a protection gas mix with 80% argon – 20% carbon dioxide. A certified AWS A5.18 0.045" diameter 70 ksi S-3 wire was used in the process. Tensile tests as per ASTM A370, (2003) in the rolling, transverse and at 45° from the rolling direction confirmed the low anisotropy of the steel used.

Results and Discussion

Table 1 summarizes the test results for stub, full scale joists and girder experiments. The geometry of each member is listed with the periodic characteristics for the single members (LENGTH, PITCH, Φ) and the batten geometry for the back-to-back specimens (THICK and DEPTH). The length of each compression member L with the slenderness coefficients considered in the calculations is also listed. For each test, the calculated value is compared against the measured failure load.

For the calculated buckling strength in the stub column tests, k values of 0.5 were chosen for the single member periodically closed to represent the welded connection completely fixed and locked in the press. In joist application with this type of web members, values of 1.0 for the channels closed using plates vs a value of 1.3 for the channels closed with 1½" weld (stitch-welded) were considered. For the stub column test of the back-to-back channels, fixity in x and y axis was assumed with $k_x = k_y = 0.5$. Only k_z was set to 0.75 to account for the possible warping of the flanges in the fully welded connection with no rotation. In full scale girder applications, values of 1.0 were implemented in Equations (10) and (11).

The case of single member periodically closed with plates overly predicts the measured failure load. It has been found that too long plates induce bending at the onset of buckling and act as a stress riser rather than a closure. This is an indicator that the proposed model using plates is invalid for boxing ratio Φ greater than 0.5 for joist members. Nevertheless, the other measured critical loads for the stitch-welded case are in good agreement with the predicted load as long as a calibrated slenderness coefficient of 1.3 is introduced.

In general, the predicted loads on stub column tests for back-to-back cold-formed channels are lower than measured. The closest value is obtained with the critical shear ratio with 88.9 kips vs a measured critical load of 95.6 kips. When tested inside full scale girder tests, values from Equation (10) and (11) of 56.1 kips and 55.9 kips respectively slightly over estimate the experimental value of 53.9 kips. Nevertheless, the critical shear approach is almost equal to this value. As outlined in the theoretical description of the Timoshenko and Bleich formulations, both methods give different predictions, but are reasonably close as long as the appropriate geometric parameters are input as explained. For the range of members tested, the critical shear method appears to be the most representative.

Conclusion

This paper outlined different design methods to predict the critical load of two types of non uniform cold-formed member loaded axially. Single web members with periodically varying cross sectional properties and back-to-back channels with batten plates were tested using stub columns tests and real joist and girder applications. The methods presented give good predictions for the range of tested specimens used in the experimental program. The proposed design schemes also offer the advantage of being implemented within the actual AISI (1996) Standard.

References

AISI (1996): "Specification for the Design of Cold-Formed Steel Structural Members," American Iron and Steel Institute.

ASTM A370 (2003): "Standard Test Methods and Definitions for Mechanical Testing of Steel Products", American Society for Testing Material, West Conshohocken, PA.

BAŽANT, Z.P., LUIGI, C., (1991): Stability of Structures Elastic, Inelastic, Fracture and Damage Theories, Oxford University Press, New-York, 984 p.

BLEICH, B., (1952): Buckling Strength of Metal Structures, McGraw-Hill Inc. New-York, 508 p.

DINEHART, D.W.., YOST, J.R., GROSS, S.P., COOK, J. (2003): "Behavior of Steel Joists with Cold-Formed Web Members", Research Report No. 4, SMI Joist/Villanova Research Partnership, Villanova University, College of Engineering, Department of Civil and Environmental Engineering.

ENGESSER, F. (1889): *Zeitschrift für Architektur und Ingenieurwesen,* 1889, p. 445

ENGESSER, F. (1895): Schweizerische Bauzeitung, 26, p. 24, 1895.

GALAMBOS, T.V., (1998): Guide to Stability Design Criteria for Metal Structures, John Wiley & Sons, Inc. New-York, 911 p.

LIN, F.J., GLAUSER, E.C., JOHNSTON, B.G. (1970): "Behavior of Laced and Battended Structural Members", *Journal of the Structural Division*, ASCE, 96, No. ST7 – p.1377.

PARENT, S. (2004): "Experimental and Numerical Study of Steel Joist and Girder Cold-Formed Web Members with Periodically Varying Section

Properties",Ph.D. Thesis (in revision), Sherbrooke University, Sherbrooke PQ, Canada

TIMOSHENKO, S. (1936): Theory of Elastic Stability, McGraw-Hill Inc., New-York, 518 p.

TIMOSHENKO, S.P. (1945): "Theory of Bending, Torsion and Buckling of Thin-Walled Members of Open Cross Section", *Journal of the Franklin Institute*, April and May 1945.

TIMOSHENKO, S. GERE, J.M. (1961): Theory of Elastic Stability, McGraw-Hill Inc., New-York, 542 p.

YU, W.W., (2000): Cold-Formed Steel Design, 3th Edition, John Wiley & Sons, Inc., New-York, 756 p.

