$ 5.00

LGSEA

TECHNICAL NOTE

On Cold-Formed Steel Construction

Light Gauge Steel Engineers Association • Washington, DC 20005 • Toll Free: (866) 465-4732 • www.LGSEA.com

COLD-FORMED STEEL FLOOR JOISTS

Introduction: Cold-formed steel (CFS) joists are becoming very popular where non-combustible material and long unsupported spans are called for in the design. The purpose of this tech note is to provide a review of the AISI design requirements of steel floor joists.

Buckling and Failure Modes of Floor Joists

CFS joist members are normally thin and consequently have a low torsional stiffness. Monosymmetric CFS sections (i.e. floor joist C-sections) have their shear center eccentric from their centroid as shown in Figure 1a. This eccentricity produces considerable torsional deformations in thin-walled sections (see Fig. 1b). Floor joist members may also undergo flexural-torsional mode of buckling (as in Figure 1f) because of their low torsional stiffness. Figures 1c through 1e demonstrate the different ways in which a joist channel section may buckle when subjected to major axis bending moment. Local buckling mode (Fig. 1b) only involves the web, the compression flange, and the lip. Distortional buckling (Fig. 1d) occurs when the compression flange and the lip rotate about the flange web junction with the web providing some elastic restraint to rotation. This mode of buckling is also referred to as local-torsional buckling. When the tension flange is subjected to torsional restraint (i.e. sheathing fastened to tension flanges) the distortional buckling mode will occur as shown in Figure 1e. Unrestrained joists (not typical of floor joists) will undergo a flexural-torsional buckling (often called lateral buckling) as shown in Fig. 1f.

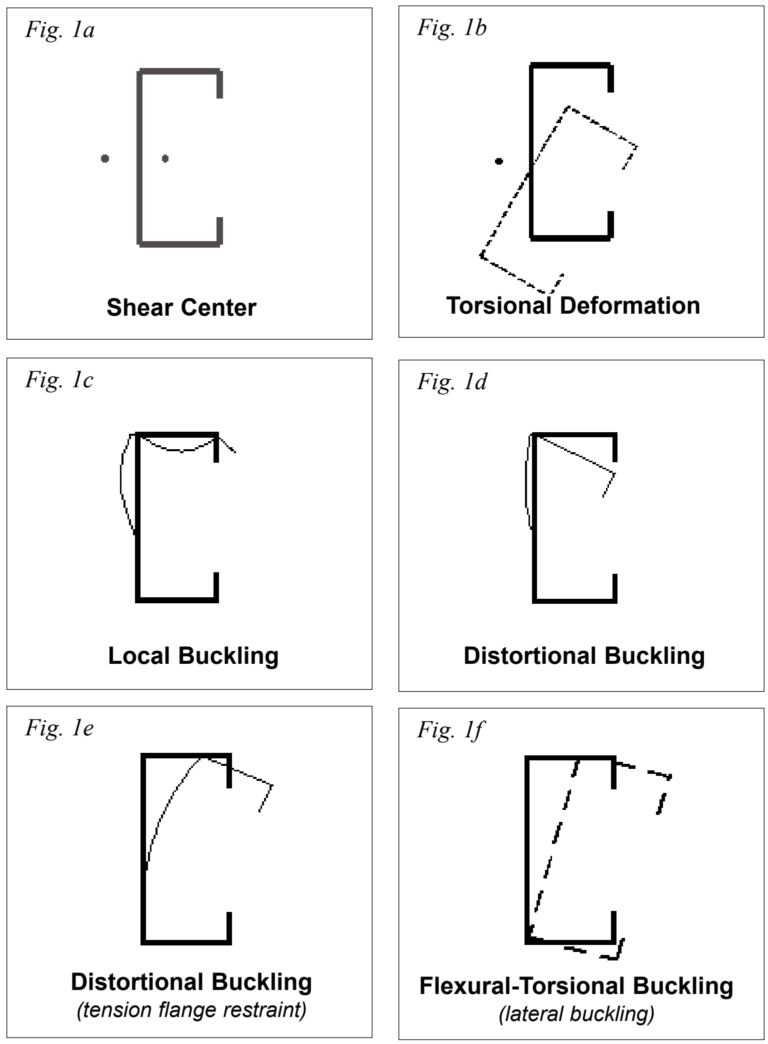


Fig. 1a Shear Center

Fig. 1b Torsional Deformation

Fig. 1c Local Buckling

Fig. 1d Distortional Buckling

Fig. 1e Distortional Buckling *(tension flange restraint)*

Fig. 1f Flexural-Torsional Buckling *(lateral buckling)*

AISI Requirements for the Design of Floor Joists

CFS floor joists are designed in accordance with the AISI *Specification for the Design of Cold-Formed Steel Structural Members*, Sections C3.1 through C3.5. Strength increase from cold work of forming (F_{ya}) can be substituted for F_y in Sections C3.1 (excluding Section C3.1.1(b)). The AISI Specification provides equations for nominal flexural strength and design flexural strength for both the ASD and LRFD design methods.

