Medeek Truss Designer TM

Truss Data

Truss Type: Fink (4/3)
Out-to-out Span: 24 ft.
Top Chord Pitch: 4 /12
Spacing: 24 in. o/c
Bearing Width: 3.5 in.
Overhang: 12 in.

Design Loads

Top Chord Live Load: 25 psf
Top Chord Dead Load: 7 psf
Bottom Chord Live Load: 0 psf
Bottom Chord Dead Load: 10 psf

Design Assumptions

Number of Plies: 1 PLY
Butt Cut: 0.25 in.
Bottom Chord Pitch: 0 /12
TC Bracing: OSB 7/16 in.
BC Bracing: 10 ft. o/c

Design Options

Truss Plate Mfr.: Mitek
Peak Joint Fixity: Rigid
DOL Lumber: 1.15
DOL Plates: 1.15
Rep Stress Incr: YES
CSI/JSI Limit: 1.00/1.00

Geometry

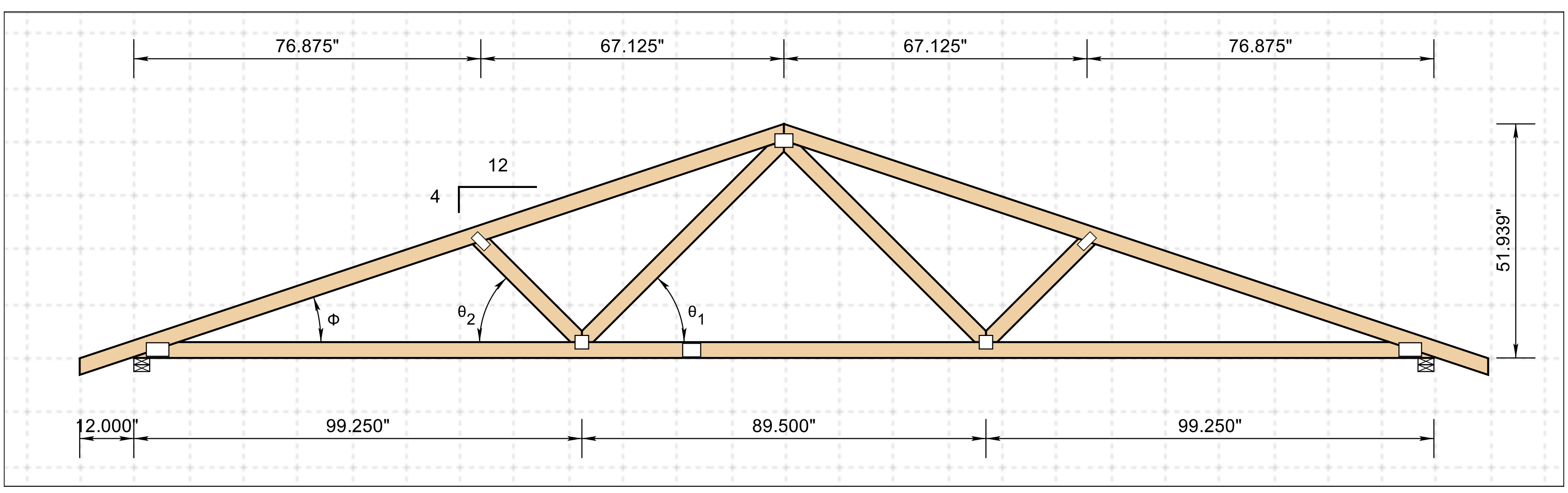
Total Scarf: 9.750 in.
Adj. Scarf: 6.250 in.
Number of Top Chord Panels: 4
Top Chord Panel Lengths: 67.125 in.
Number of Bottom Chord Panels: 3
Bottom Chord Panel Lengths: 89.500 in.

Φ: 18.43 deg.
θ_2: 45.00 deg.
θ_1: 45.00 deg.

LUMBER (Engineered)		**CSI**
Top Chord:	2 X 4 HF No.1	0.90
Bot. Chord:	2 X 4 HF No.1	0.96
Webs (tension):	2 X 4 HF Stud	0.26
Webs (compression):	2 X 4 HF Stud	0.10
Bearing (@ Heel):	2 X 4 HF No.1	0.51

DEFLECTIONS
Δ_{LL} = 0.122 in. $(L/d)_{LL}$ = 2365.0
Δ_{TL} = 0.244 in. $(L/d)_{TL}$ = 1178.6

Weight: 75 lbs



Loads and Forces

ASD DESIGN METHOD: SF(D) + (L)

SF (Slope Factor) = 1/Cosine(Φ) = 1.05

Top Chord: DL(SF) + LL = 7(1.05) + 25 = 32.38 psf = 64.76 plf
Bottom Chord: DL(SF) + LL = 10(1.00) + 0 = 10.00 psf = 20.00 plf

$R_1 = R_5$: 1081.84 lbs.
$P_1 = P_5$: 354.89 lbs.
$P_2 = P_4$: 388.54 lbs.
P_3: 362.24 lbs.
$P_6 = P_7$: 157.29 lbs.

$F_{12} = F_{45}$: 2298.83 lbs.
$F_{23} = F_{34}$: 2021.83 lbs.
$F_{17} = F_{56}$: 2180.86 lbs.
F_{67}: 1490.86 lbs.
$F_{27} = F_{46}$: 399.53 lbs.
$F_{37} = F_{36}$: 626.50 lbs.

