CHAPTER 2

COLD-FORMED STEEL DESIGN

1. INTRODUCTION. There are three design methods available to the designer: direct application of the AISI Specification, prescriptive methods, and testing of designed assemblies. The AISI specification can be used for the bulk of design calculations when selecting beams, columns, web stiffeners, and effective section properties. AISI has guides for shear walls and cold-formed steel trusses. These design guides can be used for Corps of Engineers projects, but are limited to all steel designs. The use of plywood and oriented strand board are not allowed in permanent Military design. When a designed assembly does not fall within the limits of these design guides, those assemblies must be tested. The prescriptive method that follows should only be used within its stated limits, and was created to serve the single family housing industry.

2. PRESCRIPTIVE METHODS.

a. General. One method of designing one and two family homes is to use the Prescriptive Methods for residential Cold-Formed Steel Framing, by the U.S. Department of Housing and Urban Development and the National Home Builders Association. The limitations for the prescriptive method of design are summarized in the table below and apply to residential type construction. Building loads are determined by using the ASCE 7-93 Minimum Load Assumption for Buildings.

Table 2-1. Summary of Prescriptive Methods Limitations

Web depths		Deflection Limits	
Bearing Walls	Heights	Walls	Remarks
100 mm (4")	2.4 to 3.0 m (8 to 10 ft)	L/240	Total Load
150mm (6")	2.4 to 3.0 m (8 to 10 ft)	L/360	Live Load
Joists/Rafters	Spans	Joists	
300mm (12")	2.4 to 4.9m (8 to 16 ft)	L/240	Total Load
350mm (14")	2.1 to 6.4m (7 to 21 ft)	L/480	Live Load
400mm (16")	3.9 to 7.9m (13 to 26 ft)	Rafters	
Ceiling Joists	Spans	L/180	Total Load
100mm (4")	2.4 to 4.9m (8 to 16 ft)	L/240	Live Load
150mm (6")	2.7 to 7.0m (9 to 23 ft)	Ceiling Joists	
200mm (8")	3.0 to 7.9m (10 to 26 ft)	L/240	Total Load
250mm (10")	3.7 to 8.2m (12 to 27 ft)	Headers	
300mm (12")	3.7 to 8.8m (12 to 29 ft)	L/240	Total Load
Flange 41 mm (1.625") Lip 13 mm (0.5")	Max House Width 11 m (36 ft)	L/360	Live Load

Notes:
1. Self-drilling and tapping screws: SAE J78, use a minimum edge and end distance of 3 diameters, a minimum of three exposed threads, and components in contact with each other, no gaps between connected parts. In connections that are in shear only, a minimum edge distance of 1.5 diameters in the direction of the force shall be used.

2. Dimensions in this table were originally in U.S. Customary units. Light gage steel materials are available as soft metric equivalents.

b. Material Thickness. Designers need to specify the Design or Nominal thickness of the material required. However, the minimum thickness of material that can be delivered to the job site is 95% of the design thickness specified, see Table 1-1. When using this method of specifying materials the designer will always get a material thicker than the minimum shown in the Table 1-1, and the design factor of safety, Ω and the resistance factor, ϕ accounts for this difference in thickness. Then the material should be coated with a minimum of ASTM A294, G60 galvanized coating for corrosion protection. The following is a listing of the material's design thickness used in the Prescriptive Methods code. Also, this code has developed a standard method for marking each stud. The industry identifier of 350S162-068 is read as a 39 mm (3.50 ") Stud with a 41 mm (1.62 ") flange and a material thickness of 1.81 mm (68 mils). In general studs used in this code are the same overall dimensions as their wood stud counterpart.

- Bearing Walls: 0.84, 1.09, 1.37, 1.72, 2.45 mm (33, 43, 54, 68, 97 mils)
- Nonstructural Walls: 0.46, 0.68 mm (18, 27 mils)
- Joists/Rafters: 0.84, 1.09, 1.37, 1.72, 2.45 mm (33, 43, 54, 68, 97 mils)
- Ceiling Joists: 0.84, 1.09, 1.37, 1.72, 2.45 mm (33, 43, 54, 68, 97 mils)
- Strapping: 0.84 mm (33 mils)

c. Engineered Portions of the Prescriptive Code. When using the Council of American Building Officials (CABO) Prescriptive Code (National Association of Home Builders report) there are many conditions that need to be checked by the design engineer. When using the Prescriptive method, the design engineer shall check the selection of the buildings main structural members, as this method uses building loads from ASCE 7-93. The design loads will be upgraded to the most current ASCE 7 standard. The following components are engineered portions of the prescriptive code.

- Sheathing selection and design when over 145 Km/hr (90 mph) winds and seismic zones 3 and 4
- Hybrid systems that use steel and wood
- Wall bracing, hold-downs and uplift straps for Winds loads greater than 145 Km/hr (90mph), and for Seismic Zones 3 and 4
- Pneumatically driven fasteners, powder actuated fasteners, crimping, and welding
- Overhangs, balconies, and decks with live loads greater than 1.92 KPa (40 psf)
- Floor joist splices and the design and bracing of Cold-formed Steel floor trusses
- In-line blocking every 3.7 m (12 ft) on strap braced studs
- The approval of corner framing details
- Steel strapping and 'X' bracing
- Cathedral ceilings
- Wood rafters
- Ceiling joist and rafter splices
- Steel and/or wood roof trusses and associated bracing systems
- King stud and wall stud uplift straps and end gable uplift straps
- Load carrying track members