The design of floor joists (for strength) may be governed by the interaction of yielding and buckling of (a) the most heavily loaded cross section (in bending and post-local buckling) of the elements forming the cross-section, (b) the web subjected to in-plane bending and undergoing local buckling in the compression zone, (c) the web subjected to shear force and undergoing shear buckling, or yielding in shear, (d) the web subjected to combined shear and bending, (e) the web subjected to a concentrated load at a load or reaction point, and undergoing web crippling, or (f) the whole joist in a flexural-torsional mode of buckling. The buckling modes may interact with each other to produce lower design strength if shear and bending or bending and concentrated load occur simultaneously.

AISI Specification Section C3.1.1 provides equations (Procedure I - based on Initiation of Yielding) to determine the allowable moment capacity of the section to be used in the design of (a) above. The buckling load of laterally unbraced joists subjected to flexural-torsional buckling (not typical for floor joists) is determined in accordance with Section C3.1.2 of the Specification. Joists subjected to distortional buckling with tension flanges unrestrained require a more complex analytical method to predict the buckling load.

DESIGN OF FLOOR JOISTS

CFS floor joists are normally treated as simply supported or continuous span bending members subjected to uniform and/or concentrated loads. Floor joists are also designed considering the compression flanges (top flanges) to be continually braced by the subflooring, thus providing lateral restraint. For the design of floor joists, any one of several criteria may control the design depending on the configuration of the section, thickness of material, and member length. Six design checks are usually performed when designing CFS floor joists: (1) deflection criteria, (2) bending moment, (3) shear, (4) web crippling, (5) combined bending and web crippling, and (6) combined shear and bending.

Deflection Design

Deflection limits imposed on the design of floor joists to minimize the impact on the finish material (i.e. gypsum board) can significantly impact allowable joist spans. Deflection limits have to be carefully considered and justified. The amount of deflection that a joist member experiences under a certain uniform load is determined by the joist's moment of inertia, I_x. The moment of inertia can be obtained from manufacturer's catalogs or calculated in accordance with the AISI Specification, Section B. The following formulas can be used to determine the maximum deflection of a simply supported single joist member subjected to a uniform load and a concentrated load at mid-span:

For uniformly distributed load

$$\delta = \frac{5WL^4}{384\,EI} \quad \Rightarrow L = S_d \sqrt[3]{\frac{11800000 \bullet I_x}{W \bullet D}}$$

For concentrated load at mid-span

$$\delta = \frac{PL^3}{48\,EI} \quad \Rightarrow L = 37630\sqrt{\frac{I_x}{PD}}$$

Where: L = Span, ft.
I_x = Moment of inertia, in^4.
W = Uniform load acting on joist, psf
P = Concentrated load at mid-span, lb.
D = Allowable deflection limit (i.e. 240, 360, 480, etc.)

S_d = Factor to account for joist spacing = $\sqrt[3]{16 / \text{spacing}}$
= 1.10 for 12" spacing, 1.00 for 16" spacing, 0.94 for 19.2" spacing, and 0.87 for 24" spacing.

Example:

- 8" joist, 54 mils (16 gauge), 50 ksi steel, I_x =5.47 in^4.
- W = 50 psf (10 psf dead load + 40 psf live load)
- Joist spacing = 24" o.c. Plywood sheathing attached to joist at 12-in. o.c.

Find the maximum allowable joist span due to a total load deflection limit of L/360:

$$L = 0.87\sqrt[3]{\frac{11800000 \bullet 5.47}{50 \bullet 360}} = 13.32' = 13'\text{-}4"$$

Bending Moment Design

The grade of steel (such as ASTM A653 Gr. 33) and the effective section modulus of the joist, S_x, determine the allowable bending moment. The type of steel should be obtained from the manufacturer. The section modulus can be either calculated (per AISI Section B2, B3, or B4) or obtained from the manufacturer. The calculation of the section modulus, S_x, may be complex sometimes, as the effective widths for the compression components of the web, flanges, and lips have to be determined. Consequently, the effective section involves removal of portions of the compression flange, web, and lip if each is sufficiently slender.

