Bracing Cold-Formed Steel Structures
A Design Guide

ASCE

Thomas Sputo
Jennifer L. Turner

SEI
Structural Engineering Institute

BRACING COLD-FORMED STEEL STRUCTURES
A DESIGN GUIDE

SPONSORED BY

Committee on Cold-Formed Steel and the Committee on Metals
of The Structural Engineering Institute
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EDITED BY
Thomas Sputo, Ph.D., P.E
Jennifer L. Turner

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Preface

For many practicing structural engineers, the design of structures using cold-formed steel is seen as a daunting task. The goal of this report is to remove some of the perceived mystery by providing readily useful information for bracing cold-formed steel structures. This report is not written as a highly technical research report, it is instead presented as a design guide, written by practicing structural engineers for other practicing structural engineers. Only enough theory is presented to allow the reader to understand the basis for the methods presented. The emphasis is placed on understanding the function and design of the bracing itself.

Bracing is one key to good structural design. Whether we as engineers acknowledge the fact or not, the underlying basis of steel design, for both hot-rolled and cold-formed steel structures, is the control of instability. Although little can be done to control local instabilities in a cross section, global instability of a member or an entire structure can be controlled by the proper design and implementation of adequate bracing. While structural engineers have long understood the need to provide adequate bracing, the 3rd edition of the American Institute of Steel Construction *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC 1999) only recently included design requirements for the bracing itself. The applicable specification for cold-formed steel, the American Iron and Steel Institute *North American Specification for the Design of Cold-Formed Steel Structural Members, 2001 Edition with Supplement 2004* (AISI 2004c), however, provides little in the way of general provisions.

This report documents in a concise manner the current practices related to bracing cold-formed steel structural elements and systems. Currently this information is fragmented and sometimes not readily accessible to the practicing engineer. This report provides summaries of cold-formed steel bracing practices and design examples with references to sources of additional information. This will enable the practicing engineer to readily access this information, resulting in cold-formed steel structures with greater levels of reliability, safety, and economy.

Fortunately, over the past decade, professional and industry organizations, including the Light Gauge Steel Engineers Association (LGSEA), the Steel Stud Manufacturers Association (SSMA), and the American Iron and Steel Institute (AISI) among others, have made available design publications and computer software that make design less onerous for the typical practicing engineer in consulting practice. This report makes ready reference to many of these readily available sources of information, and the reader is encouraged to study these primary sources of information in greater depth.

The authors of this report gratefully acknowledge the assistance of the members of the ASCE-SEI Committee on Cold-Formed Steel in preparing this report. Many other individuals also contributed ideas that are included in this report. Their help was likewise invaluable. Several organizations, including the American Iron and Steel Institute, the Steel Framing Alliance, the Steel Stud Manufacturers Association, and the Light Gauge Steel Engineers Association likewise contributed their expertise. The cooperation of the Department of Civil and Coastal Engineering at the University of Florida, where the first author is a part-time lecturer and the second author was a graduate student while this report was being written, is also acknowledged.

The Technical Activities Committee of the Structural Engineering Institute of ASCE provided funding for this work through a special project grant.

The authors hope that this report will contribute in some small way to the furtherance of the structural engineering professions goal of creating safer, more economical structures.

Thomas Sputo, Ph.D., P.E., M.ASCE
Sputo Engineering

Jennifer L. Turner, SM.ASCE
Sputo Engineering

Gainesville, Florida, USA
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Contents

Appendices

Chapter 1. Introduction to Bracing Design

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 2004c) contains design requirements for several specific cold-formed steel assemblies, as have previous American Iron and Steel Institute Specifications (AISI 1986, 1991, 1996), and Canadian Standards Association S136 Specification (CSA 1994). Whereas the 1999 AISC-LRFD specification currently provides general guidance which 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 monograph seeks to look at current North American practice related to the design of bracing for cold-formed steel elements and structures; to report on the methods of implementation of existing specification requirements; and to report on how current practice has filled the void where specific specification guidance is lacking.

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.

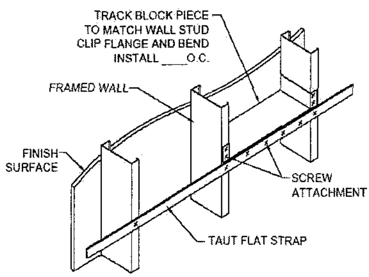


Figure 1-1 Single Flat Strap with Bracing (SSMA 2000a)

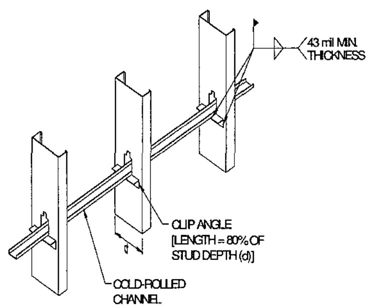


Figure 1-2 Cold-Rolled Channel with Clip Angle (SSMA 2000a)

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.

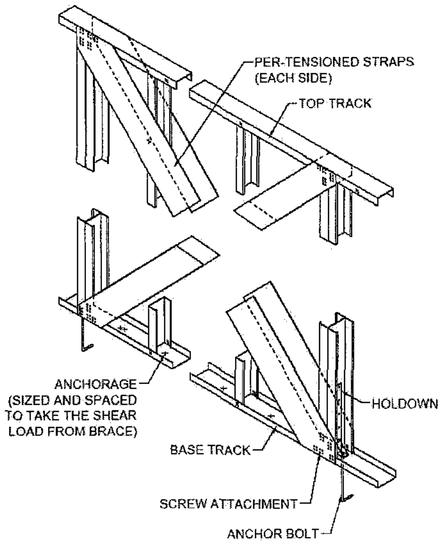


Figure 1-3 Shearwall X- Bracing (SSMA 2000a)

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. Both examples will be discussed further in Chapters 2 and 3.

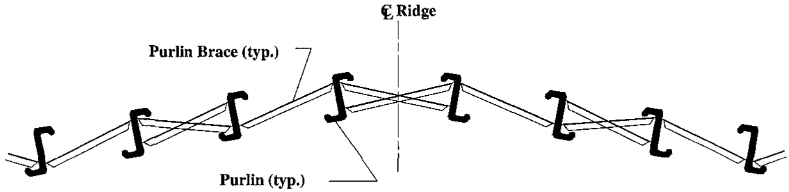


Figure 1-4 Purlin Bracing

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.

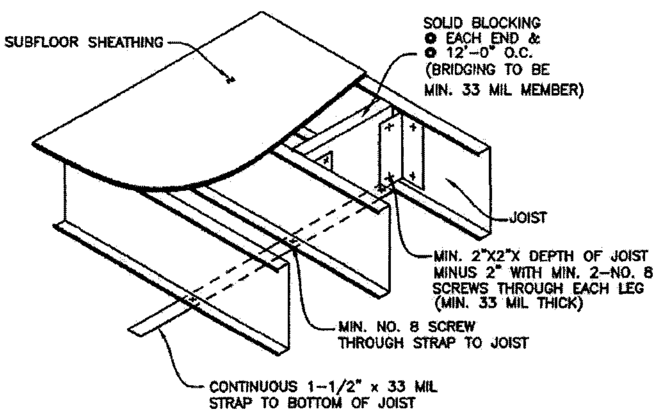


Figure 1-5 Floor Joist Bracing (NASFA 2000)

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).

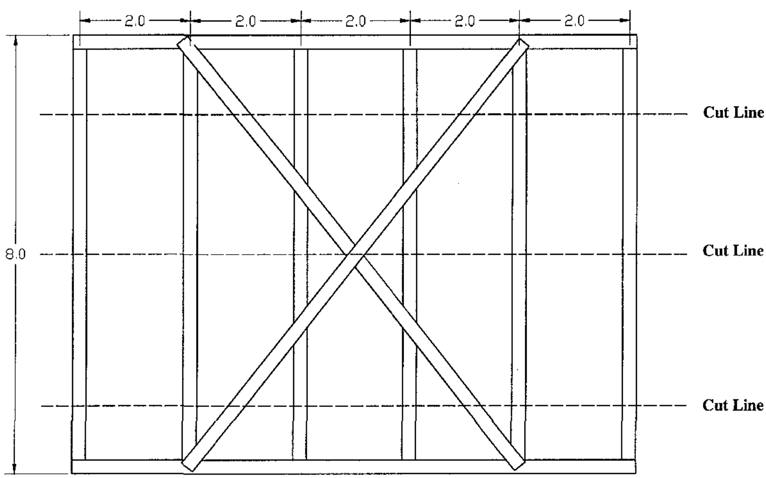


Figure 1-6 X Strapped Shearwall as a Relative Brace

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, which will be described in more detail in Chapter 3, 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).

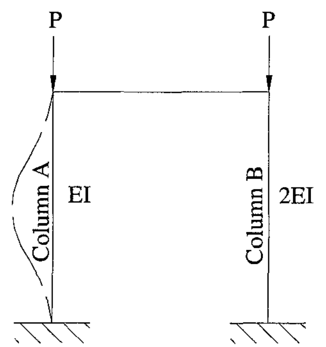


Figure 1-7 No Sway Buckling Mode

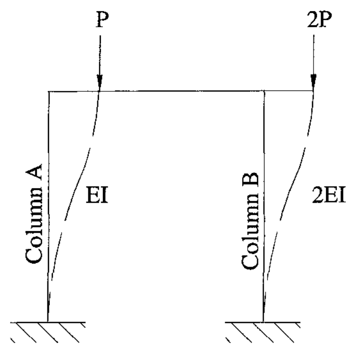


Figure 1-8 Sway Buckling Mode

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}.

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

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) (See Appendix C for Winter's paper). The initial out-of-straightness that a member possesses is delta, Δ_i, with brace stiffness as beta, 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-Δ 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-LRFD specification (1999) 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_{cre}/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.

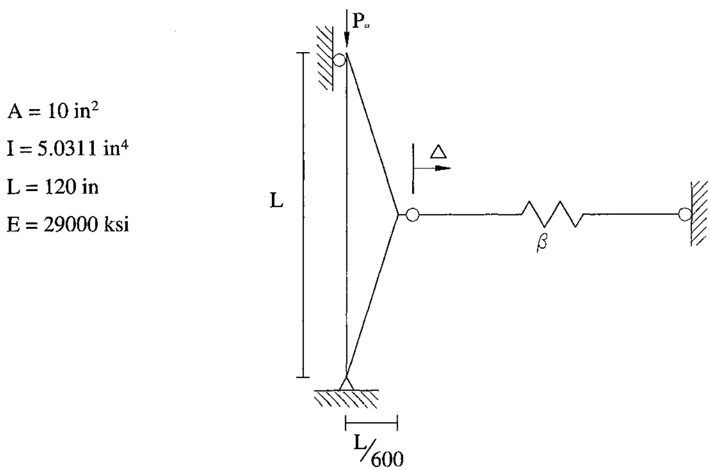


Figure 1-9 Imperfect Column Model

Table 1-1 Imperfect Columns

Beta (k/in.)	Pcr (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 x β_i
13.92	400.08	1.032	0	
13.63	400.08	1.028	0	β_i
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.

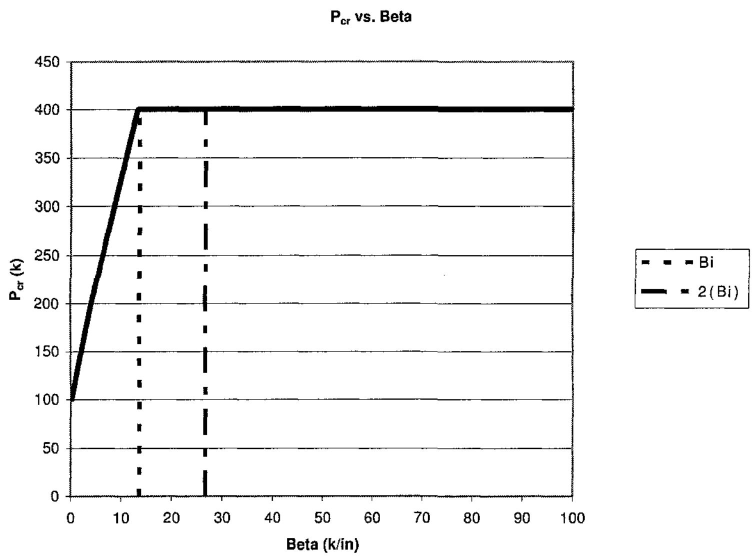


Figure 1-10 P_{cr} vs Beta

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.

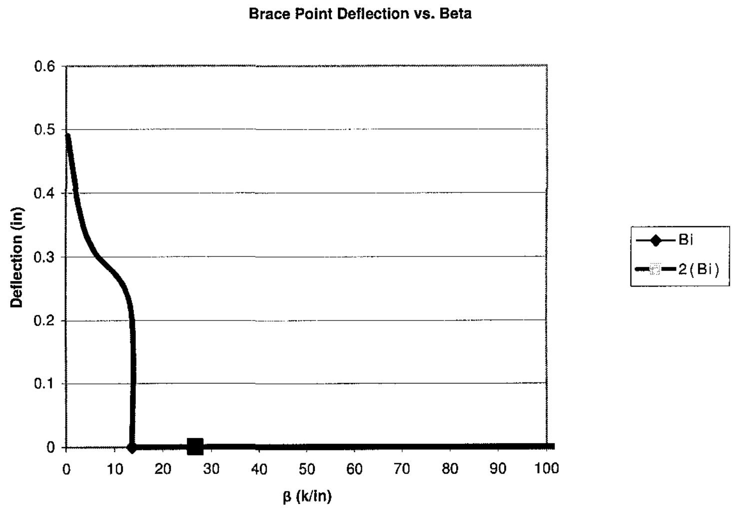


Figure 1-11 Brace Point Deflection vs. Beta

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).

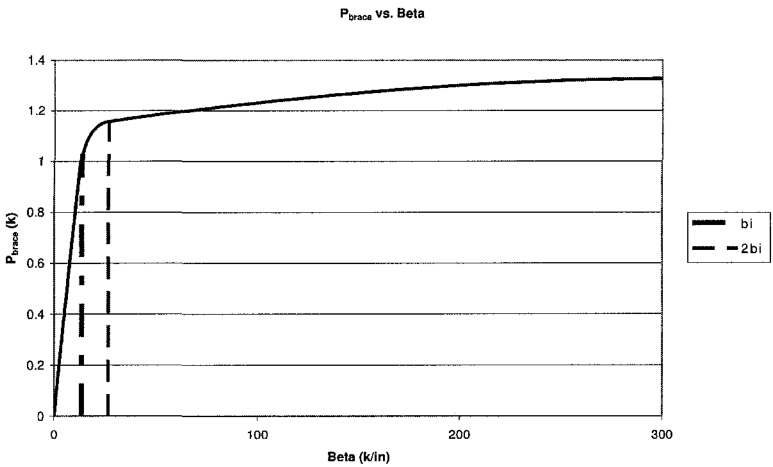


Figure 1-12 P_{brace} vs β

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.

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

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.

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

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.

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

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 have 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-LRFD commentary (AISC 1999).

In 2005, AISC will issue the first combined LRFD / ASD specification which includes provisions for bracing in LRFD and ASD designs.

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-LRFD (1999) 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 bucking) 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 (1989, 1999) 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.

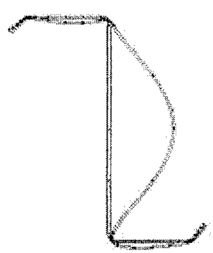


Figure 1-13 Local Buckling in Compression

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.

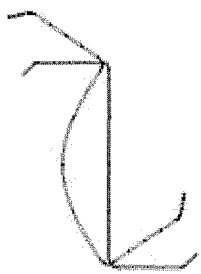


Figure 1-14 Distortional Buckling in Compression

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 2004c)

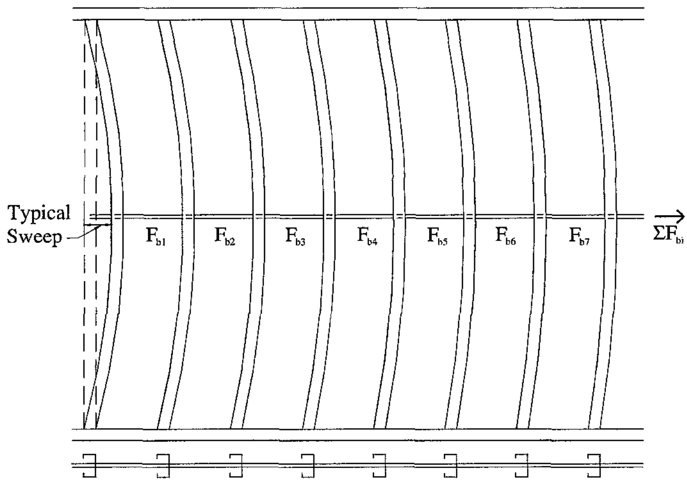


Figure 1-15 Systemic Sweep in Wall Studs

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 monosymetric 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* (2004c) provides no specific guidance on requirements for bracing compression members. The Canadian appendix requires that column braces be designed to resist 2% of the compression force in the column, without a stiffness limitation, and without reference to cross sectional properties.

The "2 percent" rule appears to be based on long standing experience without experimental or analytical verification or basis. Research conducted at the University of Florida (Green, Sputo, and Urala 2004) indicates that the following criteria is acceptable for the design of axially loaded C sections using ASD design criteria.

$P_{br} = 0.01\ P_n$ (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.

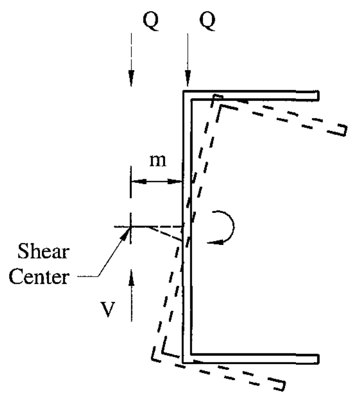


Figure 1-16 Rotation of C Section

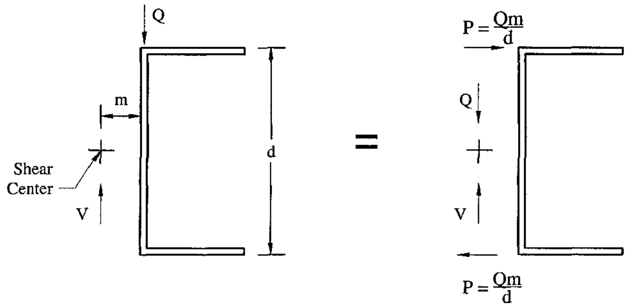


Figure 1-17 Resulting Brace Forces for C Section

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.

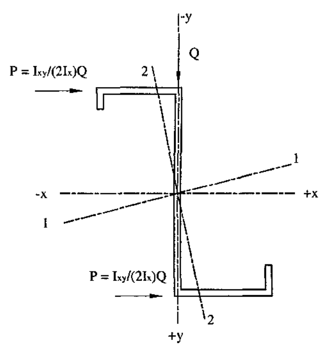


Figure 1-18 Principal Axes and Resulting Brace Forces for Z Section

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 2004c) 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.

1.6 Thickness of Steel Components

Historic practice has been to designate steel thickness by reference gauge. This practice, however, has lead to ambiguity as to the design thickness, due to varied reference gauge schedules. The preferred practice is to designate thickness either in inches or in mils (0.001 inches). The AISI *Specification* allows fabricated components to have a minimum thickness up to 5 percent less than the thickness considered in design.

The metal building industry typically designates cold-formed steel components by the design thickness in inches, while the cold-formed steel framing industry designates thickness as a minimum thickness in mils. Table 1-2 lists minimum thicknesses, design thicknesses, and traditional gauges for reference purposes.

Table 1-2 Thickness – Steel Components

Minimum Thickness[1] (mils)	Design Thickness (in)	Reference Gauge
18	0.0188	25
27	0.0283	22
30	0.0312	20 – Drywall
33	0.0346	20 – Structural
43	0.0451	18
54	0.0566	16
68	0.0713	14
97	0.1017	12

[1] Minimum Thickness represents 95% of the design thickness and is the minimum acceptable thickness delivered to the job site based on Section A3.4 of the 2004 AISI Specification.

Chapter 2. Cold-Formed Framing

2.1 Introduction

Cold-formed steel members and assemblies utilized primarily in the construction of load bearing walls, curtain walls, and trusses are typically referred to as "cold-formed framing." The design of these members is generally governed by the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI 2004c). Although the need for bracing is envisioned by this document, except for limited situations this document is silent as to the actual design provisions for bracing. To provide guidance to designers, industry and professional associations have developed documents that are indicative of acceptable practice.

As noted in Chapter 1, initial imperfections in structural members have an influence on the magnitudes of the required brace strength and stiffness to adequately brace a member. ASTM standards (ASTM 2000a, ASTM 2000b) allow for the imperfection limits for steel studs and track, as noted in Table 2-1.

Table 2-1. Cold-Formed Framing Imperfection Limits

Imperfection	Definition	Limits
Camber	Strong axis out-of-straightness	1/32" per foot (1/2" maximum)
Bow	Weak axis out-of-straightness	1/32" per foot (1/2" maximum)
Crown	Web out-of-flatness	1/16"

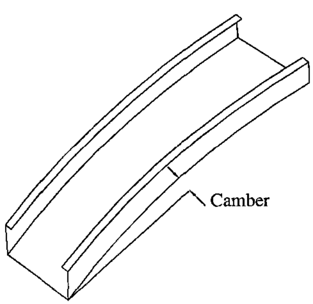


Figure 2-1a Manufacturing Camber Tolerances

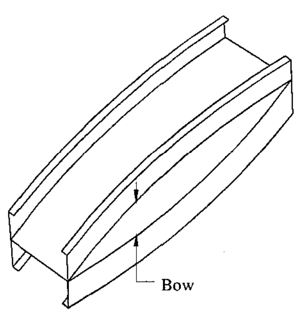


Figure 2-1b Manufacturing Bow Tolerances

These out-of-straightness limits are equivalent to an imperfection of L/384. Equivalent limits for hot-rolled steel members are L/1500. These less stringent limits are the result of the inherent imperfections induced in these thin walled members during the roll forming process.

Most cold-formed framing members used in North American practice are some form of an un-lipped or lipped cee section. Within the past ten years however, several proprietary truss systems have emerged which use tubes and various hat shaped sections for chord and web members.

Two methodologies exist for the design of braced cold-formed framing: sheathed design and "all-steel" design. Sheathed design relies on the applied sheathing (gypsum wall board, plywood, etc.) to provide bracing to compression elements, while the all-steel approach neglects the structural contribution of the attached sheathing and instead utilizes steel bracing members to provide the necessary bracing action. While the aims of both methods of design are similar (to provide bracing of adequate strength and stiffness to develop the design load in the braced member), the methods of providing that bracing are different.

2.2 Sheathing Braced Designs

Sheathing braced designs rely on the attachment of the applied sheathing to brace the steel members through diaphragm action. Traditionally, provisions for sheathing braced designs have been found in the AISI specification (AISI 2004c). Recently however, these provisions have been relocated to alternate documents pertaining solely to cold-formed framing design (AISI 2004a).

Sheathing braced designs have the advantage of providing nearly continuous support to the studs, while providing for a direct load transfer path for the developed bracing forces into the top and bottom tracks, and therefore into the primary structure. And since the sheathing is usually required for other purposes, it is available for bracing purposes at little or no additional cost.

2.2.1 Wall Studs in Compression

The *Standard for Cold-Formed Steel Framing – Wall Stud Design* (AISI 2004a) provides guidance for the design of wall studs in compression using sheathing as bracing. This standard requires that the design axial strength of the stud be calculated in accordance with the AISI Specification (AISI 2004c). Furthermore, the *Standard* requires that each face of the wall shall be braced with identical sheathing connected to both bottom and top wall tracks. If different sheathing is used on opposing faces of the wall, a design may be performed assuming that both faces of the wall are sheathed using the weaker of the two materials, or a rational analysis may be performed. Due to additional torsional demand being placed on the sheathing, studs with single sided sheathing must be designed using all-steel principles.

Sheathing braced design in the *Standard* is based on rational analysis assuming that the sheathing braces the stud at the location of the sheathing fastener (AISI 2004a). Therefore, the unbraced length with regard to the major axis, L_x, is equal to the overall length of the member, and the unbraced length with regard to the minor axis, L_y, and with regard to torsion, L_t, is taken to be twice the distance between sheathing fasteners. This requirement takes into consideration the possibility that an occasional screw attachment is completely inoperative.

Using bracing principles developed by Winter (1958), the brace force is defined as:

$$F_{br} = K(\Delta + \Delta_0) = 0.02P \qquad \text{(Eq. 2-1)}$$

Where : $\Delta = \Delta_0 = L/384$

L = Total stud height

Using this model, and applying a factor of safety of 2.0, the *Standard* (AISI 2004a) provides a table of maximum obtainable nominal stud loads considering 1/2" or 5/8" gypsum wall board sheathing which is attached using #6 or #8 screws at a maximum spacing of 12 inches on center.

The *Standard* (AISI 2004a) does allow for rational analysis for other sheathing materials, such as plywood. The commentary to the *Standard* has an example illustrating the use of rational analysis for ½ inch plywood sheathing attached to each face of the wall.

Additionally, the *Standard* (AISI 2004a) requires that the wall studs be evaluated without sheathing for loads that may occur during construction or in the event that the sheathing is removed or ineffective (either during renovations or during a fire). The recommended LRFD load combination is:

$$1.2D + (0.5L \text{ or } 0.2S) + 0.2W \qquad \text{(Eq. 2-2)}$$

2.2.2 Curtain Wall Studs

Under lateral load applied to the strong axis, C-studs will tend to rotate due to the applied load (which is centered on the flange) being eccentric from the shear center of the stud (outside the web) (LGSEA 2001a). This tendency to twist, as shown in Figure 2-2, must be resisted by the action of the sheathing.

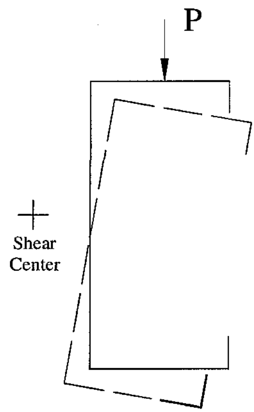


Figure 2-2 Load Eccentric to Shear Center

Two schools of thought prevail on this point. First, that the sheathing possesses sufficient rigidity to resist this torsion. However in limited research, gypsum wallboard has been shown to be insufficient to brace studs with thicknesses exceeding 33 mils (AISI 2002e). The alternate view is to neglect any torsional restraint provided by the sheathing and to provide mechanical bracing as necessary to develop the design strength (AISI 2004e). The most prevalent design practice at this time is to provide mechanical bracing (either flat strap or cold-rolled channels) for all curtain wall studs when the thickness of the stud exceeds 33 mils. In this case, the studs are designed as unbraced flexural members in accordance with the AISI Specification (AISI 2004c).

Flexural design equations for unbraced members require that the ends of the members be restrained against torsion. When the top of the wall is installed with a single slip-track connection to allow for vertical movement, the top of the stud is not considered to be torsionally braced at this end; therefore, mechanical bracing is required to be installed for this purpose, usually within 12 inches as shown in Figure 2-3a. This problem is eliminated if a double slip track (Figure 2-3b) is used (LGSEA 2001a).