3. COLD-FORMED STEEL FRAMING.

a. General. There are many differences when steel designers start using lightweight steel framing systems. Generally cold-formed sections are shaped and formed from flat sheets, their original mechanical properties are changed during cold-forming, there are no standard shapes, the thickness of materials are generally less then 3 mm (1/8 inch) thick, and the predominate failure mode is buckling followed by a post-buckling strength increase.

b. Design Guidance. Guidance from the American Iron and Steel Institute (AISI) applies to materials with a thickness that is less than 5 mm (3/16 inch) thick for connections, and plate up to 25 mm (1 inch) can be cold-formed. Guidance from the American Institute of Steel Construction (AISC) applies to thicker materials. When using cold-formed stainless steel, design requirements can be found in American Society of Civil Engineers (ASCE) criteria. The 1996 AISI specification covers material usage, loading combinations (ASCE 7), the structural analysis of elements, members, and assemblies, the design of connections and joints, and the testing for special cases.

(1) Section A, General Provisions. This section discusses the limits of applicability, materials, loads allowable stress design (ASD), load resistance factor design (LRFD), strength increase due to cold working, and serviceability.

(2) Section B, Elements. This section discusses dimensional limits, the effective widths of stiffened and unstiffened elements, and stiffeners. This analysis considers the flat widths and thickness of the flange, the web, and the lip, along with the effects of any intermediate stiffeners. The element analysis is used to determine the effectiveness of the element and the design stress level for that element.

(3) Section C, Members. This section goes into the calculation of section properties, the design of tension members, flexural members, concentrically loaded members, members in combined axial and bending, and cylindrical tubular members. In Figure 2-1 is shown the common section symmetries used in cold-formed steel design. Section symmetries are defined by using the relative positions of the center of gravity (CG) and shear center (SC) of the sections geometry. Sections having separate locations for the CG and SC are singly symmetric. When they are collocated at the same point they are doubly or point (Z sections of equal flange width) symmetric.

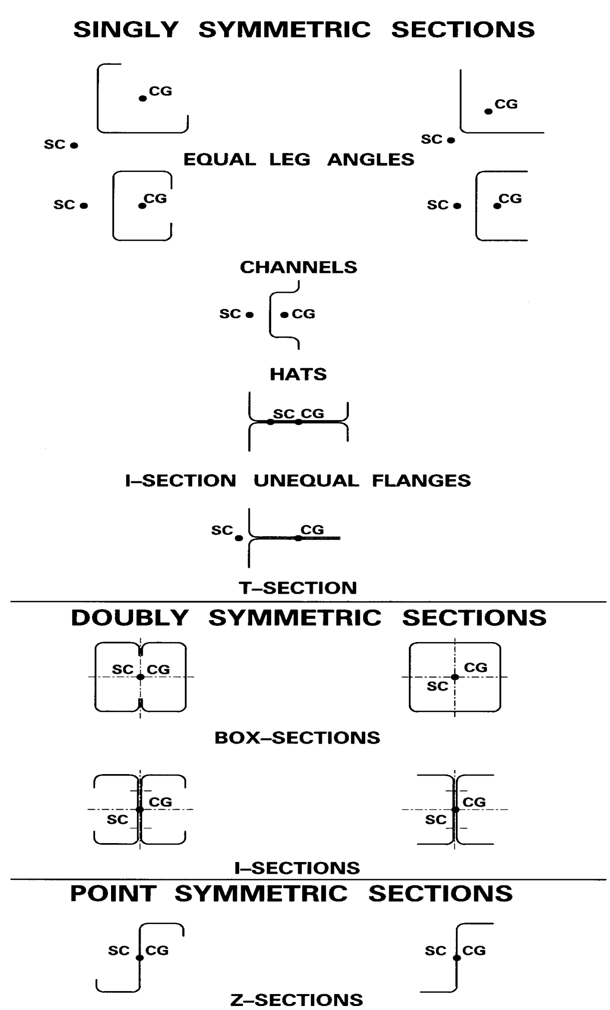

Figure 2-1 Typical Cold-Formed Section Symmetries

(4) Section D, Structural Assemblies. This section deals with structural assemblies. This section includes discussions on built-up sections, mixed systems, lateral bracing, wall stud systems, and floor, roof or wall steel diaphragm construction.

(5) Section E, Connections and Joints. This section covers the design of connections and joints, and includes welded connections, bolted connections, screw connections, shear rupture, and connection to other materials. The provisions for screw connections is new with this edition.

(6) Section F, Test for Special Cases. This section discusses testing methods and procedures for the conditions not specifically covered in the specification. This would include such tests for determining structural performance, confirming structural performance, and determining mechanical performance. Design guidance for shear walls will be performed in accordance with chapter 3 of this manual.

c. Wall Studs and Roof Trusses. Wall stud and roof framing sections are typically made from C shaped sections. These wall stud sections should always have a stiffening lip to aid in the development of the flange flat width

Table 2-2. AISI Approved Steels (Section A3.1)

