

Introduction to Bracing Design For Cold-Formed Steel Structures

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1.1 General Information

While structural engineers have designed bracing elements into their structures for centuries, it is only over the past fifty years that a developing awareness of the means and methods for properly proportioning that bracing has developed. Winter (1958) published what is considered by many to be the seminal paper on bracing theory. Derivatives from that paper have formed the basis for most current theories, guidelines, and specification provisions for the design of bracing elements in current structural and cold-formed steel practices.

The 1999 *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC 1999) was the first North American steel design specification to provide comprehensive general design provisions for the strength and stiffness requirements for bracing elements. The current cold-formed steel design specification, the *North American Specification for the Design of Cold-Formed Steel Members* (AISI 2007) contains design requirements for several specific cold-formed steel assemblies, as have previous American Iron and Steel Institute Specifications (AISI 1986, 1991, 1996, 2004), and Canadian Standards Association S136 Specification (CSA 1994). Whereas the 2010 AISC specification (AISC 2010) currently provides general guidance that may be utilized in nearly any situation encountered by a practicing design engineer, the AISI and CSA specifications only provide guidance which is limited to specific situations, leaving the designer to their own resources for bracing situations and conditions not specifically covered by these specifications.

When a designer of a cold-formed steel structure or element determines that bracing is appropriate or required for a particular application, but the applicable design specification is silent as to how to proportion that bracing, how are they to proceed?

Current design practice has adapted to this void, utilizing designs developed from first principles of mechanics, modifications of existing procedures for similar hot-rolled or cold-formed steel assemblies, or past experience, “engineering common sense”, or empiricism.

This paper looks at current North American practice related to the design of bracing for cold-formed steel elements and structures.

1.2 Categorization of Bracing

Braces may be categorized by function, by performance criteria, or by method of interaction between braced points. These categories exist for defining usage of the brace system, however, they are not mutually exclusive. A brace may perform multiple functions under multiple loading situations, or even under a single load condition.

1.2.1 Function: Stability, Strength, and Serviceability

For purposes of determining required strength and stiffness, braces may be categorized relative to the function that the brace serves in the structure. The function of a single brace may change under different loading conditions, and a single brace may serve multiple purposes.

1.2.1.1 Stability

Stability bracing serves to ensure the stability, or resistance to buckling, of an individual member or the entire structure. When applied to an individual member, this bracing is typically designed to ensure that a particular member buckles in a higher buckling “mode.” For instance, a functioning mid-point brace in an elastic column will serve to reduce the unbraced length by 50%, thereby increasing the buckling resistance by 400%. An example of this type of bracing in cold-formed steel structures is weak-axis longitudinal bracing of axially loaded steel studs by either flat straps on the stud flanges

(Figure 1-1), or cold-rolled channels through the stud webs (Figure 1-2), or continuous sheathing attached to the stud flanges.

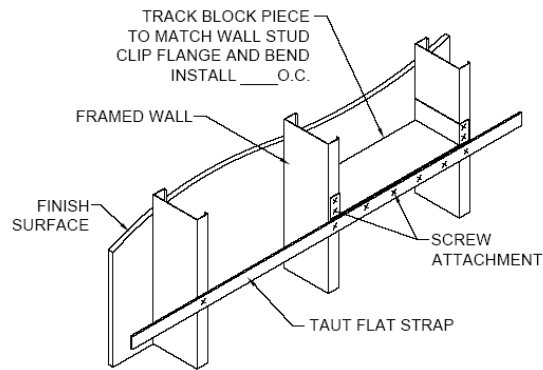


Figure 1-1 Single Flat Strap with Bracing (SSMA 2000)

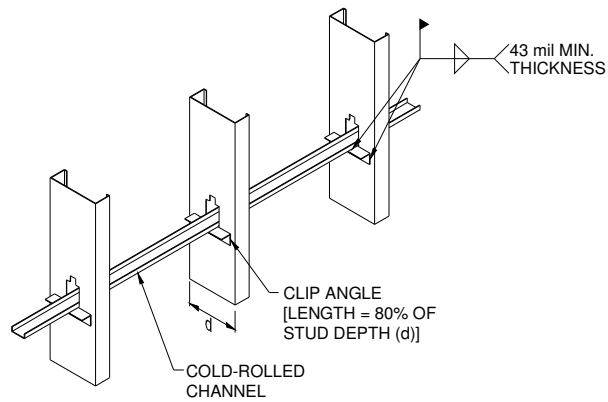


Figure 1-2 Cold-Rolled Channel with Clip Angle (SSMA 2000)

Stability bracing can also provide for the global stability of the entire structure. The longitudinal forces that develop in the aforementioned stud bracing must be resolved out of the structure to ensure that the entire line of studs does not buckle laterally as an entire unit. These forces may be resolved using X-Bracing straps (Figure 1-3) or sheathed shearwalls.

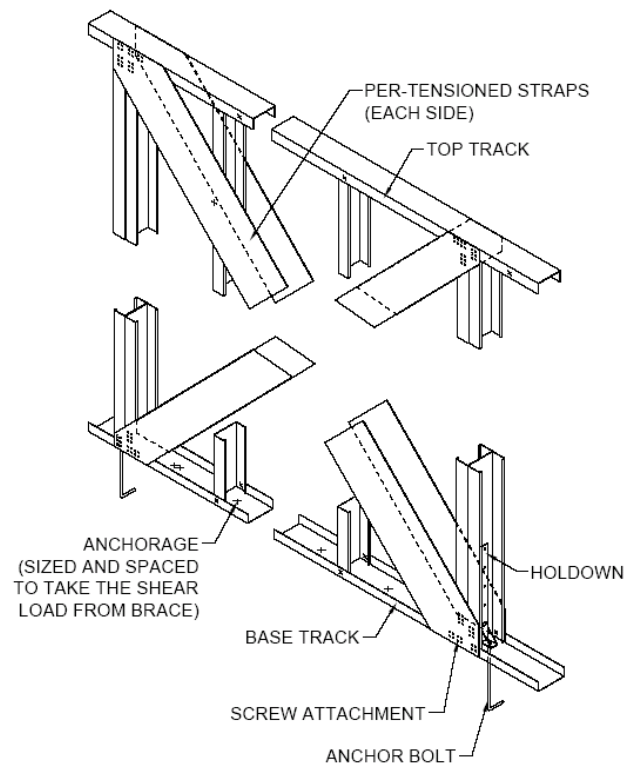


Figure 1-3 Shearwall X- Bracing (SSMA 2000)

1.2.1.2 Strength

Whereas stability bracing is designed to resist the effects of forces that develop internal to a structure, strength bracing exists to resist the effects of externally applied forces, such as lateral load effects due to wind and seismic events. Man-made causes of lateral loads include equipment impact and non-symmetric or eccentric loading.

