Bracing Design for Torsional Bucking of Cold-Formed Steel Wall Stud Columns

C. D. Moen

Wei-Wen Yu International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri, U.S.A., November 7 & 8, 2018

Bracing design for torsional buckling of cold-formed steel wall stud columns

C.D. Moen[1]

Abstract

A method is presented for calculating the required brace stiffness and strength to limit torsional buckling deformation in cold-formed steel wall stud columns. The bracing (bridging) design method utilizes recent insight from classical stability solutions that define twist of singly and doubly-symmetric columns with an initial twist imperfection as a function of column compressive load. A wall stud design example is provided.

Introduction

Singly-symmetric cold-formed steel C-section wall studs are the bread and butter of the light steel framing industry, and to ensure these studs are working well together in a wall system, discrete bracing (bridging) is provided to limit stud twisting and bending. A stud tends to twist under a compressive load because the flexural and torsional buckling modes are coupled when the centroid is offset from the center of twist. The goal of this paper is to provide a method to calculate the torsion bracing demand and the stiffness required to limit this twist to a reasonably small magnitude.

While flexural bracing of compression members has been widely studied both analytically and experimentally, stability research leading to recommendations for torsional buckling deformation of compression members is limited. Torsional bracing stiffness equations for I-section columns were developed from an elastic torsional buckling solution including the torsion brace as a rotational spring (Helwig and Yura 1999). Recent work experimentally validated an energy solution that demonstrates how to use the critical elastic column buckling load including a discrete torsion brace (Blum and Rasmussen 2016) to calculate the capacity of steel portal frames. The connection between column torsional buckling twist deformation, initial imperfections, and required bracing stiffness

[1] CEO and President, NBM Technologies, Inc., cris.moen@nbmtech.com

and strength still remain elusive however, primarily because analytically predicting the tendency of a column to twist under a compressive load is cumbersome.

Flexural column bracing strength and stiffness requirements in current codes and standards stem from classical stability studies (e.g., Winter 1958). The required brace force and stiffness to limit mid-height deflection of an imperfect column is calculated with a free body diagram assuming column double curvature and a hinge (no moment in the column) at the brace. The allowable column deflection at the brace is typically assumed as the same magnitude as the initial flexural imperfection.

Extending this style of flexural bracing design approach to torsional bucking is challenging because the mechanics of coupled flexural-torsional buckling modes requires more bookkeeping since there are three governing differential equations (not one), and the relationship between the applied compressive column load and the twist and flexural deformation considering initial geometric imperfections is more complicated. Flexural-torsional buckling deformation equations derived by Chen (1977, Section 4.5) were recently confirmed by Moen and Plaut (2018) for the case of an imperfect pinned warping free column. The twist deformation equation from Moen and Plaut (2018) is utilized herein to propose a bracing design methodology for cold-formed steel wall stud columns that tend to develop torsional buckling deformation under load.

Torsion bracing design for cold-formed steel stud columns

It is proposed that discrete mid-height torsion bracing for stud columns can be designed in two steps. The first step is to calculate the twist, θ, at the mid-height of the stud, that develops as it is loaded in compression to its nominal unbraced capacity i.e., when $P=P_{n,unbraced}$. The twist θ can be calculated with a solution of the governing differential equations defining equilibrium of an imperfect column (Moen and Plaut 2018):