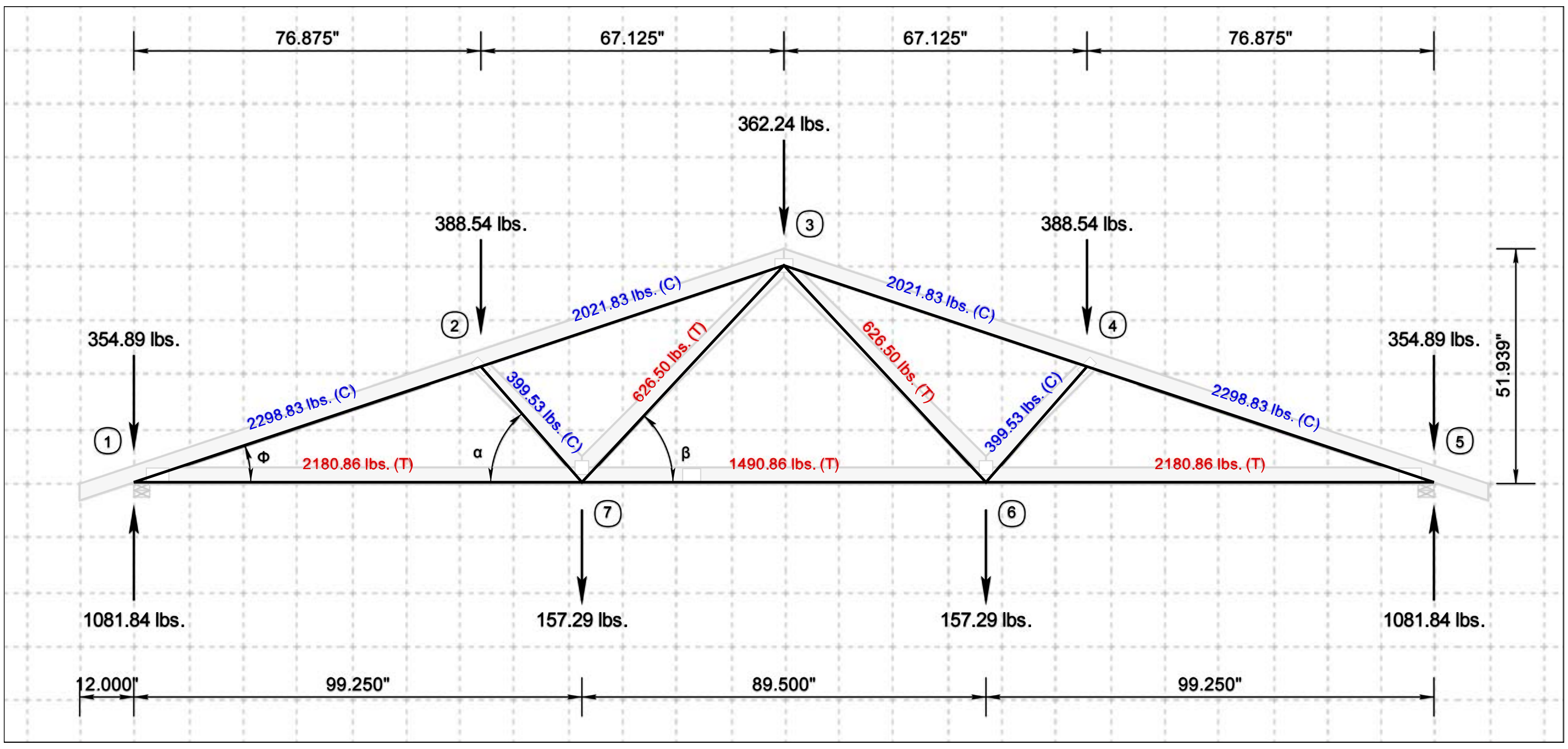
Φ: 18.43 deg.
α: 48.87 deg.
β: 47.01 deg.

PLATES (Engineering Under Construction)

Joint	Type	Plate Size	X	Y	JSI
1	MT20	3.0 x 5.0	CTR	CTR	0.98
2	MT20	2.0 x 4.0	CTR	CTR	0.32
3	MT20	3.0 x 4.0	CTR	CTR	0.79
4	MT20	2.0 x 4.0	CTR	CTR	0.32
5	MT20	3.0 x 5.0	CTR	CTR	0.98
6	MT20	3.0 x 3.0	CTR	CTR	0.70
7	MT20	3.0 x 3.0	CTR	CTR	0.70
8	MT20	3.0 x 4.0	CTR	CTR	0.58

*Plates have not been designed to provide for placement tolerances (C_q=1). It is the responsibility of the fabricator to increase plate sizes to account for this factor.

MiTek
ICC-ES Report: *ESR-1988*



Moment Calculations

SIMPLIFIED METHOD: $M = \frac{w(QL)^2}{}$

Bending moment forces in the top and bottom chords are calculated in accordance with the simplified design method outlined in Appendix D of *ANSI/TPI 1-2002*. This method is only applicable for statically determinate trusses and is deemed to be conservative.

where:

M = Top or bottom chord bending moments due to uniformly distributed loads (in.-lbs.)
w = Uniform distributed load (LL + DL) along chord member (lbs./in.)
Q = Bending and moment length factor as per Table D1 and D4 (dimensionless)
L = Panel length as per Table D1 and D4 (in.)