Steels that are allowed and are not shown must comply with the ductility requirements of Sections A3.2 and A3.3. The typically provided by the manufacturer is Other Steel in the AISI Specification and requires the use of material coupons to verify material properties.		
ASTM A653, Grades **33**, 37, 40, **50** **(Most common)**	F_y =228 to 345 Mpa (33 to 50 ksi)	F_u = 310 to 483 MPa (45 to 70 ksi)
ASTM A653 Grade 80 (Deck and Panels)	F_y = 552 MPa (80 ksi)	F_u = 565 MPa (82 ksi)
ASTM A500 Tubes only	F_y =228 to 345 MPa (33 to 50 ksi)	F_u = 310 to 427 MPa (45 to 62 ksi)

d. Effective Width. The major cold-formed steel design concept is the “Effective Design Width” of flat elements. These flat elements can be stiffened, partially stiffened, or unstiffened. Using a bent edge that can form a lip as on a C or Z section stiffens flat elements. Also, using an intermediate crease in the middle of the flat element on the top of a hat section, or using another flat element such as the sides of a hat section. When the stiffening element is not fully effective such as an overly long lip on a C or Z section the flat element is partially stiffened. An unstiffened element would be a C or Z section that does not have an edge lip. Cold-formed steel sections have very high width to thickness ratios which means elements within the section are susceptible to local buckling. Therefore, only that portion of the section that is capable of taking load is included in the effective width of the flat element. Currently, only the AISI approach to determine effective section properties is allowed, other methods that have been used include finite strip analysis and finite element methods.

e. Advantages of Cold-Formed Steel. The advantages of cold-formed steel systems occur because of several unique material characteristics such as: the recyclable nature of the material, low weight, high strength, custom shapes, rounded corners with no fillets, post buckling strength, and element stiffening characteristics.

f. Limit States. Designers should recognize the two limit states used in the AISI specification: the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS). The first limit state is a function of strength, the second is structural function. AISI has very limited coverage of serviceability limit states since they are different for each owner, user, and designer, see Paragraph 4.j. for further guidance on live load deflection limits. Typical design considerations for a member in bending would show how these two limit states are used. The strength limit states for beams would include: flexure, shear, web crippling, and shear lag, while the serviceability limit state would include deflections and flange curling. Many times section will look distressed, buckled or distorted and still have a great deal of strength remaining. Therefore, serviceability limit states are set to reduce distortion effects.

g. Design Thickness. Designers need to calculate the sectional properties using the design or nominal thickness of the material. The AISI design equations do account for the minimum or delivered thickness difference. When using the AISI specification the equation for calculating the nominal strength of the section are the same for Allowable Strength Design (ASD) or Load Factor Resistant Design (LRFD) methods. The method of calculation of section properties is unique to the AISI specification as effective sections properties are used in many of their procedures. Thin materials and high strength steels combine to make sections that are subject to not only local buckling, lateral buckling, but also lateral-torsional buckling, and therefore, sections are not always fully effective. Non-fully effective sections need to have their effective section properties calculated. This calculation is a iterative process for C and Z sections, when the neutral axis is located nearer to the tension flange of the section, and the section properties are calculated based on the compression flange yielding first. The effective section is based on the effective flat widths of the compression-flange, the flange-lip, and the section's web. All sections are to be designed to develop their full strength using an all steel design. This means that all cold-formed steel sections are to be braced with steel to prevent lateral or lateral-torsional buckling created by lateral or twisting loads. Sheathing and gypsum wall board are not allowed to brace the stud section. When the section is fully effective the nominal moment capacity is equal to the fully effective section modulus times the yield strength of the steel. Typically, cold-formed sections are too thin to develop plastic sections and cannot redistribute plastic moments. Also, web crippling is usually required at concentrated loads and at beam supports, and web reinforcement may be required around pipe openings. Around these areas of the beam designers should check the requirements for beam web stiffening with reinforcement plates at pipe opening or web crippling at supports and concentrated loads as necessary.

4. DESIGN OF STRUCTURAL ELEMENTS

a. AISI Specification. The AISI Specification for the Design of Cold-Formed Structural Members-1996 applies to the design of all cold-formed steel members.

b. Preliminary Member Selection. A good report to use when designing cold-formed steel sections is the AISI Report CF 93-1, "Preliminary Design Guide for Cold-Formed C and Z Members", June 1993. This report uses Gross section properties and reduced stresses to size cold-formed sections. It has in it design procedures for strength in bending, strength in shear, strength in combined bending and shear, strength in web crippling, strength of concentrically loaded compression members, and strength in combined axial and bending. Designers using this procedure will normally be conservative by 5 to 15%.

c. Element Behavior. Designers will quickly learn about the characteristics of thin compression elements, elastic buckling, and post buckling strength. Within the AISI criteria stresses are referred to as flat plate elements and plate stresses, these are in reference to the flat rolled elements of the cross section being analyzed. Designers should note that AISI assumes that flat plate stresses are assumed to be uniform across the plate element's width as the plate reaches the critical plate buckling stress, which has a nonuniform distribution. Sine wave ripples along the length of the member characterize buckling of a thin compression element. Thin plate elements go into an elastic buckling mode when the longitudinal plate stresses exceed $f_{cr,}$ the critical buckling stress of the element as calculated by Equation 2-1. As flat elements buckle the plate stresses are redistributed to the stiffened edges of the plate, and edge stresses approach the yield stress of the steel. Therefore, the effective width is a function of the design stress. This redistribution effect was developed by Mr. George Winter at Cornell University, and has been used by the airline industry since the 1940's. This is the basis of the effective width concept used to design cold-formed steel section. The constant λ (lambda) as shown in Equation 2-2 is used to calculate the effective width of the flat element. Values of λ less than or equal to 0.673 indicate a fully effective element, otherwise they are less than fully effective. While such an effect may appear to be detrimental there can still be a significant amount of strength remaining. The elastic critical buckling factor, k of a stiffened element can increased by a factor of 9.3 over an unstiffened element.