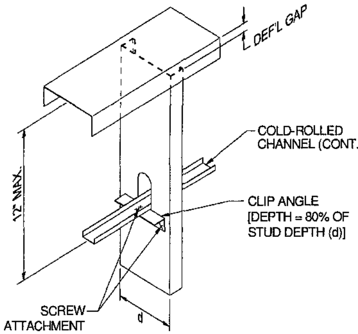


Figure 2-3a Single Track Slip (SSMA 2000a)

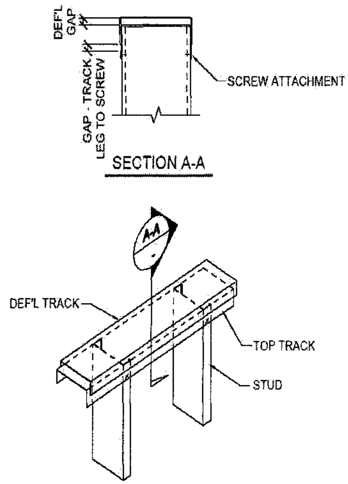


Figure 2-3b Double Track Slip (SSMA 2000a)

2.3 Mechanically Braced Studs

Mechanically braced framing designs, also referred to as "all steel designs", rely on discrete steel bracing elements installed primarily to provide resistance to long-wave buckling (flexural, torsional, or torsional-flexural). There are several industry-typical bracing (or bridging) details which are widely used, and which have proven to be adequate by many years of acceptable use. Additionally, several proprietary bridging systems have been developed during recent years. The basic premise of each of these bridging designs is that the system provides sufficient strength and stiffness to allow the stud system to realize its design strength.

2.3.1 Industry Typical Bridging Systems

Through use over many years, several typical bridging systems have emerged that are widely used in the industry. The details shown in this document are not to be considered to be absolute. Variations in dimension, connector placement, and element thickness are common in practice.

2.3.1.1 Screwed Through the Punchout Bridging

It is common practice to bridge studs using 1-1/2" x ½" x 54 mil cold-rolled channels installed through the web punchouts. The channel is connected to the stud web by a cold-formed angle, typically 1-1/2" x 1-1/2" in dimension, although 2" x 2" angles are also commonly used. Thicknesses range from 33 mils to 68 mils, with 54 mils being the most common. Angle lengths are usually ½" shorter than the stud depth, although shorter or longer lengths are also encountered.

The channel is commonly connected to the angle using (2) #10 self-drilling (SD) screws, and the angle is connected to the stud web also using (2) #10 SD screws. The spacing between screws is critical to develop moment resisting connections.

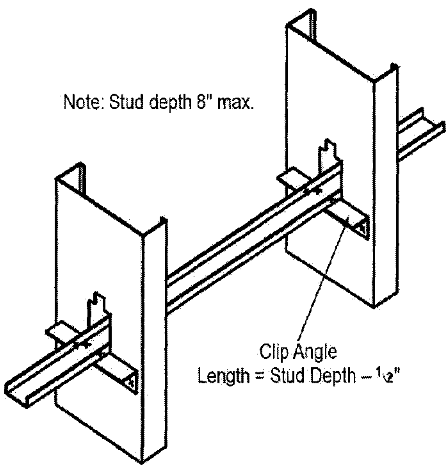


Figure 2-4 Cold Rolled Lateral Bracing (SSMA 2000c)

This bridging is designed to form a moment connection between the stud and the continuous bridging channel. This configuration has the advantage that it is relatively stiff, due to the fact that the angle is connected to the web close to the flanges, thereby effectively bracing the flanges. This type of bridging is easily installed from one side, an advantage in exterior wall construction in multistory buildings. Additionally, screwed construction is oftentimes seen as less costly than welded construction. The tensile strength of the screw connection to the web and the shear strength of the screw connection to the channel are limiting factors.

One disadvantage to this bridging method is that the web punchouts must align along the run of the wall in order to allow the bridging channel to be continuous.

Example 2 illustrates the design of screwed through-the-punchout-bridging for curtain wall studs.

2.3.1.2 Welded Through the Punchout Bridging

A modification to the screwed connection is one where welds are substituted for the #10 SD screws. Due to limitations in welding thin sheet steel materials, a practical lower limit of 43 mils for all welded elements is usually followed, although this is not an absolute limit.

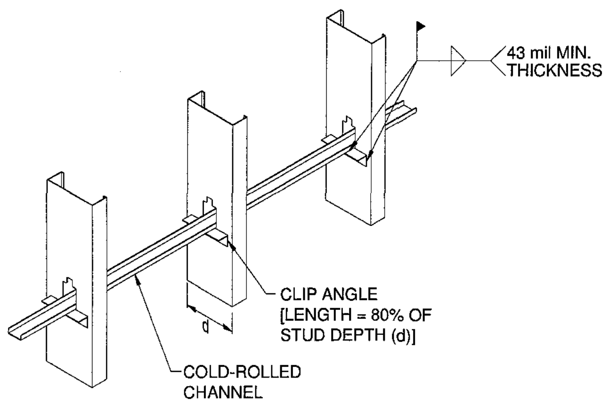


Figure 2-5 Welded Bridging with Clip Angle (SSMA 2000a)

This configuration also has the advantage that it is relatively stiff, due to the fact that the angle is connected to the web close to the flanges, thereby effectively bracing the flanges. However, welded construction is often seen as more costly than screwed construction, particularly in field-installed systems. Yet, welded assemblies are oftentimes cost competitive in shop panelized assemblies. Example 3 illustrates the design of welded through-the-punchout bridging for axially loaded studs.

2.3.1.3 Cold-Rolled Channel Directly Welded to Stud Web

In instances of welded construction using studs with thicknesses of 54 mil or greater, the cold-rolled channel may be directly welded to the stud web. This detail is typically not used for studs with web depths exceeding 8 inches, however testing at the University of Florida (Green, et. al. 2004) tends to indicate that the stud depth does not control the performance of this bridging connector. This assembly is usually more flexible than those using angles, due to the lack of stiffening to the stud web otherwise afforded by the angle. Additionally, the flanges may not be adequately braced in deeper stud sections, due to the distance from the brace point to the flanges.

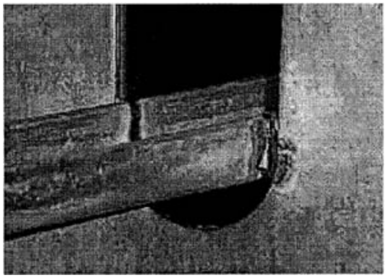


Figure 2-6 Cold-Rolled Channel Directly Welded to Stud Web (University of Florida)

2.3.1.4 Flat Straps Mechanically Attached to Stud Flanges

Another common method of bracing stud flanges is attaching flat straps to each stud flange, typically at intervals of 4 feet as illustrated in Figure 2-7. The flat straps are typically 33 mil in thickness, with widths ranging from 1 inch to 2 inches. Designs using straps with thicknesses up to 98 mils and widths up to 4 inches do exist, however, they are not common. The strap connection to each stud is commonly a #10 screw, however arc spot welds may also be utilized in welded construction.

Flat strap bridging is designed to act in tension only. Because the studs have a tendency to twist in the same direction, the straps must be periodically anchored to the primary structure or shear blocks must be installed at intervals between the studs to remove longitudinal forces from the system. These shear blocks are typically track or stud sections, installed between the studs at intervals determined by calculation.

Flat strap bridging has the advantage that it may be installed independent of punchout locations with only 2 screws per stud. Additionally, this form of bridging is very stiff, even if installed with some initial slackness. An additional concern is that the face mounted straps can cause cosmetically undesirable "bumps" in the finished wall surface. The design of this bridging is illustrated in Example 1.

Because access to both sides of the wall is required to install the straps, this may be a disadvantage in some installations. Additionally, this type of bridging is more prone to field abuse than channel bridging.

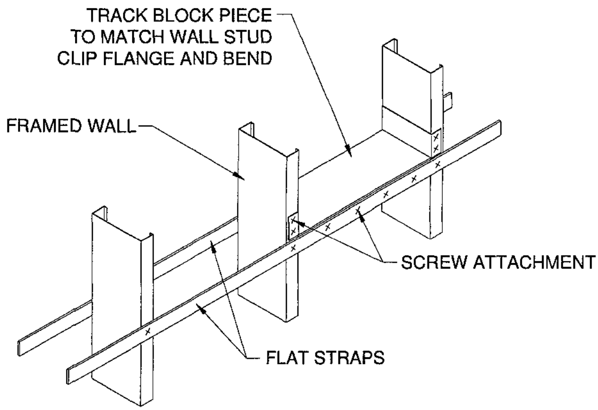


Figure 2-7 Bridging Double Flat Strap with Blocking (SSMA 2000a)

2.3.2 Axially Loaded Mechanically Bridged Studs

Mechanical bridging for axially loaded steel studs may take the form of any of the industry typical configurations or any one of several proprietary systems. The basis for the bridging design is to limit weak-axis lateral and torsional displacements so that the long-wave buckling design capacity predicted by the unbraced length may be attained. LGSEA Technical Note 559 (LGSEA 2001c) provides one method of determining the required lateral brace force demand using the method of discrete bracing found in the *Guide to Stability Design Criteria for Metal Structures* (Galambos 1998).

$$P_{brace} = 0.004\left(4 - \frac{2}{n}\right)\left(\frac{P}{2}\right) \quad \text{(Eq. 2-3)}$$

$$k_{brace} = \left(4 - \frac{2}{n}\right)\left(\frac{2}{L}\right)\left(\frac{P}{2}\right) \quad \text{(Eq. 2-4)}$$

Where:
- P = Axial load in the stud
- L = Unbraced length
- n = Number of intermediate braces

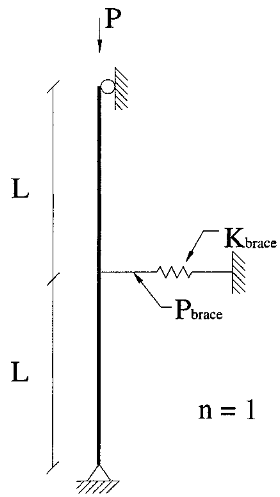


Figure 2-8 Stud with Brace

Note that as n approaches infinity, 2/n goes to zero. Thus the maximum value of the term (4-2/n) is 4.0. This value may conservatively be used to estimate brace requirements for strength and stiffness. Testing undertaken at the University of Florida (Green and Sputo 2004) has shown that this method is somewhat conservative, but acceptable for design practice.

The *Standard for Cold-Formed Steel Framing – Wall Stud Design* (AISI 2004a) conservatively specifies an ASD brace design strength of 2% of the axial load in the stud, based on long standing industry practice.

Due to the systematic nature of imperfections in the studs, bracing forces will accumulate over the run of wall, and must be periodically removed from the bridging system. No criteria currently exists for determining the maximum number of studs over which forces must be assumed to accumulate, therefore current thought is to sum the bracing forces over the total number of braced studs between points where the force is removed from the system. This force may be removed from the system by relying on flat strap cross braces at periodic intervals (See Figure 2-9) (AISI 2002e).

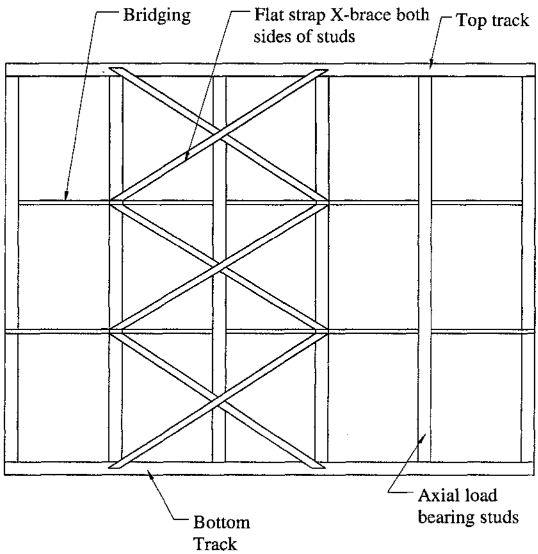


Figure 2-9 Bracing for Axial Load Bearing Studs

The *Cold-Formed Steel Framing Design Guide* (AISI 2002e) provides an excellent example of designing bridging for axially loaded stud walls. The reader of this document is strongly encouraged to study this guide.

2.3.3 Mechanically Bridged Curtain Wall Studs

Mechanical bridging for curtain wall studs may take the form of any of the industry typical configurations or any one of several proprietary systems. The basis for the bridging design is to prevent flexural-torsional buckling of the studs. Both the *Standard for Cold-Formed Steel Framing – Wall Stud Design* (AISI 2004a) and LGSEA Technical Note 559 (LGSEA 2001c) recommend using the provisions of Section D3.2.2 of the AISI Specification (AISI 2004c) to determine the required bracing forces that must be resisted.

The *Standard for Cold-Formed Steel Framing – Wall Stud Design* (AISI 2004a) is silent as to the required brace stiffness. Technical Note 559 (LGSEA 2001c) recommends a minimum brace stiffness to limit secondary stresses in the stud due to deformation of the brace. Assuming a maximum overstress of 15% and a 0.026-radian rotation of the section (Yu 2000), the axial stiffness of the brace may be computed as:

$$k_{brace} = {}^{2P_{brace}}\!/_{0.026d} \qquad \text{(Eq. 2-5)}$$

The *Cold-Formed Steel Framing Design Guide* (AISI 2002e) provides several excellent examples of designing bridging for curtain wall studs. The reader of this document is strongly encouraged to study that guide.

Non-structural interior walls which are required to resist only a 5-pound per square foot (psf) lateral load are a particular subset of curtainwalls. When clad with sheathing on both sides, the wall may be considered as a sheathing braced design. When clad on only one flange, however, mechanical bridging must be installed. SSMA Technical Note 2 (SSMA 2000b) provides design guidance and nomographs for determining bridging spacing and details for using mechanical bridging.

2.3.4 Mechanically Bridged Studs under Combined Bending and Axial Loads

For studs with combined bending and axial loads, the *Standard for Cold-Formed Steel Framing – Wall Stud Design* (AISI 2004a) recommends using the more stringent brace strength requirement calculated from either bending or axial load. The required capacities are to be summed. While the *Standard* is silent as to the required brace stiffness in this instance, it is also accepted practice to utilize the higher of the required stiffnesses.

2.3.5 Prescriptive Methods of Bracing Studs

The AISI *Standard for Cold-Formed Steel Framing – Prescriptive Method for One and Two Family Dwellings* (AISI 2002d) provides prescriptive guidelines for bracing structural steel studs for use in one and two family dwellings. While the title of this document refers to dwellings, the scope of it is such that it may be applied to other structures of similar size and configuration. This document was developed to provide a "pre-engineered" alternative to custom engineered designs.

This standard requires that the flanges of structural studs (axial or curtainwall) be braced in one of the following ways.

1. Sheathing applied to both sides of the wall, either gypsum wall board or structural sheathing.
2. Horizontal steel straps (1-1/2" x 33 mil minimum), attached to each stud flange using a #8 screw. These straps shall be installed at the mid-height of 8 foot tall walls, and at the third-points of other walls 10 foot or less in height. In-line shear blocks shall be installed at the termination of all straps, and at intervals not to exceed 12 feet.
3. A combination of sheathing and straps.

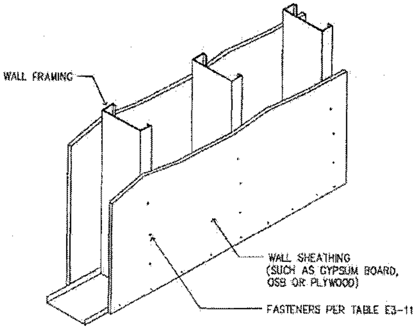


Figure 2-10a Stud Bracing with Sheathing Material Only (AISI 2002d)

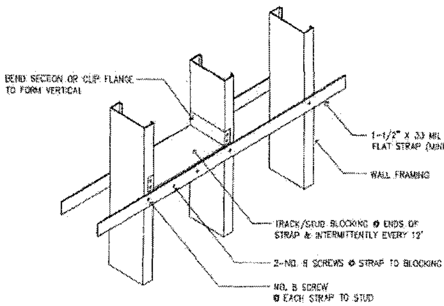


Figure 2-10b Stud Bracing with Strapping Only (AISI 2002d)

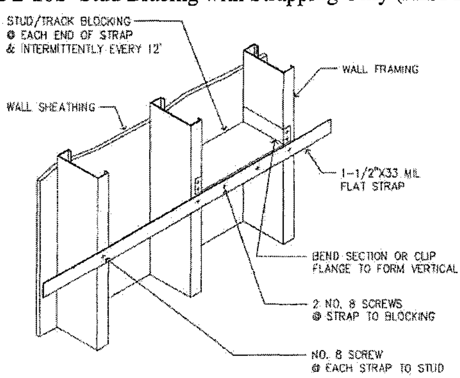


Figure 2-10c Stud Bracing with Strapping and Sheathing Material (AISI 2002d)

2.4 Floors or Roof Joist

Cold-formed cee sections are utilized as floor or roof joist members. As such, the top flange of the joist is usually attached to a rigid diaphragm material (plywood or profiled steel deck), and the bottom flange often has gypsum wallboard attached to it. Much like the previously discussed curtain wall studs, loading eccentric to the shear center of the joist will cause the joist to twist under load. While independent mechanical bracing may not be required to resist lateral-torsional buckling if the compression flange is braced, bridging or bracing is usually installed at intervals not to exceed 8 feet to control visually objectionable twisting of the cross section and to provide stability during erection. This bridging must be anchored to walls or solid blocking to provide a path for the restrained twisting forces to be removed from the floor system. (AISI 2002e).

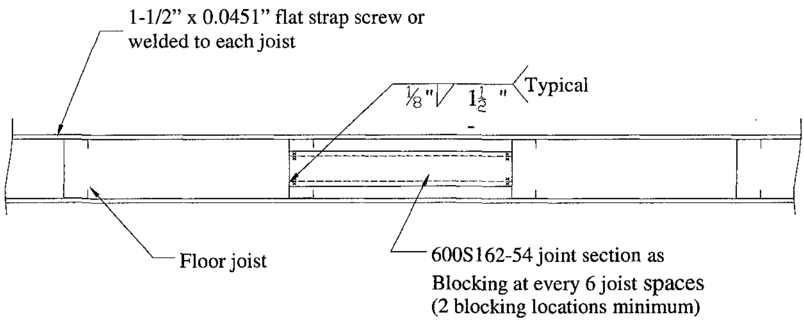


Figure 2-11 Floor Joist Bridging

If joists are continuous over multiple spans, points of contraflecture develop where the bottom, or unbraced flange goes into compression under gravity loads. Common practice is to provide mechanical bracing at or near the inflection point, and at other intervals necessary to develop the required design strength in the region of negative bending.

2.4.1 Prescriptive Methods of Bracing Floor Joists

The AISI *Standard for Cold-Formed Steel Framing – Prescriptive Method for One and Two Family Dwellings* (2002d) provides prescriptive guidelines for bracing and blocking floor joists. Like the prescriptive guidelines for structural studs, this information has application beyond dwellings.

This standard allows for bracing floor joists as follows:

1. The top flanges shall be braced by floor sheathing.
2. For joists with spans exceeding 12 feet, the bottom flange shall be braced using one of the following methods:
   a. Directly attached gypsum wall board.
   b. Continuous steel straps and blocking at intervals not to exceed 12 feet.

2.5 Cold-Formed Truss Framing

2.5.1 Introduction

Currently the use of cold-formed roof trusses is increasing. One reason for this increased is the development of custom truss web and chord profiles. Also, the specification of steel trusses has become very similar to wood trusses (Allen 2004).

2.5.2 Cold Formed Truss Bracing

Truss bracing is required for two general purposes. First, it is used to hold the truss upright, straight and in place, and second, to transfer loads through the structure. Truss bracing that is used to provide temporary stability to the trusses during construction is referred to as erection, construction, or temporary bracing. The proper design and installation of this type of bracing is essential in ensuring that trusses will remain stable until the roof sheathing and other permanent bracing is installed. Construction bracing is typically the responsibility of the truss erector, who should ensure that the bracing design is compatible with all the elements of the structure. Bracing used to brace individual elements in the truss and to distribute loads through the structure once construction is complete, is referred to as permanent bracing. This type of bracing must be a part of the system for the life of the structure. Permanent bracing is the responsibility of the truss designer or the Engineer of Record depending on its function. Several different types of bracing are defined below. Some bracing may be used initially as construction bracing and then, if the bracing is not removed, it doubles as permanent bracing. Often construction bracing is left in the system unless it obstructs the installation of the permanent bracing or other elements of the structure (LGSEA 2002b).

1. Lateral bracing - Bracing placed perpendicular to the plane of the truss attached to either the chord or the web of adjacent trusses. It is generally a C-shape, angle, or tube that is connected directly to the truss chord or web. It is utilized as construction bracing to maintain the trusses upright and keep them at the desired spacing. This type of bracing may also be utilized as permanent bracing to reduce the buckling length of the truss. It is the only bracing to be specified on the design drawings (AISI 2002c).

2. Ground bracing – Bracing spanning from the ground to the truss supporting, no more than six trusses prior to the

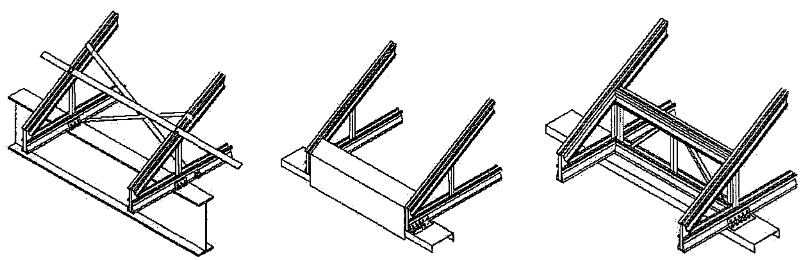
installation of the top chord diagonal bracing. Used to prevent buckling of the truss system.

3. Sway bracing – Diagonal braces utilized during construction. Used to prevent tipping of the trusses.

4. Diagonal bracing – Also known as cross bracing, is bracing placed between lateral braces, in the same plane, and between chords and webs of trusses (LGSEA 2002b). It is provided to prevent buckling of the truss system by transferring loads from the lateral brace to adjacent walls that are able to resist the loads.

5. Bridging – Bracing that connects the top chord of one truss to the bottom chord of an adjacent truss.

6. Blocking – A diagonal strap, brake formed shape or truss (Figure 2-12) used to transfer loads from the sheathing or roof deck to the walls below (LGSEA 2002b).



Flat Straps on Beams Cold-Formed Steel Brake Shape Truss as Blocking

Figure 2-12 Blocking Types (LGSEA 2002b)

Bracing design is complex and time consuming if the designer is new to bracing design. Not only is the design detailed, but specifications for the design of truss bracing have yet to become standardized making it difficult to know what procedure to follow. Advisory guidelines have been published by the Light Gage Steel Engineers Association (LGSEA). Although intended only as a guide, they greatly simplify the task by stating that bracing should be designed using a minimum of 2 percent of the full member axial force (LGSEA 1998b).

2.5.2.1 Construction Bracing

GUIDELINES FOR TRUSS CONSTRUCTION BRACING DESIGN (LGSEA 1998a)

1. Determine the axial forces the truss member will experience during construction, using a load equal to the greater of 5 psf or the truss span in feet divided by 7.

2. Determine the maximum spacing allowed for lateral bracing using the calculated axial loading and cross section size of the top chord. The provisions of Chapter C of the AISI *Specification* (AISI 2004c) should be utilized for this.

3. Design the top chord lateral brace and connection at each truss for a force equal to 2% of the top chord axial force.

4. Design diagonal braces to transfer the load from the lateral brace to the exterior walls. Determine the spacing of the braces by summing the capacity of the diagonal brace and the connection strength, then dividing it by the length of the lateral brace.

5. Determine the number of trusses that need to be supported by ground bracing. Design the ground bracing members and connections for the cumulative force of each line of top chord lateral bracing. Compare this load to a load applied to the trusses from a 50 mph wind (use required local conditions if greater).

6. Design bottom chord lateral and diagonal brace in a manner similar to the top chord bracing design.

7. Design cross bracing in the web plane at each end of the building and at 20 feet on-center.

Ground bracing should be installed to stabilize the first truss or system of trusses that is installed. The bracing should consist of vertical and diagonal braces. It is considered unsafe practice to only install the vertical brace according to *Recommended Design Specification for Temporary Bracing of Metal Plate Connected Wood Trusses (DSB-89)* (TPI 1989). This bracing should be in line with the same braced points as the top chord lateral bracing when the required brace spacing permits. There also needs to be a lateral brace at each diagonal ground brace as well as at ends of the lateral brace as shown in Figure 2-13 (LGSEA 1998a). Design calculations for wood ground bracing are given in DSB-89 and can be used as a guideline for cold-formed steel bracing design.

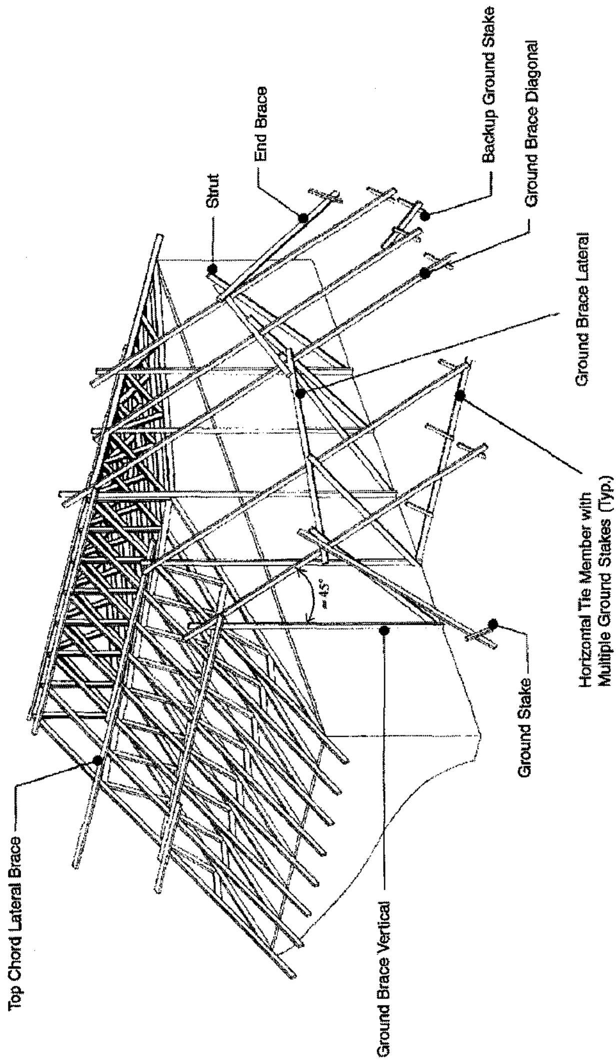


Figure 2-13 Ground Bracing Description (LGSEA 1998a)

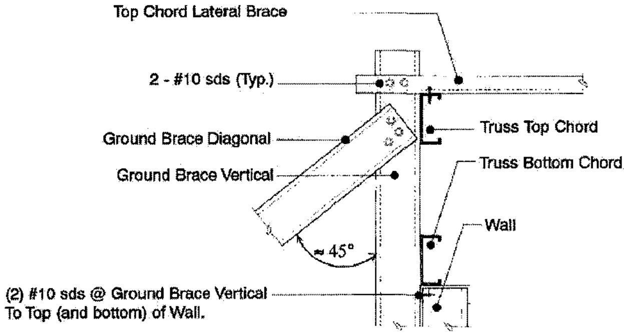


Figure 2-14 Ground Bracing Connection Details (LGSEA 1998a)

Bottom chord bracing, also known as tension chord bracing, is used to properly maintain the spacing of the trusses as well as provide lateral support to resist buckling forces due to wind uplift (WTCA 1993). The bracing may be designed using the same procedures as the top chord or conservatively may be designed using these general requirements (LGSEA 1998a).