Top Chord @ Panel Point:

w_{tc} = 5.40 lbs./in.
Q_{pp} = 0.9 *(Table D1)*
L = $(L_i+L_a)/2$ = 67.125 in. *(Table D1)*

$$M_{pp} = \frac{w(QL)^2}{8} = \frac{5.40(0.9 \times 67.125)^2}{8} = 2461.9 \text{ in.-lbs.}$$

Top Chord @ Mid-Panel:

c = 0.5 *(Table D1, Footnote 4)*
S_a = 6.250 in.
w_{tc} = 5.40 lbs./in.
$Q_{mp} = 0.58(\cot\Phi)^{0.23}$ = 0.747 *(Table D1, Footnote 2)*
L = $(L_i+L_a)/2 + c(S_a)$ = 70.250 in. *(Table D1)*

$$M_{mp} = \frac{w(QL)^2}{8} = \frac{5.40(0.747 \times 70.250)^2}{8} = 1856.3 \text{ in.-lbs.}$$

Bottom Chord:

w_{bc} = 1.67 lbs./in.
Q_{bc} = 1.0 *(Table D4)*
$L_i = L_{bc} + S_a$ = 89.500 + 6.250 = 95.750 in.
L = $(L_i+L_a)/2$ = 92.625 in. *(Table D4)*

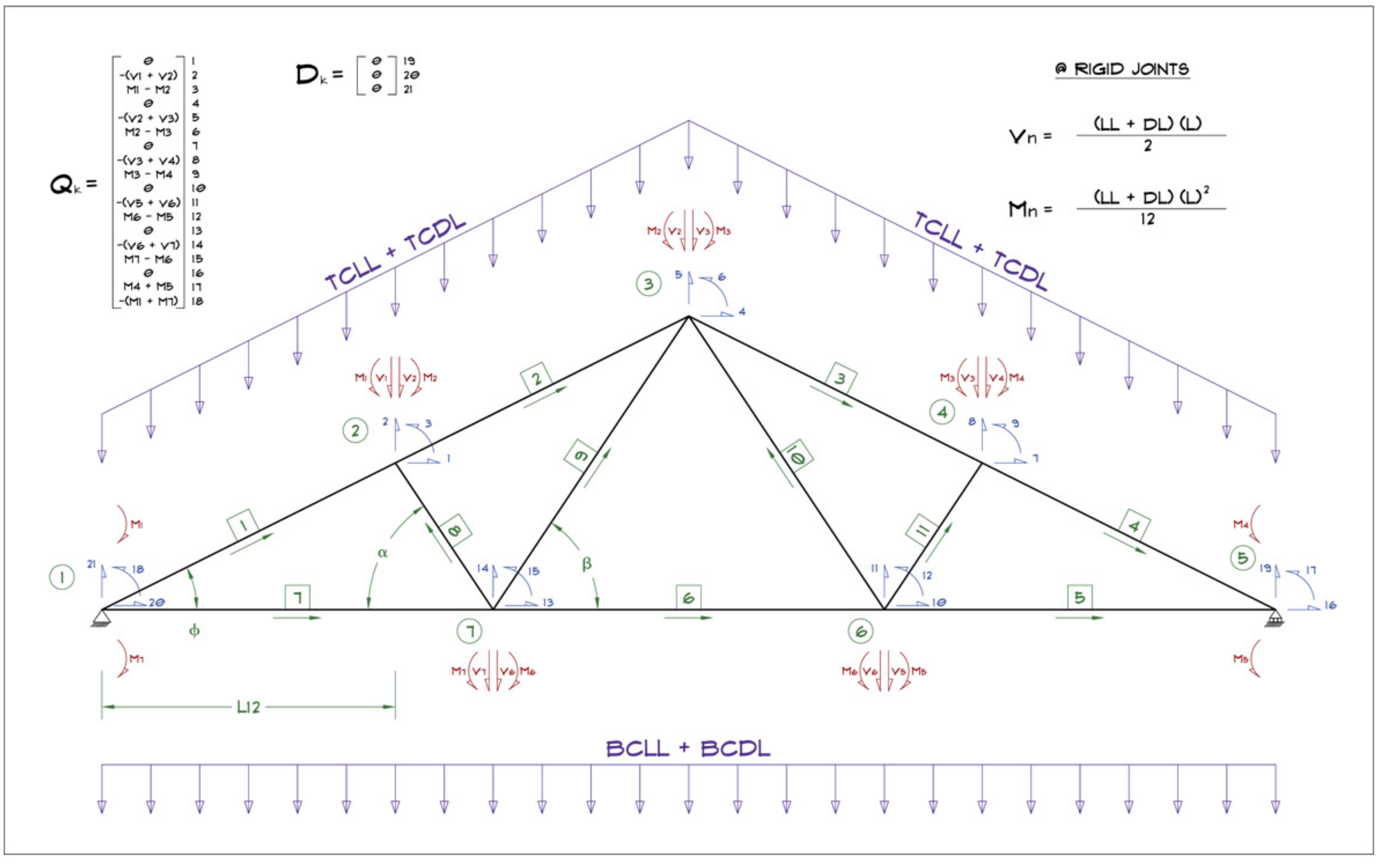
$$M_{bc} = \frac{w(QL)^2}{8} = \frac{1.67(1.0 \times 92.625)^2}{8} = 1787.4 \text{ in.-lbs.}$$