Shearwalls in steel stud bearing walls are one example of strength bracing required to resist lateral loads due to wind or earthquake. Another example is metal building roof purlin bracing (Figure 1-4) which is designed to resist anchorage forces which develop in roof systems.

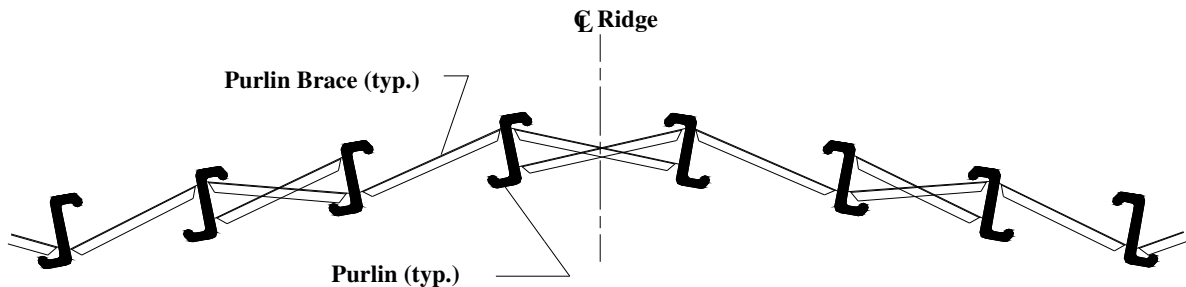


Figure 1-4 Purlin Bracing

1.2.1.3 Serviceability

Some bracing is not designed to resist specific internal forces or external loads. Bracing which exists primarily to control deflections or deformations may be referred to as serviceability bracing. Oftentimes, strap or diaphragm bracing is installed between cold-formed steel joists in floor systems to restrain potential member rolling under load (Figure 1-5). Similar bracing is often installed in metal building roof systems between purlins for the same reasons (Ellifritt et al. 1992).

In design, the typical assumption for instances similar to Figure 1-5 is that the compression flange is braced and the bending is about the geometric x-axis. If blocking is eliminated and rolling occurs, cross-axis bending begins to occur, which can significantly reduce both strength and bending stiffness of the floor system.

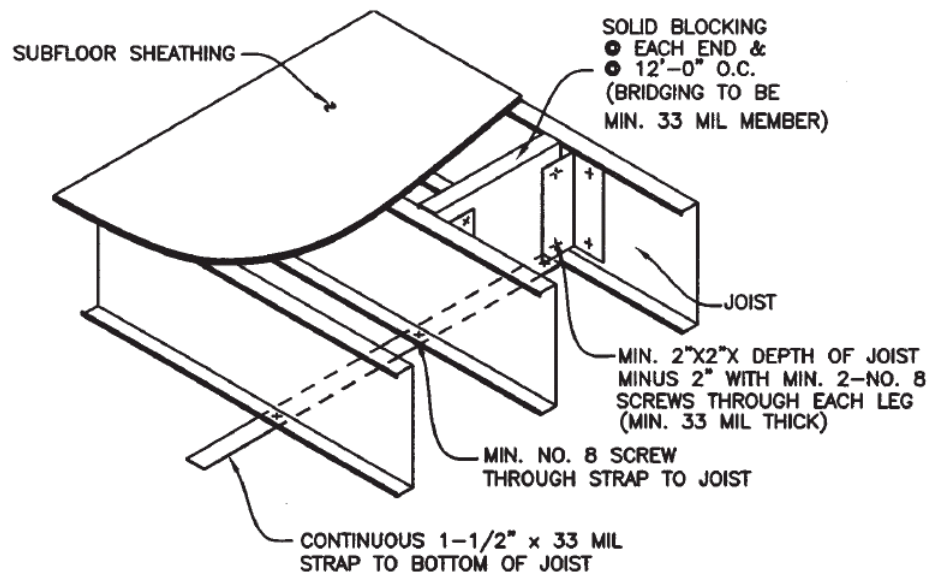


Figure 1-5 Floor Joist Bracing (NASFA 2000)

1.2.2 Stability Performance Criteria: Relative, Discrete, Continuous, and Lean-on

Yura classified bracing systems into four main categories: relative, discrete, continuous and lean-on (Galambos 1998).

A relative brace controls the movement of the adjacent stories or other brace points along the beam or column affected (Galambos 1998). A commonly occurring relative brace in cold-formed steel construction is an X-strapped shearwall in a cold-formed stud bearing wall structure.

A simple way to determine if a brace is a relative brace is to cut the braced member anywhere along its length, and the cut will pass through the brace itself (Figure 1-6).