$$f_{cr} = \frac{k\pi^2 E}{12\left(1-\mu^2\right)\left(\frac{w}{t}\right)^2} \qquad \text{(Eq 2-1)}$$

$$\lambda = \left(\frac{1.052}{\sqrt{k}}\right)\left(\frac{w}{t}\right)\sqrt{\frac{f}{E}} \qquad \text{(Eq 2-2)}$$

when

$$\lambda \leq 0.673... \Rightarrow b = w \qquad \text{(Eq 2-3)}$$

when

$$\lambda > 0.673... \Rightarrow b = \rho w \qquad \text{(Eq 2-4)}$$

$$\rho = \frac{\left(1-\frac{0.22}{\lambda}\right)}{\lambda} \qquad \text{(Eq 2-5)}$$

Where:

- λ = an element slenderness factor
- k = a plate buckling coefficient = 4 for a stiffened element, and 0.43 for an unstiffened element
- w = the flat width of the element
- t = the thickness of the element
- f = the design stress in the element determined at the Nominal Moment (M_n) based on the effective section properties
- f_{cr} = the critical elastic buckling stress for the plate
- E = the modulus of elasticity of the element
- ρ = an effective width factor
- μ = Poisson's ratio (0.3 for steel) in the elastic range

d. Element Slenderness. A review of the slenderness factor shows the relationship of the width to thickness ratio to design stress. Very thin members are less effective at higher stresses. The AISI specification limits element w/t ratios to be less then 60 for flanges that are stiffened with simple lips (C and Z sections), and flanges that are stiffened with elements that are stiffened (hat sections) to less than 500. When elements have a w/t ratio greater than 30 and 250 they will display a noticeable waviness before reaching their design capacity, this does not effect the members final design capacity.

e. Simplified Section Properties. A simplified approach to calculating section properties uses the centerline of each section element. Section properties are calculated by using the corners and flat element dimensions. When calculating the moment of inertia the corners are so small they can be ignored. When a lip is used to stiffened an element the moment of inertia of the edge stiffener must be greater than the required moment of inertia per AISI Section B2, *Effective Widths of Stiffened Elements*. When the neutral axis is closer to the tension flange or at mid-depth on a symmetrical section, the design stress in the tension flange is at yield, and λ is calculated based on the compression stress in the flange. Should the neutral axis be closer to the compression flange than the solution becomes iterative to locate the neutral axis, compressive stress, and λ.

f. Members.

(1) Properties of Sections, Section C1. Typically full cross section properties are used except where reduced or effective element widths are required. A computer program is needed to efficiently calculate effective section properties

(2) Tension Members, Section C2. Common rules for hole placement and largest hole reduction is in AISI specification. Currently there are no shear lag provisions in the specification. However, there will be in the future manual. Also, A307 Bolts are commonly used in Cold-Formed Steel design.

(3) Flexural Members, Section C3.

(a) Strength for Bending Only, Section C3.1. Reference Yura on member bracing. For screw down roof systems, brace the C's and Z's at 1/3 points for uplift loads only.

(b) Strength for Shear Only, Section C3.2

(c) Strength for Combined Bending and Shear, Section C3.3.

(d) Web Crippling Strength, Section C3.4.

(e) Combined Bending and Web Crippling Strength, Section C3.5.

(4) Concentrically Loaded Compression Members, Section C4. Torsional-Flexural Buckling is unique to cold-formed columns and beam-columns. and the Factor of Safety for Columns is equal to 1.80. Torsional-Flexural Buckling controls most columns.

(a) Sections Not Subject to Torsional or Torsional-Flexural Buckling, Section C4.1.

(b) Doubly or Singly Symmetric Sections Subject to Torsional or Torsional – Flexural Buckling, Section C4.2

(c) Nonsymmetrical Sections, Section C4.3.

(d) Compression Members Having One Flange Through-Fastened to Deck or Sheathing, Section C4.4.

(5) Combined Axial Load and Bending, Section C5.

(a) Combined Tensile and Axial Load and Bending, Section C5.1.

(b) Combined Compressive Axial Load and Bending, Section C5.2

(6) Cylindrical Tubular Members, Section C6.

(a) Bending, Section C6.1.

(b) Compression, Section C6.2

(c) Combined Bending and Compression, Section C6.3.

g. Wall Studs and Wall Stud Assemblies. Section D4. Use only all steel designs. Therefore only paragraph (a) is allowable. The effective area, A_e and the nominal buckling stress, F_n are to be calculated in accordance with Section B.

(1) Wall Studs in Compression. Section D4.1. Studs need to be checked for column buckling between fasteners (a) and flexural and or torsional column buckling (b). Paragraph (c) is not to be used since it deals with sheathing for bracing. Studs used in wall sections will be firmly placed in the track prior to attachment of the stud track units. No gaps will be allowed between the stud web/flange and the track being assembled.

(2) Wall Studs in Bending. Section D4.2. Wall studs should always have stiffened or partially stiffened compression flanges. Ignore the values shown for unstiffened compression flanges. The provisions of Section C3.1 apply to the bending strength of the member except for Section C3.1.2 Lateral Buckling Strength. Calculations should be based on stiffened and partially stiffened compression flanges when determining the nominal moment capacities M_{nxo} and M_{nyo}. Anchor bridging to solid blocking is critical

(3) Wall Studs with Combined Axial Load and Bending. Section D4.3. Interaction equations for this loading condition are in Section C5.