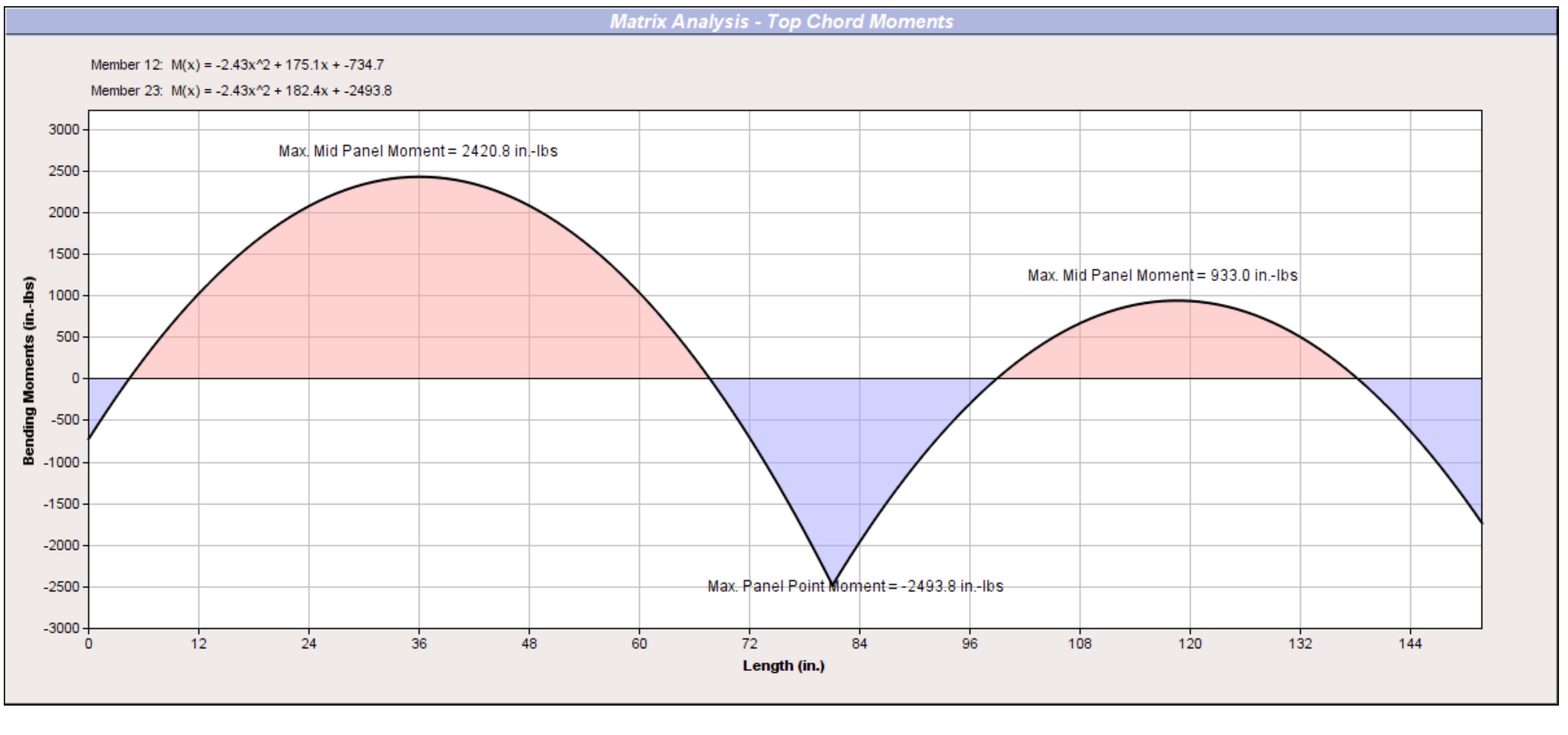
MATRIX ANALYSIS METHOD:

In accordance with the structural analysis approach outlined in Chapter 7 of the Commentary of the *ANSI/TPI 1-2007* bending moment forces in the top and bottom chords are calculated using matrix analysis in combination with accurate analog assumptions. To obtain an accurate picture of the forces present within a truss we use the following assumptions: Top and bottom chords (including top chord overhangs) are modeled as beams while web members are modeled as simple axial members. The heel joint is modeled as a rigid joint to more accurately reflect the actual behavior of the truss. The peak joint fixity is user configurable and can be either modeled as a rigid, semi-rigid or pinned joint. Web members are pin jointed to chords and other adjoining web members and can only transfer axial loads.