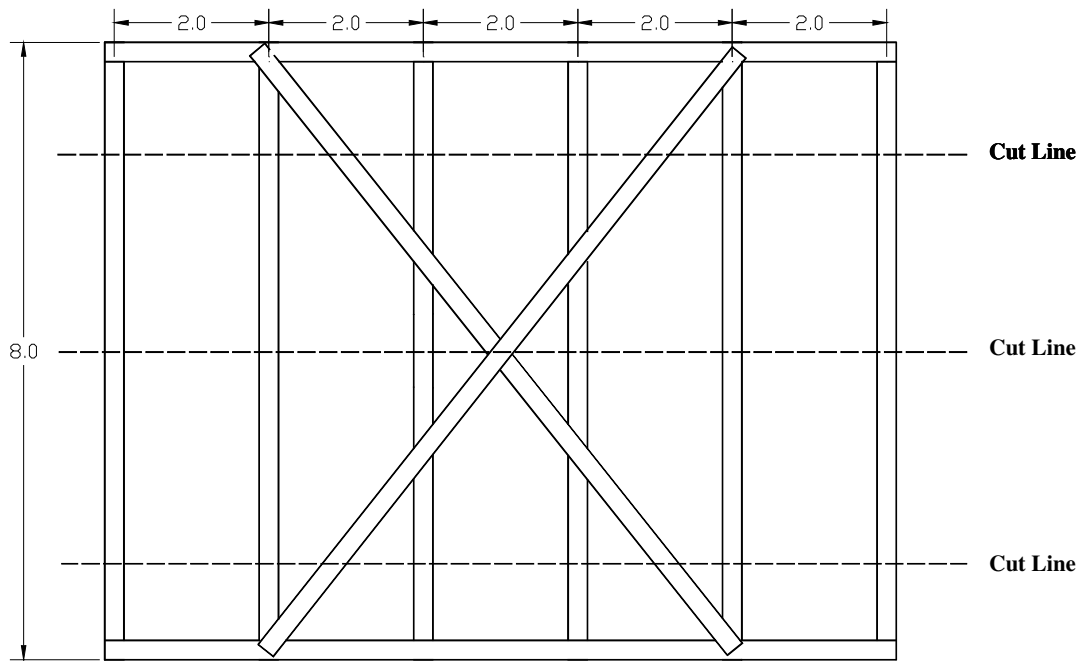


Figure 1-6 X Strapped Shearwall as a Relative Brace

A discrete brace can take a multitude of different forms, such as a metal building endwall girt connecting to a cold-formed endwall column, a cold-formed purlin connecting to a cold-formed endwall rafter, or a cold-rolled channel bracing a stud. All these types of bracing are discrete because the movement is controlled only at the particular brace point (Galambos 1998). The required bracing force and stiffness for this type of brace is dependent upon the number of braces provided.

A continuous brace is any brace that creates a column or beam with a theoretical unbraced length of zero about the braced axis (Galambos 1998). Examples include attaching siding to columns or placing decking on girders. The brace stiffness for this system is directly related to the buckling load. As the stiffness of the continuous brace increases, the load required to buckle the column or beam also increases.

A lean-on brace system occurs when a beam or column relies on adjacent connected structural members for support (Galambos 1998). Examples of lean-on

systems in cold-formed steel structures are columns in racks and strut-purlins in metal buildings. A strut-purlin relies on lean-on bracing from the adjacent, parallel, non-axial loaded purlins to resisting buckling at the restrained strap point of the purlins

An effective lean-on brace causes members in a frame to buckle in a no-sway mode, rather than a sway mode. When a member buckles in a no-sway mode (Figure 1-7) there is no lateral displacement of the frame. Once a column buckles, it loses all lateral stiffness; therefore one column of the frame must remain unbuckled allowing the buckled columns in the frame to lean on it. This system will remain stable as long as the sum of the loads is less than the sum of the critical buckling loads, P_{cr} applied to the frame (Yura 1971). Once P_{cr} is reached and all columns have buckled the frame is now considered to be in the sway mode of buckling. In a sway mode there is lateral displacement of the frame (Figure 1-8).

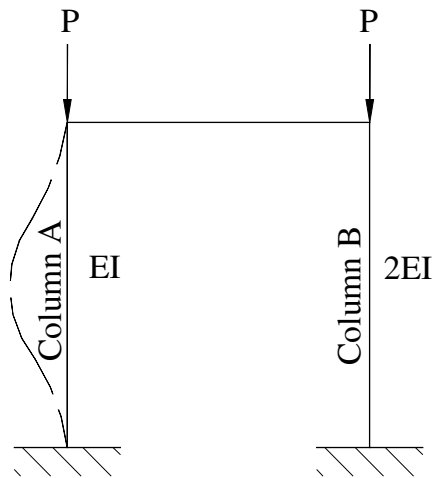


Figure 1-7 No Sway Buckling Mode

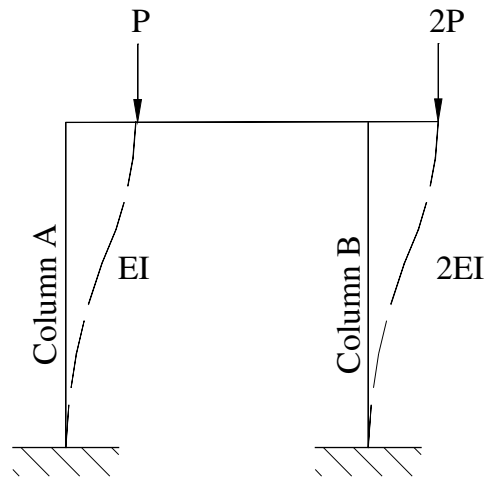


Figure 1-8 Sway Buckling Mode

The bracing system for the frame in Figure 1-8 has failed. The columns are no longer able to rely on other columns for support and the system has become unstable. In Figure 1-7, the load is equally applied to both columns. Column A has a stiffness of EI while column B has a stiffness of $2EI$. Column A has buckled and now is leaning on column B. As the load is increased on column B, it buckles, failing to act as a brace, not allowing column A to use it for support, causing the system to now buckle in a sway mode.

1.2.3 Method of Interaction: Relative and Nodal

In cold-formed steel structures, a brace or a system of braces are typically connected to multiple braced members within that structure. Depending on how the brace controls the motion of the braced point relative to the rest of the structure determines how the brace is categorized.

A relative brace controls the movement of the braced point with respect to adjacent braced points. The bracing connects several braced elements or members, with the relative motion of each member being identical. Relative and lean-on bracing systems are examples of relative braces.

A nodal brace controls the movement of the braced point without interaction with adjacent braced points. Continuous braces and discrete braces are examples of nodal braces.

1.3 Basic Bracing Models

The simplest method to analyze a column for buckling is to assume a perfectly straight, plumb and elastic member. Using these simplifications, the member will buckle, or bifurcate, at the Euler buckling load, P_{cre} .

$$P_{cre} = \frac{\pi^2 EI}{(KL)^2} \quad (\text{Eq. 1-1})$$

As long as the load does not exceed P_{cre} the member will remain stable and in equilibrium, without lateral displacement.