$$\theta = -(\pi^4 I_1 I_2 a_2 A\mathrm{E}^2 L^2 P x_o - \pi^4 I_1 I_2 a_1 A\mathrm{E}^2 L^2 P y_o + \pi^2 I_2 a_3 A\mathrm{E} L^4 P^2 x_o^2 - \pi^2 I_1 a_2 A\mathrm{E} L^4 P^2 x_o + \pi^2 I_1 a_3 A\mathrm{E} L^4 P^2 y_o^2 + \pi^2 I_2 a_1 A\mathrm{E} L^4 P^2 y_o + a_3(-A) L^6 P^3 x_o^2 - a_3 A L^6 P^3 y_o^2 - \pi^4 I_1 I_2 a_3 \mathrm{E}^2 L^2 I_o P + \pi^2 I_1 a_3 \mathrm{E} L^4 I_o P^2 + \pi^2 I_2 a_3 \mathrm{E} L^4 I_o P^2 - a_3 L^6 I_o P^3)/(\pi^6 I_1 I_2 A\mathrm{E}^3 C_w - \pi^4 I_1 A\mathrm{E}^2 L^2 P C_w - \pi^4 I_2 A\mathrm{E}^2 L^2 P C_w + \pi^2 A\mathrm{E} L^4 P^2 C_w + \pi^4 I_1 I_2 A\mathrm{E}^2 G J L^2 - \pi^2 I_1 A\mathrm{E} G J L^4 P - \pi^2 I_2 A\mathrm{E} G J L^4 P - \pi^2 I_2 A\mathrm{E} L^4 P^2 x_o^2 - \pi^2 I_1 A\mathrm{E} L^4 P^2 y_o^2 + A G J L^6 P^2 + A L^6 P^3 x_o^2 + A L^6 P^3 y_o^2 - \pi^4 I_1 I_2 \mathrm{E}^2 L^2 I_o P + \pi^2 I_1 \mathrm{E} L^4 I_o P^2 + \pi^2 I_2 \mathrm{E} L^4 I_o P^2 - L^6 I_o P^3) \tag{1}$$

The axial load P is applied at the column centroid (+ magnitude is compression), E is the steel modulus of elasticity, $G=E/(2(1+\nu))$ is the steel shear modulus, and ν is Poisson's ratio for steel. The eccentricities from the column centroid to the column center of twist along the principal axes are x_o and y_o, L is the unbraced stud height, I_1 and I_2 are moments of inertia of the column cross-section about principal axes 1 and 2 with the axis origin at the column centroid, A is the column cross-sectional area, C_w is the warping torsion constant for the cross-section, J is the St. Venant torsion constant for the cross-section, and the polar moment of inertia about the center of twist $I_o=I_1+I_2+A(x_o^2+y_o^2)$. Flexural initial imperfection magnitudes at mid-height of the column are a_1 and a_2 (units of length) measured along the 1 and 2 principal axes, and a_3 is the initial twist imperfection magnitude (in radians) at column mid-height.

The proposed torsion bracing design criterion is that the twist, θ, calculated in Eq. (1) resulting from $P=P_{n,unbraced}$ is limited by the bracing, or in other words, the bracing should react back to limit the twist to an acceptably small value, θ_n. The flexural reaction in the brace, M_n, i.e., the torsion reaction applied to the column, that limits the twist to θ_n is calculated as (AISC 1997, Case 3):

$$M_n=\left(\theta-\theta_n\right)\frac{GJ}{L}\left[\left(1-\alpha\right)\frac{z}{L}+\frac{a}{L}\left(\frac{\sinh\frac{\alpha L}{a}}{\tanh\frac{L}{a}}-\cosh\frac{\alpha L}{a}\right)\sinh\frac{z}{a}\right]^{-1} \tag{2}$$

where $\alpha L=0.50L$ is the mid-height bracing location, $z=L/2$, and $a=(EC_w/GJ)^{0.5}$. The magnitude of M_n in Eq. (2) is the required strength of the torsion brace to develop the column braced nominal capacity, $P_{n,braced}$, and $k_T=M_n/\theta_n$ is the required brace stiffness to limit the twist at the brace to θ_n when the column is loaded to $P_{n,braced}$.

Example – bracing design for limiting torsional buckling deformation

The following example presents the proposed torsion bracing design methodology. The specific torsion bracing magnitudes and conclusions should not be used in design since the approach has yet to be verified experimentally or with simulation.

Mid-height torsion bracing is designed for a typical cold-formed steel stud in this section. The stud column – a 362S162-54 lipped Cee cross-section (SSMA 2011), has a singly-symmetric cross-section where A=272mm^2, I_1=363370mm^4

and I_2=64100 mm^4 are the principal moments of inertia about the cross-section centroid, C_w=120572604 mm^6, J=188 mm^4, x_o=-33.4 mm, y_o=0 mm and the column length is L=2438 mm. The polar moment of inertia about the center of twist is $I_o=I_1+I_2+A(x_o^2+y_o^2)$=730902 mm^4. The initial flexural imperfection magnitudes at column mid-height are assumed as $a_1=a_2=L/1000$. The initial twist imperfection magnitude at column mid-height is assumed as a_3=0.00628 radians x (L/2/1000mm)=0.00766 radians, determined based on an imperfection study in Zeinoddini and Schafer (2012) where the average twist for cold-formed steel studs was reported as 0.00628 radians over a meter of length. The elastic modulus for steel is E=200 kN/mm^2.