The maximum allowable simply supported joist span due to maximum bending moment is:

$$L = \sqrt{\frac{1152 F_b S_x}{W}}$$ For joist subjected to uniformly distributed load

$$L = \frac{F_b S_x}{3P}$$ For joist subjected to concentrated load at mid-span

Where: L = Span, ft.
F_b = Allowable bending stress, psi
S_x = Effective section modulus, in^3.
P = Concentrated load at mid-span, lb.
W = Uniform load acting on joist, psf
S_b = Factor to account for joist spacing = $\sqrt{16 / \text{spacing}}$ = 1.15 for 12" spacing, 1.00 for 16" spacing, 0.91 for 19.2" spacing, and 0.82 for 24" spacing.

Example:

- 8" joist, 54 mils (16 gauge), 50000 psi steel, S_x = 1.304 in^4.
- Joist spacing = 24-in. o.c., F_b = 19800 psi, Plywood sheathing attached to joist at 12-in. o.c.

Find the maximum allowable span due to a uniformly distributed load of 50 psf (10 psf dead load + 40 psf live load)

$$L = 0.82\sqrt{\frac{0.5 \bullet 19800 \bullet 1.304}{50}} = 13.18' = 13'\text{-}2"$$

Shear Design

Web shear normally does not control the design of CFS floor joists for simple spans unless the members have a large web height to steel thickness ratio (h/t). However, the AISI Specification requires web shear to be checked for all floor joists. AISI Section C3.2 is used to calculate the shear strength of CFS joists. The nominal shear strength, V_n, of a web consisting of two or more sheets are calculated as the sum of the nominal shear strength of each sheet (web). Each sheet is considered as a separate element carrying its share of the shear force.

The allowable span of a uniformly loaded singly supported joist span as limited by the shear, can be determined from the following equation:

$$V_a = WL/2 \qquad \Rightarrow \qquad L = \frac{288V_a}{S_c W}$$

Where: L = Span, ft.
V_a = Allowable shear strength, lb.
W = Uniform load acting on joist, psf
S_c = Factor to account for joist spacing
=16/spacing = 1.33 for 12" spacing, 1.00 for 16" spacing, 0.83 for 19.2" spacing, and 0.67 for 24" spacing.

Example:

- 8" simply supported joist [8" web, 1-5/8" flange, ½" lip, R = 3/32", 54 mils (16 gauge)]
- F_y = 50000 psi, Joist spacing = 24" o.c., Plywood attached to joists at 12-in. o.c.
- W = 50 psf (10 psf dead load + 40 psf live load)

Calculate shear strength per Section C3.2 of the AISI Specification:

h = 8 - 2(0.054 + 0.09375) = 7.705 in.
h/t = 7.705 / 0.054 = 142.7

$0.96EK_v / F_y = 0.96[(29500)(5.34) / 50]^{1/2} = 53.89$
$1.415EK_v / F_y = 1.415[(29500)(5.34) / 50]^{1/2} = 79.42$

$h/t > 1.415EK_v / F_y \quad \Rightarrow \quad V_n = 0.905\, EK_v\, t^3/h$

Equation C3.2-3

$V_n = 0.905\,(29500000)(5.34)(0.054)^3/\,7.705 = 2914$ lb.
$\Omega_v = 1.67$
$V_a = 2914 / 1.67 = 1745$ lb.

Calculate allowable span due to V_a:

$$L = 0.67\ \frac{1.5 \bullet 1745}{50} = 35.1 \text{ ft.} = 35'\text{-}1"$$

Web Crippling Design

CFS joists have thin webs that are susceptible to localized web buckling at concentrated loads or bearing locations. AISI Specification Section C3.4 provides design equations for single-web and I-sections with stiffened or partially stiffened flanges and unstiffened flanges. These equations are limited to joists with web height to thickness (h/t) ratio of less than or equal to 200. Joists with h/t ratios greater than 200 shall be provided with adequate means of transmitting concentrated loads and/or reactions directly into the webs (i.e. adding bearing stiffeners). The height of the web, the thickness of the steel, and the length of the bearing are the most critical components in the AISI web crippling equations. Other factors such as steel material, and bend radius play a lesser role.