Models should incorporate member out-of-straightness and out-of-plumbness. All real structural members have a natural out-of-straightness due to the manufacturing process and an out-of-plumbness due to erection. Out-of-plumbness and out-of-straightness may be taken into account using a model described by Winter (1958) and Yura (1996). The initial out-of-straightness that a member possesses is Δ_i , with brace stiffness as β , and an axial load, P . The axial load that is applied to the imperfect member will create a moment causing the member to buckle at lower load, P_{cr} . This $P-\Delta$ behavior may be calculated using any one of many elastic second order analysis software packages.

In most design specifications, however, the design equation used to predict column instability and the critical load, is a modified Euler buckling equation applied to a perfectly straight member. The AISC specification (2010) takes member imperfections into account by modifying the Euler buckling equation to include factors that incorporate these effects. A severe limitation that must be considered when designing a column using the ideally straight and plumb model is that the affect of the brace strength and stiffness is negligible until the column bifurcates. For an actual column, if the brace stiffness is ignored, the brace has no effect until bifurcation occurs. The effects of the brace strength and stiffness will not be noticed unless a second order elastic analysis is performed. If a design is to be performed assuming a perfectly straight column the bracing for the column must be sufficient. A sufficient brace is recommended by Winter to be twice the ideal brace stiffness. The ideal brace stiffness is the Euler buckling load divided by the length, P_{cr}/L .

The effect of brace stiffness on elastic column buckling can be seen in the model of a column with an out-of-straightness of 1/600, as illustrated in Figure 1-9. The column and brace was modeled in MASTAN2, Version 2.0, (MASTAN2 2002) using elastic beam elements. The brace stiffness was varied by changing the cross sectional area appropriately. A second order elastic buckling analysis was performed with these varied brace stiffnesses. The results of the analysis are provided in Table 1-1.

$A = 10 \text{ in}^2$
 $I = 5.0311 \text{ in}^4$
 $L = 120 \text{ in}$
 $E = 29000 \text{ ksi}$

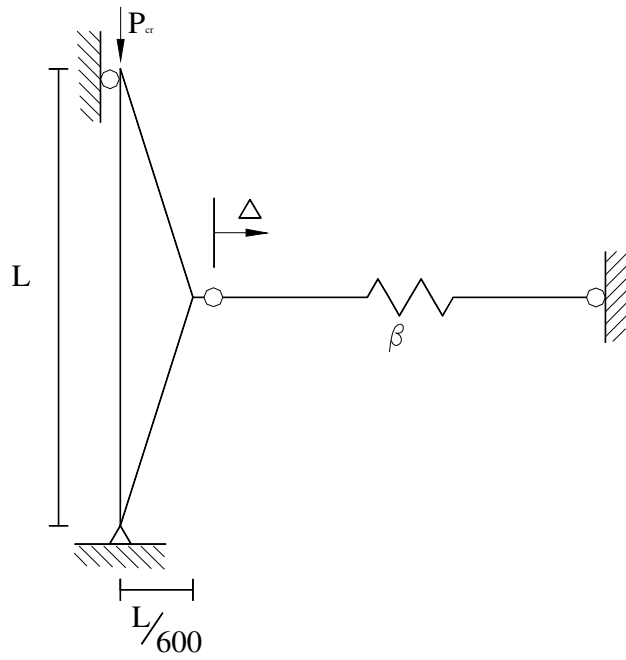


Figure 1-9 Imperfect Column Model

Table 1-1 Imperfect Columns

Beta (k/in.)	P _{cr} (k)	P brace (k)	Defl (in.)	Note
2900	400.08	1.331	0	
290	400.08	1.326	0	
26.68	400.08	1.157	0	$2 \times \beta_t$
13.92	400.08	1.032	0	
13.63	400.08	1.028	0	β_t
13.34	399.95	1.022	0.2236	
5.8	237.2	0.460	0.31	
2.9	169.6	0.230	0.37	
0.29	100.7	0.0023	0.49	

As seen in Figure 1-10, as the brace stiffness increases from zero to β_i , the buckling capacity increases from that of a column of length L to that of a column of length $L/2$. Increasing the stiffness of the brace beyond β_i has no further effect on the buckling capacity of the column.

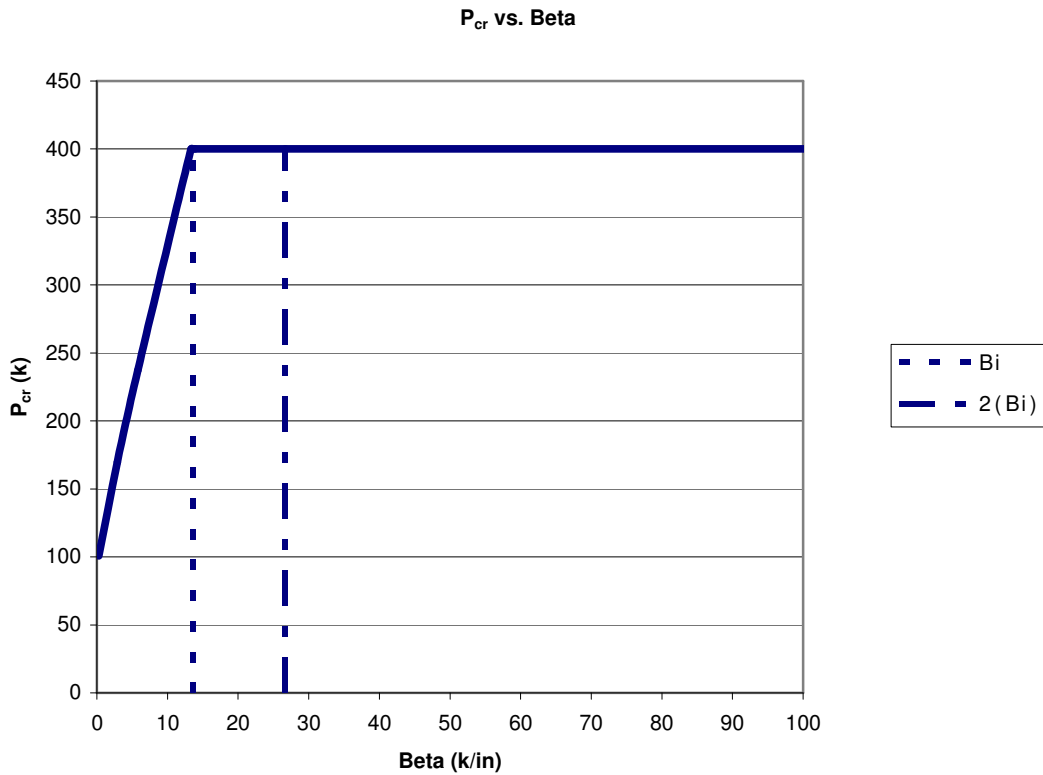


Figure 1-10 P_{cr} vs Beta

Figure 1-11 illustrates the relationship between brace stiffness and lateral deflection of the brace point before bifurcation occurs. Again, as the brace stiffness increases, the lateral deformation decreases until the brace stiffness reaches that of β_i .