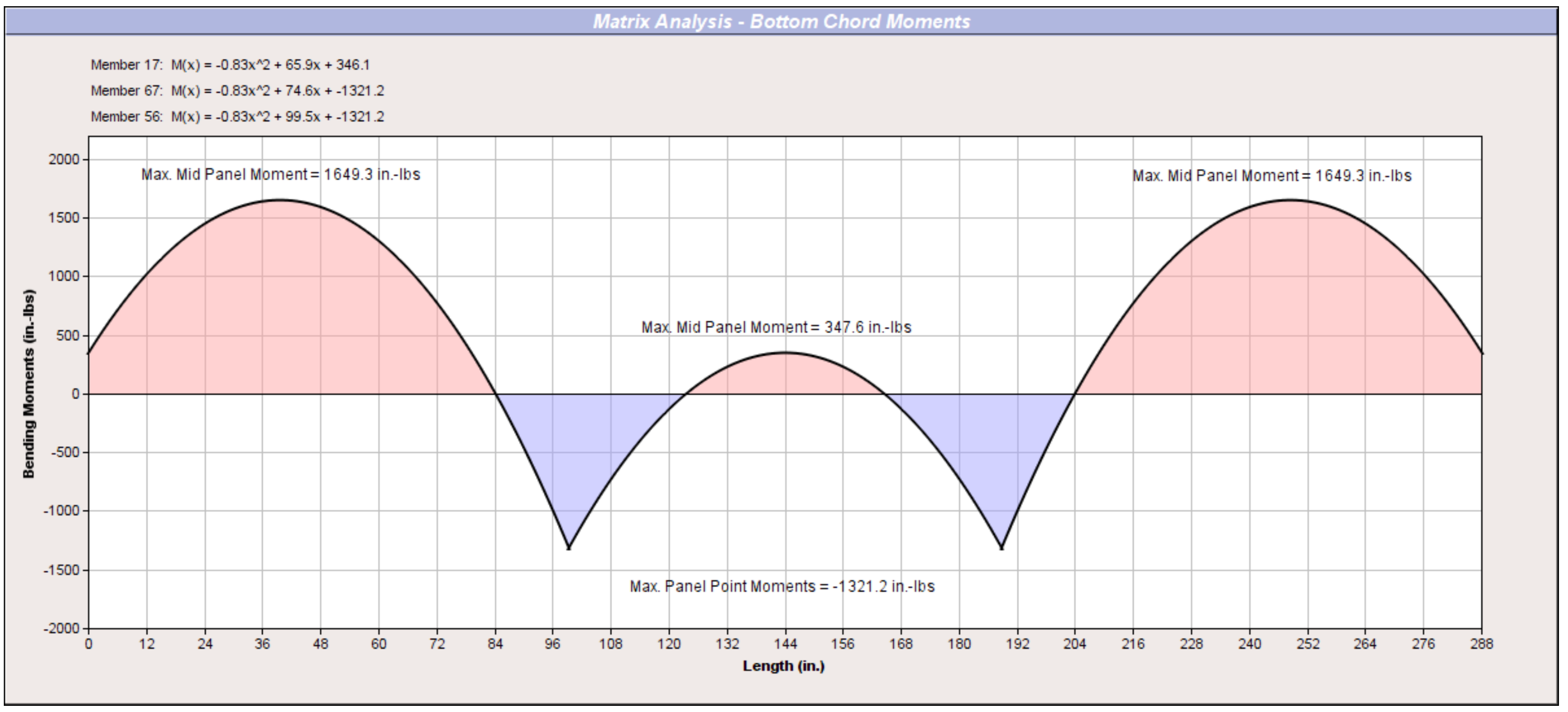
The moments present on the top and bottom chords are heavily influenced by the amount of fixity assigned to the heel and peak joints. In general modeling the joints as more rigid decreases the lumber strength required and increases the plates size whereas modeling the joints as pinned decreases the plate size and increases the required lumber strength.

Typically the axial forces calculated using Matrix Analysis are only 5-8% higher than the numbers given using classical truss analysis. As such the classical axial calculations will be used in the member design. The simplified method of moment calculations shown above is for reference while the moments derived from the matrix analysis will be used in the member design calculations below:

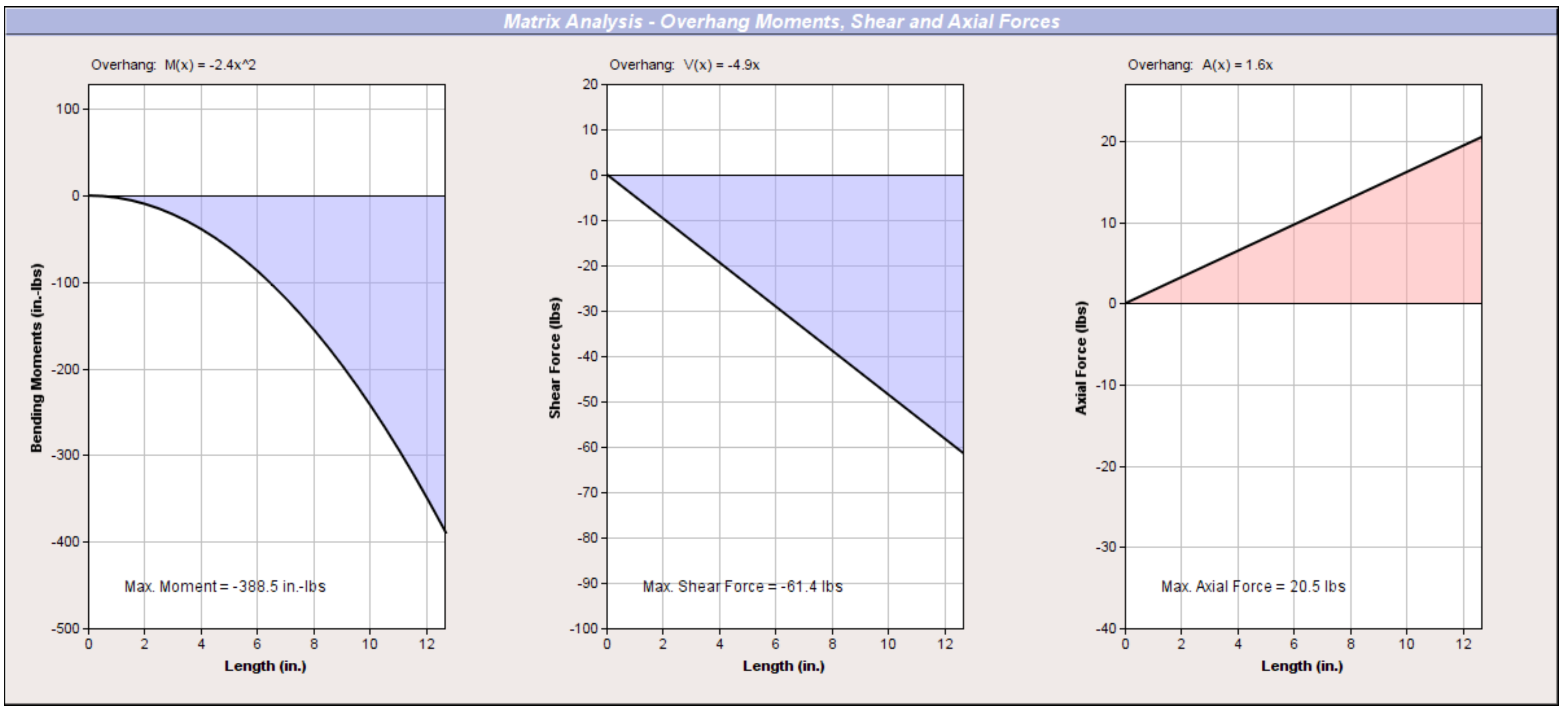




Axial Diagram Shear Diagram



Axial Diagram Shear Diagram



Member Design

Given the calculated axial forces and uniform distributed loading, it is possible to determine the lumber that will be required to resist those forces, using the ASD design methodology as outlined in the Member Design Procedures (Ch. 7) of *ANSI/TPI 1-2007* in conjunction with reference design values from the AWC 2012 NDS Supplement.
For this analysis the wet service factor (C_M), temperature factor (C_t) and incising factor (C_i) are all assumed as unity.
Common practice in the truss industry is to show a combined stress index (CSI) for each member, as determined by the ratio of the actual stress to the adjusted design value. The tension web, compression web, and top and bottom chords are considered below:

1.) Tension Web Design (37) (36):

The L'/d ratio for tension members must be checked to ensure it does not exceed 80 in order to account for short-term stress reversals due to wind or seismic loads. K_w, the length adjustment factor for webs assumes a certain amount of fixity at each end due to the metal connector plates.

$L'/d \le 80$ *(Sec. 7.3.6)*

where:

$L' = K_w(L_w)$

given:

L' = Effective buckling length (in.)
d = Web Thickness for buckling out of plane (in.)
K_w = Length adjustment factor for webs (dimensionless)
L_w = Web length (in.)

$K_w = 0.8$ *(Sec. 7.2.2.2)*
d = 1.5 (in.)
L_w = 63.29 in.

$$L'/d = \frac{K_w(L_w)}{d} = \frac{0.8(63.29)}{1.5} = 33.8 \le 80 \quad \textbf{OK}$$

. . . .

$f_t \le F_t'$ *(Sec. 7.3.1)*

where:

$f_t = P_t / A$
$F_t' = F_t(C_D)(C_M)(C_t)(C_r)(C_i)$

given:

f_t = Actual axial tension stress in web member (psi)
P_t = Axial tension in web member (lbs.)
A = Cross-sectional area of web member (in.^{2})
F_t' = Adjusted ASD tension design value (psi)
F_t = Reference ASD tension design value from NDS Table 4A (psi)
C_D = Load duration factor (dimensionless)
C_M = Wet service factor (dimensionless)
C_t = Temperature factor (dimensionless)
C_r = Repetitive member factor (dimensionless)
C_i = Incising factor (dimensionless)

Try 2 X 4 HF Stud:

F_t = 400 psi *(Table 4A)*
C_D = 1.15
C_r = 1.0 *Web Member (Sec. 6.4.2.1)*
F_t' = 400(1.15)(1.0)(1.0)(1.0)(1.0) = 460.0 psi

$$f_t = \frac{P_t}{A} = \frac{626.50}{5.25} = 119.3 \text{ psi} \le 460.0 \text{ psi} \quad \textbf{OK}$$

$$CSI = \frac{f_t}{F_t'} = \frac{119.3}{460.0} = \mathbf{0.26}$$

2.) Compression Web Design (27) (46):

The L'/d ratio for compression members must be checked to ensure it does not exceed 50 in order to account for long-term compressive loads. K_w, the length adjustment factor for webs assumes a certain amount of fixity at each end due to the metal connector plates.

$L'/d \leq 50$ *(Sec. 7.3.6)*

where:

$L' = K_w(L_w)$

given:

L' = Effective buckling length (in.)
d = Web Thickness for buckling out of plane (in.)
K_w = Length adjustment factor for webs (dimensionless)
L_w = Web length (in.)

K_w = 0.8 *(Sec. 7.2.2.2)*
d = 1.5 (in.)
L_w = 31.64 in.

$$L'/d = \frac{K_w(L_w)}{d} = \frac{0.8(31.64)}{1.5} = 16.9 \leq 50 \quad \textbf{OK}$$

. . . .

$f_c \leq F_c'$ *(Sec. 7.3.2)*

where:

$f_c = P_c / A$
$F_c' = F_c^*(C_p)$
$F_c^* = F_c(C_D)(C_M)(C_t)(C_r)(C_i)$

and:

$$C_p = (K_f) \times \left[\left(\frac{1 + F_{cE}/F_c^*}{2c} \right) - \sqrt{\left(\frac{1 + F_{cE}/F_c^*}{2c} \right)^2 - \left(\frac{F_{cE}/F_c^*}{c} \right)} \right]$$

$$F_{cE} = \frac{(.822) \times E'_{min}}{(L'/d)^2}$$

$E'_{min} = E_{min}(C_T)(C_M)(C_t)(C_r)(C_i)$

given:

f_c = Actual axial compressive stress in web member (psi)
P_c = Axial compression in web member (lbs.)
A = Cross-sectional area of web member (in.^{2})
F_c' = Adjusted ASD compressive design value parallel to grain (psi)
F_c^* = Adjusted ASD compressive design value parallel to grain @ zero slenderness ratio (psi)
F_c = Reference ASD compressive design value parallel to grain from NDS Table 4A (psi)
C_D = Load duration factor (dimensionless)
C_M = Wet service factor (dimensionless)
C_t = Temperature factor (dimensionless)
C_r = Repetitive member factor (dimensionless)
C_i = Incising factor (dimensionless)
C_T = Buckling stiffness factor (dimensionless)
K_f = Multiple ply reduction factor (dimensionless)
C_p = Column stability factor (dimensionless)
F_{cE} = Euler critical buckling stress
E_{min} = Modulus of elasticity for stability calculations (psi)
E'_{min} = Adjusted modulus of elasticity for stability calculations (psi)
c = Buckling and crushing interaction factor (dimensionless)