The critical location for web crippling is at the member's ends for single joist spans and at the member's ends and intermediate supports for multiple spans. Most manufacturers tabulate the web crippling strength of simply supported joist spans in their technical catalogs. The design equations of the AISI Specification Section C3.4 can be used if this information is not readily available.

The allowable span of a uniformly loaded simply supported joist as limited by web crippling at the ends can be determined from the following equation:

$$L = S_c\ \frac{1.5P_{allow}}{W}$$

Where: L = Span, ft.
P_{allow} = Allowable end reaction web crippling strength, lbs.
W = Uniform load acting on joist, psf
S_c = Factor to account for joist spacing
=16/spacing = 1.33 for 12" spacing, 1.00 for 16" spacing, 0.83 for 19.2" spacing, and 0.67 for 24" spacing.

Example:

For the shear design calculation example (above) with bearing length (N) = 1.5", calculate the maximum allowable span due to web crippling strength.

Calculate web-crippling capacity of the joist per Section C3.4 of the AISI Specification:

$$P_n = t^2\, k\, C_3\, C_4\, C_9\, C_\theta\, [331 - 0.61(h/t)][1 + 0.01(N/t)]$$

Equation C3.4-1

$k = 894F_y/E = 1.515$ *Equation C3.4-21*

$C_3 = 1.33 - 0.33k = 0.830$ *Equation C3.4-12*

$C_4 = 1.15 - 0.15R/t \leq 1.0$ *Equation C3.4-13*

$C_4 = 0.890$ (not less than 0.5)

$C_9 = 1.0$

$C_\theta = 0.7 + 0.3(\theta/90)^2 = 1.0$ *Equation C3.4-20*

$N/t = 27.78$ (1.5 in. bearing length)

$P_n = 1.017$ kips = 1,017 lb.

$P_a = P_n / \Omega_w = 1017/1.85 =$ kips = 550 lb.

Calculate allowable span due to P_a:

$$L = 0.67 \frac{1.5 \bullet 550}{50} = 11.05 \text{ ft.} = 11'\text{-}1''$$

Combined Bending and Web Crippling Design

AISI Specification Section C3.5 requires a combined bending and web crippling check for unreinforced joist webs. Three equations are provided in AISI Specification Section C3.5: Single unreinforced webs, Multiple unreinforced webs (such as I-beams), and Two nested Z-shapes (support point). Interaction equation for shapes having single unreinforced webs is normally used to design floor joists. The ASD format of the interaction equation is shown below:

$$1.2\left(\frac{\Omega_w P}{P_n}\right) + \left(\frac{\Omega_b M}{M_{nxo}}\right) \leq 1.5$$

Where:
- M = Flexural strength
- P = Required strength for the concentrated load or reaction in the presence of bending moment.
- P_n = Nominal strength for concentrated load or reaction in the absence of bending moment per Section C3.4.
- M_{nxo} = Nominal flexural strength per section C3.1.1
- Ω_b = Factor of safety = 1.67
- Ω_w = Factors of safety per section C3.4.

Combined Shear and Bending Design

The combination of a bending stress and a shear stress in a web produces a further reduction in the capacity of the web. The degree of the reduction depends upon whether the web is stiffened or not. The presence of negative moment (reversed bending) at the interior supports of multiple span joists require certain measures to be taken to ensure the joist is not overstressed. AISI Specification Section C3.3 requires combined shear and bending check for multiple span joists. Two equations are given for this check.

The interaction equation for unstiffened webs is a circular formula based upon an approximation to the theoretical interaction of local buckling resulting from shear and bending. The interaction equation for stiffened webs is linear and not as severe because of the greater post buckling capacity in the combined shear and bending buckling mode. The unreinforced web equation is normally used when designing floor joists. The ASD format of that equation is shown below:

$$\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2 \leq 1.0$$

Where:
- M = Flexural strength
- M_{nxo} = Nominal flexural strength about the centroidal axis per section C3.1.1.
- V_n = Nominal shear force when shear alone exists.
- Ω_b and Ω_v are factors of safety per sections C3.1.1 and C3.2 respectively.