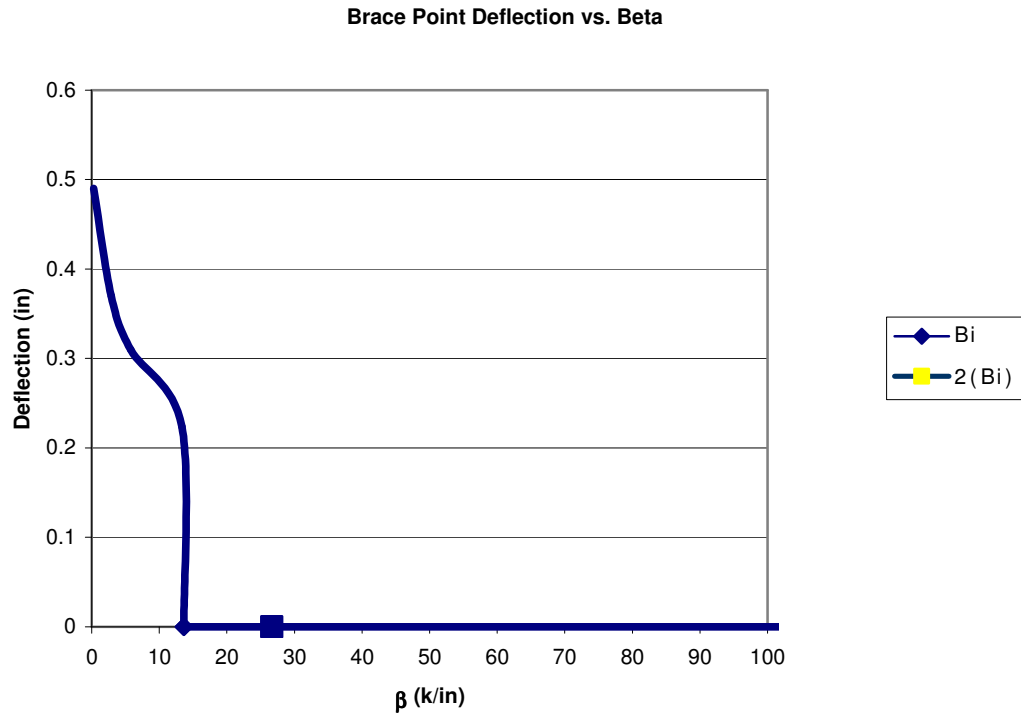


Figure 1-11 Brace Point Deflection vs. Beta

Figure 1-12 is an illustration of the effect of brace stiffness on brace force. In this instance, at levels of brace stiffness less than β_i , the brace force at bifurcation is linearly related to the brace stiffness, and reaches approximately 80% of the maximum brace force at a brace stiffness of β_i . Increasing the brace stiffness to $2\beta_i$ increases the brace force by approximately 20%. For further discussion on this topic, see Galambos (1998).