1. Lateral bracing – Space at 10' on-center, maximum.
2. Diagonal bracing – Space at 30' on-center, maximum (every 15 trusses)
3. Attach lateral and diagonal bracing to each truss with a minimum of 2-#10-16 x ¾" self drilling screws (SDS), except at the ends of the diagonal braces, use 3-#10 SDS.

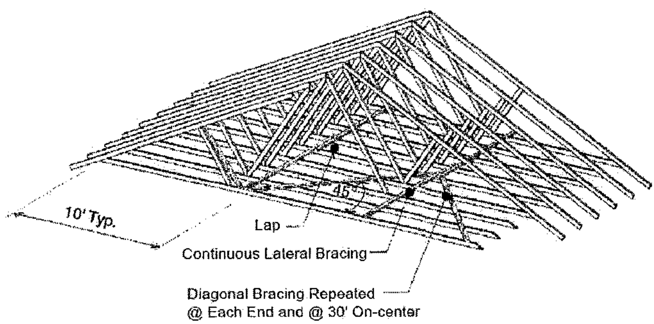


Figure 2-15 Bottom Chord and Web Bracing Detail (LGSEA 1998a)

The design procedures for web bracing are general requirements similar to the method for designing bottom chord bracing. Web bracing is installed in plane with web and in line with the bottom chord lateral bracing. The general requirements (LGSEA 1998a) are

1. Cross bracing shall be installed at each end of the building and at 20’ on-center throughout the remainder of the building.
2. Cross bracing shall be attached to each truss with a minimum 2-#10-16 x ¾” SDS.

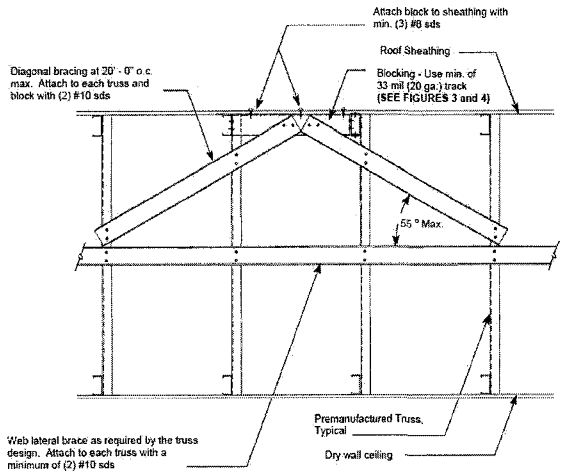


Figure 2-16 Web Lateral and Diagonal Bracing (LGSEA 1998b)

It is essential that the trusses do not become bent or out-of-plumb during delivery and placement. To prevent deforming the trusses while they are lifted into place, either use crane spreader bars for lifting or install special stiffening in the truss. The trusses must be fastened to the bearing walls while the construction bracing is installed. This bracing must be able to resist the loads of the erectors as well as any additional forces necessary for truss stability until the permanent bracing is installed. If sheathing is to be placed directly on the truss then the erection bracing must be removed. If roof purlins are installed due to truss spacing greater than 24 inches on-center, it is not necessary to remove the construction bracing if it does not interfere with the permanent bracing.

Example 7 illustrates the design of construction bracing for a cold-formed steel roof truss.

2.5.2.2 Permanent Bracing

Permanent bracing is used to integrate the truss into the structure. The general design procedure for permanent bracing is based on trusses with spacing up to 24 inches on-center. These are guidelines that are meant to serve as a guide to the engineer (LGSEA 1998b). Other bracing designs based on the principles illustrated here can be developed.

GENERAL DESIGN PROCEDURE FOR PERMANENT BRACING

1. Determine the location of the required permanent bracing by reviewing the truss design. The truss design will indicate the locations of the bracing required to develop the needed strength in the individual members of the truss.

2. Determine the maximum axial force in each truss member that requires permanent bracing.

3. Determine the design force required for each line of lateral bracing. The design force should be equal to a minimum of 2 percent of the member axial force.

4. Design the top chord lateral bracing. Design the diagonal bracing to transfer the cumulative forces from the lateral braces to the shear walls receiving this force.

5. Design the connections for the top chord lateral and diagonal bracing. Anchorage should be either to the roof or ceiling diaphragms, or anchored at the ends of a solid wall.

6. Design bottom chord lateral and diagonal bracing in a manner similar to the top chord bracing design.

7. Design the web lateral bracing, connections to truss, and anchorage. Anchorage should be to the roof or ceiling diaphragm or anchored at the ends to a solid wall.

For a truss spacing no greater than 24 inches on-center, structural sheathing attached directly to the top chord and gypsum board sheathing attached directly to the bottom chord will typically provide adequate lateral support. Examples of structural sheathing include plywood, corrugated metal deck, or any rigid system that is delivered in sheets and directly attached to the truss top chord. Standing seam roofs typically do not provide sufficient lateral bracing of the top chord (LGSEA 2004). If the trusses are spaced greater than 24 inches on-center or if structural sheathing is not used, purlins need to be designed to provide lateral bracing for the roof. The purlins will not provide adequate diagonal bracing by themselves, therefore diagonal bracing will also need to be designed and installed (LGSEA 2004). The total force which the lateral brace will need to carry is 2% of the compression force of the truss members times the number of trusses, N, between the lines of diagonal braces (LGSEA 1998b).

$$\text{Brace Force} = 0.02\text{N} \qquad \text{(Eq. 2-6)}$$

If bracing for the top chord (compression chord) is not specified it is usually assumed that sheathing provides adequate lateral bracing.

To ensure that adequate bracing has been designed and installed there must be clear communication between the building designer, truss designer, and the contractor. One of the primary reasons for truss failures is when the members of the design-build team each assume that someone else has taken responsibility for the bracing design, when actually no one has considered bracing and the trusses collapse due to loads that were not properly transferred throughout the structure.

2.6 Shearwalls and Roof Diaphragms in Cold-Formed Framed Construction

The analysis and design of shearwalls and roof diaphragms in cold-formed framed construction should seem very familiar to any engineer who is knowledgeable in the design of these same components in wood-framed construction. The mechanics of flexible diaphragms (shearwalls) is identical between the materials, and many design provisions for these elements in cold-formed steel systems are modifications of the provisions developed for wood-frame systems. Because of these similarities, this section will assume basic familiarity with shearwalls and diaphragms, and will instead concentrate on the differences between the materials, and relevant advances in the design and analysis of these systems.

Design provisions for shearwalls and diaphragms are developed from the results of monotonic and dynamic test programs, along with rational analysis using first principles of mechanics. A large number of research projects over the past ten years have resulted in a much better understanding of the behavior of steel framed shearwalls and diaphragms, thereby producing more robust specifications and design provisions.

2.6.1 Shearwalls

The *Standard for Cold-Formed Steel Framing – Lateral Design* (AISI 2004b) provides basic ASD and LRFD design requirements for cold-formed steel shearwalls sheathed with gypsum wallboard, plywood, oriented strand board or sheet steel for both wind (static) and seismic (dynamic) applications. While the *Standard* allows for design based directly on principles of mechanics, the greatest utility of this *Standard* are the tables of nominal shear strengths for the various sheathing materials. Provisions for Type I (solid shearwalls) and Type II (perforated shearwalls) are provided, similar to wood-framed walls. Most provisions in the *Standard* were drawn from the *Shear Wall Design Guide* (NASFA 1998). This document is an excellent reference to the background for the design provisions and a concise synopsis to the underlying research. Also, the LGSEA

Technical Note 550 (LGSEA 2001b) is an additional source of design values for various shearwall sheathing materials and systems.

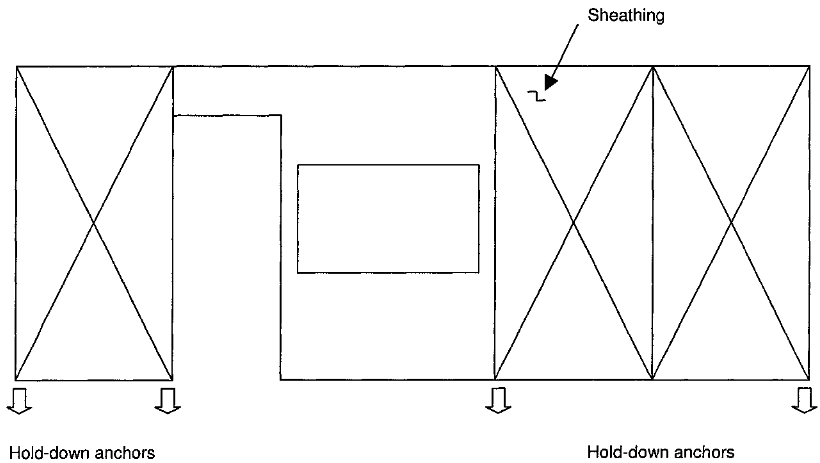


Figure 2-17a Type I Shearwall

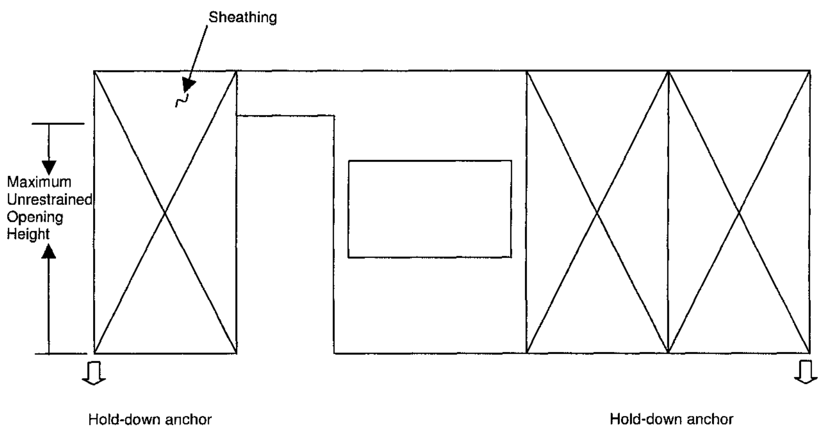


Figure 2-17b Type II Shearwall

The design guidance provided in most publications considers the use of #6, #8, or #10 self-drilling screws (SDS) for the attachment of the sheathing materials to the framing. However, several proprietary fastening systems, primarily using nails or pins which are either powder or pneumatically driven, exist for fastening sheathing. These systems often have the advantage of rapid installation, and are becoming more popular. The strength of these assemblies may be determined from principles of mechanics or through published test data available from the product manufacturer.

In addition to strength provisions, the *Standard* has equations for calculating design deflections, based on recent research.

While the basics of load transfer in steel-framed shearwalls is similar to that encountered in wood-framed shearwalls, the thin-walled nature of the framing elements requires some extra consideration in the design of the boundary elements and load-transfer mechanisms (See Figure 2-18). The *Standard* specifically notes that boundary members, chords, collectors, and anchorage must be proportioned to transmit the induced forces, using the strength provisions of the AISI Specification (2004c). While this same requirement exists for wood-framed shearwalls, designers often do not specifically check the design of the chords and boundary elements, because they usually do not control the design of the shearwalls. The design of chords in cold-formed walls is especially critical, due to local buckling concerns in the studs. LGSEA Technical Note 556a-6 (LGSEA 1997b) is an especially good reference that provides design guidance, details, and a design example for boundary elements in shearwall systems. Examples 4 and 5 in this document illustrate the design of Type I and Type II shearwalls, respectively.

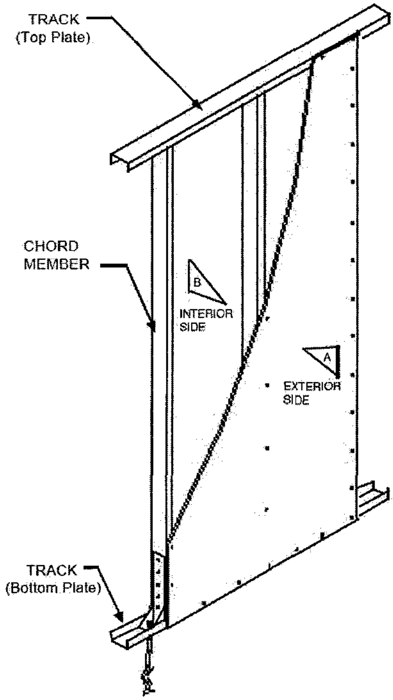


Figure 2-18 Vertical Lateral Force Resisting System Framing (LGSEA 1997b)

2.6.1.1 Strap Braced Shearwalls

An alternative to sheathed shearwalls that is particularly utilized in "all-steel" design is the use of strap braces to create vertical trusses in the plane of the wall, as illustrated in Figure 2-19. Instead of relying on the in-plane shear resistance of the sheathing, a sheet steel strap is designed to act as a tension member transferring the in-plane force into the foundation. This vertical truss requires that the chord stud at the termination of the strap be designed to resist the often substantial compression force that develops.

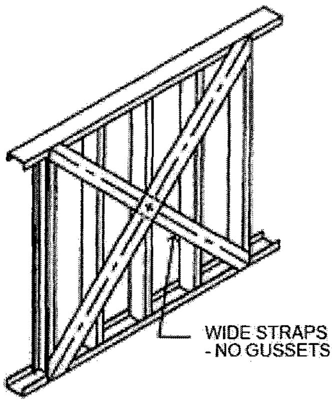


Figure 2-19 Strap Bearing Wall (LGSEA 2001b)

Due to the large number of screws typically required to transfer the tension force at the ends of the strap, most common designs utilize wide straps to eliminate the need for gussets, however gussets are sometimes used.

LGSEA Technical Note 550 (LGSEA 2001b) recommends that engineers also be aware of the following considerations when using straps.

1. Attention must be given to the development of high drag strut forces in the track as a member of the vertical truss. Reinforcement of the track for these compression forces may be required.

2. When straps are used on only one side to the wall, consideration should be given to possible out-of-plane bending in the chord studs and top track that may result from this eccentric loading.

2.6.2 Diaphragms

The *Standard for Cold-Formed Steel Framing – Lateral Design* (AISI 2004b) also provides ASD and LRFD design requirements for diaphragms in cold-formed steel structures for both wind (static) and seismic (dynamic) applications. The *Standard* requires that the strength of the diaphragm be calculated using principles of mechanics, but alternatively does allow the strength of diaphragms sheathed with wood structural panels to be determined using tabulated strengths. LGSEA Technical Note 558b-1 (LGSEA 1998c) provides current design guidance on the proper application of mechanics to the design of diaphragms.

The transfer of shear forces from a roof diaphragm into the shearwalls is a greater concern in cold-formed framed construction than it is in wood-framed construction.

While the mechanics are the same, the rollover resistance of cold-formed steel truss seats is lower than that of an equivalent wood truss. Additionally, wood trusses are initially installed to the wood top plate using (2) 12d common nails, which provide a substantial amount of in-plane force transfer, where these nails are not present with steel trusses. Therefore, the development of an adequate drag-strut to transfer these forces is necessary. LGSEA Technical Note 556a-4 (LGSEA 1997a) provides technical guidance on designing these details, as shown in Figure 2-20. Example 6 illustrates the design of a dragstrut and shear block.

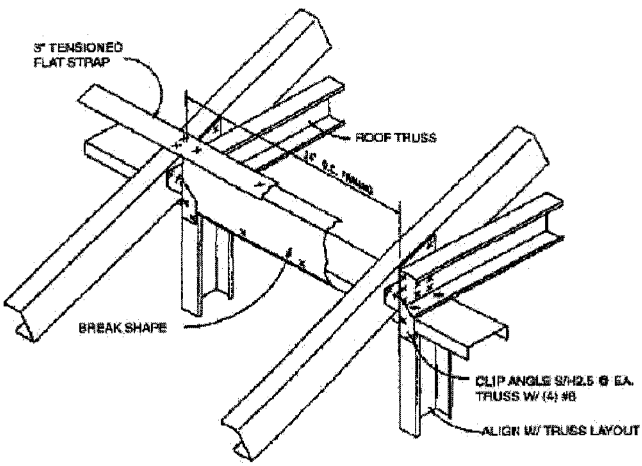


Figure 2-20 Drag Strut and Shear Block (LGSEA 1997a)

2.6.3 Prescriptive Guidance for Shearwalls and Diaphragms

The AISI *Standard for Cold-Formed Steel Framing – Prescriptive Method for One and Two Family Dwellings* (2002d) provides prescriptive guidelines for shearwalls in one and two family dwellings. While the title of this document refers to dwellings, the scope of it is such that it may be applied to other structures of similar size and configuration. This document was developed to provide a "pre-engineered" alternative to custom engineered designs for both wind and seismic resisting applications.

The *Standard* allows for both Type I and Type II shearwalls (Figures 2-17a and 2-17b), sheathed with either OSB or plywood structural sheathing, and maximum wall heights of 10 foot. Using the limitations on building configurations, minimum lengths of shearwalls and required anchorages are tabulated for these building configurations. The design of shearwalls is simplified to determining required shearwall lengths and complying with the associated details. Similar details and prescriptive requirements exist for roof diaphragms sheathed with minimum 3/8" structural board sheathing.

Chapter 3. Cold-Formed Steel in Metal Building Systems

3.1 Introduction

Cold-formed steel members and assemblies are major components of most metal building systems (also referred to as pre-engineered metal buildings). Cold-formed steel elements are typically compose the secondary framing (purlins and girts), wall and roof sheathing, and often in the endwall framing. Occasionally cold-formed steel members are found in the primary framing of short span buildings.

The most commonly used sections are C and Z sections, typically with equal, parallel flanges, however sections with nonparallel and unequal flanges are sometimes used for eave struts. The design of these members is governed by the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI 2004c). The *Specification* provides design requirements for several common bracing applications found in metal buildings. Although most consulting engineers will not design these elements, occasionally they will be tasked with modifying existing metal building systems, therefore having a working understanding of the bracing requirements for the cold-formed steel components is useful. Additionally, roof or wall systems comprised of cold-formed steel components, similar in construction to metal buildings, may be designed for integration into more traditional structural systems.

As noted in Chapter 1, initial imperfections in structural members have an influence on the magnitudes of the required brace strength and stiffness to adequately brace a member. The Metal Building Manufacturers Association (MBMA 2002) has established the following recommended imperfection limitations for purlins and girts in metal building systems shown in Table 3-1.

Table 3-1 MBMA Cold-Formed Purlin and Girt Imperfection Limits

Imperfection	Definition	Limits
Camber	Strong axis out-of-straightness	0.025" per foot (No maximum specified)
Bow	Weak axis out-of-straightness	Not Specified

For imperfections not found in the MBMA guidelines, the limits of Table 3-2 may be considered to be representative of good practice.

Table 3-2 Suggested Imperfection Limit

Imperfection	Definition	Limits
Bow	Weak axis out-of-straightness	1/32" per foot
Crown	Web axis out-of-straightness	1/16"

Most cold-formed steel components in metal buildings are directly or indirectly connected to wall or roof sheathing which may be assumed to provide some degree of restraint to the member, however certain beams and columns, particularly in endwall frames must be designed using discrete (or point) braces. Additionally, discrete point bracing may be needed to augment bracing provided by sheathing.

C and Z sections used as flexural members have a tendency to "roll" under vertical load. C sections roll due to the load application not coinciding with the shear center of the section (Figure 3-1) while Z sections roll because the principal axis is inclined to the direction of loading. Lateral bracing restrains rolling and reduces the secondary stresses associated with rolling. Because of possible failure of the purlins or the girts due to wind uplift or increased gravity loads due to snow or ponding, it is essential that lateral bracing be provided to ensure stability of C or Z sections.

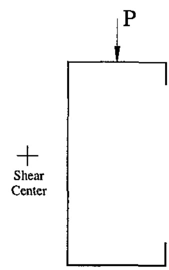


Figure 3-1 Gravity Load not Concentric with Shear Center

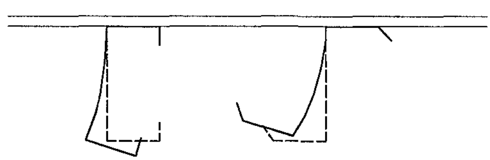


Figure 3-2 Purlin "Rolling" under Gravity Load

Purlins and girts may be braced by a through-fastened metal roof or by installing angles or flat straps that run between eaves and attach to each purlin and eave girt, as shown in Figures 3-3 and 3-4.

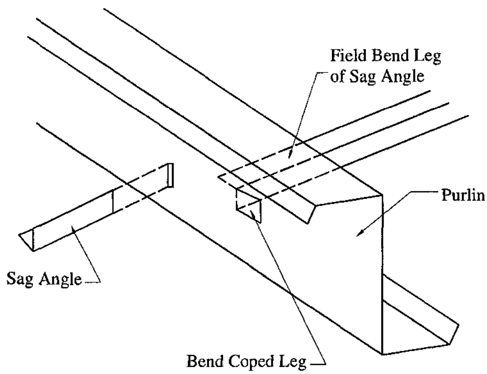


Figure 3-3 Sag Angle Braces for Purlin

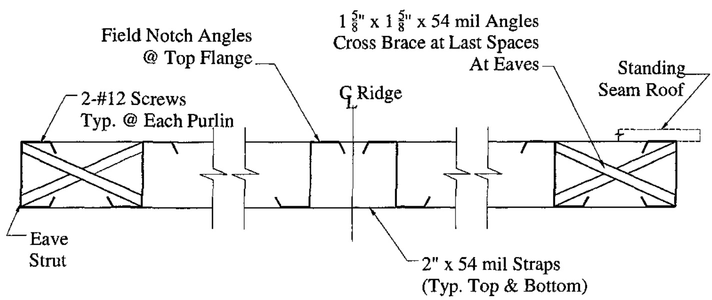


Figure 3-4 Purlin Bracing Using Angles and Straps

This chapter will comment on and provide examples of bracing guidelines for cold-formed steel elements that are specific to metal building systems.

3.2 Purlins and Girts with One Flange Through-Fastened to Sheathing

The top flange of purlins under gravity load is typically in compression, except for the region near the supports. A typical through fastened roof can safely be assumed to provide adequate bracing against global buckling to the sections of the purlin where the roof deck is in contact with the compression flange. The roof deck, however, provides no restraint against local or distortional buckling of the purlin. Under wind loading which produces a net uplift force on the roof (a very common situation), the inner flange that is not in contact with the roof panel is in compression. In this condition, the inner (compression) flange is elastically braced by the tension flange, which is in turn braced by the sheathing (Figure 3-5).

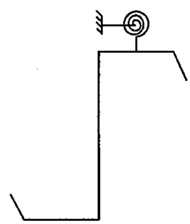


Figure 3-5 Elastically Braced Compression Flange

The AISI *Specification* (AISI 2004c) provides a reduction factor to account for this elastic restraint.

$$M_n = R\, S_e\, F_y \qquad \text{(Eq 3-1)}$$

Where: R = Reduction Factor
S_e = Effective Section Modulus
F_y = Material Yield Stress

Because this design provision was developed based on test results, the limitations regarding purlin dimensions, insulation thickness, span lengths, and material strengths must be adhered to. This design is illustrated in Example 9. If an assembly does not comply with these limitations, or if the roof sheathing does not provide sufficient restraint against uplift, a separate bracing system must be provided to brace the inner purlin flange against uplift.

Occasionally roof sheathing is replaced on existing metal buildings, with an architect or consulting engineer being responsible for the specification of the retrofit roofing. If a through fastened (or screw-down) roof panel was originally installed, it is safe to assume that the sheathing was designed to provide restraint to the inner flange in uplift loading conditions. If the new sheathing is to be a standing seam metal roof (SSMR), a new and independent bracing system for the purlins will need to be designed and installed.

3.3 Purlins with One Flange Fastened to Standing Seam Metal Roofing (SSMR)

3.3.1 SSMR Systems Under Gravity Load

Provisions for gravity-loaded roofs sheathed with standing seam metal roofing (SSMR) are similar to those for through fastened roofing undergoing wind uplift. The SSMR may be assumed to provide some finite bracing to the compression flange of the purlin in contact with the SSMR. This bracing cannot be assumed to be complete, as it was for through fastened roofs. When standing seam metal roofing is used, a reduction factor may be calculated using the results from the AISI "*Base Test Method for Purlins Supporting A Standing Seam Roof System*" (AISI 2002a), or discrete point bracing may be designed which ignores any contribution of the SSMR to the bracing of the purlins. Reduction factors are not tabulated in the AISI *Specification* (AISI 2004c) due to the wide variation in bracing provided by differing SSMR systems. In Canadian practice, any bracing contribution of the SSMR must be ignored, and discrete point bracing must be installed.

Designers proposing to replace existing SSMR systems are cautioned that the bracing provided to the purlins from the replacement panels may differ significantly from the original roof, and supplemental bracing may be required to be installed.

3.3.2 SSMR Systems Under Uplift Load

Due to the wide variation in capacity of SSMR systems under wind uplift load, one of which is the vast degree of difference in bracing provided to the purlins by the SSMR, testing must be performed on the system in accordance with one of several test protocols, including the aforementioned "*Base Test*" (AISI 2002a). The reader is referred to one of several references for additional information (CCFSS 2003).

3.4 Strut-Purlins

Lateral forces due to wind or seismic loading normal to the building frames are transferred through the roof system until those forces are removed from the system in a braced bay. Typical practice is to design several purlins as compression struts to carry this force to the braced bay. These purlins are referred to as strut-purlins, and are designed to resist axial and flexural forces.

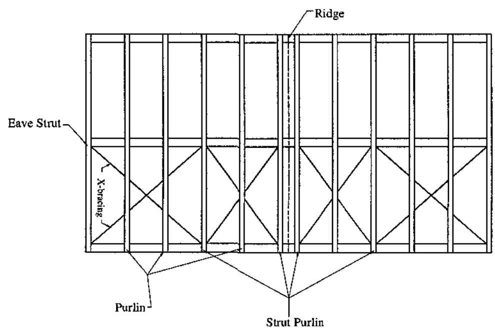


Figure 3-6 Roof Framing with Strut-Purlins Indicated

While the flexural capacity of the strut-purlin is determined by other provisions of the AISI *Specification* (AISI 2004c), the axial capacity of this member is calculated by provisions developed through testing of these compression members which have only one flange attached to through fastened sheathing. The specification lists ten criteria that must be met in order to utilize these provisions, which consider that the roof panel to purlin connection provides some degree of rotational stiffness. The design of a strut-purlin is illustrated in Example 10.

When reroofing a metal building, a designer must consider if the strut-purlins were designed considering partial weak-axis bracing being provided to the strut. Standing seam metal roofing cannot be assumed to provide any weak-axis bracing to the strut, therefore supplemental discrete bracing must be provided if through fastened roofing is replaced by standing seam roofing.

3.5 Anchorage of Roof Systems Under Gravity Load with Purlin Top Flange Connected to Sheathing

In metal roof systems consisting of C or Z purlins, the roof system as a whole will tend to move laterally unless external restraint is provided to brace against this movement. This anchorage will typically consist of elements designed to resist the forces developed by the restrained lateral movement, often referred to as anti-roll clips, as shown in Figure 3-7. The lateral forces that develop in the roof system are typically carried through the roof deck by diaphragm action to the building frames, where it is removed from the system, although the use of strap bracing is also occasionally used for this purpose.