The first torsion bracing design step is to calculate how much the unbraced stud wants to twist when loaded to its nominal unbraced capacity, $P_{n,unbraced}$. The stud critical elastic local buckling load is $P_{cr\ell}$=70.9 kN, the critical elastic distortional buckling load P_{crd}=108 kN, the critical elastic global buckling load P_{cre}=18.8 kN calculated assuming the unbraced length is L, and the stud squash load is P_y=93.8 kN assuming the steel yield stress F_y=345 MPa. Using the AISI Direct Strength Method (AISI 2016), the global buckling ultimate limit state capacity P_{ne}=16.5 kN, the local-global buckling ultimate limit state capacity is $P_{n\ell}$=16.5 kN, and the distortional buckling ultimate limit state capacity P_{nd}=74.4 kN. The nominal stud capacity is P_n=min(P_{ne}, $P_{n\ell}$, P_{nd})=16.5 kN. Substituting $P=P_{n,unbraced}$ into Eq. (1) results in a mid-height twist of θ=0.2584 radians when the stud is not torsionally braced.

The moment in the brace required to resist the mid-height twist is calculated with Eq. (2) assuming that $\theta_n=a_3$, i.e., the twist at the braced location is the same as the initial twist imperfection magnitude. This assumption is consistent with flexural bracing design (Winter 1958). The resulting M_n=27.9 kN-mm from Eq. (2) is the flexural demand on the brace as the stud is loaded to its braced capacity, $P_{n,braced}$. The required brace stiffness that restrains twist from flexural-torsional buckling to θ_n is $k_T=M_n/\theta_n$=3642 kN-mm/rad. The stud braced capacity is obtained by recalculating P_{cre}=63.6 kN for a braced length of L/2, resulting in P_{ne} =50.6 kN and $P_{n\ell}$=48.0 kN. The braced column capacity $P_{n,braced}$=min(P_{ne}, $P_{n\ell}$, P_{nd})=48.0 kN.

Conclusion

A design method and equations are proposed for calculating the required stiffness and strength of mid-height torsion bracing in cold-formed steel wall studs columns. The bracing is designed to restrain the tendency of the stud to twist from torsional buckling. The calculation methodology is just theoretical at this point and needs to be validated with experiments and simulation.

Acknowledgements

The author is grateful for the thoughtful comments and discussions related to this work that was provided by the members of the American Iron and Steel Institute Torsion Bracing Task Group in 2017 and 2018.

References

AISI-S100-16 (2016). North American Specification for the Design of Cold-Formed Steel Structural Members. American Iron and Steel Institute, Washington, D.C.

Blum, H.B., Rasmussen, K.J.R. (2017). "Design Method for Columns with Intermediate Elastic Torsional Restraint." Proceedings of the Structural Stability Research Council Annual Conference, San Antonio, TX.

Chen W., and Atsuta T. (1977). Theory of beam–columns. Space behavior and design, vol. 2. McGraw-Hill, p. 539–78.

Helwig, T.A., Yura, J.A. (1999). "Torsional Bracing of Columns." ASCE Journal of Structural Engineering, 125(5), pp. 547-555.

Moen, C.D., Plaut, R.H. (2018). "Calculation Of Twist And Flexural Deformations For A Thin-Walled Column With Initial Imperfections." Proceedings of the International Conference on Thin-Walled Structures (ICTWS), Lisbon, Portugal.

SSMA (2011). Steel Stud Manufacturers Association, Product Technical Information, ICBO ER-4943P, <http://www.ssma.com >, (December 15, 2011).

Winter, G. (1958). ''Lateral bracing of columns and beams.'' Trans., ASCE, 125, Part 1, 809–825.

Zeinoddini V., Schafer B., (2012). "Simulation of geometric imperfections in cold-formed steel members using spectral representation approach." Thin-Walled Structures, 60, pp. 105–117.