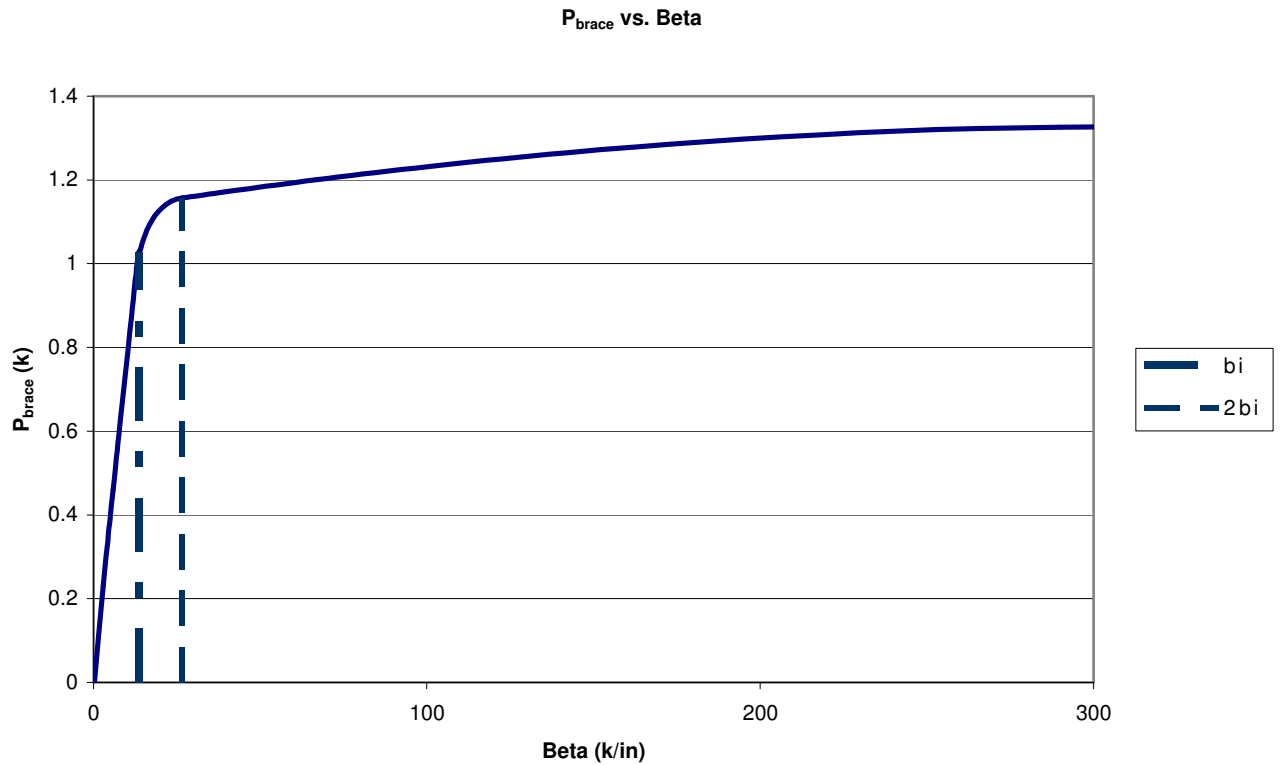


Figure 1-12 P_{brace} vs β

To create an adequate brace system, a combination of strength and stiffness must be provided. It has been proven through tests that an exact calculation of strength and stiffness is not necessary (Winter 1958). Historically, a recommended brace strength of 2% of the braced member compressive force has been used in design. As a practical matter, the “2% Rule” for strength typically lead to bracing with adequate stiffness.

A brace may be physically modeled as a system of springs, either in series or in parallel, depending on the configuration of the brace. The stiffness of a brace must consider not only that of the brace member itself, but also the connection to the braced member. For example, the axial stiffness of a tension brace member is the cross-sectional area times the elastic modulus divided by the brace length.

$$\beta = \frac{AE}{L} \quad (\text{Eq. 1-2})$$

The connections at each end of the physical brace have associated stiffnesses that must be considered, including fasteners and the connected materials. The fasteners and connected materials must be analyzed for yield, hole deformation, shear, and other considerations. The results calculated for the stiffness of the connection and the brace should be used in the following equation to calculate the overall stiffness of the system.

$$\frac{1}{\beta_{\text{system}}} = \frac{1}{\beta_{\text{brace}}} + \sum \frac{1}{\beta_{\text{connection } i}} \quad (\text{Eq. 1-3})$$

A system of braces in parallel occurs when two or more braces are placed parallel to each other and their stiffnesses are summed.

Most braces in parallel are actually a number of braces in series. Bracing that is installed in parallel still must be connected to the braced member. This connection stiffness makes each individual brace a series brace and the grouping of braces parallel braces.

$$\beta_{\text{system}} = \sum \beta_i \quad (\text{Eq. 1-4})$$

To this point, only flexural buckling has been considered, but in thin walled open cold-formed steel sections, torsional and combined torsional-flexural buckling must be considered. Therefore, stability braces are needed to resist not only flexural buckling, but also torsional effects. For a column this brace is required to resist twisting and torsional buckling, as well as lateral displacement and flexural buckling. An example of potential torsional buckling in a cold-formed steel structure is when siding is placed on only one side of an endwall column. The siding provides lateral support against flexural buckling because of the fasteners used to attach the siding to the column. When fasteners are only attached to one flange of the column, torsional buckling may often occur and bracing to resist that effect must be provided.

1.4 Provisions for Bracing Hot-Rolled Steel Structures

With the issuance of the Third Edition of the AISC-LRFD specification (AISC 1999), structural steel designers in North America first had codified guidance available for the design of bracing elements for their structures. Equivalent ASD guidance is available from several sources, including Galambos (1998) and Yura (1993, 1995).

These AISC provisions are based primarily on Winter (1958) as modified by Yura. The derivations of these provisions are beyond the scope of this monograph, but have been well documented and are readily available in several sources, particularly Galambos (1998), and the AISC commentary (AISC 2010).

1.5 Differences Between Hot-Rolled and Cold-Formed Steel Bracing Design

The engineer who is conversant with design of hot-rolled steel structures and their bracing using the AISC (2010) and AISC-ASD (1989) specifications will find both similarities and differences when designing bracing for cold-formed steel elements and structural systems.

1.5.1 Global Stability

Hot-rolled structures typically consist of doubly symmetric members (wide flange or hollow sections) with locally stable (compact) plate elements. When considering stability, the prevalent consideration in hot-rolled steel structures is global stability, be it individual member (column flexural buckling or beam lateral-torsional buckling) or overall structural stability.

When designing in cold-formed steel, the hot-rolled norm is the exception. Singly-symmetric (cee) and point-symmetric (zee) sections are the most commonly used

column and beam cross-sections. In most cases, efficiently loading these cross-sections through their centroids (for columns) or shear centers (for beams) is nearly impossible. Therefore, unavoidable secondary torsion and flexure develop, which must be considered in bracing design. In fact, bracing is often designed and installed to prevent these secondary forces from developing.

Additionally, the stability limit states of torsional and torsional-flexural buckling which are uncommon in hot-rolled wide-flange columns are common limit states for cold-formed cee and zee columns, with attendant torsional bracing considerations.

Long-wave buckling for columns (flexural, torsional, and torsional-flexural), and beams (lateral-torsional and torsional) can be effectively restrained through the use of properly proportioned and detailed bracing.

1.5.2 Local Stability

With the exception of built-up metal building frame sections, most structural steel designed using the AISC specifications (2007) consists of locally stable (or non-slender) cross sections, where the effects of local buckling are not a design consideration. In cold-formed steel, the locally stable cross-section (consisting of non-slender plate elements) is the exception, not the rule.

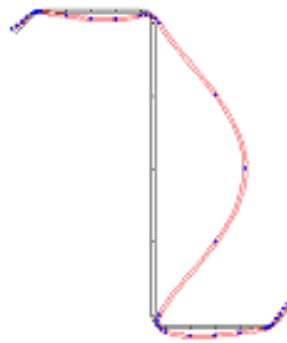


Figure 1-13 Local Buckling in Compression

Local buckling of the plate elements that make up cold-formed cross-sections, as shown in Figure 1-13, cannot be controlled by the addition of bracing, except in the most limited circumstances. However, once cold-formed sections undergo local buckling, they exhibit post-buckling strength increases, therefore elimination of local buckling through the use of braces is not desirable.