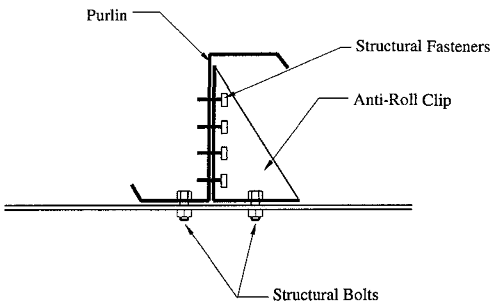


Figure 3-7 Roof System Anchorage

The AISI *Specification* (AISI 2004c) provides equations for calculating the magnitude of the anchorage forces to be restrained, developed by a first-order elastic stiffness model, and verified by experimental testing. While initially developed for through fastened roof systems, later research determined that the design provisions were also applicable to standing seam roof systems. Calculation of anchorage forces is illustrated in Example 8.

Chapter 4. Miscellaneous Cold-Formed Steel Elements and Systems

4.1 Introduction

Industrial rack systems and cold-formed steel diaphragms are structural elements and systems that structural engineers may encounter at some point in their careers. While industrial racks are somewhat specialized in their design and not usually designed by the typical consulting engineer, steel diaphragms are utilized as bracing elements in many steel structures, both hot-rolled and cold-formed.

4.2 Rack Systems

Industrial rack systems are engineered structures consisting of both cold-formed steel shapes and hot-rolled sections. Although most consulting engineers will never design these structures, occasionally consultants will be asked by facility owners to design modifications to existing rack systems to accommodate future needs, or to determine the maximum load carrying capacity of an existing rack. Therefore, a basic understanding of how these structures derive their stability through proper bracing is useful.

Although racks are three-dimensional structures as shown in Figure 4-1, they are usually designed as a series of two-dimensional planar frames with semi-rigid joints. The base design specification for industrial racks is the *Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks,* published by the Rack Manufacturers Institute (RMI 2002). The RMI *Specification* provides for cold-formed steel elements in racks to be designed by the provisions of the AISI *Specification* (AISI 2004c), as modified by the RMI document. Any engineer considering modifying a rack structure is encouraged to obtain a copy of the RMI document.

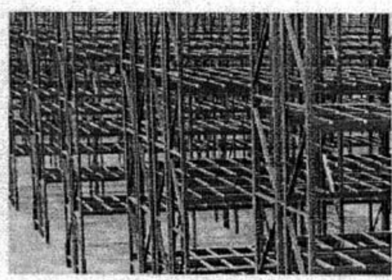


Figure 4-1 Cold-Formed Steel Industrial Rack (Unarco Material Handling)

Bracing for rack systems may be conveniently divided into bracing required for the stability of either individual members or the system, or bracing designed to resist lateral forces applied to the completed system, herein referred to as strength bracing. Bracing elements in racks are most often cold-formed channels or closed tube sections. While a single bracing element may fulfill multiple needs, stability bracing and strength bracing will be considered independently.

4.2.1 Stability Bracing

Stability bracing in rack systems is designed and installed to provide for the stability of individual members, or the entire rack structure itself.

4.2.1.1 Stability Bracing of Individual Members

Rack posts are designed as compression members that can buckle either in pure flexure (bending about a principal axis without twist) or in a torsional-flexural mode (bending accompanied by twisting of the post). Channels with wide flanges, such as the typical rack post, tend to buckle in a torsional-flexural mode, as illustrated in Figure 4-2. Therefore, bracing details that effectively restrain torsion are necessary, as shown in Figures 4-3a, 4-3b, and 4-3c.

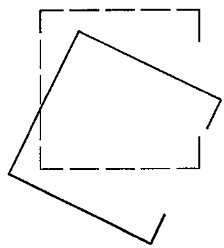


Figure 4-2 Torsional-Flexural Buckling of a Rack Column

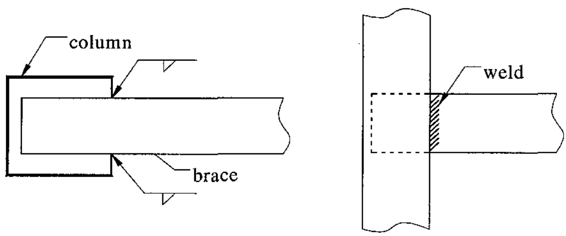


Figure 4-3a Joint Detail Resisting Post Twisting (Brace Narrower than Post)

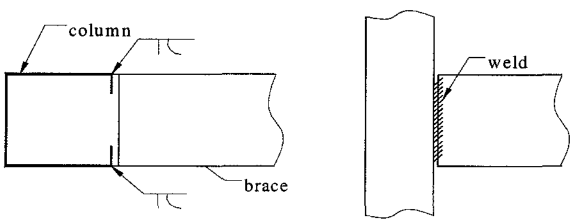


Figure 4-3b Joint Detail Resisting Post Twisting (Brace Matches Post Width)

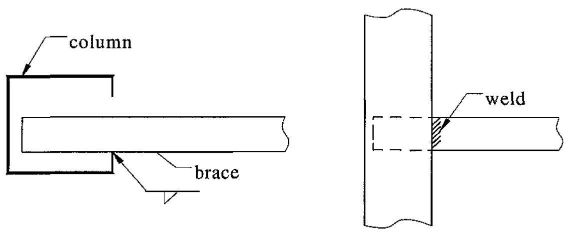


Figure 4-3c Joint Detail Ineffective in Resisting Post Twisting

Beams in racks are typically designed as laterally unsupported flexural members. In pallet racks however, it is a common industry design assumption that the pallets provide some definable bracing to the beams through friction.

4.2.1.2 Stability Bracing of Rack Frames

Depending on the type of rack and the use of that rack, the upright frames may be designed as unbraced frames with semi-rigid joints or as braced frames. A single rack system may be designed as an unbraced frame in one direction and as a braced frame in the orthogonal direction, due to the absence or presence of diagonal bracing.

To be considered as braced against sidesway, a rack structure must have diagonal bracing in the vertical plane of the portion of the rack under consideration, in order to restrain the columns in the braced plane. In order to restrain columns in other planes, the shelves need to either be rigid, or have diagonal bracing in the horizontal plane, as illustrated in Figure 4-4.

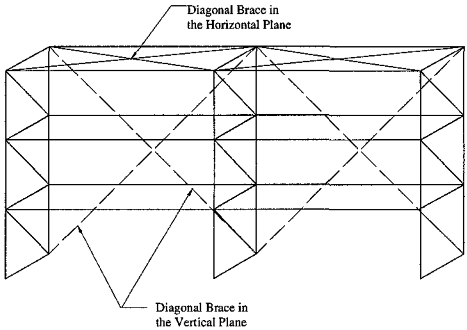


Figure 4-4 Horizontal and Vertical Bracing

Various arrangements of diagonal bracing are possible, as shown in Figure 4-5. When bracing does not extend for the full height of the frame, levels without bracing will need to be considered to be unbraced.

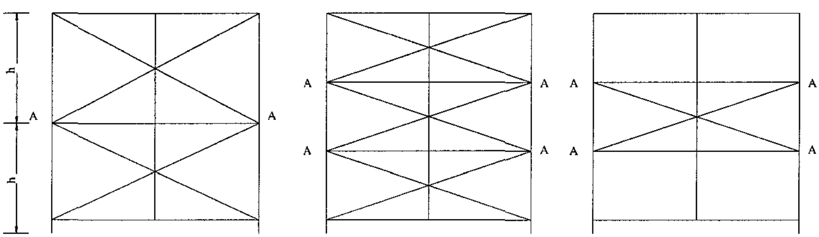


Figure 4-5 Sideway Bracing

4.2.2 Strength Bracing

Most racks are not subject to environmental loads. When required by the controlling building code, however, the rack structure must be designed to resist lateral loads resulting from seismic or wind loading. The rack structure may be designed to resist these loads as either a semi-rigid frame, or as a braced frame with diagonal bracing. Stability bracing may be utilized to resist these laterals loads. The design of frame structures to resist lateral loads is a subject familiar to most engineers. The RMI

Specification (RMI 2002) provides very specific guidance for calculating these loads on the rack structure.

4.3 Shear Diaphragms

A cold-formed steel diaphragm is a structural assembly consisting of cold-formed steel panels that provide in-plane shear strength. Diaphragms typically carry lateral loads, such as wind loads or earthquake loads, but may resist gravity loads in limited instances. Diaphragms eliminate the need for separate bracing systems to resist lateral loads.

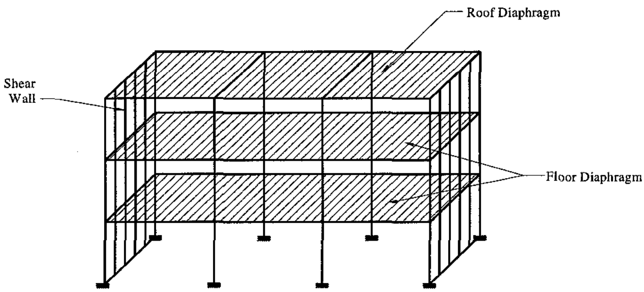


Figure 4-6 Shear Diaphragms

The behavior of a diaphragm is dependent on the type of panels used, the interlocking (or sidelap) connection between the panels, the fastening of the panels to the supporting structure, and the geometry of the described area (SDI 1987).

Types of cold-formed steel diaphragms include (Fisher et. al. 2002):

1. Steel deck without fill
2. Steel deck in combination with insulating fill
3. Concrete slab on steel form deck
4. Composite steel deck with lightweight concrete
5. Composite steel deck with normal weight concrete

A properly designed steel deck can independently resist lateral loads. The addition of fill materials increases the strength and stiffness of the diaphragm.

4.3.1 Diaphragm Strength and Stiffness

Several factors contribute to the strength and stiffness of a cold-formed steel diaphragm. Key elements are the panel configuration, panel span, panel thickness and material strength, attachment of the panels to the structure, and the type and thickness of fill material (if used).

4.3.1.1 Diaphragm Strength

The strength of the diaphragm is controlled by panel configuration, panel span, spacing of the supporting members, panel thickness, types and arrangements of fasteners, and the fill used, if any.

1. *Panel Configuration* as determined by the height of the panel ribs and width of the panel flats has significant impact. The height of the panel ribs is more crucial than the width.

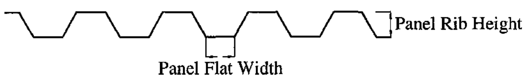


Figure 4-7 Diaphragm Panel

A panel with deeper ribs is more flexible, resulting in greater distortion of the panel, which in turn is more noticeable at the ends of the diaphragm where the in-plane shear is greatest. Depending on the span of the panels, a rib height can be selected that is sufficiently strong with little resulting deformation. A wider panel gives the diaphragm increased strength due to less side laps along the diaphragm (Yu 2000).

2. *Shorter Span Panel* could provide a somewhat higher shear capacity than longer span panels. Tests show that strength is not overly dependent on panel span (Yu 2000).

3. *Decreased Purlin Spacing* increases the strength of the panels and the diaphragms as a whole. This is more noticeable with thinner sheets.

4. *Greater Material Thickness* increases the strength of the diaphragm (Yu 2000). As the material thickness increases, the diaphragm strength also increases, but not at a linear rate.

5. *Types and Arrangement of Fasteners* affect the strength of the diaphragm. It is not only affected by the fastener type and arrangement, but by the material surrounding the fasteners. When insufficient fasteners are utilized to attach the diaphragm to the structural system, or the fasteners are inadequate in size, the diaphragm may fail due to separation of the fasteners from the structural system or tearing of the surrounding material. If the correct number and size of fasteners are utilized, though they are spaced incorrectly, the diaphragm may fail due to elastic buckling (Yu 2000). To prevent failure, the fasteners must be sized and spaced appropriately, adding additional side lap fasteners and end connectors as necessary.

6. *Concrete Fill* on steel panels provides a rigid and effective diaphragm. The strength of the concrete and its bond to the panels influences the strength of the diaphragm. Lightweight concrete may be used to increase the stiffness of the diaphragm, though it does not provide as much strength to the diaphragm as does normal-weight concrete.

4.3.1.2 Diaphragm Stiffness

The stiffness of a diaphragm is a measure of how much the diaphragm resists deflection under load. The deflection of an unfilled diaphragm system is affected by flexural stress, shear stress, seam slip, and local distortion of the panels. The shear stiffness, G', of the diaphragm is determined by placing a load, P, on the structure and calculating its deflection, Δ.

$$G' = \frac{P}{\Delta}\left(\frac{a}{L}\right) \qquad \text{(Eq. 4-1)}$$

Where a = height of the structure
L = length of the structure parallel to the lateral force, P

This stiffness can be converted into an effective shear modulus $G = G'/t$.

The stiffness of the diaphragm may also be affected by what are known as warping relaxations. These relaxations occur when end closures are removed or are not effective while the load is still maintained. Another factor affecting stiffness is the discrete connection at the panel sidelaps. These deflections need to be added to the deflection determined previously to calculate an accurate shear stiffness (SDI 1987).

$$G' = \frac{Pa/L}{\Delta_s + \Delta_d + \Delta_c} \qquad \text{(Eq. 4-2)}$$

Where Δ_s = total shear deflection
Δ_d = total warping relaxations

Δ_c = total discrete connections deflection

The overall deflection of the diaphragm is greatly reduced by the use of concrete fill.

4.3.1.3 Analytical Methods for Determining Diaphragm Strength and Stiffness

In North American practice, there are two generally accepted analytical methods for determining the strength and stiffness of the diaphragm, the Steel Deck Institute (SDI) method and the Tri-Service method. The SDI method is described in the *SDI Diaphragm Design Manual* (SDI 1987) while the Tri-Service method is discussed in *Seismic Design for Buildings* (Army 1982). For further details on these methods the reader is referred to the referenced documents.

4.3.2 Connections

Connections are one of three critical components of a diaphragm. In addition to panel properties and span layout (Figure 4-8), connections contribute to the diaphragm strength and stiffness. Connections are most commonly created by welds, screws, or power driven fasteners.

A structural connection is used for attaching the panels to the structural framing. Sidelap connections, also known as stitch connections, connect panel sheets together, while not attaching them to the frame (SDI 1987). Due to the variety of fasteners and their strengths, the choice of fastener depends on the shear requirements and preference of the designer.

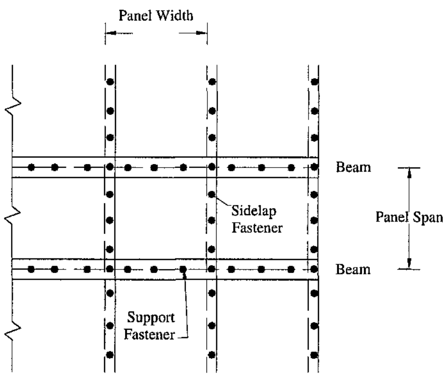


Figure 4-8 Support and Sidelap Fasteners

Currently, welding is the most common structural connection used to connect panels to the structural frame. Weld washers are recommended for panels thinner than 0.028 inches to prevent burn though of the sheet. For panels thicker than 0.028 inches a weld washer is not recommended. It is more efficient to place a 5/8" diameter arc spot weld on the thicker panels (SDI 1987). Other methods of connecting panels to structural framing are power driven fasteners or self drilling screws. The advantages to these mechanical fasteners are that they are easy to install, they leave a clean appearance, and they have fewer quality control concerns (Fisher et al. 2002). The disadvantage of these fasteners is that their shear capacities are less than that for comparable welds.

Sidelap connections may be made using either welds, button punching, or self drilling screws. Welding panels thinner than 0.0295 inches are not recommended due to the possibility of burn through. Where welding is possible, the welds have a strength of approximately 75% of the structural connector strength (SDI 1987). Button punching is the most unreliable form of sidelap connection. Ensuring the upstanding leg is fully inserted into the folded-over double element is critical to ensuring the strength of the button punch. If the upstanding leg is not fully inserted, the two panels will not be attached properly. Even if the button punch connection is done correctly, it only stabilizes the panel edges. It provides little diaphragm strength. Self drilling screws are the most reliable form of sidelap connection. The strength of the screw is limited by bearing fracture, or a tearing fracture of the panel surrounding the screw.

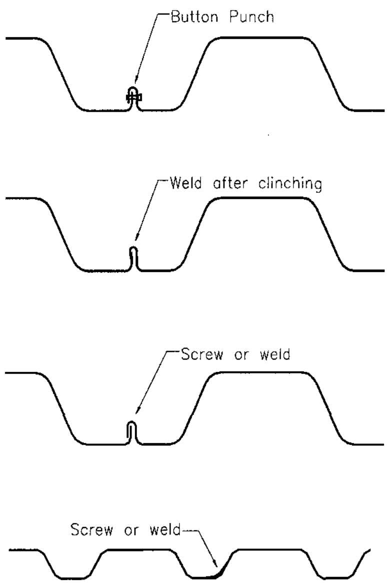


Figure 4-9 Sidelap Connection

4.3.3 Special Considerations

There are several factors that require special attention to ensure that a diaphragm has the appropriate strength and stiffness. Yu lists six considerations that will help to ensure a sound diaphragm (2000).

1. If purlins are framed on top of a perimeter beam, the shear force in the panels may cause the purlins to tip or roll-over due to the eccentric loading. The tipping may be prevented by adding a rake channel or other member to transfer the force from the panels to the structural frame.

2. Panel interruptions by openings or non-structural panels need special attention.

3. A special note needs to be added to the drawings if the panels are to be used as diaphragms, prohibiting the removal of any panels unless alternate bracing is provided.

4. Due to the dependence of the diaphragm on the type, spacing, and strength of the fasteners, no substitution should be allowed from the fasteners specified.

5. The diaphragm is not effective until all components are in place and properly attached. Temporary bracing should be provided to ensure the diaphragm is in the proper alignment until all the panels are installed and connected. Temporary bracing should also be provided anytime a panel needs to be removed.

6. Proper inspection and quality control procedures need to be implemented to verify the size and spacing of the fasteners.

4.3.4 Design Procedure for Shear Diaphragms

Shear diaphragms are designed to resist horizontal load. The design is based on the shear strength and stiffness needed to resist the applied loading and on the design provisions of the local building code. The design shear strength using the Allowable Strength Design (ASD) method is (AISI 2004c):

$$S_d = \frac{S_n}{\Omega_d} \qquad \text{(ASD)} \qquad \text{(Eq. 4-3)}$$

$$S_u = \phi_d S_n \qquad \text{(LRFD)} \qquad \text{(Eq. 4-4)}$$

Where S_d = design shear strength for diaphragm, lb/ft
S_n = in-plane diaphragm nominal shear strength established by calculation or test, lb/ft
S_u = LRFD design strength for diaphragm, lb/ft
Ω_d = factor of safety for diaphragm shear (See Table 4-1)
ϕ_d = resistance factor for diaphragm shear (See Table 4-1)

Table 4-1 Factors of Safety and Resistance Factors for Diaphragms (AISI 2004c)

Load Type or Combinations Including	Connection[1] Type	Limit State			
		Connection Related		Panel Related[2]	
		USA & Mexico		USA & Mexico	
		Ω_d	ϕ_d	Ω_d	ϕ_d
Earthquake	Welds	3.00	0.55	2.00	0.80
	Screws	2.50	0.65		
Wind	Welds	2.35	0.70		
	Screws				
All Others	Welds	2.65	0.60		
	Screws	2.50	0.65		

[1] When fastener combinations are used within a diaphragm system, the more severe factor is used.
[2] Panel buckling is out of plane buckling and not local buckling at fasteners. The more severe factor limit state controls the design.

In addition to designing the diaphragm for strength, deflection of the diaphragm must be considered (AISI 1967).

$$\Delta_{total} = \Delta_b + \Delta_s \qquad \text{(Eq. 4-5)}$$

Where Δ_{total} = total deflection of shear diaphragm, in.

Δ_b = bending deflection, in.

Δ_s = shear deflection, including deflection due to seam slip and local distortion , in.

Table 4-2 Deflection of Shear Diaphragms (AISI 1967)

Type of Diaphragm	Loading Condition	Δ_b	Δ_s [a]
Simple Beam (at center)	Uniform load	$\dfrac{5wL^4(12)^2}{384EI}$	$\dfrac{wL^4}{8G'b}$
	Load P at center	$\dfrac{PL^3(12)^3}{48EI}$	$\dfrac{PL}{4G'b}$
	Load P applied at each 1/3 point	$\dfrac{23PL^3(12)^3}{648EI}$	$\dfrac{PL}{3G'b}$
Cantilever beam (at center)	Uniform Load	$\dfrac{wa^4(12)^3}{8EI}$	$\dfrac{wa^2}{2G'b}$
	Load P applied at free end	$\dfrac{Pa^3(12)^3}{3EI}$	$\dfrac{Pa}{G'b}$

[a] When the diaphragm is constructed with two or more panels of different lengths, the term $G'b$ should be replaced by $\Sigma G'b$ where G' and b are the shear stiffness and the length of a specific panel, respectively.

This total deflection must be within the allowable limits of the building code or other design provisions.

Chapter 5. Summary

This report has discussed current industry practice for bracing of cold-formed steel structural members and structural assemblies within the framework of providing practical design guidance to practicing engineers. Hopefully this report has helped the reader to develop an appreciation of bracing design. In most situations, the design of bracing is driven by basic engineering mechanics principles that are acquired at the undergraduate level.

The current state-of-the-art is not static. Organizations such as the American Iron and Steel Institute, the Light Gauge Steel Engineers Association, and the Steel Stud Manufacturers Association among others, are pursuing research and development efforts to advance current practice through the development of new knowledge of bracing systems. The field is rapidly changing and continued study is necessary to remain abreast with current knowledge. The reader is encouraged to study the reference publications cited in this report for further and more complete information.

Appendix A

Unit Conversions

To Convert	To	Multiply by
inch (in.)	mm(millimeter)	25.4
kilogram force (kgf)	N (newton)	9.81
kilogram-force-meter	N-m (newton-meter)	9.81
pound force(lbF)	N (newton)	4.45
pound force/inch	N/m (newton/meter)	1.75
pound force per square inch (psi)	kPa (kilopascal)	6.89
pound mass (lbm)	kg (kilogram)	0.454

Appendix B

Design Examples

Example 1

Face Mounted Strap for Mechanically Braced Axially Loaded Studs (ASD) (Section 2.3)

Required: Design face mounted flat straps to brace 600S162-43 studs under the stated loading conditions

Reference: LGSEA Technical Note 559 (LGSEA 2001c)

Service Wind load = 25 psf
Service Axial load = 1900 lbs per stud

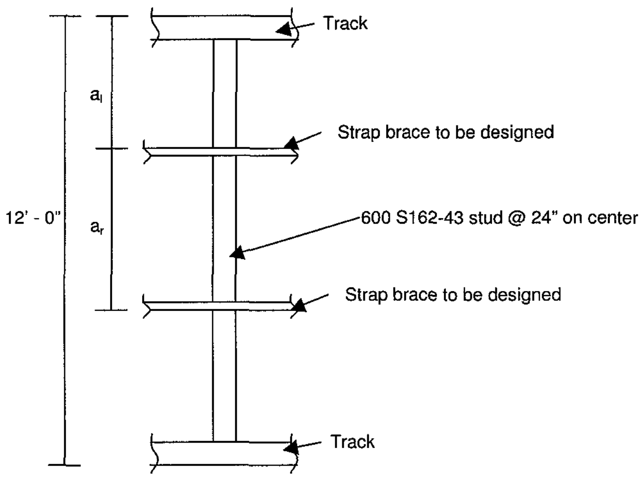


Figure B-1 Exterior wall stud in axially load bearing wall

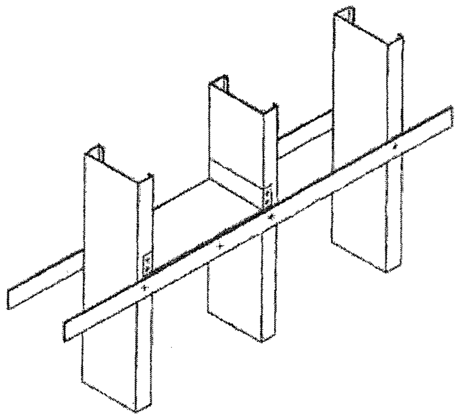


Figure B-2 Strap Braces and Shear Block (AISI 2002d)

Step 1: Calculate Bracing Requirements for Flexure
(AISI Specification, Section D3.2.2) (AISI 2004c)

Input Data:

Service applied uniform load (plf), w	50 plf
Dist. Between braces to the left of this brace, a_l	48 inches
Dist. Between braces to the right of this brace, a_r	48 inches
Stud web depth – out-to-out, d	6 inches
Stud thickness, t	0.0451 inches
Stud inside corner radius, r	0.0712 inches
Stud flange width, b_f	1.625 inches
Stud gross moment of inertia, I_x	2.316 in^4
Overall out-to-out depth of stud lip, D	0.5 inches

Computed Values:

Horizontal projection of stud flange from web inside face — 1.51 inches

$$w_f = b_f - t - r$$

Distance from stud shear center to web mid-plane — 0.67 inches

$$m = \frac{w_f(d)(t)}{4(I_x)}\left[(w_f)(d) + 2(D)\left(d - \frac{4(D^2)}{3(d)}\right)\right]$$

$$P_{\text{twist brace}} = 1.5\left(\frac{m}{d}\right)\left(\frac{w}{12}\right)\left(\frac{a_l}{2} + \frac{a_r}{2}\right)$$ 33.0 pounds

$K_{\text{twist brace}} = \dfrac{2(P_{\text{twist brace}})}{0.026 * d}$ 423.6 lbs/inch

Step 2: Calculate Bracing Requirements for Axial Load
(SSRC Guide, Section 12.5) (Galambos 1998)

Input Data:

Applied service axial load, P	1900 pounds
Stud unbraced length, L	48 inches
Number of braces (installed mechanical braces), n	2

Computed Values:

$P_{\text{axial brace}} = 0.004\left(4 - \dfrac{2}{n}\right)\left(\dfrac{P}{2}\right)$ 11.4 pounds

$k_{\text{axial brace}} = \left(4 - \dfrac{2}{n}\right)\left(\dfrac{2}{L}\right)\left(\dfrac{P}{2}\right)$ 118.3 lbs/inch

Step 3: Total Bracing Requirements per Stud

$P_{\text{brace total}} = \sum\left(P_{\text{twist brace}} + P_{\text{axial brace}}\right)$ 44.4 pounds

$k_{\text{brace total}}$ = Maximum of $k_{\text{twist brace}}$ or $k_{\text{axial brace}}$ 423.6 lbs/inch

Step 4: Required Capacity of Shear Blocking
(AISI Specification, Section C3.2) (AISI 2004c)

Shear blocking to be piece of stud: 600S162-33

Input Data:

Blocking yield strength, F_y	33 ksi
Blocking thickness, t	0.0346 inches
Stud spacing, s	24 inches
Stud flange width, b_{flange}	1.625 inches
Stud depth out-to-out, d_{stud}	6 inches
Modulus of elasticity, E	29500 ksi

Blocking inside corner radius, r	0.0764 inches

<u>Computed Values:</u>

Shear Buckling Factor

$a = s - b_{flange}$ 22.38 inches

$h = d_{stud} - 2(r) - 2(t)$ 5.78 inches

$a/h =$ 3.87

$k_v = 5.34 + \dfrac{4.00}{\left(\dfrac{a}{h}\right)^2}$ if $\dfrac{a}{h} > 1.00$ 5.61

$= 4.00 + \dfrac{5.34}{\left(\dfrac{a}{h}\right)^2}$ if $\dfrac{a}{h} \le 1.00$

Shear Strength of Block:

web slenderness $= \dfrac{h}{t}$ 166.99

$S_1 = \sqrt{\dfrac{E(k_v)}{F_y}}$ 70.80

$S_2 = 1.51\sqrt{\dfrac{E(k_v)}{F_y}}$ 106.90

$V_n = h(t_w)(0.60)(F_y)$ if $\dfrac{h}{t} \le \sqrt{\dfrac{E(k_v)}{F_y}}$

$= (h)(t_w)\dfrac{0.60\sqrt{\dfrac{E(k_v)}{F_y}}}{\left(\dfrac{h}{t}\right)}$ if $\sqrt{\dfrac{E(k_v)}{F_y}} < \dfrac{h}{t} \le 1.51\sqrt{\dfrac{E(k_v)}{F_y}}$

$= \dfrac{h(t_w)(0.904)(E)(k_v)}{\left(\dfrac{h}{t}\right)^2}$ if $\dfrac{h}{t} > 1.51\sqrt{\dfrac{E(k_v)}{F_y}}$ 1070 lbs.