Holes (punchouts) in Floor Joists

AISI Specification Section B2.2 addresses only small circular holes. Steel manufacturers typically punch standard size holes, 1-1/2” wide x 4" long along the centerline of the web, in the joist to facilitate the passing of utilities. The span tables in the *Prescriptive Method for Residential Cold-Formed Steel Framing* (and in the ICC and the IRC steel sections) were developed assuming standard holes spaced at 24 in. on center.

AISI's *Design Guide For Cold-Formed Steel Beams With Web Penetration* (Publication RG-9712) provides a design method in an allowable stress design (ASD) format applicable to members with large penetrations. The Design Guide limits the applicability of the recommended method to C-sections with web depth-to-thickness ratio (h/t) not exceeding 200 and a web perforation depth-to-height ratio (a/h) not exceeding 0.75. Furthermore, web perforations shall comply with the following limitations:

- Hole center-to-center spacing ≤ 24” ("h" is the flat depth of the web)
- Length of circular holes ≤ 6 in.
- Length of non-circular holes ≤ 2.67a
- Holes located along the centerline of the web.

The AISI Design Guide provides provisions for bending, shear, combined bending and shear, and web crippling equations that takes into effect the size of the hole.

Example:

For the design calculation example (above) with bearing length (N) = 1.5", calculate the allowable shear capacity and web crippling strength assuming a 5 inch circular hole located 10 inches from edge of bearing and along the centerline of the joist.

Calculate web crippling strength of the joist (with holes):

For end-one-flange (EOF) loading conditions, when the hole is not within the bearing length, the AISI Design Guide provides a reduction factor that must be applied to the web crippling strength calculated per Section C3.4 of the Specification.

Check if the AISI Design Guide equations are applicable:

- $a/h = 5.0/7.705 = 0.65 < 0.75$ ok.
- $R_c = 1.01 - 0.325(a/h) + (0.083(x/h)$ (x is the nearest distance between hole and bearing edge.)
- $R_c = 1.01 - 0.325(5/7.705) + 0.083(10/7.705) = 0.6914$
- $P_a = 550$ lb. (previously calculated)
- $P_{allow} = P_a R_c = 550 \times 0.6914 = 380$ lb.

Calculate shear capacity of the joist (with holes):

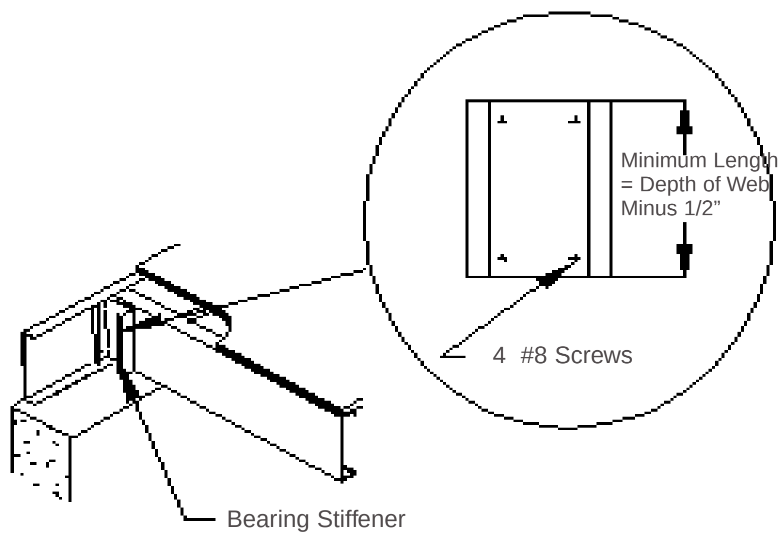
- $V_{allow} = q_{s1}q_{s2}V_a$
  (Where $V_a = 1745$ lb. as calculated before)
- $c = h/2 - a/2.83$ (for circular holes)
- $c = 7.705/2 - 5/2.83 = 2.086$
- $c/t = 2.086/0.054 = 38.63$
- $5 \leq c/t = 38.63 \leq 54 \Rightarrow q_{s1} = c/54t$ and $q_{s2} = 1.5(V_1/V_2) - 0.5 \leq 1.3$
- V_1/V_2 is the variation in shear along the longitudinal axis of the web hole. V_1 is the larger shear and V_2 is the smaller shear at the edge of hole. In this simply supported joist example, $V_1/V_2 = 1$
- $q_{s1} = 2.086/54(0.054) = 0.715$
- $q_{s2} = 1.5(V_1/V_2) - 0.5 \leq 1.3 = 1.0$
- $V_{allow} = (0.715)(1.0)(1745) = 1248$ lb.