1.5.3 Distortional Buckling

The limit state of distortional buckling is not often seen in hot-rolled steel sections, and is usually particular to cold-formed cross-sections. Whereas local buckling is limited to the deformation of individual plate elements, distortional buckling involves the distortion of the cold-formed cross-section shape, as shown in Figure 1-14.

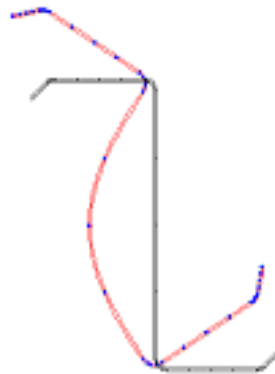


Figure 1-14 Distortional Buckling in Compression

The limit state of distortional buckling cannot typically be controlled through the use of bracing, however some bracing installations can actually precipitate distortional buckling.

1.5.4 Brace and Joint Considerations

The brace system used in a typical hot-rolled steel structure is required to resist higher brace forces and possesses higher stiffness than most braces in cold-formed steel structures. This occurs due to the fact that most cold-formed steel components carry lower loads, and therefore place lower demands on the required bracing systems. Even though the demands are lower, the design of efficient braces in cold-formed steel structures requires attention to detail primarily due to differences in the effectiveness of the connection details.

Braces in hot-rolled steel structures are often connected using either welds or fully-pretensioned bolts in slip-critical connections, thus limiting joint deformation and increasing stiffness. A typical cold-formed steel brace is constructed using either screws or snug-tight bolts. Joint flexibilities thus exist due to screw tilting and bolt slip under load in typical cold-formed steel bracing systems.

An additional source of joint flexibility comes from the relative thinness of the sheet steel. Both elastic and inelastic deformations occur due to hole deformation caused by fastener (bolt or screw) bearing against the hole. Added to this is the fact that in certain widely-used details, both elastic and inelastic deformations of the connected parts can occur due to bending of the material.

Since most cold-formed braces are of a series nature, the addition of these inherent flexibilities must be considered in evaluating brace stiffness per Equation 1-3.

1.5.5 Systematic Effects

Cold-formed structural systems often consist of multiple elements installed in close proximity to each other (wall studs, metal building purlins and girts) where the installed bracing must be designed considering both the individual member and the entire system.

By its nature, the roll-forming process will reproduce defects in multiple members that may be installed consecutively in a wall or roof assembly. When installed, the defects (sweep, camber, or bow) will most likely be installed in the same direction, creating a systematic bias towards buckling in one direction. Since these brace forces are not “canceled out” between members, the bracing forces necessarily accumulate, as shown in Figure 1-15. Although these forces are known to accumulate, the proper number of individual members to consider for the accumulation of force is still the subject of ongoing research. The only currently available design guidance in this area pertains to metal building roof purlins braced by steel roof deck, where the brace force is considered to accumulate over a maximum of 20 purlin lines. (Murray and Elhouar 1985, AISI 2007)

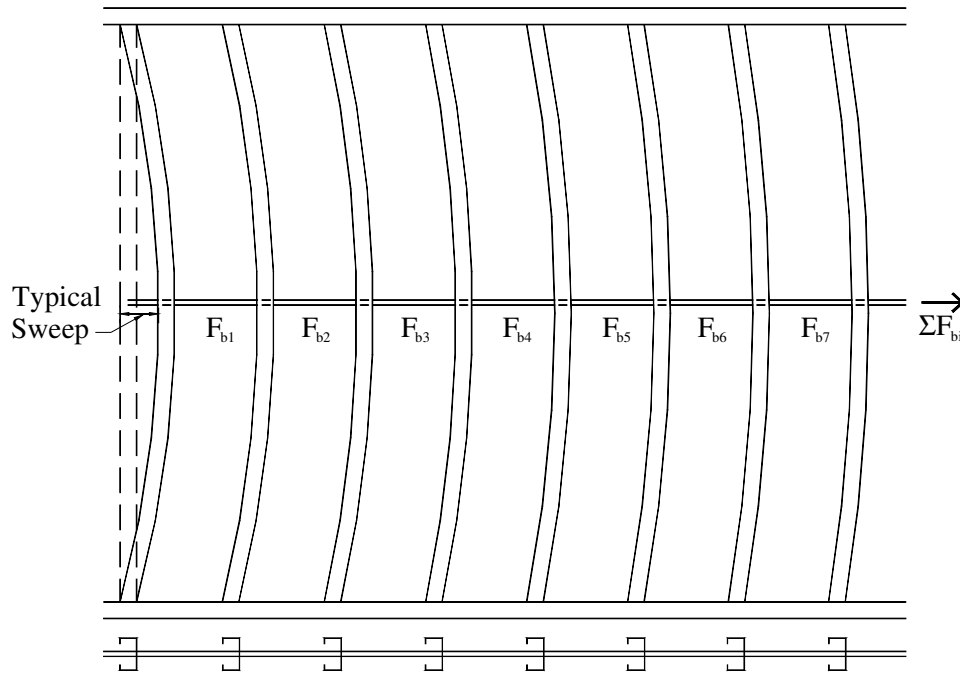


Figure 1-15 Systemic Sweep in Wall Studs

1.5.6 Bracing of Unsheathed Beams and Columns

When neither flange of a beam or column is attached to sheathing, discrete braces may be designed to increase buckling strength. Cold formed C and Z cross sections, respectively monosymmetric and point symmetric, respond differently to load than doubly symmetric hot-rolled wide flange sections. Unless restrained, these sections have a pronounced tendency to roll when loaded, resulting in secondary stresses that can be substantial.

1.5.6.1 Unsheathed Columns

The main body of the *AISI Specification (2007)* provides specific guidance on requirements for bracing compression members.

$$P_{br} = 0.01 P_n \quad (\text{Eq. 1-5})$$

$$\beta_{req} = \frac{2(4 - 2/n)P_n}{L_b} \quad (\text{Eq. 1-6})$$

Where: P_n = Column nominal axial strength considering braces to be effective

P_{br} = Required ASD brace strength

β_{req} = Required brace stiffness

n = Number of approximately equally spaced braces

L_b = Spacing between braces

1.5.6.2 Unsheathed Beams

An unsheathed C or Z flexural member will tend to twist under load, due to the fact that the load is typically not applied through the shear center of the member (for C

sections) or because the principal axes of the section are inclined to the line of action of the load (for Z sections).