V_n allowable = V_n/Ω	670 lbs.
$\Omega = 1.60$	

Step 5: Final Design of Bracing System

Maximum number of studs which may be supported by one block

$= 670/44.4 = 15.10$ studs

Input Data:

Design number of studs to be braced by one block	12
Strap yield strength, F_y	33 ksi
Strap fracture strength, F_u	45 ksi
Strap width, w	1 inches
Strap thickness, t_s	0.0346 inches
Framing screw diameter (#10 screw), d_s	0.183 inches

Computed Values:

Required design force in strap — 533.33 pounds

$P_{strap} = P_{brace\ total}$ (12 studs)

$k_{strap} = \dfrac{A_g(E)}{L}$ — 3544.10 lbs/inch

$A_g = w\,(t_s)$

L = (Stud spacing)(Design number of studs to be braced by one block)

CHECK:	$k_{strap} > k_{brace\ total}$	3544.10 > 423.6	***OKAY***

Strap nominal fracture strength

$P_{n\ strap} = A_n\,(F_y)$ — 1272.07 pounds

$A_n = (w - d_s)t_s$

Strap allowable fracture strength

$= \dfrac{P_{n\ strap}}{\Omega}$ — 636.03 pounds

$\Omega = 2.00$

Strap nominal yield strength

$P_{n\ strap} = A_g\,(F_y)$ — 1141.80 pounds

Strap allowable yield strength

$= \dfrac{P_{n\ strap}}{\Omega}$ 683.71 pounds

$\Omega = 1.67$

Strap allowable

= smaller of *Strap allowable fracture strength* or *Strap allowable yield strength* 636.03 pounds

Strap allowable > P_{strap} 636.03 > 533.33 ***OKAY***

Example 2

Through-the-Punchout Bridging Design For Mechanically Braced Curtainwall Studs (ASD) (Section 2.3)

Required: Design through the punchout bridging to brace 600S162-43 studs under the stated loading conditions.

Assume: 600S162-43 Studs @ 12'-0" long @ 24" o.c.
Braced at 4'-0" intervals
Wind Pressure = 15 psf

Reference: Cold-Formed Steel Framing Design Guide (AISI 2002e)

Step 1: Bridging Channel Design

The bridging channel is modeled as a continuous beam supported by the major axis bending strength of each stud and loaded by the twisting moment from each stud, as illustrated in Figure B-3. Assume a 5 span (i.e. 5 stud spaces) condition as shown in Figure B-4.

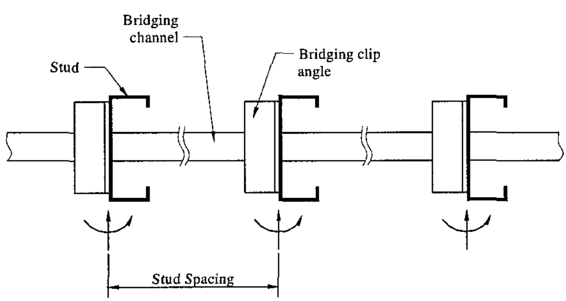
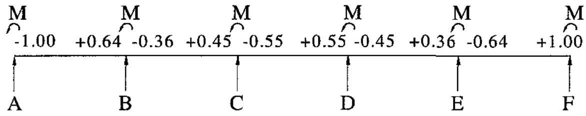


Figure B-3 Curtainwall Studs Braced by Bridging Channel



Moment = Coefficient x M
(+ = tension bottom fiber)

Figure B-4 Five Span Bridging Channel

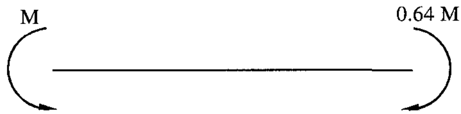


Figure B-5 Critical Span

The outside span is critical, as shown in Figure B-5. The moment, M, is derived from the top and bottom flange brace requirements given in the AISI Specification D3.2.2 (AISI 2001).

$P_L = 1.5K'wa = 1.5wam/d$

where:

w = load/ft on the stud for strength = (24/12)(15) = 30 lb/ft
a = bridging spacing = 4.00 ft.
m = stud web center line to shear center = 0.670"
d = 6"

$P_L = 1.5(30)(4.00)(0.670/6)$
$= 20.1$ lb

The moment resisted by the bridging channel is given by the flange brace couple with a lever arm equal to the depth of the stud (Figure B-6).

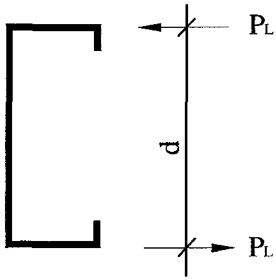


Figure B-6 Twisting Moment on Channel

M = 20.1(6) = 120.6 in.-lb. The resulting moment values are illustrated in Figure B-7.

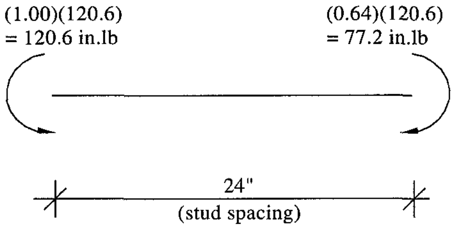


Figure B-7 Bridging Channel Design Moment

Try 150U50-54 bridging channel (Fy = 33ksi).

Per SSMA (2001),
$M_{allow} = 1227.6$ in.-lb.

$120.6 < 1227.6$ ***OKAY***

Step 2: Welded Bridging Connection

Twisting moment transferred from stud to bridging channel:

M_{req} = 120.6 in.-lb.

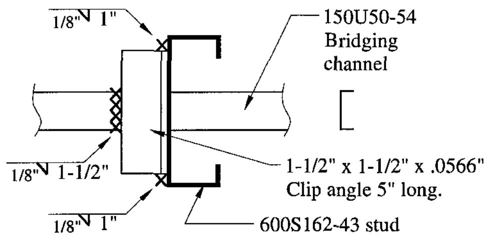


Figure B-8 Welded Connection

Clip angle size

Based on field experience, it is recommended that the thickness of the bridging clip angle be the greater of the thickness of the stud or 0.0566" (for wind bearing applications).

Use t = 0.0566"

Bridging channel to clip angle weld (See Figure B-8):

Clip angle	t = 0.0566"	F_u = 45 ksi
Bridging channel	t = 0.0566"	F_u = 45 ksi

Using linear analysis, the maximum required load per inch of weld length may be calculated by:

$$q_{req} = \frac{M_{req}}{S_{weld}} = \frac{120.6}{(1)(1.5)^2/6} = 322 \text{ lb/in.}$$

Allowable weld capacity per inch of weld length is:

$$q_{all} = 0.75tLF_u/\Omega$$

= 0.75(0.0566)(1)(45)(1000)/2.50
= 764 lb/in > 322 lb/in **_OKAY_**

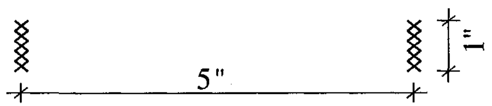


Figure B-9 Clip Angle to Stud Weld

Clip angle to stud weld (See Figure B-9):

Clip angle	t = 0.0566"	F_u = 45 ksi
Stud	t = 0.0451"	F_u = 45 ksi

Weld group allowable moment (stud material governs):

$M_{all} = (5.00)0.75tLF_u/\Omega$
$= 5.00(0.75)(0.0451)(1)(45)(1000)/2.5$
$= 3044$ in.-lb. >> 120.6 in.-lb. **_OKAY_**

Step 3: Screwed Bridging Connection

See Figure B-10. For member sizes not shown, see Figure B-8.

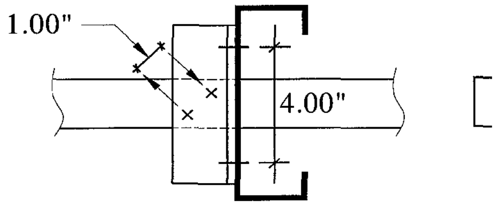


Figure B-10 Screwed Connection

Bridging channel to clip angle screws:

Required shear per screw:

V_{req} = 120.6/1.00 = 120.6 lb.

Allowable shear per screw assuming #10-16

t_1 = 0.0566" F_{u1} = 45 ksi
t_2 = 0.0566" F_{u2} = 45 ksi
d = 0.190"

t_1/t_2 = 1.0 therefore choose the governing P_{ns} from AISI Equations E4.3.1-1, E4.3.1-2 and E2.3.1-3 (AISI 2004c).

$P_{ns} = 4.2(t_2^3 d)^{1/2} F_u = 1109$ lb.
$P_{ns} = 2.7 t_1 d F_{u1} = 1307$ lb.
$P_{ns} = 2.7 t_2 d F_{u2} = 1307$ lb.

Therefore:

P_{ns} = 1109 lb. controls

$V_{all} = P_{ns} / \Omega$
$= 1109/3.0 = 370$ lb. > 120.6 lb.

Clip angle to stud screw assuming #10-16

Required pull-out:

T_{req} = 120.6/4 = 30.1 lb

Allowable pull-out:

From AISI Equation E4.4.1-1

$t_c = t_2 = .0451"$
$F_{u2} = 45$ ksi
$d = 0.190"$

$P_{not} = 0.85 t_c d F_{u2}$
= 328 lb

$T_{all} = P_{ns} / \Omega = 328/3 = 109$ lb

= 109 lb > 30.1 lb ***OKAY***

Example 3

Through – the – Punchout Bridging Design for Mechanically Braced Axially Loaded Studs (Section 2.3)

Required: Design bridging to resist torsion induced in the studs by wind load (AISI Specification D3.2.2) and weak axis buckling of the studs.

Assume: Service Axial Dead Load = 317 lb/stud
Service Axial Live Load = 1583 lb/stud
Service Wind Load = 15 psf
Studs = 600S162-43 @ 12'-0" long @ 24" o.c.

Reference: Cold-Formed Steel Framing Design Guide (AISI 2002e)
Standard for Cold-Formed Steel Framing-Lateral Design (AISI 2004b)

Step 1: Applied Loads

i) Bridging axial load

Assume bracing force equal to 2% of stud axial load

Required bridging axial load = (0.02)(required stud axial load)(number of studs braced)

ii) Bridging major axis moment, M_x

Bridging moment is taken from Figures B-3, B-4, B-5 (Example 2)

Bracing torsional force is defined by AISI Specification D3.2.2 as follows:

$$P_L = 1.5K'wa = 1.5wam/d$$

Where: w = load/ft on the stud = (24/12)(15) = 30 lb/ft
a = bridging spacing = 4.00 ft
m = web center line to shear center = 0.670"
d = 6"

$$P_L = 1.5(30)(4.00)(0.670/6)$$
$$= 20.1 \text{ lb}$$

The moment resisted by the bridging channel is calculated from the flange brace force couple with a lever arm equal to the depth of the stud. (Figure B-6, Example 2)

M = 20.1(6) = 120.6 in.-lb.

The resulting moment values in the outside span are illustrated in Figure B-11.

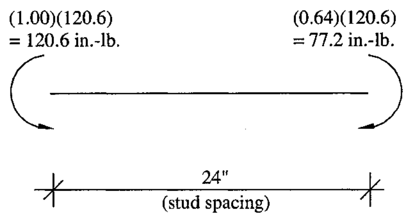


Figure B-11 Bridging Channel Design Moment

iii) Bridging Minor Axis Moment, M_y

Bridging minor axis moment is illustrated in Figure B-12

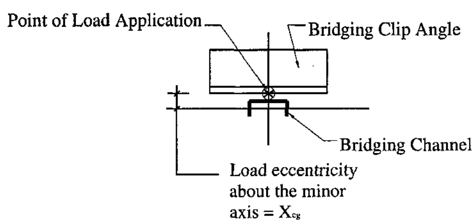


Figure B-12 Bridging Minor Axis Moment

Note: The axial load in the bridging channel accumulates over the number of studs between bridging anchorage points. While each increment of axial load is applied with a minor axis eccentricity, the accumulated axial load is assumed to be concentric.

Step 2: Design Strengths

Try 150U50-54 bridging channel (F_y = 33 ksi)

M_{allow} = 1227.6 in.-lb. (SSMA 2001)

P_{allow} = 671 lbs. (AISIWIN 2003)

Assume bridging will be anchored at 28 feet o.c. (every 14 studs)

P_{brace} = (14)(0.02)(1900) = 532 lbs.

$$\frac{120.6}{1227.6}+\frac{532}{671}=0.89<1.0$$ ***OKAY***

Step 3: Flat Strap X-Bracing

The bridging will be anchored every 14 studs. See Figure B-13 for a suggested anchorage detail using flat strap X-bracing.

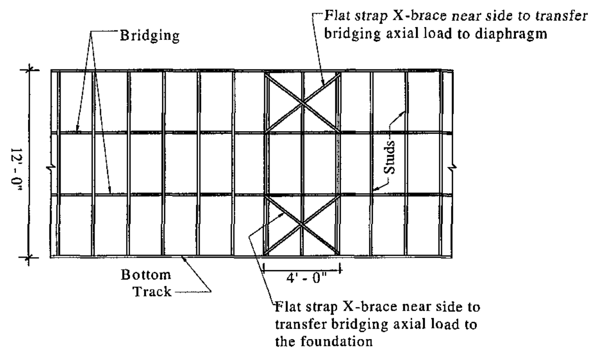


Figure B-13 Bridging Anchorage

P_{brace} = (14 studs)(0.02)(1900 lb) = 532 lb

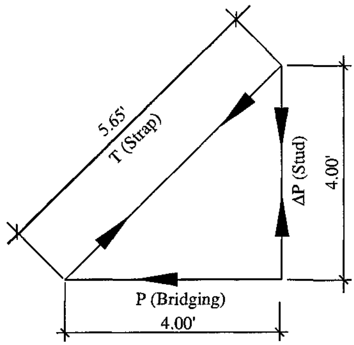


Figure B-14 Flat Strap X-Bracing

i) Flat strap size

$$T_{strap} = \frac{5.65}{4.00} \times 912 \text{ lb} = 1288 \text{ lb}$$

By AISI Specification C2

$$A_{req} = {T_{req}\Omega_t}/{F_y}$$
$$= (1288)(1.67)/33000$$
$$= 0.0652 \text{ in}^2$$

Use 1½" x 0.0451" flat strap with F_y = 33 ksi

$A = 1.50(0.0451) = 0.0676 \text{ in}^2 > 0.0652 \text{ in}^2$ ***OKAY***

ii) Weld required end of each strap

$$V_{all} = {0.75tLF_u}/{\Omega}$$
$$= 0.75(0.0451)(1)(45)/2.50$$
$$= 0.609 \text{ kips/in of weld length}$$

Required length = 1288/609 = 2.11 in.

Use welds illustrated in Figure B-15

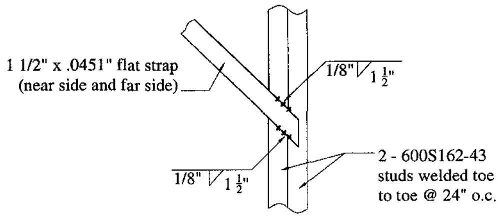


Figure B-15 Required weld at end of each strap

iii) X-bracing vertical reaction

The vertical component of force in the flat straps increases the stud axial load:

Increase in stud force = 912(4.00/4.00) = 912 lbs

The double stud is ***OKAY*** by inspection

iv) X-bracing horizontal direction

The top and bottom tracks and their connections to the diaphragms resist the horizontal component of force in the flat straps – not checked here.

Step 4: Bridging Clip Angle at Double Stud Anchorage Point

In axial load bearing construction to insure a stiff connection detail, use clip angles one thickness heavier than the thickness of the stud. This "rule" is based on engineering judgment and has not been confirmed by test.

Use 1-1/2" x 1-1/2" x 0.0566" bridging clip angles (Figure B-16)

The welds shown in Figure B-16 are ***OKAY*** by inspection

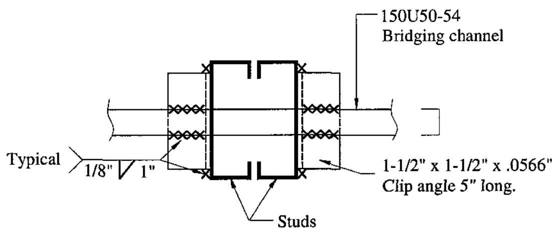


Figure B-16 Bridging Clip Angle at Double Stud Anchorage Point

Step 5: Typical Bridging to Stud Welded Connection
See Figure B-17 for the stud to bridging connection detail.

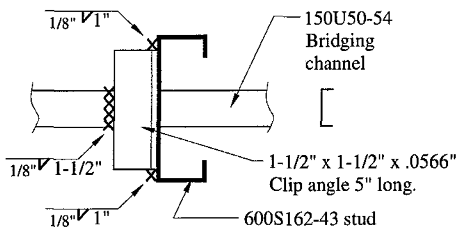


Figure B-17 Bridging to Stud Connection

Maximum twisting moment transferred from the stud to the bridging channel due to wind load:

M_{req} = 120.6 in.-lb.

The axial load transferred from stud to bridging channel

Load Case I   1.0(L+D)

$P_{req} = 0.02P$
$= 0.02(1900 \text{ lb})$
$= 38.0 \text{ lb}$

Load Case II   D+0.75(L+W)

$P_{req} = 0.02(317+0.75(1583))$
$= 30.1 \text{ lb.}$

Step 6: Bridging Channel to Bridging Clip Angle Weld

For the weld shown in Figure B-17, using a linear method of weld analysis:

$S_{weld} = (1/6)(1.5)^2 = 0.375 \text{ in}^2$
$A_{weld} = 1.5 \text{ in}$

Load Case I   1.0 x (L + D)

$M_{req} = 0$
$P_{req} = 38.0 \text{ lb}$

$$q_{req} = \frac{P_{req}}{A_{weld}} = \frac{38.0}{1.5} = 25.3 \text{ lb./in.}$$

Load Case II   D + 0.75(L+W)

$M_{req} = 0.75(120.6) = 90.4 \text{ in.-lb.}$
$P_{req} = 0.02(317+0.75(1583)) = 30.1 \text{ lb}$

$$q_{req} = \frac{P_{req}}{A_{weld}} + \frac{M_{req}}{S_{weld}} = \frac{30.1}{1.5} + \frac{90.4}{0.375} = 261 \text{ lb/in}$$

$q_{req} = 261$ lb/in. controls

$q_{all} = 0.75tLF_u/\Omega$ (AISI Eq. E 2.4-4)
$= 0.75(0.0566)(45)(1000)/2.55$
$= 749 \text{ lb/in} > 261 \text{ lb/in}$ ***OKAY***

Step 7: Bridging Clip Angle to Stud Weld

For the welds shown in Figure B-18, using the linear method:

$S_{weld} = (1/6)(1)^2 = 0.167 \text{ in}^2$

$A_{weld} = 1$ in

The required load on each weld in lb/in. is given by:

$$q_{req} = \frac{P_2 + P_1/2}{A_{weld}} + \frac{(P_1/2)(0.500)}{S_{weld}}$$

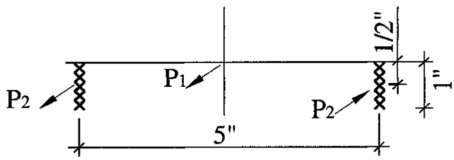


Figure B-18 Welds at end of bridging clip angles

<u>Load Case I</u> 1.0 x (L + D)

$P_1 = 38.0$ lb
$P_2 = 0$

Substituting:

$$q_{req} = \frac{38.0/2}{1} + \frac{(38.0/2)(0.500)}{0.167}$$

$= 75.9$ lb/in

<u>Load Case II</u> D + 0.75(L + W)

$P_1 = 30.1$ lb

$P_2 = 0.75(120.6)/5.00 = 18.1$ lb

Substituting:

$$q_{req} = \frac{18.1 + 30.1/2}{1} + \frac{(30.1/2)(0.500)}{0.167}$$

$= 78.2$ lb/in

$q_{req} = 78.2$ lb/in controls

$q_{all} = 0.75tLF_u/\Omega$
$= 0.75(0.0451)(45)(1000)/2.55$
$= 597 > 78.2$ lb/in ***OKAY***

Example 4

Shear Wall Design - Type I
(Section 2.6.1)

Given: Solid shear wall framed with 362S162-33 studs at 24 inches o.c. sheathed with 7/16 inches OSB on 1 face with #8 screws at 6 inches o.c. at sheet edges, 12 inches o.c. in the sheet field.

Required: Design this Type I solid shearwall to resist a 6000 lb. service load due to wind.

(1) 7/16" OSB on 1 face with #8 screws at 6 inches o.c. at sheet edges, 12 inches o.c. in the sheet field.
(2) Strap Bracing

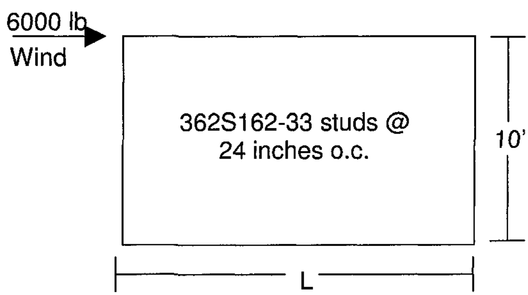
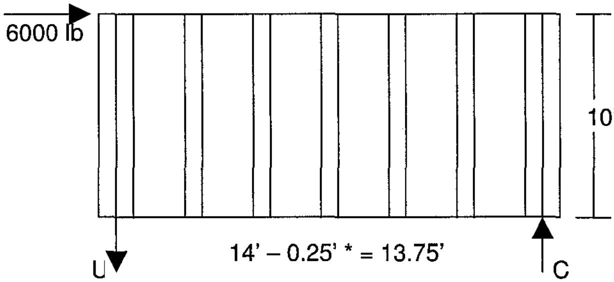


Figure B-19 Type I Solid Shearwall

Reference: Standard for Cold-Formed Steel Framing-Lateral Design (AISI 2004b)

Step 1: Sheathing Approach

V_n = 910 lb/ft (Nominal Strength) (AISI 2004b)
Ω = 2.00 (Factor of Safety for wind) (Note: Ω = 2.50 for seismic forces)
$V_a = 910/2.00 = 455$ lb/ft (Allowable Strength)
$L_{req} = 6000/455 = 13.2$ ft - say 14 ft
$h/L = 10/14 < 2/1$; per limits in AISI (2004b) ***OKAY***



* Assumed centerline dimensions to chord studs.

Figure B-20 Sheathing Braced Shearwall

Calculate End Reactions

$$\sum M_{@U} = 0 = 6000(10) - C(13.75)$$

C = 4364 lb Compression
U = 4364 lb Tension

Select a hold-down anchor to resist minimum force of 4364 lbs. to transfer uplift force into foundation.

Design Compression Chord Studs: (2) 362S162-33 studs back to back, braced at midheight by bridging

P_{allow} = 4972 lb (AISIWIN 2003) ***OKAY***

Step 2: Strap Based Approach

Straps are typically installed at 45° if window or door framing allows.

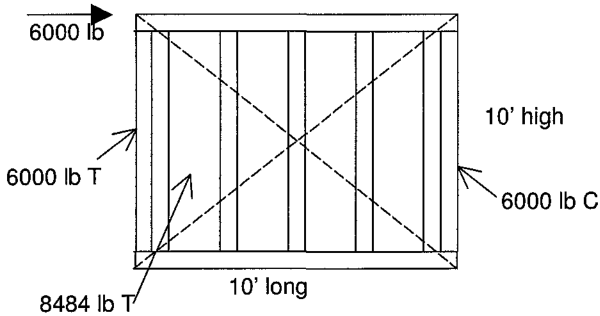


Figure B-21 Strap Braced Shearwall

Flat straps may be on one face or both faces of wall.

Try 68 mil x 8" flat strap (t = 0.0713"), 33 ksi on one face of the wall.

$$T = {A_g F_y}/{\Omega} = {8(0.0713)(33000)}/{1.67}$$

$= 11271$ lb $>$ 8484 lb ***OKAY***

Design chord studs: (2) 362S162-43 studs, back to back, braced at midheight

$P_{allow} = 6755$ lb (AISIWIN 2003) ***OKAY***

Design anchorage of straps to chord studs

8484 lb to 43 mil studs (t = 0.0451")

#10 screw in 43 mil material = 263 lb shear capacity (SSMA 2001)

$8484 \text{ lb}/263 \text{ lb} = 33$ screws in each end of strap ***NOT PRACTICAL***

Try welds: SSMA (2001)
Fillet weld (per inch) = 609 lb allowable on 43 mil, 33 ksi material

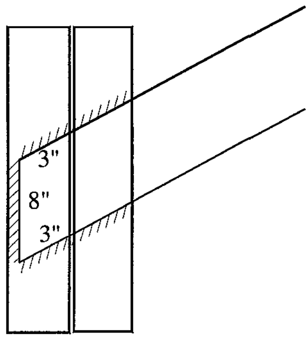


Figure B-22 Welded Connection at End of Strap

Weld Capacity = 14" (609lb/in) = 8526 lb > 8484 lb ***<u>OKAY</u>***

Select a hold-down anchor to resist a minimum force of 6000 lbs. to transfer uplift force into foundation.

Example 5

Shearwall Design - Type II
(Section 2.6.2)

Given: 362S162-33 studs at 24 inches o.c. and 7/16 inches OSB on 1 side of wall with #8 screws at 6 inches o.c. at sheet edges, 12 inches o.c. at sheet field.

Required: Design this Type II (Perforated) shearwall to resist a 6000 lb. service load due to wind.

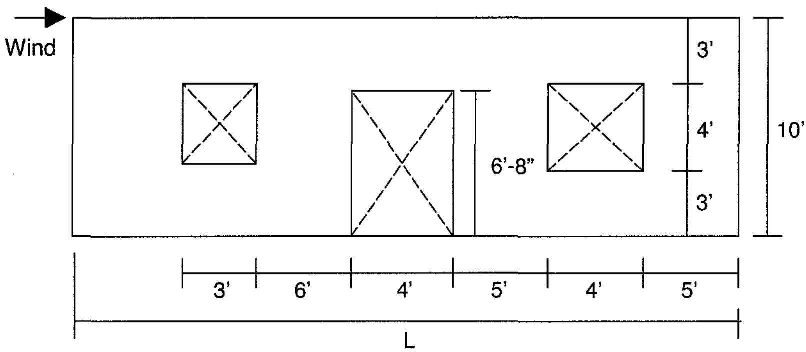


Figure B-23 Type II Perforated Shearwall

Reference: Standard for Cold-Formed Steel Framing – Lateral Design (AISI 2004b)

Step 1: Calculate Geometry of Shearwall

Aspect ratio between openings $10\text{ ft}/6\text{ ft} = 1.67:1$

Maximum Allowed: 2:1 ***OKAY***

Maximum unrestrained opening height for this wall = 3ft + 4ft = 7ft

$$\text{opening}/\text{wall height} = 7\text{ ft}/10\text{ ft} = 0.70$$

Length of unsheathed wall = 3ft + 4ft + 4ft = 11ft

Step 2: Assume L_{min}

Try L = 34ft

Percent of wall with full height sheathing = $\left(34\ \text{ft} - 11\ \text{ft}\right)/34\ \text{ft} = 68\%$

Shear Resistance Adjustment Factor (AISI 2004b)

$C_a = 0.74$ (by interpolation)

$V_n = 910$ lb/ft

$V_a = V_n/\Omega = 910/2.0 = 455$ lb/ft Allowable

$V_{a\ adj} = 455\ (0.74) = 336$ lb/ft

$6000/336 = 17.8$ ft required $<$ 34ft assumed ***OKAY***

Step 3: Calculate Required Holddown Capacity

6000(10') = T(28' - 0.25')

T = 2162 lb required

Select holddown anchors to resist 2162 lbs.