(4) Floor, Roof or Wall Steel Diaphragm Construction. Section D5. This section provides design values for various diaphragm conditions.

h. Design Guide for Cold-Formed Steel Trusses - AISI RG-9518. Trusses will be designed using the following guidance.

(1) Member ends will be assume to be pinned.

(2) Webs of members are to be pinned.

(3) Unbraced lengths will be:
(a) Continuous chord members,
- K_x, K_y, K_t = 0.75 when appropriate sheathing is attached to the top and bottom flange of those chords.
- K_x, K_y, K_t for other conditions = unity or 1.0.

(b) Compression web members K = 1.0.

(c) End moment coefficient, C_m = 0.85, and strength amplification factor, C_b per AISI C3.1.2

(d) Compression chord with combined axial and bending use section C5 at panel points Equation C5.2.1-2, and between panel points equation C5.2.1-7.
Where:
L_x, L_t = panel length
L_y = distance between sheathing connections

i. Shear Wall Design Guide - AISI RG-9604 and Chapter 3 of this manual.

(1) Braced shear wall systems will be designed for strength and stiffness as follows. The greater requirement will control the size of the bracing system. Also, the designer will consider the seismic bracing guidance in Chapter 3 of this technical instruction.

(2) Strength requirements for a brace force of, F_{br} as follows for ASD or LRFD design methods:

$$F_{br} = 0.004P\left(L/B\right) \qquad \text{(Eq 2-6)}$$

Where:

F_{br} = Brace force in consistent units.
P = Share of the gravity load supported by the shear wall frame. Should two frames support the entire floor load P is ½ the floor load.
L = Length of diagonal brace.
B = Width of the frame bent.

(3) Brace Stiffness will meet the following requirement:

$$A_b = \frac{C(P)(L^3)}{E(h)(B^2)} \qquad \text{(Eq 2-7)}$$

Where:

A_b = Cross sectional area of the brace in consistent units.
C = Constant: ASD = 4, LRFD = 2.67.
P = Share of gravity load supported by the braced frame for lateral stability.
L = Length of diagonal brace.
E = 29,500 Ksi, or 203 395 MPa.
h = Height of braced frame.
B = Width of braced frame.

j. Serviceability Deflection Limits

(1) Live Load Deflections. Deflection criteria for buildings under load are generally established to ensure functional performance and economy of design. Many of the deflection limits used today have been established based on past performances and the perception of the occupants in a building. A consideration for deflection limits is often tied into the performance of the attached claddings. A stiffer cladding requires a stiffer backup to prevent damage to the cladding. In the case of floor joists, the limit may be established to reduce vibrations (see paragraph 2.4.m) which are unacceptable to the occupants. In any case, consideration should be given to all serviceability issues. The following list is a compilation of suggested deflection limits by manufacturers, trade groups and building officials that may be applied to light gauge metal framed structures. The list below assumes limits at full dead plus live load; full dead plus snow; or full dead plus wind load unless noted otherwise.

Exterior wall finishes

Brick veneer over sheathed framing	L/600
Stucco over sheathed or open framing	L/360
Exterior Insulation Finish Systems	L/240
Cement board sheathing over framing	L/360
Stone (marble, granite, limestone)	VERIFY WITH STONE SUPPLIER
Plywood and Oriented strand board	L/240
Gypsum sheathing over framing	L/240
Metal or vinyl siding over sheathed framing	L/240

Interior wall finishes

Plaster	L/360
Ceramic tiles	L/360

Gypsum drywall	L/240
Wood finishes	L/240

Floors

Residential	L/360 total load; L/480 live loads
Office and storage	L/240 total load; L/360 live loads

Roofs

Trusses	L/240 total load; L/360 live load
Rafters	L/180 total load; L/240 live load

(2) Drift Limits. See Chapter 3 for drift limits guidance when using cold-formed steel shear walls. Deflection states for buildings broadly encompass a variety of design considerations the designer should be aware of but are not covered in this brief synopsis. The deflection limits stated above are the limit states for various building elements under load. There are also limits for lateral deflections (story drifts) due to wind or seismic loading of the building, these are best determined by review of the applicable model building code. Another consideration for the designer is long term deflection (creep). While this often does not effect the design, creep should be considered when a large proportion of the total load is do to permanent (dead) loads.

k. Continuous Beams and Joists. *"Effective Lengths for Laterally Unbraced Compression Flanges of Continuous Beams Near Intermediate Supports"* by J. H. Garrett, Jr., G. Haaijer, and K. H. Klippstein, Proceedings, Sixth Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla / American Iron and Steel Institute

l. Effect of Holes. Proposed additions to the Specification for the Design of Cold-Formed Steel Structural Members and accompanying Commentary, Sections B2.4, C3.2.2, and C3.4.2, based on University of Missouri-Rolla, Department of Civil Engineering, Reports on Behavior of Cold-Formed Steel Sections with Web Openings, by Roger A. LaBoube. Table 2-3 shows the typical dimensions of holes that can be placed in various size joist members.