Try 2 X 4 HF Stud:

F_c = 800 psi *(Table 4A)*
E_{min} = 440000 psi *(Table 4A)*
C_D = 1.15
C_T = 1.0 *Web Member, No Structural Panel Sheathing Attached (Sec. 6.4.5.1)*
C_r = 1.0 *Web Member (Sec. 6.4.2.1)*
K_f = 1.0 *Single Ply Member*
c = 0.8 *for sawn lumber*

E'_{min} = 440000(1.0)(1.0)(1.0)(1.0)(1.0) = 440000 psi

F_c^* = 800(1.15)(1.0)(1.0)(1.0)(1.0) = 920.0 psi

$$F_{cE} = \frac{(.822) \times E'_{min}}{(L'/d)^2} = \frac{(.822)(440000)}{(16.9)^2} = 1269.9 \text{ psi}$$

$$C_p = (1.0) \times \left[\left(\frac{1 + 1269.9/920.0}{1.6} \right) - \sqrt{\left(\frac{1 + 1269.9/920.0}{1.6} \right)^2 - \left(\frac{1269.9/920.0}{0.8} \right)} \right] = 0.79$$

$F_c' = F_c^*(C_p)$ = 920.0(0.79) = 726.1 psi

$$f_c = \frac{P_c}{A} = \frac{399.53}{5.25} = 76.1 \text{ psi} \leq 726.1 \text{ psi} \quad \textbf{OK}$$

$$CSI = \frac{f_c}{F_c'} = \frac{76.1}{726.1} = \mathbf{0.10}$$

3.) Top Chord Design (12) (45):

Top chord design requires that two checks be made: one for panel point conditions and the other at mid-panel. The mid-panel check is for a positive moment and the axial force imposed by the load while the panel point check is for the negative moment that occurs across the panel point due to the support of the web members intersecting with the chord at that location and the axial load.
The critical top chord member is (12) because it has the highest axial force. The top chord will experience both axial compression and bending, therefore a combined stress analysis is required.

The L'/d ratio for the top chord (compressive member) must be checked to ensure it does not exceed 50 in order to account for long-term compressive loads. K, the effective buckling length adjustment factor for chords for buckling in the plane of the truss is now given by Kavanagh's equation (Sec. 7.2.1.1) in the *ANSI/TPI 1-2007*.

$L'/d \leq 50$ *(Sec. 7.3.6)*

where:

$L' = K(L_t)$

and:

$$K = \sqrt{\frac{(\pi^2 + 2N_a)(\pi^2 + 2N_b)}{(\pi^2 + 4N_a)(\pi^2 + 4N_b)}}$$

$$N_a = \frac{\left(\frac{4E_a I_a}{L_a}\right)}{\left(\frac{E_t I_t}{L_t}\right)}$$

$$N_b = \frac{\left(\frac{4E_b I_b}{L_b}\right)}{\left(\frac{E_t I_t}{L_t}\right)}$$

given:

L' = Effective buckling length for chord member(in.)
d = Web Thickness for buckling in plane (in.)
K = Effective length adjustment factor for chords (dimensionless)

L_t = Unbraced length of member for which K and L' is being determined (in.)
E_t = Modulus of Elasticity of chord member for which K is being determined (psi)
I_t = Moment of inertia of chord member for which K is being determined (in^4)

L_a = Unbraced length of member adjacent to member for which K and L' is being determined (in.)

E_a = Modulus of Elasticity of member adjacent to chord member for which K is being determined (psi)
I_a = Moment of inertia of member adjacent to chord member for which K is being determined (in^4)

L_b, E_b, I_b = The same as L_a, E_a, I_a except for the adjacent chord member to the opposed side of the member for which K is being determined. If there is no such adjacent member , N_b shall equal zero.

Try 2 X 4 HF No.1:

L_t = 70.756 in.
L_a = 70.756 in.
d = 3.5 in.
$E_t = E_a$ = 1500000 psi *(Table 4A)*
$I_t = I_a = I_x$ = 5.36 in^4
N_a = (4 x 1500000 x 5.36/70.756)/(1500000 x 5.36/70.756) = 4.0
N_b = 0.0

$$K = \sqrt{\frac{(\pi^2 + 2(4.0))\ (\pi^2 + 2(0))}{(\pi^2 + 4(4.0))\ (\pi^2 + 4(0))}} = 0.831$$ (Sec. 7.2.1.1)

$$L'/d = \frac{K(L_t)}{d} = \frac{0.831(70.756)}{3.5} = 16.8 \le 50$$ **OK**

@ Mid-Panel (12) (45):