Example 6

Drag Strut and Truss Blocking Design (Section 2.6.2)

Given: 48 foot long wall carrying in-plane load of 6000 lbs due to wind

Required: Design the top track for this wall to function as a drag strut to transfer a 6000 lb. wind force into the shearwall. Wall studs and roof trusses are spaced at 2 feet o.c.

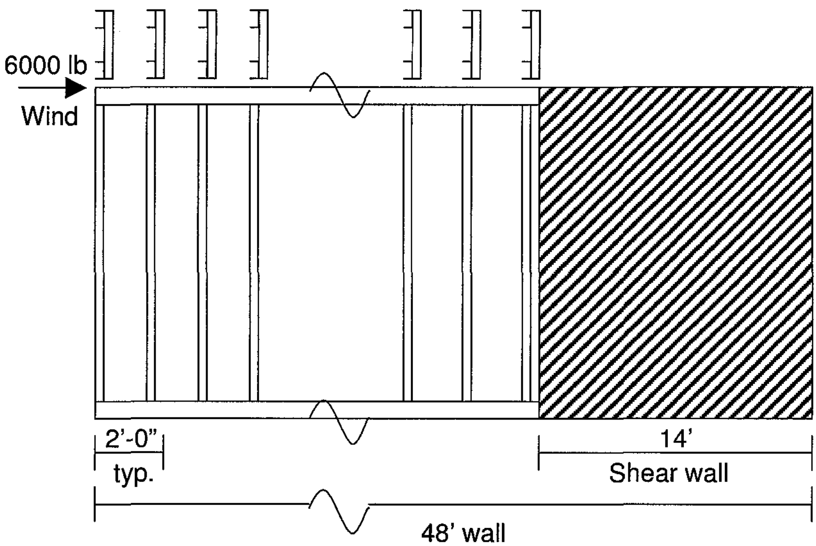


Figure B-24 Wall with Partial Shearwall

Reference: LGSEA Tech Note (556a-4) (LGSEA 1997a)

Step 1: Determine loads

Drag strut load = $6000\text{lb}/48\ \text{ft} = 125$ lb/ft

Shear wall load = $6000\text{lb}/14\ \text{ft} = 429$ lb/ft

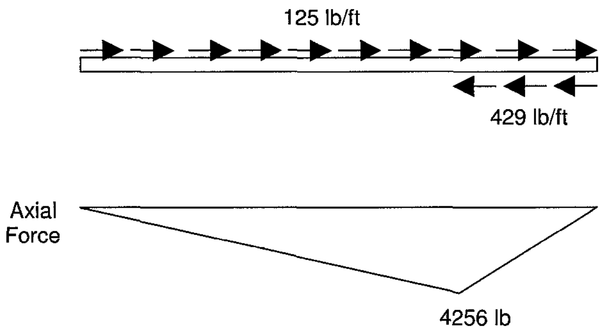


Figure B-25 Forces in Drag Strut

Maximum Axial Force = (429-125) x 14 ft = 4256 lb

Step 2: Determine Required Track Size

Try: 362T125-43 (33 ksi) track - to match studs
Assume unbraced length = 2.0 ft

P_{allow} = 1156 lb (AISIWIN 2003) ***NO GOOD***

Try: 362T125 – 97 (50 ksi) track
Assume unbraced length = 2.0 ft

P_{allow} = 5687 lb (AISIWIN 2003) ***OKAY***

Step 3: Track Splice

Drag strut is of significant length and requires thoughtful detailing, including splices.

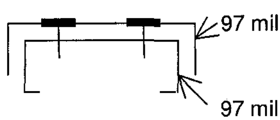


Figure B-26 Track Splice Detail

per SSMA (2001)

#10 screw in 97 mil steel = 1130 lb allowable shear

$^{4256\text{lb}}\!/_{1130\text{lb}} = 3.8$ → Say 4 screws each side of splice

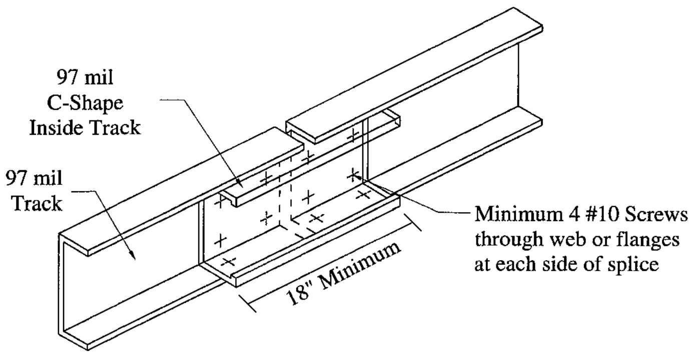


Figure B-27 Track Splice Detail

Step 4: Blocking between Trusses

Assume 2500 lb of 6000 lb is transferred into the wall through the roof diaphragm

14' Shearwall/Trusses at 2' o.c. = 7 spaces

Use 7 shear blocks to match shear wall length and use 43 mil blocking

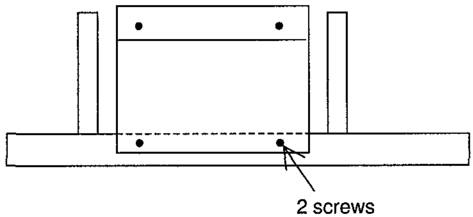


Figure B-28 Shear Block

$2500\text{lb}/7 = 357$ lb required per screw per SSMA (2001)

#10 screw in 43 mil steel = 263 lb allowable shear → use 2 screws minimum per shear block.

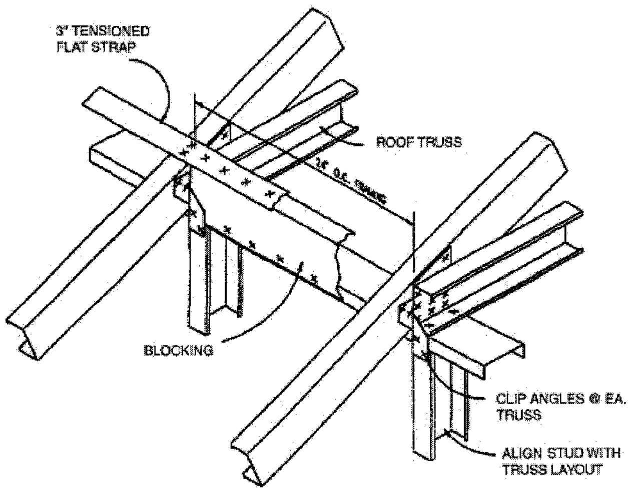


Figure B-29 Typical Shear Blocking Detail (LGSEA 2003)

Example 7

Construction Bracing for Cold-Formed Steel Roof Truss (Section 2.5)

Given: Truss Span = 48 ft

Truss chord forces (from full design loads = 40 psf)

Top chord = 3820 lbs. maximum compression
Bottom chord = 3575 lbs. maximum tension

Top & Bottom chord are 54 mil thickness (t = 0.0566"), 33 ksi material

Capacity of #10-16 x ¾" SDS = 276 lbs. each (33 to 54 mil material) (AISI 2002a)

Required: Design temporary construction bracing for this truss.

Reference: Design Guide for Construction Bracing of Cold-Formed Steel Trusses (LGSEA 1998a)

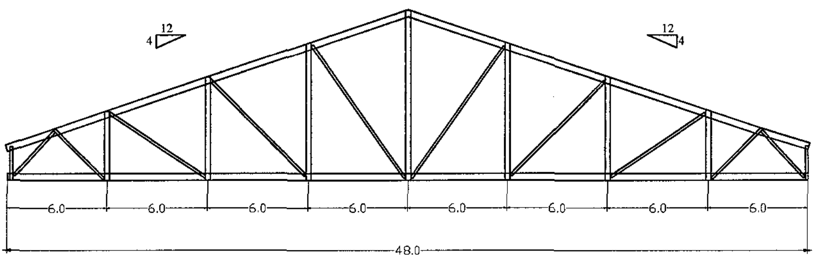


Figure B-30 Truss Layout

Step 1: Determine Construction Load

Use the greater of 5 psf or L/7 psf

Construction design load = 48 ft. span/7 = 6.85 ~ 7 psf > 5 psf
Use 7 psf

Using a design load of 7 psf, re-analyze the truss and determine the maximum allowable unbraced length of the truss top chord, considering the compression force in the chord at this reduced load. For this example, the maximum allowable unbraced length is assumed to be 6 feet at this reduced construction load.

Step 2: Determine Truss Forces

Truss top chord forces (due to reduced construction load)

Top chord: $3820 \text{ lb.} \left({}^{7\text{psf}}\!\big/_{40\text{psf}} \right) = 668 \text{ lb. compression}$

Lateral brace forces: per truss

One-half truss span = 24 feet

${}^{24 \text{ ft}}\!\big/_{6 \text{ ft}} = 4$ lines of bracing

Assume required brace force is 2% of axial force in chord.

(P_{chord}) (2% x number of lines of bracing)

(668 lb)(0.02)(4 lines) = 54 lb per truss (on each slope)

Diagonal brace force (maximum) = ${}^{54 \text{ lb.}}\!\big/_{\cos 45°} = 76$ lb per truss (assuming diagonal brace installed at 45 degrees)

Step 3: Determine Design Load for Diagonal Brace

Try spacing diagonal braces at 20 ft o.c (every 10 trusses)

Maximum diagonal brace force = (76 lb per truss) (10 trusses) = 760 lb.

Quantity of screws required to attach diagonal brace = 760 lb/276 lb= 2.8
- use a minimum of 3 screws

Maximum lateral brace force = (668 lb)(0.02) = 14 lb per truss
(14 lb per truss)(10 trusses) = 140 lbs.
- use 2 - #10 SDS

Design the diagonal brace for a minimum load of 760 lb (not shown)

Summary

Top chord lateral and diagonal braces for this design: (Figure B-31 & B-32)

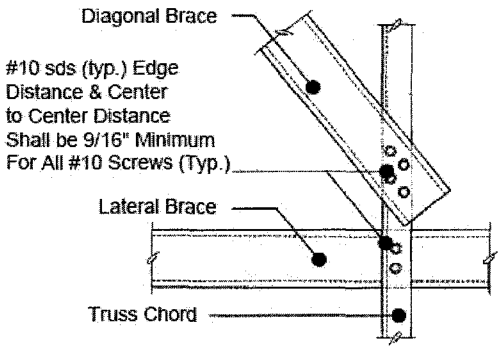


Figure B-31 Typical Brace Connection (LGSEA 1998a)

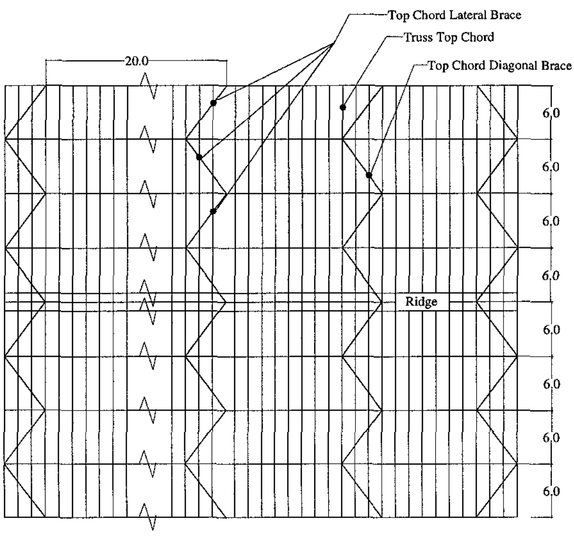


Figure B-32 Plan View of Roof

Lateral braces – space at 10 ft o.c., maximum. Typical conservative practice is to provide a brace near ridge and at the exterior wall. Connect the brace to each truss with a minimum of 2-#10-16 x ¾" SDS. Lap lateral braces a minimum of 2 trusses at all splices.

Diagonal braces – space at 20 ft o.c. maximum. Connect brace to each truss with minimum of 2-#10-16 x ¾" SDS, except at each end of the braces in the outside bay (nearest the exterior wall) where 3-#10-16 x ¾" SDS are used. Do not splice the diagonal braces.

Step 4: Ground Bracing

Supporting trusses with ground bracing prior to the installation of top chord diagonal bracing (Figure B-33 & B-34)

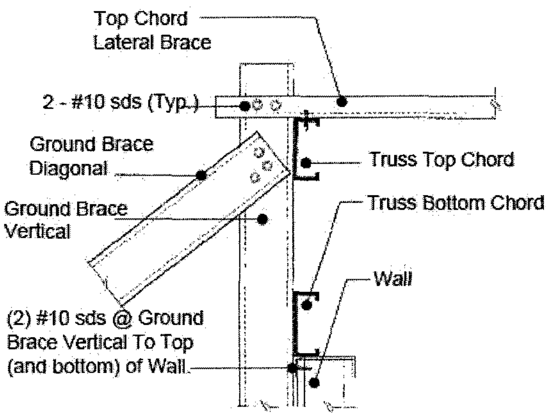


Figure B-33 Ground Brace Connections (LGSEA 1998a)

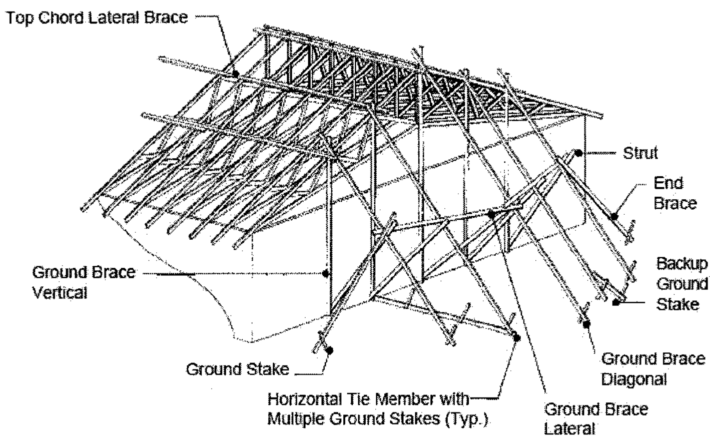


Figure B-34 Ground Bracing (LGSEA 1998a)

Ground Brace Force = (14 lb per truss)(8 trusses)/ $\cos\ 45°$ = 158 lb per brace

Design each ground brace, ground stake, etc. for a force of 158 lb.

Summary

Install ground brace diagonals in line with each row of top chord lateral bracing (angle of the ground brace diagonal should not exceed 45 degrees).

Install ground bracing when the first truss is installed.

Install a maximum of 8 trusses before installing the truss top chord diagonal bracing.

Provide a lateral brace on each diagonal ground brace at mid-span and an end brace at ends of the lateral brace.

Step 5: Bottom Chord Bracing

Bottom chord members are in tension. The same design procedures used for the top chord bracing system could be conservatively used to design the bottom chord bracing system.

Lap all lateral braces a minimum of 2 trusses at all splices.

Example 8

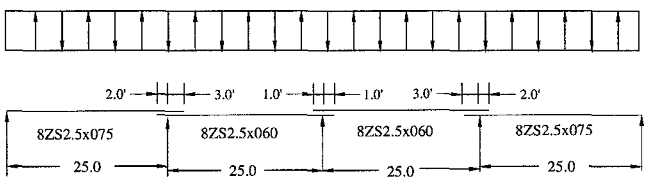
Anchorage Forces in Roof Purlin System
(Section 3.5)

Given: 1. Four span Z-purlin system using laps at interior support points to create continuity
2. Roof covering is attached with through fasteners along entire length of purlins
3. $F_y = 55$ ksi
4. Roof slope = 0.5/12; all purlins face uphill
5. No discrete bracing lines: anti-roll clips at each support at every fourth purlin line

Required: Design anchorages to resist in-plane forces in a metal building roof system.

Reference: AISI Manual of Cold-Formed Steel Design (AISI 2002a)

Dead Load = 15 plf, Roof Live Load = 100 plf, Wind Uplift = 115 plf



For: 8ZS2.5x060
$t = 0.060$ in
$R = 0.1875$ in
$I_x = 8.146$ in^4
$S_f = 2.037$ in^3
$S_e = 1.708$ in^3
$I_y = 1.225$ in^4
$b = 2.50$ in
$d = 8.00$ in

For: 8ZS2.5x075
$t = 0.075$ in
$R = 0.1875$ in
$I_x = 10.10$ in^4
$S_f = 2.524$ in^3
$S_e = 2.195$ in^3
$I_y = 1.510$ in^4
$b = 2.50$ in
$d = 8.00$ in

Step 1: Compute the anchorage forces at the supports

Compute the forces at the supports with anchorage at every fourth purlin ($n_p = 4$)

Multiple-Span System with Restraints at the Supports.

$$P_L = C_{tr}\left[\frac{0.053 b^{1.88} L^{0.13} \cos\theta}{n_p^{0.95} d^{1.07} t^{0.94}} - \sin\theta\right]W \qquad \text{(AISI Eq. D3.2.1-5)}$$

$$\theta = \tan^{-1}(0.5/12) = 2.39 \ \text{degrees}$$

Gravity Case

W = n(L)(DL+LL) = (4)(25)(15+100) = 11,500 lb

End Span:

t = 0.075 in

$$P_L = C_{tr}\left[\frac{0.053(2.5^{1.88})(25\text{x}12)^{0.13}\cos(2.39)}{4^{0.95} 8.0^{1.07} 0.075^{0.94}} - \sin(2.39)\right]11{,}500$$

$$= C_{tr}\,1887 \qquad \text{(AISI Eq. D3.2.1-5)}$$

Interior Span:

t = 0.060 in

$$P_L = C_{tr}\left[\frac{0.053(2.5^{1.88})(25\text{x}12)^{0.13}\cos(2.39)}{4^{0.95} 8.0^{1.07} 0.060^{0.94}} - \sin(2.39)\right]11{,}500$$

$$= C_{tr}\,2438 \qquad \text{(AISI Eq. D3.2.1-5)}$$

At outside supports, $C_{tr} = 0.63$

$P_L = (0.63)(1887) = 1189$ lb

At interior supports, average the contributions from adjacent purlins

At first interior support, $C_{tr} = 0.87$
$P_L = 0.87(1887 + 2438) / 2 = 1881$ lb

At center support, $C_{tr} = 0.81$
$P_L = 0.81(2438 + 2438) / 2 = 1975$ lb ← Worst Case

Wind Uplift Case
W = n(L)(D – W) = (4)(25)(15 - 115) = -10,000 lb vs. 11,500 lb Gravity
By inspection, the gravity case controls.

Step 2: Design Anti-Roll Clip

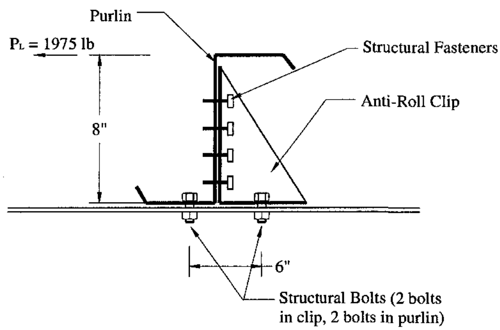


Figure B-35 Roof System Anchorage

$M_{OT} = (1975 \text{ lb})(8 \text{ in.}) = 15800 \text{ in-lb}$

$M_R = (T)(6)$

$15800 = 6T$

$T = 2633 \text{ lb}$ or 1317 lb / bolt

½" A307 bolt in tension = $0.20 \text{ in}^2 \times 20 \text{ ksi}$
= 4000 lb / bolt Allowable

1317 lb < 4000 lb ***OKAY***

Note: Anti-roll clips usually have capacity based on testing, not calculation

Comment

The calculation of anchorage forces is not readily calculatable by first order analysis. The AISI provisions (AISI 2004c) are a simplification based on a regression analysis of test data. Similar forces may be determined from a second order elastic finite element analysis of the roof system which considers flexibilities of the sheeting, purlin cross-sections and anchorages. In practice, however, this is rarely done.

Example 9

Tension Flange of Roof Purlin Braced by Sheathing (Section 3.2)

<u>Given</u>: 1. Four span Z-purlin system using laps at interior support points to create continuity
2. Roof covering is attached with through fasteners along entire length of purlins
3. $F_y = 55$ ksi
4. Roof slope = 0.5/12; all purlins face uphill

<u>Required:</u> Determine if the purlin line possesses adequate strength.

Reference: AISI Manual of Cold-Formed Steel Design (AISI 2002a)

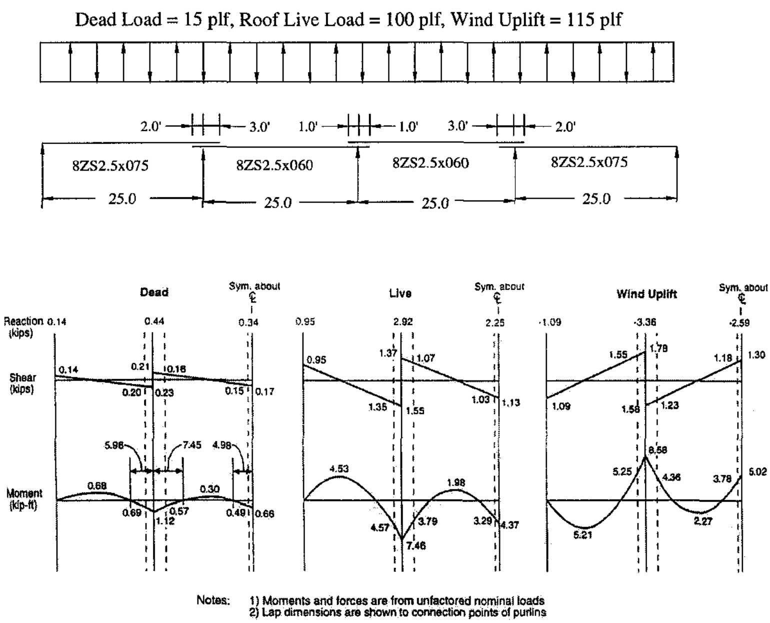


Figure B-36 Shear and Moment Diagrams (AISI 1997d)

For: 8ZS2.5x060	For: 8ZS2.5x075
$t = 0.060$ in	$t = 0.075$ in
$R = 0.1875$ in	$R = 0.1875$ in
$I_x = 8.146\ in^4$	$I_x = 10.10\ in^4$
$S_f = 2.037\ in^3$	$S_f = 2.524\ in^3$
$S_e = 1.708\ in^3$	$S_e = 2.195\ in^3$
$I_y = 1.225\ in^4$	$I_y = 1.510\ in^4$
$b = 2.50$ in	$b = 2.50$ in
$d = 8.00$ in	$d = 8.00$ in

Check the design for uplift loads using ASD method.

Step 1: Calculate Design Uplift Loads

Strength for Bending Only (AISI Section C3.1.3)

Required Strength

By inspection, D + W controls

$M = (M_D + M_W)$

End Span

Moment near center of span:

$M = (0.68 - 5.21) = 4.53$ *kip-ft*

Interior Span

Moment near center of span:

$M = (0.30 - 2.27) = 1.97$ kip-ft

Step 2: Check Allowable Strength

$M_n = R\, S_e\, F_y$ (AISI Eq. C3.1.3-1)

$R = 0.70$ (for continuous span z-sections)

End Span
For t = 0.075 in

$$M_n = (0.70)(2.195)(55) = 84.51 \text{ kip-in} = 7.04 \text{ kip-ft}$$

$$\frac{M_n}{\Omega_b} = \frac{7.04}{1.67}$$

$$= 4.22 \text{ kip-ft} < 4.53 \text{ kip-ft}$$ ***NO GOOD***

Interior Span
For t = 0.060 in

$$M_n = (0.70)(1.708)(55) = 65.76 \text{ kip-in or } 5.48 \text{ kip-ft}$$

$$\frac{M_n}{\Omega_b} = \frac{5.48}{1.67}$$
$$= 3.28 \text{ kip-ft} > 1.97 \text{ kip-ft}$$ ***OKAY***

Other Comments

All other regions of the system have their compression flange braced by the roof panel and would be checked for strength by AISI *Specification* Section C3.1.1. Since the magnitude of the shears, moments and reactions are approximately 65 percent of those of the gravity case, it can be concluded that the design satisfies the *Specification* criteria for uplift.

Example 10

Purlin with One Flange Through Fastened to Sheathing – Strut-Purlin (Section 3.4)

Given: 1. Steel: $F_y = 50$ ksi
2. Span: 25 ft. = 300 in.
3. Section: 8ZS2.5x060
$D = 8.0$ in.
$B = 2.5$ in.
$t = 0.060$ in.
$A = 0.847$ in.^{2}
$r_x = 3.10$ in.

Required: Determine the compression design strength of this purlin braced by sheathing on one flange.

Reference: AISI Manual of Cold-Formed Steel Design (AISI 2002a)

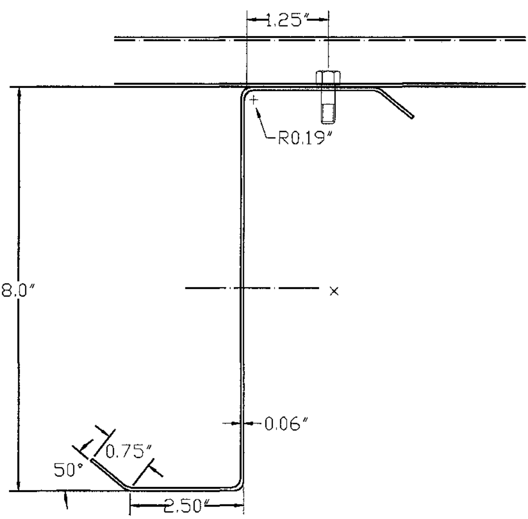


Figure B-37 Purlin with One Flange Through Fastened to Sheathing

<u>Step 1: Nominal Axial Strength, P_n – Flexural Buckling about the X-axis</u>

Note: The sheathing is assumed to restrain Y-axis flexural buckling, allowing for only X-axis buckling.