WEB STIFFENERS

Webs of thin-walled CFS members may cripple or buckle locally under a concentrated load location or at a bearing reaction. The purpose of a web stiffener (also called bearing stiffener) is to ensure that the full capacity of the member is achieved before failure. It is therefore necessary to adequately size web stiffeners. Web stiffeners are designed in accordance with AISI Specification Section B6.1.

Web stiffeners are usually specified for all floor joists bearing point locations. Stiffeners are typically cut from a stud or track section of equivalent thickness to that of the floor joist or the stud above. Web stiffeners can be installed on either side of the web and shall extend the full depth of the web. Stiffeners are typically fastened to the web with a minimum of four screws. A typical web stiffener detail is shown in Figure 2.

Figure 2



Web Stiffener Detail

FLOOR JOIST SPAN TABLES

CFS floor joist span tables can be found in many publications. Most steel manufacturers or rollformers have technical catalogs tabulating joist dimensions, section properties, and allowable spans based on certain deflection limits. Most manufacturer's catalogs however, cater to engineers or designers, and may not contain all design checks required by AISI. AISI's *Prescriptive Method for Residential Cold-Formed Steel Framing*, on the other hand contains allowable span tables for 30 psf and 40 psf live loads for both single and multiple spans. These tables are prescriptive and can be easily used by non-engineers. The ICC *One- and Two-Family Dwelling Code* (and the IRC Final Draft) also contain CFS allowable joist span

DESIGN EXAMPLE

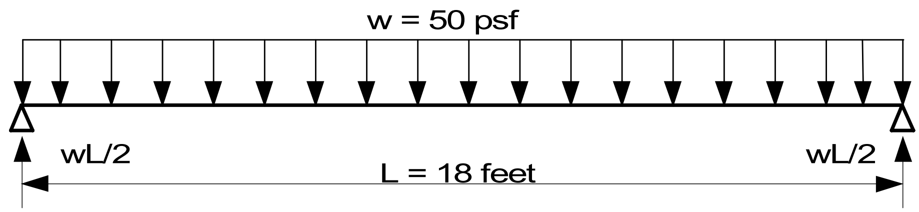
Design a simply supported floor joist span of 18 feet supporting the top floor of a two-story building.

- Joist spacing = 24" on center with top flanges braced by plywood sheathing
- $t = 0.0557$ in. (thickness)
- $F_y = 50$ ksi
- Dead load = 10 psf
- Live load deflection limit = L/360
- Live load = 30 psf
- Total load deflection limit = L/240

The design of such a floor joist requires first a selection of a joist member, calculating its capacity (or selecting the capacity from a manufacturer's catalog), and finally checking this capacity against the applied loads.

Select a 1000S162-54 (2 x 10 x 54 mil) joist. Calculate also the allowable load due to the shear and bending capacities. Web crippling capacity and combined web crippling and bending will not be calculated, as the joist will be stiffened at reaction points. Use the AISI Design Specification [1996].

Figure 3



I. Allowable Shear Capacity:

AISI Design Specification (Section C3.2):

$h = 10 - 2(0.0557 + 0.125) = 9.639$ in.
$h/t = 9.639 / 0.0557 = 172.90$

$0.96[Ek_v / F_y]^{1/2} = 0.96[(29{,}500)(5.34) / 49.745]^{1/2}$
$= 54.022$
Where $k_v = 5.34$ for un-reinforced webs

$1.415[Ek_v / F_y]^{1/2} = 1.415[(29{,}500)(5.34) / 49.745]^{1/2}$
$= 79.63$

$h/t > 1.415[Ek_v / Fy]^{1/2} \quad => \quad V_n = 0.905\ Ek_v\ t3/h$
Equation C3.2-3

$V_n = 0.905\ (29{,}500{,}000)(5.34)(0.0557)^3/ 9.639$
$= 2{,}563$ lb.

$\Omega_v = 1.67$ (factor of safety)

$V_a = 2{,}563 / 1.67 = 1530$ lb.

$V_a = wL/2$

Maximum span due to allowable shear = $L = 2V_a/w$

L = 2(1530)/(40x2) = 38.25 feet = 38'-3" > required 18 feet OK.

II. Allowable Moment Capacity

AISI Design Specification (Section C3.1):

$M_n = S_eF_y$

To calculate Se, the effective portions of the compression flange, edge stiffener, and web should be calculated.