Table 2-3: Pipe Openings: Maximum Pipe Opening and Web Reinforcement

Joist Size with 41.3 mm (1-5/8") Flange mm (in)	"d" Max hole Diameter mm (in)	"b" Plate Size mm (in)	"a" Screw Hole Spacing mm (in)	"c" Edge Distance mm (in)	"A" Min Distance to Concentrated Load or Support
292 (11.5)	175 (6-7/8)	229 (9)	67 (2-5/8)	14 (9/16)	L/6
235 (9.25)	133(5-1/4)	229 (9)	57 (2-1/4)	24 (15/16)	L/25
203 (8.0)	108 (4-1/4)	178 (7)	48 (1-7/8)	17 (11/16)	L/16
184 (7.25)	108 (4-1/4)	178 (7)	48 (1-7/8)	17 (11/16)	L/16
152 (6.0)	89 (3-1/2)	133 (5-1/4)	37 (1-7/16)	12 (15/32)	L/10

Notes:
1. L = joist span.
2. b = reinforcement plate dimensions for a square plate.
3. a = spacing between screw holes that attach the reinforcement plate to the joist.
4. c = Minimum edge distance of screws from the edge of the reinforcement plate.

m. Floor Vibrations. Light weight floors and short spans can lead to vibration problems in floor systems. The following methods uses the dimensions and cross-sectional properties of the floor and the predicted central floor deflection. This procedure was developed by Kraus and Murray for residential floor systems.

(1) Calculate the critical central floor deflection, y_{crit} (in) from Onysko's Criterion (Onsyko 1995):

$$y_{crit} = \frac{37.32}{L^{1.3}} \quad \text{(Eq 2-8)}$$

Where:

L = Floor span in inches

(2) Calculate the predicted deflection of a single joist, Δ_{ot} due to a 255 lb concentrated load at midspan:

$$\Delta_{ot} = \frac{255L^3}{48EI} \quad \text{(Eq 2-9)}$$

Where:

L = Floor span in inches.
E = Modulus of Elasticity of the joist.
I = Moment of Inertia of a single joist.

(3) Calculate the number of effective joist, N_{eff} from the Steel Joist Institute (SJI) equation:

$$N_{eff} = 1 + 2\sum\left(\cos\frac{x\pi}{2x_o}\right) \quad \text{(Eq 2-10)}$$

Where:

x = Distance from the center joist to the joist under consideration (inches).
x_o = Distance from center joist to the edge of the effective floor = 1.06eL (inches).
L = Joist span (inches).
e = $(D_x/D_y)^{0.25}$
D_x = Flexural stiffness perpendicular to the joist = $E_c t^3/12$
D_y = Flexural stiffness parallel to the floor joists = EI_t/S
E_c = Modulus of elasticity of the sub-flooring.
E = Modulus of elasticity of the joists.
t = sub-flooring thickness.
I_t = Moment of inertia of joists alone.
S = Joist Spacing.

(4) Calculate the predicted central floor deflection, Δ_o:

$$\Delta_o = \frac{\Delta_{ot}}{N_{eff}} \quad \text{(Eq 2-11)}$$

Where:

Δ_o = Deflection of floor at mid-bay.
Δ_{ot} = Deflection of a single joist due to 255 lb. concentrated load at midspan.
N_{eff} = Number of effective joists in the floor system.

(5) Compare the Δ_o value to the critical deflection, y_{crit}:

If	$\Delta_o < y_{crit}$:	Acceptable
If	$y_{crit} < \Delta_o \le 1.1\ y_{crit}$:	Marginal
If	$\Delta_o > 1.1\ y_{crit}$:	Unacceptable

5. FASTENERS AND CONNECTIONS

a. Sheet Metal Screws. AISI Specification for the Design of Cold-Formed Steel Structural Members and accompanying Commentary - Section E4. This section is applicable to screws with a nominal diameter of 2.03 mm (0.08 in) $\leq$ d $\leq$ 6.35 mm (0.25 in). The nominal diameter is measured across the threads and will be thread forming or thread cutting. Screws may be used with or without a self drilling point. Table 2-4 gives some suggested loading values for screw connections. Pullout values are for attaching facing materials and are not to be used for connection design.

b Bolts. AISI Specification for the Design of Cold-Formed Steel Structural Members and accompanying Commentary - Section E3. Bolts are designed for sheets with the thinnest sheet being less than 4.8 mm (3/16 in). When the thinnest sheet is greater than 4.8 mm (3/16 in) use the AISC specification. Four design conditions need to be considered:

- Longitudinal shearing of the sheet parallel to the through the end of the sheet,
- Bearing or piling up behind the bolt,
- Tearing through the net section,
- Shearing of the bolt.

Table 2-4: Suggested Capacities for Screw Connectors in kN (lbs)

Steel Nominal / Design Thickness mm (in)	No. ¼-14		No. 12-14		No. 10-14		No. 8-14		No. 6	
	Shear or Bearing	Pullout	Shear or Bearing	Pullout	Shear or Bearing	Pullout	Shear or Bearing	Pullout	Shear or Bearing	Pullout
2.583 (0.1017)	2.60 (585)	1.57 (352)	2.00 (450)	1.44 (324)	1.45 (327)	1.40 (314)	NA	1.35 (303)	NA	NA
1.811 (0.0713)	2.27 (511)	1.08 (242)	1.83 (412)	0.96 (215)	1.27 (286)	0.91 (205)	NA	0.89 (200)	NA	NA
1.438 (0.0566)	1.89 (426)	0.71 (159)	1.68 (377)	0.68 (153)	1.16 (261)	0.67 (151)	1.05 (236)	0.63 (142)	NA	0.59 (132)
1.146 (0.0451)	1.34 (301)	0.45 (101)	1.23 (276)	0.45 (101)	1.17 (263)	0.44 (98)	1.10 (248)	0.42 (94)	0.84 (188)	0.37 (83)
0.879 (0.0346)	0.69 (154)	0.32 (71)	0.64 (143)	0.31 (70)	0.63 (141)	0.31 (69)	0.62 (140)	0.30 (68)	0.59 (133)	0.24 (53)