$K = 1$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad \text{(AISI Eq. C4.1-1)}$$

$$= \frac{\pi^2 (29500)}{(300/3.10)^2} = 31.1 \text{ ksi}$$

$$\lambda^2 = \sqrt{\frac{F_y}{F_e}} \qquad \text{(AISI Eq. C4-4)}$$

$$= \sqrt{\frac{50}{31.1}} = 1.27 < 1.50$$

$$F_n = (0.658^{\lambda_c^2})F_y \qquad \text{(AISI Eq. C4-2)}$$

$$= (0.658^{1.27^2})50 = 25.5 \text{ ksi}$$

$$P_n = A_e F_n \qquad \text{(AISI Eq. C4-1)}$$

A_e is calculated as 0.603 in^2 at F_n = 25.5 ksi per AISI (2004c)

$P_n = (0.603)(25.5) = 15.4$ kips

<u>Step 2: Nominal Axial Strength, P_n – Flexural Torsional Buckling</u>

$$P_n = C_1 C_2 C_3 AE/29500 \qquad \text{(AISI Eq. C4.6-1)}$$

$$x = a/b \qquad \text{(AISI Eq. C4.6-5)}$$

$$= 1.25/2.50 = 0.50$$

$$C_1 = 0.79x + 0.54 \qquad \text{(AISI Eq. C4.6-2)}$$

$$= (0.79)(0.50) + 0.54 = 0.935$$

$C_2 = 1.17\alpha t + 0.93$ (AISI Eq. C4.6-3)

$= (1.17)(1.0)(0.06) + 0.93 = 1.00$

$C_3 = \alpha(2.5b - 1.63d) + 22.8$ (AISI Eq. C4.6-4)

$= 1[(2.5)(2.50) - (1.63)(8.00)] + 22.8 = 16.0$

$P_n = (0.935)(1.00)(16.0)(0.847)(29500)/29500$ (AISI Eq. C4.6-1)

$= 12.7$ kips

Step 3: Governing Limit State is Flexural-Torsional Buckling

P_n = minimum of 12.7 kips or 15.4 kips = 12.7 kips

Step 4: ASD Allowable Design Strength

$P_n/\Omega = 12.7/1.80 = 7.06$ kips

Appendix C

Lateral Bracing of Columns and Beams
By
George Winter

George Winter's paper, "Lateral Bracing of Columns and Beams" (Winter 1958) is considered by many to be the paper that provided the basis for modern design provisions for bracing of beams and columns. Using columns formed from cold-formed steel channels placed back-to-back to form an I-section, and bracing constructed using cardboard, Winter experimentally demonstrated that effective bracing must not only possess a definable strength, but must also possess sufficient stiffness. Using these tests as a basis, Winter developed mathematical models for required bracing strength and stiffness for beams and columns.

Since this paper is not readily available to most engineers, it is reproduced in this report for background information. The version of the paper contained here was published by ASCE as Transactions Paper No. 3044 in 1960.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3044

LATERAL BRACING OF COLUMNS AND BEAMS

George Winter,[1] F. ASCE

With Discussion by Messrs. Anthony A. Chibaro; Giles G. Green; Bruce G. Johnston; Marvin A. Larson; William Zuk; and George Winter

SYNOPSIS

There are many situations in which it is necessary to determine the characteristics required of lateral bracing in order to counteract buckling of columns or beams, or to decide whether a given bracing system is adequate to provide the required lateral support. In this paper a simple elementary method is developed that permits the lower limits of the strength and rigidity of lateral support to be computed in order to provide "full bracing" to columns and beams. "Full bracing" is defined as equivalent in effectiveness to immovable lateral support.

INTRODUCTION

The two kinds of structural bracing are: that provided to resist secondary loads, such as windbracing, and that provided to increase the strength of structural members by preventing them from deforming in their weakest direction. This paper refers only to the latter kind of bracing. Here, again, there are two different types of problems. The first of these is bracing applied to counteract stable but detrimental types of deformation. As examples one might mention beams of channel or Z-shape, loaded in the plane of the web. Such beams, unless properly braced, twist and deflect sideways with consequent loss of strength.[2,3] The second such problem is in regard to bracing applied to prevent buckling in the weakest direction, and thereby to increase the usable strength

Note.—Published, essentially as printed here, in March, 1958, in the Journal of the Structural Division, as Proceedings Paper 1561. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

1 Prof. and Head, Dept. of Struct. Engrg., Cornell Univ., Ithaca, N. Y.

2 "Performance of Laterally Loaded Channel Beams," by George Winter, Warner Lansing, and R. B. McCalley, Research, Engineering Structures Supplement, Colston Papers, Vol. II, London, 1949, p. 179.

3 "Unsymmetrical Bending of Beams with and Without Lateral Bracing," by Lev Zetlin and George Winter, Proceedings Paper 774, ASCE, August, 1955.

of the member. It is this last kind, bracing against buckling, that is the subject of the present investigation.

To resist secondary loads, and to counteract stable but detrimental deformations, it is only necessary to provide bracing of sufficient strength. Given the loads, there is no basic difficulty in computing the internal forces in the bracing system and to dimension it accordingly. In contrast, for bracing against buckling to be effective, it must possess not only the requisite strength but also a definite minimum rigidity. As will be seen, neither the requisite strength nor the necessary rigidity can be computed uniquely except on the basis of assumed imperfections of shape or loading of the member to be braced.

Mention of a few frequent problems of such bracing will show the variety of such situations. To prevent buckling of the compression chord of a roof truss out of its plane, is the strength and rigidity of the roof proper sufficient (steel roof deck, or other planking adequately connected to form a diaphragm), or must special bracing members be provided. For a laced wall column in a mill building, where the outer leg is adequately braced in the plane of the wall, how must horizontal braces be positioned and dimensioned to prevent buckling, parallel to the wall, of the free-standing inner leg. In order for a beam or girder to be fully braced, must its compression chord be embedded in a concrete floor slab. If not, what is the required minimum strength and rigidity of the floor system to provide full bracing. These few cases will suffice to show the problem of bracing against buckling.

Notation.— The letter symbols adopted for use in this paper are defined where they first appear, in the illustrations or in the text, and are arranged alphabetically for convenience of reference, in the Appendix.

CHARACTERISTICS OF EFFECTIVE BRACING

Bracing of this type is of considerable consequence because very large increases in carrying capacity can be achieved by very light and inexpensive bracing. This will be illustrated by selected results from a series of tests that have been reported[4] in detail.

Specimens consisted of two channels cold-formed of 16—gage sheet steel and spot welded back-to-back to form an I-shape 4 in. deep and 2 in. wide. They were tested as columns 12 ft 3 in. long between knife edges. Whereas one of the columns was tested unbraced, for the other identical specimens intermediate bracing was provided at equidistant points. This bracing, at each point, consisted of two strips of thin, 10-ply cardboard, 3/4 in. wide and 2 in. or 15 in. long whose combined tensile strength was 103 lb per pair. Fig. 1 shows one of the studs under test, with four pairs of strips in place, the right part a portion of a failed specimen with two pairs of broken cardboard strips. (These photos happen to show tests where the strips were 1.5 in. wide.) Table 1 gives a representative selection of test results.

These results qualitatively illustrate a number of important points:

1. In these tests the usable strength of the columns could be increased as much as fifteen times by using bracing as weak as that provided by the described cardboard strips.

---

[4] "Light Gage Steel Columns in Wall-Braced Panels," by G. G. Green, George Winter, and T. R. Cuykendall, Bulletin No. 35/2, Engrg. Experiment Sta., Cornell Univ., Ithaca, N. Y.

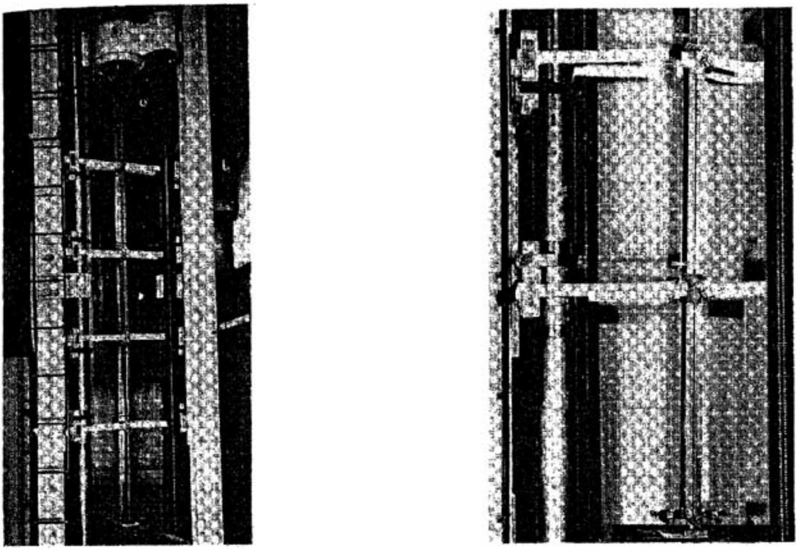


FIG. 1.—TEST OF STEEL STUD BRACED BY CARDBOARD STRIPS

TABLE 1.—TESTS ON CARDBOARD BRACED LIGHT GAGE STEEL COLUMNS.

Test[b]	Cardboard Strips		Measured Ultimate Load, in pounds	Brace Strength, in Percent of Column Strength	Failure[a]
	Number of Pairs	Length, inches			
0	0	—	1098	—	—
4	1	15	4650	2.2	column buckled
8	2	15	8600	1.2	strips broke
9	2	4	9900	1.0	column buckled
15	3	15	10,450	1.0	strips broke
16	3	2	15,200	0.7	strips broke
20	4	15	11,600	0.9	strips broke
22	4	2	17,000	0.6	column buckled

[a] When failure occurred by buckling, the bracing strips remained intact.
[b] Test numbers agree with those used by Green, Winter, and Cuykendall.

2. In those tests in which column failure was accompanied by fracture of the bracing strips, the strength of one brace was of the order of only 1% of the strength of the column.

3. The efficiency of bracing depends not only on its strength but also on its rigidity. In fact, the strength per brace was the same in all these tests, 103 lb. However, because the longer strips possess proportionately larger extensibility, the shorter the strips, the greater the rigidity of the bracing. Comparison of the paired tests 8, 9; 15, 16; and 20, 22 shows, in each case, that the greater rigidity produced the larger column capacity. In addition, tests 8, 9 and 20, 22 illustrate that, other things being equal, the greater the rigidity of the bracing, the smaller the strength required of it to produce a given column capacity. Indeed, in each of these pairs, the softer (longer) bracing broke whereas the more rigid (shorter) bracing not only produced a larger column load but did not break at that load. Although the strips broke in both tests 15 and 16 it is seen that here, too, of two bracings of the same strength, the more rigid one produced the higher column load before failure occurred by fracture of the bracing.

It is possible, in rather elaborate mathematical ways, to investigate this situation accurately by an analysis of elastically supported columns with initial imperfections. On the basis of somewhat approximate assumptions a few special cases of this kind have been analyzed.[5,6] Unfortunately such analyses lead to results far too complex for direct practical application. However, the small magnitudes of both rigidity and strength of the bracing that are sufficient to produce extremely large effects, as illustrated in Table 1, suggest that it is not necessary to compute these two characteristics with great accuracy. Instead, it is sufficient and practical, to devise a simple method which permits: (1) Computing a safe lower limit, rather than an exact value, of the necessary rigidity of the bracing such that the strength of the braced member will attain its maximum possible value; and (2) determining a safe, lower limit, rather than an exact value, of the strength required of a bracing of rigidity equal to or larger than that so computed.

POINT-BRACED COLUMNS

Analysis.—As an acceptable approximation, the performance of practical means of bracing can be regarded as elastic. Consequently, for ideal columns (straight, concentrically loaded, and so forth) the well established theory of columns on elastic supports permits one, for a given column, to determine[4,6,7] the relation between support rigidity (spring constant) and column load, at least for simple cases of equidistant supports. These investigations give no information on required strength of support, because for such ideal conditions, the reaction at the support is zero up to the moment of buckling, and indeterminate when buckling occurs.

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5 "Lateral Bracing Forces on Beams and Columns," by W. Zuk, Proceedings Paper 1032, ASCE, July, 1956.

6 "Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill Book Co., Inc., New York, 1936.

7 "Lateral Buckling of Elastically Braced Columns," by G. G. Green, thesis presented to Cornell University, at Ithaca, in 1948, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

However, for most design situations the question is not that of determining the buckling load for a given rigidity of support. The former more or less steadily increases with the latter up to a certain limit. The question is, much more simply, to determine the minimum rigidity that will make the actually elastic bracing equivalent, in effect, to an unyielding support. Such bracing will evidently produce the maximum column load for the given location of brace or braces and will be called full bracing. From the data of Table 1 it is evident that relatively low rigidities and strengths are required to provide such full bracing. Consequently, any bracing of lesser effectiveness will hardly be less costly, and is therefore uneconomical. For this reason, and in accord with the points previously enumerated, it is the main aim of the present investigation to compute the required characteristics of such full bracing.

If an ideal column with hinged ends is furnished with an unyielding, knife-edge support at midlength (Fig. 2 (a)), it buckles in two half sine-waves. At the intermediate support the second derivative of y with respect to x $y'' = -M/EI = 0$, in which M is the moment of force, I is the moment of inertia, and E is the modulus of elasticity. Consequently, if a real or fictitious hinge were introduced at the support in the continuous column, as shown in Fig. 2 (b), nothing would be changed. In particular, the column, with or without such hinge, buckles at the Euler-Shanley load

$$P_e = \frac{\pi^2 E I}{L^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which L is the unbraced length of the column, P_e is the column load and where, in the elastic domain, E is Young's modulus and in the plastic domain it is the tangent modulus.

If this ideal column is braced by an elastic support and if this support is rigid enough to provide full bracing, the column buckles in exactly the same manner as for unyielding support, and again a fictitious hinge can be introduced at the support (Fig. 2 (c)).

To obtain information for practical design of bracing it is now necessary to investigate real columns. This is necessary because, as previously mentioned, the theory of ideal columns would indicate that supports of infinitesimally small strength are sufficient, which is evidently not a permissible idealization and is at variance with the information of Table 1. Also, as will become apparent, the minimum rigidities computed for full bracing of ideal columns are not sufficient to achieve full bracing of real, that is, imperfect columns. To simplify analysis it will be assumed that actual column imperfections (eccentricity, curvature, and so forth) can be represented by an equivalent, initial crookedness or bow, whose amplitude d_o is small as compared to L.

Such a column, with single elastic support, is shown in Fig. 2 (d). As the column is loaded, the elastic brace with spring constant k deflects together with the column as is shown in broken lines, with a consequent support reaction $F = k\, d$. If full bracing is provided, final failure occurs at a load equal, or almost equal, to P_e, at which load the column snaps into the two-half-wave mode shown solidly. In this case, as before, a fictitious hinge can be introduced at the support with negligible error.

Taking moments about this hinge $M = \frac{F\,L}{2} - P_e\left(d_o + d\right) = 0$. Introducing $F = k\,d$ one obtains the spring constant required to produce full bracing:

$$k_{req} = \frac{2\,P_e}{L}\left[\left(d_o/d\right) + 1\right] \quad \ldots\ldots\ldots\ldots\ldots (2)$$

For an ideal column $d_o = 0$ and one has

$$k_{id} = \frac{2\,P_e}{L} = \frac{2\,\pi^2\,E\,I}{L^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

This value is identical with that given by the exact theory of elastically supported columns, for that amount of rigidity that is required to make an elastic support

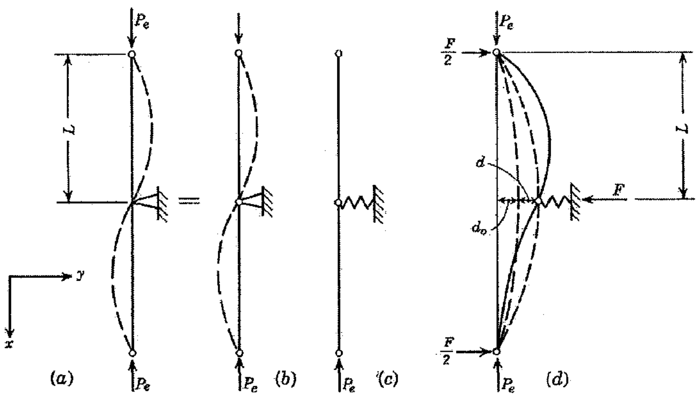

FIG. 2.—IDEAL COLUMN WITH KNIFE EDGED AND ELASTIC SUPPORT

as effective as an unyielding support. It will be observed that the rigidity required to produce full bracing in imperfect columns (Eq. 2) exceeds that required for the ideal column (Eq. 3). The larger the imperfection d_o, the more this is so.

If one knows or estimates the initial imperfection d_o (being guided, for instance, by specified tolerances for crookedness) and if one stipulates the maximum permissible deflection, d, immediately preceding failure, (for example $d = L/500$, conservatively), one can then compute the required spring constant k_{req}, which is a measure of the rigidity of the bracing.

The required strength of bracing (S) is equal to the reaction F at, or immediately preceding, buckling, that is,

$$S_{req} = k_{req}\,d = \frac{2\,P_e}{L}\left(d_o + d\right) = k_{id}\left(d_o + d\right) \quad \ldots\ldots\ldots (4)$$

It can now be said that "full bracing" has been provided if $k_{act} \geqq k_{req}$ and $S_{act} \geqq S_{req}$.

Columns with two and three equal and equidistant supports will be considered briefly, whereupon certain generalizations will become apparent. A column with two equidistant supports, when loaded with P_e so that fictitious hinges can be assumed at these supports, could buckle in one of the two modes shown on Fig. 3 (a) or 3 (b). To decide which of these modes is more unfavorable, that is, which requires the larger support rigidity, k, the case of the ideal column will be investigated first. From Fig. 3 (a) $F\ L - P_e d = 0$ which, with $F = k_{id} d$ gives $k_{id} = \frac{P_e}{L}$. On the other hand, from Fig. 3 (b) $\frac{F}{3} L - P_e\ d = 0$ which, with $F = k_{id} d$ gives

$$k_{id} = \frac{3\ P_e}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

It is seen that k_{id} for the mode of Fig. 3 (b) exceeds considerably that for Fig. 3 (a). This means that if k actually had only the value given for Fig. 3 (a), then the column could not reach the value P_e and buckle in the mode of Fig. 3 (a), because at some lower load it would snap into and buckle in the mode of Fig. 3 (b). It is, consequently, the latter mode that governs and determines the magnitude of k_{id}. The truth of this can also be shown by simple energy considerations or from the complete theory of elastically supported columns summarized in the Appendix. (The method of fictitious hinges for finding k_{id} for full bracing of ideal columns has been used by Fr. Bleich.[8])

To obtain values for k_{req} and S_{req} it is now necessary, as in the previous case, to assume an initially curved shape. Because the ideal column buckles according to Fig. 3 (b), it is evidently most unfavorable if the initial shape happens to be affine to this buckling shape. It is improbable that a fabricated column will be bowed exactly in such a symmetrical S-shape. Making such a simplifying assumption is, therefore, on the conservative side.

Assuming the initial and final shape of Fig. 3 (c), which, for simplicity, only the chords connecting the hinges have been shown instead of the curved column, one has $\frac{F}{3}\ L - P_e\ \left(d_o + d\right) = 0$ and $F = k_{req} d$. Then,

$$k_{req} = \frac{3\ P_e}{L}\ \left[\left(d_o/d\right) + 1\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

and,

$$S_{req} = \frac{3\ P_e}{L}\ \left(d_o + d\right) = k_{id}\ \left(d_o + d\right) \quad \ldots\ldots\ldots\ldots (7)$$

in view of Eq. 4.

Finally, for three intermediate supports it can be shown similarly that of the three possible modes again the zigzag shape in which consecutive supports are displaced in opposite directions requires the largest rigidity of support. This situation is shown schematically in Fig. 4. It is assumed that k is the same for all three supports, but this does not mean that the deflections, d, are the same. Taking moments about hinge a of the forces acting on spans 1 and 2,

8 "Stahlhochbauten," by F. Bleich and Springer, Vol. 1, Berlin, 1932, p. 182.

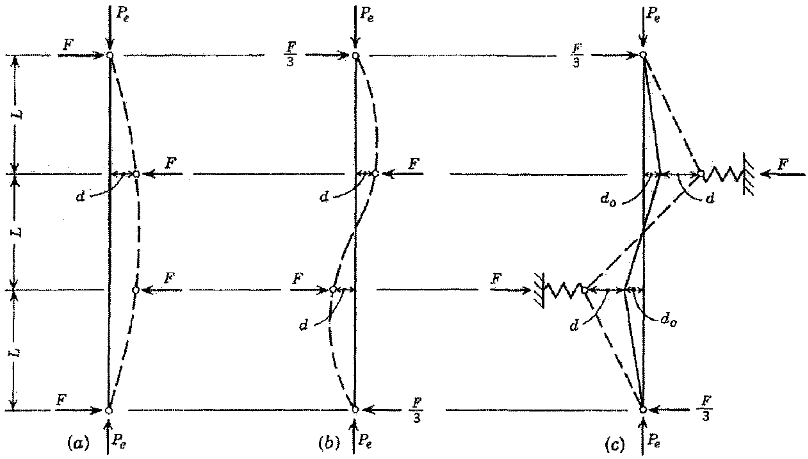

FIG. 3.—COLUMN WITH TWO EQUIDISTANT SUPPORTS

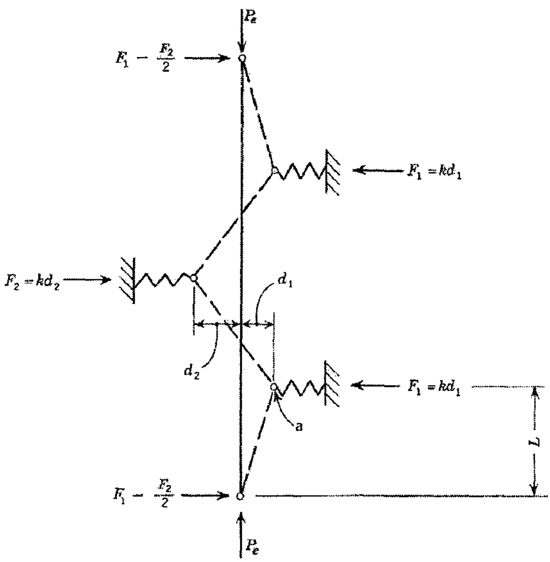

FIG. 4.—COLUMN WITH THREE INTERMEDIATE SUPPORTS

respectively, $P_e d_1 - [F_1 - (F_2/2)]L = 0$ and $P_e(d_1 + d_2) + (F_2/2)L = 0$. Introducing $F_1 = k_{id} d_1$ and $F_2 = k_{id} d_2$, the simultaneous solution of these two equations gives a quadratic equation whose larger root is

$$k_{id} = \frac{3.41\ P_e}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

If this value is substituted in either of the two equations, one finds $d_2 = \sqrt{2}\, d_1$ which defines the ratio of the two deflections in the natural buckling mode.

If, for the same reasons as in the preceding case, one assumes the shape of the initially bowed column to be affine to this buckling mode, that is, if $d_{o2} = \sqrt{2}\, d_{o1}$, then the moment equations about the same hinge a in this column results in

$$k_{req} = \frac{3.41\ P_e}{L}\left[(d_{o1}/d_1) + 1\right] = \frac{3.41\ P_e}{L}\left[(d_{o2}/d_2) + 1\right] \quad \ldots\ldots (9)$$

$$S_{1\ req} = \frac{3.41\ P_e}{L}\left(d_{o1} + d_1\right) = k_{id}\left(d_{o1} + d_1\right) \quad \ldots\ldots (10)$$

$$S_{2\ req} = \frac{3.41\ P_e}{L}\left(\sqrt{2}\ d_{o1} + 2\ d_1\right) = k_{id}\left(d_{o2} + d_2\right) \quad \ldots\ldots (11)$$

Summary and Application.—To summarize, it was found that for n number of equally spaced, equally rigid braces the values of k_{id} are as given as follows:

n =	1	2	3	4
k_{id} =	$\frac{2\ P_e}{L}$	$\frac{3\ P_e}{L}$	$\frac{3.41\ P_e}{L}$	$\frac{3.63\ P_e}{L}$

These values agree identically with the rigorous theory of elastically supported columns. In fact, the value shown for $n = 4$, which has not been computed herein, has been taken directly from that theory.

Furthermore, in each of the analyzed cases it was found that for the initially bowed column

$$k_{req} = k_{id}\left[(d_o/d) + 1\right] \quad \ldots\ldots\ldots\ldots (12)$$

$$S_{req} = k_{req}\, d = k_{id}\left(d_o + d\right) \quad \ldots\ldots\ldots\ldots (13)$$

When the actual rigidity of bracing exceeds the required minimum value $(k_{act} > k_{req})$, as will usually be the case, then the deflection at a load almost equal to that which causes buckling between braces is found by substituting k_{act} for k_{req}. Thus,

$$d = d_o \frac{k_{id}}{k_{act} - k_{id}} \quad \ldots\ldots\ldots\ldots (14)$$

and, in this case, the required strength of the brace is

$$S_{req} = k_{act}\, d = d_o \frac{k_{id}}{1 - \left(k_{id}/k_{act}\right)} \quad \ldots\ldots\ (15)$$

Eq. 12 indicates that it is not sufficient to provide bracing of rigidity k_{id} as is sometimes maintained, because in that case, the slightest initial crookedness d_o would require bracing of infinitely large strength in order for buckling not to occur below the stipulated value P_e.

These equations and values have been derived for prismatic columns with equally spaced and equal braces. This was done primarily for simplicity and also in order to demonstrate the relation of this approach to the exact theory of elastically supported columns. By the same simple means of writing moment equations about the fictitious hinges at the braces it is also possible to analyze the required bracing for columns with unequally spaced supports or with cross-sections that differ in the individual spans. In this case it is merely necessary to introduce for each portion between braces its appropriate P_e depending on the length and cross-section of that portion.

In these cases of continuous columns the characteristics of the bracing so computed are not exact but satisfy the stipulations specified early in this paper to the effect that lower limits rather than precise values of these characteristics are desired. The method is equally applicable to the case of non-continuous columns, that is, to members that at the brace locations are spliced or otherwise connected in a manner approaching a hinge. In fact, because hinges have been assumed at these locations, the analytical results are, if anything, more accurate for these discontinuous cases than for continuous columns.

The required bracing characteristics have been seen to depend, among other things, on the assumed initial bow d_o and on the additional deflection, d, deemed permissible under a load just below the buckling load, P_e. In design work, P_e can be determined from Eq. 1. This becomes difficult when L/r is less than approximately 100, because in that case the variable tangent modulus must be used. Instead, it is satisfactory in all cases to determine P_e as the allowable load multiplied by the safety factor for columns used in the particular design code (approximately 1.9 to 2.0 in the American Institute of Steel Construction (AISC) specification). The value of the imperfection d_o eventually might be specified in codes. Lacking such provisions one can be governed by permissible tolerances. Thus, in various parts of the "Steel Construction Manual" of the AISC tolerances for columns and other compression members being crooked and being out-of-plumb vary from 1/500 to 1/1000 of the length. Assuming additional imperfections, such as eccentricities, it would seem consistent and realistic to specify d as about twice the value, which would be 1/250 to 1/500 of the length depending on the particular type of member. As regards the additional deflection, d, a rigorous requirement would be to make it equal to the assumed value of d_o. Because d is the deflection at incipient failure, this would mean that the deflection under design load would be less than half this value.

Practically, then, for a column of given characteristics and with given location of braces, one determines P_e and one specifies d_o and d from some such considerations as just shown. Then one chooses a k_{id} to match the number of braces, and from Eqs. 12 and 13 one chooses the required rigidity of the bracing and the required strength if the actual rigidity happens to be equal to the required value of rigidity. In most cases the actual rigidity will exceed this value

$(k_{act} > k_{req})$, so that correspondingly smaller actual deflections and required strengths are found from Eq. 15.

CONTINUOUSLY BRACED COMPRESSION MEMBERS

In contrast to members braced at definite intervals there are cases where elastic (or near-elastic) support against buckling is provided continuously along the entire length of the member. A case in point was mentioned in the introduction, namely the compression chord of a roof truss that, without purlins, carries roofing spanning directly from truss to truss. A similar case concerns the bracing effect on the compression chord of open web joists obtained from the floor supported by the joist. As will be seen later, probably the most important application concerns the problem of laternal support for beams and girders.