ZWILLINGER, D. (1996): Standard Mathematical Tables and Formulae, 30th Edition, CRC Press, Boston, 812 p.

List of Symbols

a	Vertical distance between the battens
$a_{n, o}$	Coefficients of a Fourier series
A_b	Cross-sectional area of the two battens
A_c	Area of a single channel
A_e	Effective area of a channel
b	Distance between the channels'centroids of a batten column
b_n	Coefficients of a Fourier series
C_w	Warping constant of torsion of cross section
DEPTH	Depth of a batten plate
E	Module of elasticity or Young's modulus (29 000 ksi)
E_t	Tangent modulus of elasticity
F_n	Nominal buckling stress
$(F_n)_e$	Nominal elastic buckling stress
$(F_n)_I$	Nominal inelastic buckling stress
F_y	Yield stress of the steel
G	Shear modulus (11 300 ksi)
I	Moment of inertia of a batten column taken as a whole
I_0	$A_c b^2/2$
I_b	Inertia of a batten plate
I_c	Inertia of a channel in a batten column
I_x	Moment of inertia about x-axis
I_y	Moment of inertia about y-axis
I_{yt}	Moment of inertia of a batten column taken as a whole
J	St. Venant torsion constant of cross section
$k_{x, y, z}$	Effective length factor in x, y and torsion-axis
K	Effective length factor from the critical shear ratios curves
L	Overall length of the member being analyzed
LENGTH	The length of the closure over the channel
$L_{x, y, z}$	Buckling length in the x, y and torsion -axis
n	Maximum number of terms included in a Fourier series Numerical factor of 1.2 for rectangular plate
P_{cr}	Critical buckling load
PITCH	The regular interval at which the centerline of a closed area repeats itself
P_e	Theoretical elastic flexural buckling load
P_n	Nominal axial strength of member

$P_{x,y}$	Euler flexural buckling load about x, y -axis
P_z	Torsional buckling load about z-axis
P_{Fy}	Squash load
p	Half period
r_b	Radius of gyration of batten elements
r_c	Radius of gyration of a longitudinal element
r_o	Polar radius of gyration of cross section about shear center
THICK	Thickness of a batten plate
x_o	x-coordinate of shear center
y(x)	Shape function for the lateral deflection
β	$1-(x_o/r_o)^2$
$\beta(z)$	Shape function for the angle of torsion
$\eta_{b,c}$	Shear shape factors for batten elements and longitudinal elements
λ_c	Column slenderness parameter
Φ	Boxing or closing ratio

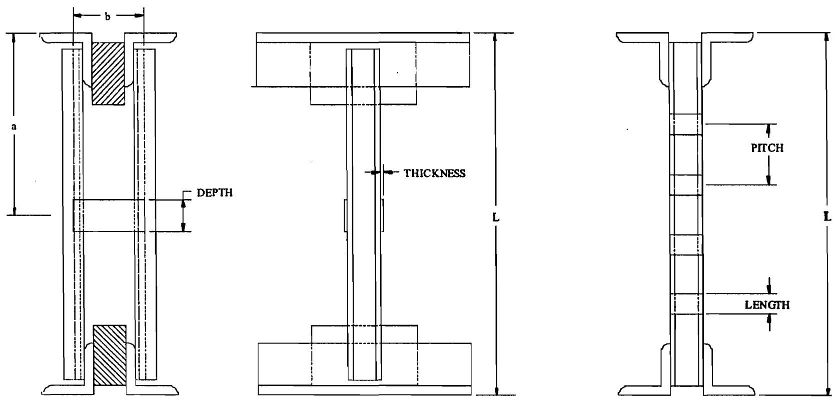

Figure 1

Stub	1 ¾	1 ¼	0	0.187	1 ¼	1 ¾	0.625	n/a	n/a	12	0.5	40.1	43.6
Stub	1 ¾	1 ¼	0	0.187	1 ½	3	0.429	n/a	n/a	18	0.5	37.3	40.0
Stub	1 ¾	1 ¼	0	0.187	2	4	0.400	n/a	n/a	24	0.5	36.1	35.6
Joist	1 ½	1 ½	¾	0.083	1 ½	10	0.10	n/a	n/a	18.5	1.3	18.5	17.3
Joist	1 ½	1 ½	¾	0.083	1 ½	20	0.04	n/a	n/a	38	1.3	12.2	12.4
Joist	1 13/16	1 3/16	0	0.187	5	8	0.50	n/a	n/a	35	1.0	21.9	13.2
											kx=0.5 ky=0.5 kz=0.75	EQ(10) 83.7	
Stub	2 ½	1 ¾	0	0.187	n/a	n/a	n/a	0.187	3 ½	24	kx=0.5 ky=0.5 kz=0.75	EQ(11) 85.0	95.6
											K=0.85	SHEAR 88.9	
											1.0	EQ(10) 56.1	
Gird	2 3/8	1 ½	0	0.187	n/a	n/a	n/a	0.187	3.5	60	1.0	EQ(11) 55.9	53.9
											K=1.28	SHEAR 53.2	

Table 1 Stub and Full Scale Test Results