Notes:
1. NA: not applicable, two thicknesses of this metal gage cannot be connected by this size screw.
2. Screw capacity is based on a minimum connected strength of F_y = 228 MPa (33 ksi). The ratio of the material ultimate tensile strength to yield strength should be equal to or greater than 1.15.
3. Screw a spacing and edge distance shall not be less than 1.5D or P/0.6F_{yt}, where D is the screw shank diameter and P is the shear load.
4. Screw capacities are based on average test results divided by a safety factor of 3.0. Test data is available from Buildex Division of ITW, Inc Itasca, Illinios; test #845, uninspected values.
5. For steels having yields other than 228 MPa (33 ksi), use the following formula:
Table Value(Actual yield strength)/(228 or 33 in consistent units) = New value

c. Welds. AISI Specification for the Design of Cold-Formed Steel Structural Members and accompanying Commentary - Section E2. The maximum thickness for the use of the AISI specification when welding sheets together is 4.6 mm (0.18 in) for the thinnest sheet. When welding thicker sheets use the AISC specification. Resistant welded sheets are limited to 3.2 mm (0.125 in) or less for the thinnest sheet. Table 2-5 gives some suggested values for fillet and flare-groove welds.

Table 2-5: Suggested Design Loads for Fillet and Flare-Bevel Groove Welds			
Design Thickness t mm (in)	Weld Size mm (in)	Weld Strength	
		Fillet N/mm (lbs/in)	Flare-Bevel Groove N/mm (lbs/in)
3.15 mm (0.1240")	4.76 (3/16)	215 (1228)	172 (982)
2.583 (0.1017)	3.97 (5/32)	176 (1007)	141 (806)
1.811 (0.0713)	3.18 (1/8)	124 (706)	99 (565)
1.438 (0.0566)	3.18 (1/8)	98 (560)	78 (448)
1.146 (0.0451)	3.18 (1/8)	78 (447)	63 (358)

Notes:
1. Welds can be positioned in shear or tension.
2. Weld strength for fillet = 0.3 F_y t, where t = minimum welded material thickness.
3. Weld Strength for flare-bevel groove = 0.3 F_y t/1.25.
4. Values shown are for F_y = 228 MPa (33 ksi). For F_y = 278 MPa (40 ksi) multiply tabulated values by 1.33. For F_y = 345 MPa(50 ksi) multiply tabulated values by 1.52.
5. Flare-bevel groove welds occur between the outside radius of one piece and a flat surface of another piece.

d. Anchors. ASTM F 1554 for Anchor Bolts covers straight, bent, headed, and headless bolts for anchoring the structural support to the foundation. Bolts covered have diameters from 6.35 mm (¼ in) to 101.6 mm (4 in) and yield strengths of 248, 379, and 724 MPa (36, 55 and 105 ksi).

i) Expansion Anchors or similar devices will be designed as bolted connection between the anchor and the cold-formed structure. In lieu of specific anchor data the suggested values in table 2-6 may be used. Designer must assure that the anchors as supplied meet the design requirements.

Table 2-6: Suggested Capacity for Expansion Anchors in Stone Aggregate Concrete							
Anchor Diameter mm (in)	Minimum Embedment mm (in)	Type of Loading	Concrete Strength MPa (psi)			Minimum Anchor Spacing mm (in)	Minimum Edge Distance mm (in)
			13.8 (2000) kN (lbs)	27.6 (4000) kN (lbs)	41.4 (6000) kN (lbs)		
6.35 (¼)	64 (2-1/2)	Pullout Shear	1.45 (325) 1.69 (380)	1.89 (420) 2.89 (650)	1.87 (420) 2.89 (650)	64 (2-1/2)	32 (1-1/4)
13 (½)	70 (2-3/4)	Pullout Shear	2.96 (665) 7.6 (1710)	4.00 (900) 9.25 (2080)	5.38 (1210) 10.3 (2320)	127 (5)	64 (2-1/2)
19 (¾)	83 (3-1/4)	Pullout Shear	4.16 (935) 13.6 (3050)	5.65 (1270) 19.0 (4270)	6.0 (1360) 20.0 (4510)	191 (7-1/2)	95 (3-3/4)
25 (1)	114 (4-1/2)	Pullout Shear	7.2 (1610) 27.9 (6280)	8.9 (2000) 29.9 (6720)	11.3 (2530) 35.4 (7960)	254 (10)	127 (5)

Notes:
1. Pullout values listed may be doubled with special inspection.
2. Values may not be increased 1/3 for wind for seismic loads.
3. ICBO uninspected values - Hilti/ICBO report #2156.

ii) Powder driven pins may be used to attach cold-formed members to concrete or structural steel typical suggested capacities are shown in tables 2-7 and 2-8.

e. Connections and Joints. In Cold-formed steel design the AISI specification is to be used when the thickness of the thinnest sheet being connected is less than 4.8 mm (3/16"), when the thickness of the thinnest material is greater than 4.8 mm (3/16") use the AISC specification. AISI uses only bearing connections, snug tight fit. In cold-formed design washers are typically not used in cold-formed steel construction. The primary mode of failure is sheet tearing for AISI designed connections. Two limit states are considered in the design of cold-formed steel connections: tilting of screw and subsequential tearing of sheet, and tension pull over are common. Riveted and crimped fasteners are proprietary connections, they are not to be used in COE designs. Table 2-9 gives some suggested loads per fastener for joist clip angles.