It is not possible to treat this case by basic methods as elementary as those for point-support. However, the results[6] of the well-established theory of columns on continuous elastic foundation can be approximated by convenient, algebraic expressions which will permit formulating requirements for such bracing in very simple terms.

An ideal elastic column on a continuous, elastic foundation buckles into m number of half-waves, in which the number m depends on the relative rigidities of the column and the foundation, and is obtained from the relation

$$m^2 (m + 1)^2 = \frac{\beta_{id} L^4}{\pi^4 E I} \qquad \text{.............. (16)}$$

in which β is the modulus of the elastic foundation, and the subscript id indicates that the equation refers to the case of the ideal column. The load at which such buckling occurs is given by

$$P_{cr} = P_E m^2 + \frac{\beta_{id} L^2}{\pi^2 m^2} \qquad \text{................ (17a)}$$

in which $P_E = \pi^2 EI/L^2$, and is the Euler load of the same column without elastic support. For one extreme, namely for small values of β_{id}, the column bends into one half-wave and from Eq. 17 (a), in non-dimensional terms

$$\frac{P_{cr}}{P_E} = 1 + \frac{\beta_{id} L^2}{\pi^2 P_E} \qquad \text{................ (17b)}$$

At the other extreme, namely for very large values of β_{id}, the number m becomes so large that 1 (unity) can be neglected compared to it. Hence, an asymptotic relation for large m is obtained from Eq. 16, namely $m \rightarrow \frac{L}{\pi} \sqrt[4]{\frac{\beta_{id}}{EI}}$. For very high foundation rigidity, substitution of this expression in Eq. 17 (a) gives the asymptotic value of the critical load, in non-dimensional terms

$$\frac{P_{cr}}{P_E} \longrightarrow \frac{2}{\pi} \sqrt{\frac{\beta_{id} L^2}{P_E}} \qquad \text{............ (17c)}$$

For the values of m other than 1, or large as compared to 1, an exact value of the critical load can only be obtained by tediously computing m from Eq. 16 and substituting in Eq. 17 (a).

However, the critical load for any value of β_{id} and, consequently, of any m can be obtained with entirely satisfactory approximation from,

$$\frac{P_{cr}}{P_E} = 1 + \frac{\beta_{id}\, L^2}{\pi^2\, P_E} \quad \text{for } 0 < \frac{\beta_{id}\, L^2}{P_E} \leqq 30 \quad \ldots\ldots\ldots (18)$$

and,

$$\frac{P_{cr}}{P_E} = 0.6 + \frac{2}{\pi}\sqrt{\frac{\beta_{id}\, L^2}{P_E}} \quad \text{for } \frac{\beta_{id}\, L^2}{P_E} \geqq 30 \quad \ldots\ldots\ldots (19)$$

It will be recognized that Eq. 18 is identical with the exact Eq. 17(b) (for small values of β_{id}) and that for large values of β_{id} Eq. 17(c) is the asymptotic value of Eq. 19, as it ought to be. Numerical computation shows that the error in determining P_{cr} from Eqs. 18 or 19 is in most cases extremely small and at the worst never exceeds $\pm$ 8% (see Appendix).

By inversion one obtains the following relations which will be useful:

$$\frac{\beta_{id}\, L^2}{P_E} = \pi^2\left(\frac{P_{cr}}{P_E} - 1\right) \quad \text{for } 0 < \frac{\beta_{id}\, L^2}{P_E} \leqq 30 \quad \ldots\ldots (20)$$

$$\frac{\beta_{id}\, L^2}{P_E} = \frac{\pi^2}{4}\left(\frac{P_{cr}}{P_E} - 0.6\right)^2 \quad \text{for } \frac{\beta_{id}\, L^2}{P_E} \geqq 30 \quad \ldots\ldots (21)$$

By analogy with Eqs. 12 and 14, which have been derived for point-supported columns, it will now be assumed without proof that for imperfect columns with continuous elastic support similar relations hold:

$$\beta_{req} = \beta_{id}\,\left(d_o/d + 1\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

or by inversion

$$d = d_o\,\frac{\beta_{id}}{\beta_{act} - \beta_{id}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

These relations assume, as in the case of point support, that the initial shape of amplitude d_o is affine with the buckling shape. The relations are obviously exact in the limiting cases when $\beta_{act} = \infty$ (immovable support) or when $\beta_{act} = \beta_{id}$. For intermediate values the relations are conservative since complete affinity of the initial with the buckling shape is improbable.

With these relations one then obtains the strengths required per unit length of continuous bracing from the following equations. When $\beta_{act} = \beta_{id}$

$$s_{req} = \beta_{req}\, d = \beta_{id}\,\left(d_o + d\right) \quad \ldots\ldots\ldots\ldots (24)$$

and when $\beta_{act} > \beta_{id}$

$$s_{req} = \beta_{act}\, d = d_o \frac{\beta_{id}}{1 - (\beta_{id}/\beta_{act})} \qquad \text{. . . . . (25)}$$

As can be seen these equations are comparable to Eqs. 13 and 15.

Example.—The four-panel top chord of a triangular roof truss is 20 ft long and consists of two lipped, cold-formed $3\frac{1}{2}$ x 4 in., 12 gage channels, connected back-to-back. The relevant properties[9] are: $A = 1.69$, $r_x = 1.38$, $r_y = 1.06$, $I_y = 1.90$ (all in inch units). The truss is perpendicularly braced only at mid-span and carries a roof deck that spans directly between adjacent trusses, without purlins. The deck is suitably connected to form a diaphragm that serves to brace the top chord against buckling perpendicular to the plane of the truss. What are the required characteristics of this continuous bracing? For simplicity bending between panel points due to the uniform roof load will be neglected.

In the plane of the truss, $L/r_x = 60/1.38 = 43.5$. For this value the allowable compression stress, by AISI Specification, is 14.5 ksi. The safety factor for columns in that specification is 2.17 so that the specified ultimate load of the top chord is $P_{ult} = 2.17 \times 14.5 \times 1.69 = 53.1$ k. The Euler load of that chord for buckling out of the plane of the truss is $P_E = \pi^2 \times 29.5 \times 10^3 \times (1.90/240^2) = 9.6$ k. For the bracing to be adequate, the buckling perpendicular to the truss should not occur at a load below that for buckling in the plane of the truss, that is $P_{cr} = P_{ult} = 53.1$ k.

Assuming an initial imperfection $d_o = L/500 = 240/500 = 0.48$ in. and a maximum permissible additional deflection of the same amount immediately before failure (both deflections being perpendicular to the truss), one has from Eq. 21 $\frac{\beta_{id} L^2}{P_E} = \frac{\pi^2}{4}\left(\frac{53.1}{9.6} - 0.6\right)^2 = 60.0 > 30$. Thus, this solution is satisfactory and, $\beta_{id} = \frac{60.0 \times 9600}{(240)\,2} = 10.0$ lb per in per in. Then, from Eq. 22 $\beta_{req} = 10.0\left(\frac{0.48}{0.48} + 1\right) = 20.0$ lb per in per in. For this value β, and from Eq. 24, the required strength of bracing $s_{req} = 20.0 \times 0.48 = 9.6$ lb per in $= 115$ lb per ft. Usually the actual rigidity will exceed the required value considerably. If, for example, $\beta_{act} = 100$ lb per in per in, then from Eq. 25 $s_{req} = 0.48 \frac{10.0}{1 - (10/100)} = 5.33$ lb per in $= 64$ lb per ft.

It is seen that very low values of both rigidity and strength of the diaphragm bracing suffice to stiffen the chord adequately. It should be noted that the very light bracing required in this case is due in part to the favorable relation of r_y to r_x. For less favorable ratios a stronger and stiffer bracing would be needed as can easily be checked by substituting for the top chord a pair of plain channels (without lips) of equal x-axis properties.

LATERAL BRACING OF BEAMS AND GIRDERS

The bracing of compression members is mostly achieved by bracing systems especially provided for the purpose, even though the preceding example shows

[9] "Light Gauge Cold-Formed Steel Design Manual," Amer. Iron & Steel Inst., New York, N. Y., 1956, Table 4, p. 72.

a frequent exception from this situation. In contrast, for beams and girders the most frequent problem is that of deciding whether portions of the structure provided for other purposes give adequate, or full lateral support. Thus, it is sometimes maintained that for a floor to furnish adequate bracing to the beam on which it rests, the compression flange of that beam must be embedded in a concrete slab. The moderate required values of strength and rigidity shown so far suggest that systems weaker than actual embedment in concrete, such as floors or roofs made of a variety of prefabricated elements of lesser rigidity than cast in place concrete slabs may suffice for adequate lateral bracing of beams. Therefore, it is desirable to develop a simple method for deciding whether a lateral system whose characteristics are known with reasonable accuracy provides sufficient bracing for a beam of given properties.

The analysis of the torsional-flexural buckling of ideal beams is considerably more involved than that of the simple, flexural buckling of columns. An analysis of such buckling for elastically supported beams has been developed[5] only for a few special cases and is too complex for design use. However, for determining safe lower limits, rather than exact values, of the required bracing characteristics, as is the stated purpose of this investigation, conservative approximations can be introduced that lead to satisfactorily simple solutions.

It is known[10] that a beam is more stable against lateral buckling than is its compression portion if regarded as an independent strut free to buckle in the direction perpendicular to the plane of the loads. This can be understood qualitatively if one considers that the tension portion resists lateral bending and that, in addition, the torsional rigidity counteracts the twist which accompanies lateral beam-buckling. Consequently, these two influences reduce the buckling tendency of the compression portion and increase its stability as compared to that of the identical but isolated compression member. At the same time numerical computations show that the total force in the compression portion of beams at the instant of lateral buckling, while it is larger than the Euler column load of that portion when isolated, is still of the same order of magnitude. Consequently, if bracing is dimensioned so that it is adequate for the compression portion, assumed to be "cut off" from the tension portion, such bracing obviously will also be adequate, with some added safety, for the actual beam. This added safety, moreover, is hardly economically wasteful because the stability of the beam does not exceed that of the compression portion by a very sizable amount and because relatively weak bracing suffices for full effectiveness.

The following discussion is restricted to I-shaped beams. Rectangular or double-web beams (such as U- or hat-shapes) are so stable that no lateral bracing is required for almost any practical dimensions even though this is not explicitly recognized in some existing design specifications.

It is proposed, then, that bracing requirements for beams and girders be computed in the following manner:

1. Determine the total compression force of the fully braced beam when the allowable stress multiplied by the safety factor is reached in the outer fiber of the beam so braced (incipient failure).

2. By the methods previously developed determine the required characteristics for bracing against column buckling perpendicular to the plane of the loads

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10 "Strength of Slender Beams," by G. Winter, Transactions, ASCE, Vol. 109, 1944, p. 1321.

of the compression portion alone, considered as isolated from the tension portion and as loaded by the above force.

Example.—Determine the bracing characteristics for an 18 WF 50 beam of 30 ft span, uniformly loaded by the floor which it carries, using the AISC Specification (1949). For this beam $I_y = 37.2$ in^4; d/bt = 4.22 in^{-1}; flange area (A_f) = 4.27 in^2; area of half-web (A_w) = 3.02 in^2. Without lateral bracing the allowable stress for this beam if 7.88 ksi which is less than 40% of that for the fully braced condition (20 ksi).

At incipient failure, with the AISC nominal safety factor of 1.65, the outer fiber stress of the braced beam is 1.65 x 20 = 33 ksi. (that is the yield point of ASTM A-7 steel). The total compression force is computed from the usual linear distribution of bending stresses. Neglecting the small difference between the stress at the outer and the inner fiber of the flange, $P_{ult} = 33 \times 4.27 + (33/2) \times 3.02 = 191$ k. The Euler load for the compression half of the beam is $P_E = \pi^2 \times 29.5 \times 10^3 \times (37.2/2) / 360^2 = 41.8$ k. Proceeding now exactly as in the case of the preceding example one finds, assuming $d_o = d = L/500/0.72$ in, $\beta_{id} = 12.5$ lb per in per in; $\beta_{req} = 25$ lb per in per in and for the case that $\beta_{act} = \beta_{req}$ then $s_{req} = 18$ lb per in = 216 lb per ft. The rigidity of even quite flexible floor systems is likely to exceed β_{req} considerably. For example, if one assumes β_{act} = 200 lb per in per in one obtains $s_{req} = 9.6$ lb per in = 115 lb per ft. These, then, are the characteristics which assure bracing adequate for the unreduced bending stress of 20 ksi to be permissible, instead of 7.88 ksi for the case of no, or inadequate, bracing.

A uniformly distributed load of 1280 lb per ft is required to develop this permissible stress or, to put it differently, a load of 1.65 x 1280 = 2120 lb per ft will produce that state of incipient failure or extreme fiber yielding for which the bracing has been proportioned. It is seen that the required lateral support, s_{req}, in this example does not exceed 10% of the vertical load. Hence, mere friction between the top of the beam and the floor which rests on it would suffice to develop the force required for lateral bracing. Although this is not to suggest that such friction should actually be relied upon, the weakest connection which is practically feasible, say 1/2 in bolts at 3 ft spacing along the span, would develop a multiple of the required holding force.

SAMPLE CHARACTERISTICS OF A FLOOR SYSTEM

From 1956 to 1959 a sizable number of tests were made at Cornell University, at Ithaca, by A. H. Nilson on full-size cellular steel panel floors to ascertain their effectiveness as diaphragms, primarily for wind and earthquake bracing. The characteristics measured in these tests are, of course, the same ones which matter if such floors are to act as bracing for the supporting beams. To give some indication of actual values of horizontal strength and rigidity for floors of this kind it will suffice to quote and interpret the results of one of the weakest systems tested in that series.

Omitting irrelevant details, the test set-up is schematically shown in plan on Fig. 5(a). Three panels of steel framework, 10 ft by 12 ft each, were erected in the laboratory. Eight short stub columns, all placed on rollers, carried the shown system of beams (12 WF 27 longitudinally and 10 WF 21 transversely). The two outer transverse beams were anchored at their ends A and D. Hydraulic jacks with load cells were placed at B and C to apply horizontal forces to the inner transverse beams. Horizontal deflections were measured at E and F. Because the longitudinal beams were not continuous across the columns, the framework itself was so flexible horizontally that by pushing at B or C by hand a deflection of an inch and more could easily be produced. Cellular steel floor panels of one of the widely used makes were then erected, simulating standard practice. The individual panels, 2 ft wide, are shown in dashed lines. These panels

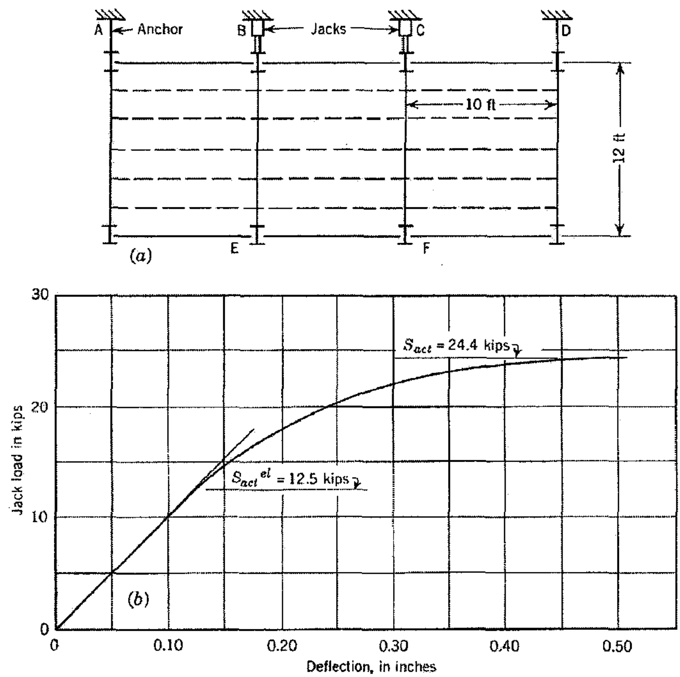

FIG. 5.—FULL SCALE TEST ON CELLULAR STEEL FLOOR

were 3 in deep, of 16 gage steel, and were welded to the beams. In the particular test being discussed, adjacent panels, along their hook joints, were connected to each other by crimping at 2 ft intervals by means of a clinching tool. Tests have shown that diaphragms of this type, together with the surrounding beams, act somewhat like plate girders, and develop considerable horizontal strength and rigidity.

Loads on jacks B and C were kept equal throughout the test, and horizontal deflections at E and F proved equal within a few thousandths of an inch. The load-deflection diagram is given on Fig. 5(b). It is seen that it is practically linear up to a load, per jack, of 12,500 lb, at which load the deflection was 0.125 in. Consequently, over the linear range the spring constant was k_{act} = 100,000 lb per in. The diaphragm failed under a horizontal load per jack, S_{act} = 24,400 lb. Even if, conservatively, one were to define the useful strength as the limit of the elastic range, one would have $S^{el}_{act} = 12{,}500$ lb.

When, in another test, the usual $2\frac{1}{2}$ in. concrete fill was applied over the steel deck, the rigidity increased to $k_{act} = 1{,}150{,}000$ lb per in, the ultimate strength to $S_{act} = 57{,}000$ lb, and the limit of the elastic range to $S^{el}_{act} = 24{,}000$ lb.

To see the significance of these values, a floor system will be assumed identical with that tested, except that a 30 ft girder from A to D will be assumed to carry the transverse beams BE and CF without intermediate support (on a 30 ft span) and that similar transverse beams frame into that girder from the other side. A simple check will show that the 18 WF 50 section of the preceding example is then satisfactory to carry, for such a system, a total floor load of 100 psf.

The girder, then, is stiffened at B and C by the transverse beams which, in turn, are supported against motion in direction of their axes by the strength and rigidity of the diaphragm formed by the panels which they support. The unsupported length of the girder is now reduced to 10 ft, resulting in $Ld/bt = 507 < 600$, so that by present AISC specifications the unreduced stress of 20,000 psi is permissible. Making use of the data of the preceding example, the girder is then to be braced for a force $P_{ult} = P_e = 191$ k. From the previously tabulated formulas for k_{id}, for $n = 2$, $k_{id} = (3 \times 191{,}000)/120 = 4770$ lb per in. Assuming again $d_o = d = L/500 = 360/500 = 0.72$ in, one has from Eq. 12 $_{req} = 4770$ $[(0.72/0.72 + 1)] = 9540$ lb per in and if, by chance, $k_{act} = k_{req}$, then from Eq. 13 $S_{req} = 9540 \times 0.72 = 6860$ lb. However, the actually measured rigidity was $k_{act} = 100{,}000$ lb per in so that, from Eq. 15, for this rigidity $S_{req} = 0.72$ $\dfrac{4770}{1 - (4770/100{,}000)} = 3620$ lb.

A comparison of these required values with the corresponding quantities obtained in the test with bare deck shows that the actual rigidity (k) was approximately ten times the required amount, and the actual usable strength S^{el}_{act} approximately three and five tenths times the required amount. It is seen, then, that this laterally relatively weak and flexible floor system supplies a multiple of the support required in this example for full lateral bracing. An even greater reserve is obtained after the usual concrete fill has been applied.

It should be observed that all the foregoing determinations have been made for the state of incipient failure, rather than for design loads. This is necessary because some of the involved phenomena are non-linear. It follows from this that the required bracing characteristics, so computed, already involve a safety factor which is, in fact, greater than that implied in the pertinent general design specification. This is so because the proposed analysis, as repeatedly emphasized, is based on a number of conservative assumptions. Hence the computed, required properties (k_{req}, S_{req}, and so forth) refer to the actual minimal requisite values and are not in need of further multiplication by a safety factor.

RESERVATIONS AND LIMITATIONS

It has been suggested that the equivalent initial crookedness, duly related to applicable straightness tolerances, might be assumed of the order of L/250 to L/500 and that the additional deflection under a load just below failure might, conservatively, be taken as equal to this amount. The three numerical examples have been computed on that basis. Whereas it is believed that these quantities are reasonable, it is evident that they should be related to imperfections and, implicitly, to safety factors assumed in the relevant design specifications. The quoted, specific values should, therefore, be understood rather as samples subject to more detailed determination by those responsible for the design specification.

The proposed method for bracing of beams and girders utilizes the proposition that a beam is more stable than its compression portion if the latter were isolated from the rest of the member. Although, this is true in the majority of cases, contrary situations may exist,[10,11] for instance, when the center of gravity of the load is located at a level very considerably above the top flange of the beam (As an illustration, consider the unlikely case of an entirely detached brick wall supported on the top flange of an unbraced girder.) In these cases, for the same reasons as stated, the usual, simplified design code provisions for lateral buckling are also inapplicable.

For the time being the proposed method should be applied only in connection with elastic design, because it has been developed to provide adequate support up to loads which cause either incipient yielding or buckling in a plane perpendicular to the bracing. It is known,[12] in regard to plastic design of steel structures, that the ultimate loads predicted by that method will not be reached unless the structures are adequately braced at least at those locations where yield hinges form. This condition is not included in the foregoing development. Hence, when applied to plastic design the proposed method of bracing analysis may require modifications which are not a subject of this paper.

CONCLUSIONS

Experimental evidence has been cited to show that very light lateral bracing (small strength and small rigidity) suffices to produce very large gains in carrying capacities of columns and beams.

A simplified analysis of initially imperfect columns with one or more intermediate supports leads to a method which permits analysis by the simplest of computations to determine lower limits for the strength and rigidity required to produce full bracing. By using two simple equations to express the results of the rigorous theory of columns on elastic foundations, a similarly simple method has been developed for determining the bracing characteristics for cases of continuous lateral support.

By an approximate method, which errs on the conservative side, the problem of laterally supported beams is reduced to that of laterally supported columns. By this means a simple procedure is developed for determining minimum characteristics for lateral support of beams and girders.

Sample computations and comparison with bracing characteristics determined in full-size tests on a light floor system show that for beams and girders lateral

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11 "Lateral Stability of Unsymmetrical I-beams and Trusses in Bending," by G. Winter, Transactions, ASCE, Vol. 108, 1943, p. 247.

12 "Behavior of Welded Single-Span Frames under Combined Loading," by C. G. Schilling, F. W. Schutz, Jr., and L. S. Beedle, Welding Research (Welding Journal, Vol. 35), 1956, p. 234.

support much lighter than that provided by flange embedment in a concrete floor is generally adequate to furnish sufficient bracing.

APPENDIX

Throughout this paper repeated reference has been made to the results of the rigorous analysis of ideal columns on elastic supports. To elaborate briefly on those features, these results are given here in a graphical non-dimensional presentation,[4,7] (Fig. 6). The value P_E used in the non-dimensional abscissa is the Euler load of the identical, full length column without intermediate supports. Consequently, this definition is identical in the elastic range with P_E as used before for continuous support in Eqs. 17 to 19. For point support, the value P_e, (Eq. 1,) had been referred to the free length between supports. Hence, for n number of intermediate equidistant point supports the relation between P_e and P_E is

$$P_e = P_E (n + 1)^2 \qquad (26)$$

For a column elastically supported at n equidistant points it is seen that the critical buckling load increases with increasing support rigidity k. For very low

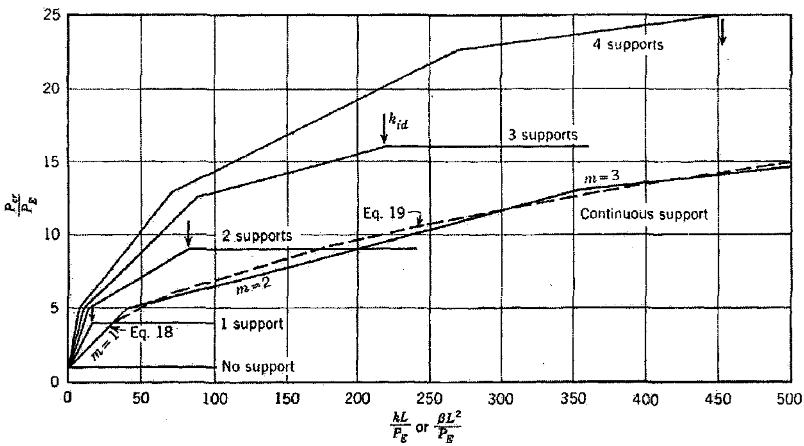

FIG. 6.—CRITICAL LOADS FOR ELASTICALLY SUPPORTED COLUMNS

values of k the column buckles in a single half-wave (m = 1). The larger k is, the larger the number, m, of half-waves and the larger P_{cr} is, up to that value of k at which the number of half-waves becomes equal to the number of free lengths between point-supports, (i.e. $m_{max} = n + 1$). For any magnitude of k equal to or larger than this particular value the column buckles as if it were held immovably at the intermediate supports, in the manner shown on Fig. 2 for n = 1. For such elastic support, then, the ideal column has attained its maximum possible carrying capacity, which represents full bracing by the definition of this paper. These particular minimum values of k which produce full bracing by the rigorous theory are identical with those values k_{id} previously tabulated, which have been determined by elementary means in this study. They are shown on Fig. 6.

For continuous, elastic foundation the solid curve of Fig. 6 gives the results of the rigorous theory and indicates, again, that as the rigidity of the foundation increases, so does the number of half-waves m as well as the theoretical buckling load P_{cr}. Of the two expressions Eqs. 18 and 19 used in this study to represent the relationship between P_{cr} and β_{id}, Eq. 18 is identical with the exact relation over this range. The second, approximate expression, Eq. 19, is shown by the broken curve. Entirely satisfactory agreement for engineering purposes, is seen to be obtained throughout, with maximum deviations of about $\pm$ 8% confined to a few short regions.

APPENDIX. NOTATION

d = amplitude of movement of member under load;

d_o = amplitude of bow in "real" member;

E = modulus of elasticity;

F = reaction;

I = moment of inertia (rectangular);

k = spring constant, measure of rigidity;

M = moment of force;

P = load on member;

P_E = Euler load of column without elastic support;

P_e = Euler load (buckling load on column);

S = strength of individual bracing;

V = total shear; and

β = modulus of elastic foundation.

Appendix D

Industry Resources

American Institute of Steel Construction

AISC
One East Wacker Drive
Suite 3100
Chicago, IL 60601
www.aisc.org

American Iron and Steel Institute

AISI
1140 Connecticut Ave.
Suite 705
Washington DC 20036
www.steel.org

Canadian Sheet Steel Building Institute

CSSBI
652 Bishop St. N.
Unit 2A
Ontario, Cambridge, Canada
www.cssbi.ca

Center for Cold Formed Steel Structures

CCFSS
Dept. of Civil, Architectural, and
Environmental Engineering
University of Missouri – Rolla
Rolla, MO 65409-0030
campus.umr.edu/ccfss

Light Gauge Steel Engineers Association

LGSEA
1201 15th Street N.W.
Suite 320
Washington DC 20005
www.lgsea.com

Metal Building Manufacturers Association

MBMA
1330 Sumner Avenue
Cleveland, OH 44115
www.mbma.com

Rack Manufacturers Institute

RMI
8720 Red Oak Blvd.
Suite 201
Charlotte, NC 28217
www.mhia.org

Steel Deck Institute

SDI
P.O. Box 25
Fox River Grove, IL 60021
www.sdi.org

Steel Framing Alliance

SFA
1201 15th Street N.W.
Suite 320
Washington DC 20005
www.steelframing.org

Steel Stud Manufacturers Association

SSMA
8 S. Michigan Ave.
Suite 1000
Chicago, IL 60603
www.ssma.com

Appendix E

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Index