When loaded through the web, C-sections will rotate about the shear center of the section, as shown in Figure 1-16. Figure 1-17 shows how the rotation of the section may be restrained by lateral braces at the flanges, with the resulting brace forces.

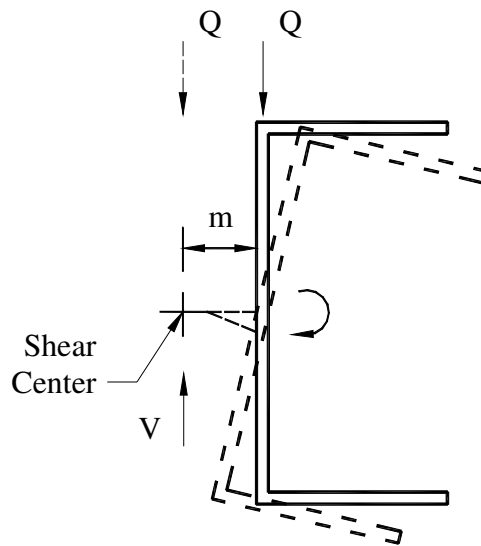


Figure 1-16 Rotation of C Section

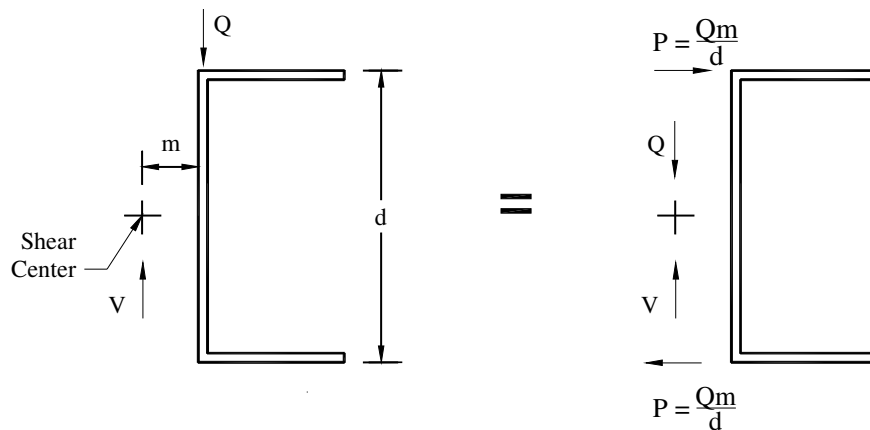


Figure 1-17 Resulting Brace Forces for C Section

For a Z section loaded through the web, the shear center and centroid are coincident, and the line of action of the load passes through this point. However, the principal axes are oblique to the web, and the applied load, resolved in the direction of the two axes produces deflections in both directions, which then may be resolved into both vertical and horizontal deflections. Through the use of mechanics, the resulting brace forces can be calculated as indicated in Figure 1-18. It should be noted that the brace forces for a Z section act in the same direction, whereas the brace forces for a C-section act in opposing directions.

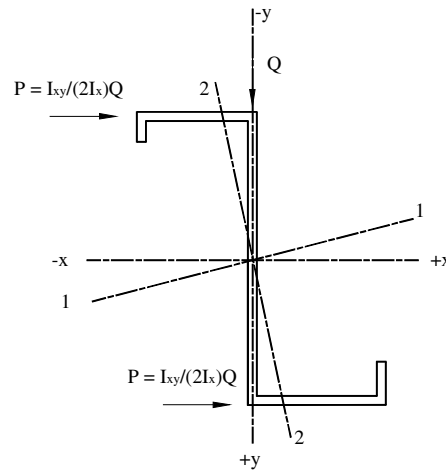


Figure 1-18 Principal Axes and Resulting Brace Forces for Z Section

Unbraced C and Z sections will develop severe rotational distortion at service load levels if the spacing interval between braces is excessive. The Canadian appendix to the *AISI Specification* (AISI 2007) requires that braces be limited to a maximum spacing of one-quarter of the beam span, to reduce distortion and resulting torsional stresses. This requirement was previously removed from the Specification for United States use, but may be used as an indication of prudent practice to limit distortion, deflection, and secondary stresses.

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QUIZ

Introduction to Bracing Design For Cold-Formed Steel Structures

1. A _____ controls the movement of the adjacent stories or other brace points along the beam or column affected.
 - a. continuous brace
 - b. discrete brace
 - c. relative brace
 - d. lean-on brace

2. A _____ controls movement only at the particular brace point.
 - a. continuous brace
 - b. discrete brace
 - c. relative brace
 - d. lean-on brace

3. A _____ is any brace that creates a column or beam with a theoretical unbraced length of zero about the braced axis.
 - a. continuous brace
 - b. discrete brace
 - c. relative brace
 - d. lean-on brace

4. A _____ occurs when a beam or column relies on adjacent connected structural members for support
 - a. continuous brace
 - b. discrete brace
 - c. relative brace
 - d. lean-on brace

5. Bracing models should incorporate out-of-straightness and _____.
 - a. out-of-plumbness
 - b. residual stresses
 - c. stiffness
 - d. none of the above

6. For a column, increasing the stiffness of the brace beyond the ideal brace stiffness _____ buckling capacity of the column

- a. has no further effect on the
- b. increases the
- c. decreases the
- d. may or may not effect the

7. For a column, as the brace stiffness increases, the lateral deformation _____ until the brace stiffness reaches that of the ideal brace stiffness.

- a. remains constant
- b. increases
- c. decreases
- d. is not changed

8. Once cold-formed sections undergo local buckling, _____ .

- a. they decrease in strength gradually
- b. they exhibit post-buckling strength increases
- c. no change in strength is exhibited
- d. none of the above

9. The limit state of distortional buckling _____ be controlled through the use of bracing

- a. can
- b. cannot
- c. may occasionally
- d. may or may not

10. An unsheathed C or Z flexural member will tend to twist under load, due to the fact that the load is typically not applied through the _____ of the member

- a. centroid
- b. shear center
- c. principal axis
- d. minor axis