Table 2-7: Suggested Capacity for Powder Driven Fasteners in Concrete

Shank Diameter Mm (in)	Minimum Penetration mm (in)	Type of Loading	Concrete Compressive Strength MPa (psi) 13.8 (2000) N (lbs)	20.7 (3000) N (lbs)	27.6 (4000) N (lbs)
3.68 (0.145)	29 (1-1/8)	Pullout Shear	0.40 (90) 0.71 (160)	0.51 (115) 1.00 (225)	0.65 (145) 1.18 (265)
4.50 (0.177)	37 (1-7/16)	Pullout Shear	0.67 (150) 1.11 (250)	0.91 (205) 1.27 (285)	1.22 (275) 1.47 (330)
5.21 (0.205)	32 (1-1/4)	Pullout Shear	0.98 (220) 1.74 (390)	1.25 (280) 1.98 (445)	1.54 (345) 2.22 (500)

Notes:
1. Capacities shown are for stone aggregate concrete and are based on a low velocity shot.
2. Minimum fastener spacing: 4"; minimum fastener edge distance: 3".
3. Values may not be increased by 1/3 for wind or seismic loads.
4. ICBO uninspected values - Hilti/ICBO research #2388.

Table 2-8: Suggested Capacity for Powder Driven Fasteners in Structural Steel

Steel Thickness mm (in)	Shank Dia: 3.68 mm (0.145") 6.35 mm (¼") kN (lbs)	9.53 mm (3/8") kN(lbs)	13 mm (½") kN(lbs)	Shank Dia: 4.50 mm (0.177") 6.35 mm (¼") kN(lbs)	9.53 mm (3/8") kN(lbs)	13 mm (½") kN(lbs)	Shank Dia: 5.21 mm (0.205") 6.35 mm (¼") kN(lbs)	9.53 mm (3/8") kN(lbs)	!3 mm (½") kN(lbs)
2.583 (0.1017)	0.93 (210)	0.93 (210)	0.93 (210)	1.49 (335)	1.76 (395)	1.76 (395)	2.16 (485)	2.34 (525)	2.94 (660)
1.811 (0.0713)	0.93 (210)	0.93 (210)	0.93 (210)	1.49 (335)	1.76 (395)	1.76 (395)	2.16 (485)	2.34 (525)	2.58 (581)
1.438 (0.0566)	0.93 (210)	0.93 (210)	0.93 (210)	1.49 (335)	1.76 (395)	1.76 (395)	2.16 (485)	2.07 (465)	2.07 (465)
1.146 (0.0451)	0.93 (210)	0.93 (210)	0.93 (210)	1.43 (321)	1.43 (321)	1.43 (321)	1.65 (372)	1.65 (372)	1.65 (372)
0.879 (0.0346)	0.88 (197)	0.88 (197)	0.88 (197)	1.10 (247)	1.07 (241)	1.07 (241)	1.24 (279)	1.24 (279)	1.24 (279)

Notes:
1. Tests were conducted with the fastener point driven completely through the back side of the hot-rolled steel member. This was necessary to obtain proper gripping force.
2. Fasteners should not be located less than 13 mm (1/2") from the edge of steel.
3. A minimum fastener spacing of 38 mm (1-1/2") is necessary.
4. Bearing strength based upon: 1.15(228 MPa)(Bearing Area) [1.15(33ksi)(Bearing Area)] for cold-formed steel.
5. Capacities shown are for either shear or pullout.
6. Values may not be increased by 1/3 for wind or seismic loads.
7. ICBO uninspected values - Hilti/ICBO research #2388

Table 2-9: Joist End Clip: Allowable Loads per fastener

Clip Length mm (in)	No. of screws in Each Leg (#10-16)	Joist Thickness 0.8879 mm (0.0346") 228 MPa (33 ksi) kN (lbs)	1.146 mm (0.0451") 228 MPa (33 ksi) kN (lbs)	1.438 mm (0.0586") 228 MPa (33 ksi) kN (lbs)
152 (6)	3	1.50 (337)	2.23(501)	4.76 (1070)
203 (8)	4	2.14 (480)	3.17 (713)	6.77 (1523)
254 (10)	5	2.78 (626)	4.14 (930)	8.83 (1985)

Notes:
1. Based on CCFSS technical bulletin vol. 2, no. 1 which outlines the proposed AISI specification provisions for screw connections.
2. F_y = 228 MPa (33 ksi) for 0.879 mm (0.0346") sheets. F_y = 345 MPa (50 ksi) for 1.438 mm (0.0566") sheets. $F_u = 1.08F_y$
3. Allowable loads based on a factor of safety of 3.0.

f. Fasteners. Information on Fasteners for Residential Steel Framing can be found in AISI RG-933.

6. USEFUL RELEVANT INFORMATION

a. Beam Diagrams and Formulas

AISC Steel Construction Manual

b. Material Weights

AISC Steel Construction Manual

c. Software. Listings of available cold-formed steel design software can be found through the CCFSS http://www.umr.edu/~ccfss/ Technical Bulletin, Center for Cold-Formed Steel Structures, University of Missouri Rolla.