Compression Flange

$R = 2 \times 0.0557 = 0.1114$ in.
$w = 1.625 -2(0.1114 + 0.0557) = 1.2908$ in.
$w/t = 23.17 < 60$

$S = 1.28[E/f]^{1/2} = 1.28[29{,}500/49.745]^{1/2} = 31.17$
$S/3 = 10.39$
$S/3= 10.39 < w/t=23.17 < S=31.17$

Therefore, AISI Design Specification Section B4.2 case II applies.

$I_a = 399t^4\{(w/t)/S) - (k_u/4)^{1/2}\}^3 = 0.00028$ in.^{4}
Equation B4.2-4
Where ku = 0.43

$d = 0.5 - 0.1115 - 0.0557 = 0.3328$ in.
$I_s = d^3t/12 = 0.00017$ in.^{4}
$I_s / I_a = 0.611$
$D/w = 0.387$

$k = C_2^n(K_a - K_u) + K_u$ *Equation B4.2-7*

$k_a = 5.25 - 5(D/w) \leq 4.0$

$k_a = 5.25 - 5(0.387) = 3.315 \leq 4.0$

$C_2^n = I_s/I_a$ but not greater than 1.0

$n = ½$

$k = 0.611(3.315 - 0.43) + 0.43 = 2.19$

$\lambda = 0.676 > 0.673$ *Equation B2.1-4*

$b = \rho w$ *Equation B2.1-2*

$\rho = (1-0.22/\lambda)/\lambda$ *Equation B2.1-3*

$\rho = 0.998$

$b = 0.998(1.2908) = 1.288$ in. (the effective width of the compression flange)

Compression Edge Stiffener

$k = 0.43$

$d/t = 5.975$

$\lambda = 0.393 < 0.673$

Therefore, $d_s' = d$

$d_s = d_s' (I_s/I_a) \leq d_s'$

Therefore, $d_s = d = 0.3328$ in. (edge stiffener is fully effective)

Web

The calculation of the effective portion of the web is an iterative process. The calculation is similar to those used for the flange end edge stiffener.

The effective widths previously calculated (flange, lip, and web) are used to calculate the effective section modulus S_e.

S_e is calculated to be 1.668 in^3; $\Omega = 1.67$

$M_n = 1.668 \times 50 = 83.40$ in.-kips.

$M_a = 83.40 / 1.67 = 49.94$ in.-kips = 4.162 ft-kips.

$M_{max} = wL2/8 \Rightarrow 50(2)(18)2/8 = 4050$ ft-lb.

= 4.05 ft-kips < 4.162 ft.-kips OK

$L_{max} = [8(4,162)/(50x2)]^{1/2} = 18'\text{-}3"$

III. Allowable Deflection Limit

For uniformly distributed load

$$\delta = \frac{5WL^4}{384EI} \Rightarrow L = S_d \sqrt[3]{\frac{11800000 * I_x}{W * D}}$$

Live load deflection:

$L = 0.87[11800000*9.774/30*360]^{1/3} = 19'\text{-}2"$
> 18'-0" required OK.

(Note, if L/480 is to be used, than the required span would be too long, and the joist would only has a maximum span of 17'-5".)

Total load deflection:

$L = 0.87[11800000*9.774/50*240]^{1/3} = 118'\text{-}6"$
> 18'-0" required OK.

Therefore, a 1000S162-54 (2x10x54) joist would be acceptable for this application.

References

1. The *BOCA National Building Code, BOCA* (1993), Building Officials & Code Administrators International, Inc., Country Club Hills, Illinois.
2. *Design Guide for Cold-Formed Steel Beams with Web Penetrations*, (1997), Publication RG-9712, American Iron and Steel Institute, Washington D.C.
3. *International Building Code* (IBC), Final Draft, July 1998, International Code Council, Falls Church, VA.
4. *International Residential Code* (IRC), Final Draft, April 1998, International Code Council, Falls Church, VA.
5. ICC *One and Two Family Dwelling Code*, International Code Council, 1998 Edition, International Code Council. Falls Church, VA.
6. *Prescriptive Method for Residential Cold-Formed Steel Framing*, Second Edition (1996), American Iron and Steel Institute, Washington D.C.
7. *Specification for the Design of Cold-Formed Steel Structural Members* , 1996 Edition, American Iron and Steel Institute, Washington D.C.

Author of this *Technical Note:* Nader Elhajj, P.E.