Interaction equation for combined bending and compression:

$$\left(\frac{f_c}{F_c'}\right)^2 + \left[\frac{f_{bx}}{F_{bx}' \times \left(1 - \frac{f_c}{F_{cEx}}\right)}\right] + \left[\frac{f_{by}}{F_{by}' \times \left[1 - \frac{f_c}{F_{cEy}} - \left(\frac{f_{bx}}{F_{bE}}\right)^2\right]}\right] \le 1.00$$ *(Sec. 7.3.5.1)*

where:

$f_c < F_{cEx}$
$f_c < F_{cEy}$
$f_{bx} < F_{bE}$

F_{bx}' and F_{by}' are the adjusted design values for bending in the plane of the truss and out of the plane of the truss, respectively. Both are calculated as F_b' with the equation shown below:

$$F_b' = F_b^* \times \left[\left(\frac{1 + \frac{F_{bE}}{F_b^*}}{1.9}\right) - \sqrt{\left(\frac{1 + \frac{F_{bE}}{F_b^*}}{3.61}\right)^2 - \left(\frac{\frac{F_{bE}}{F_b^*}}{0.95}\right)}\right]$$ *(Sec. 7.3.3.1)*

members subject to bending stresses shall be proportioned so that:

$f_b \le F_b'$ *(Sec. 7.3.3)*

where:

$f_b = M / S$
$F_b^* = F_b(C_D)(C_M)(C_t)(C_r)(C_{fu})(C_i)(K_m)$

given:

f_b = Actual bending stress in web member (psi)
M = Bending moment in web member (lbs.-in)
S = Section Modulus of web member ($in.^3$)
F_b' = Adjusted ASD bending design value (psi)
F_b^* = Adjusted ASD bending design value @ zero slenderness ratio (psi)
F_b = Reference ASD bending design value from NDS Table 4A (psi)
C_D = Load duration factor (dimensionless)
C_M = Wet service factor (dimensionless)
C_t = Temperature factor (dimensionless)
C_r = Repetitive member factor (dimensionless)
C_{fu} = Flat use factor (dimensionless)
C_i = Incising factor (dimensionless)
C_T = Buckling stiffness factor (dimensionless)
K_m = Bending capacity modification factor (dimensionless)

Since the top chord only experiences bending due to vertical loads and the chord member is braced throughout its length by continuous sheathing to prevent lateral displacement (Sec. 7.3.3.4 and Sec. 7.3.5.2) bending and buckling values need only be checked within the plane of the truss. The bending and compression interaction equation and bending equation reduce to:

$$\left(\frac{f_c}{F_c'}\right)^2 + \left[\frac{f_{bx}}{F_{bx}' \times \left(1 - \frac{f_c}{F_{cEx}}\right)}\right] \le 1.00$$

$F_b' = F_b^*$

The axial compressive side of the interaction equation is calculated similar to the web compression calculations above with the exception that C_T (buckling stiffness factor) is given by the equation below:

$$C_T = 1 + \frac{2300 \times L'}{k \times E}$$ *(Sec. 6.4.5)*

$$F_{cE} = \frac{(.822) \times E'_{min}}{(L'/d)^2}$$

$E'_{min} = E_{min}(C_T)(C_M)(C_t)(C_r)(C_i)$

given:

C_T = Buckling stiffness factor (dimensionless)
L' = Effective buckling length for chord member, but not greater than 96" (in.)
E = Modulus of Elasticity of chord member from NDS Table 4A (psi)
k = 0.59 for $COV_e \approx 0.25$ for visually graded lumber (dimensionless) *(Sec. 6.4.5.2)*
C_D = 1.15
C_r = 1.15 (bending) *(Sec. 6.4.2.1)*
C_r = 1.10 (compression & E_{min}) *(Sec. 6.4.2.1)*
K_f = 1.0 *Single Ply Member*
c = 0.8 *for sawn lumber*

Try 2 X 4 HF No.1:

F_c = 1350 psi *(Table 4A)*
F_b = 975 psi *(Table 4A)*
E = 1500000 psi *(Table 4A)*
E_{min} = 550000 psi *(Table 4A)*

L' = K(L_t) = 0.831(70.756) = 58.81 in.

$$f_c = \frac{P_c}{A} = \frac{2298.83}{5.25} = 437.9 \text{ psi}$$

$$f_b = \frac{M_{mp}}{S_x} = \frac{2420.8}{3.06} = 790.5 \text{ psi}$$

$$C_T = 1 + \frac{2300 \times (L')}{k \times E} = 1 + \frac{(2300)(58.81)}{(0.59)(1500000)} = 1.15$$

E'_{min} = 550000(1.15)(1.0)(1.0)(1.10)(1.0) = 697462 psi

F_c^* = 1350(1.15)(1.0)(1.0)(1.10)(1.0) = 1707.7 psi

$$F_{cE} = \frac{(.822) \times E'_{min}}{(L'/d)^2} = \frac{(.822)(697462)}{(16.8)^2} = 2030.8 \text{ psi}$$

$$C_p = (1.0) \times \left[\left(\frac{1 + 2030.8/1707.7}{1.6}\right) - \sqrt{\left(\frac{1 + 2030.8/1707.7}{1.6}\right)^2 - \left(\frac{2030.8/1707.7}{0.8}\right)}\right] = 0.75$$

$F_c' = F_c^*(C_p)$ = 1707.7(0.75) = 1276.2 psi

$F_b' = F_b^* = F_b(C_D)(C_M)(C_t)(C_r)(C_{fu})(C_i)(K_m)$ = 975(1.15)(1.0)(1.0)(1.15)(1.0)(1.0)(1.0) = 1289.4 psi

$$CSI = \left(\frac{437.9}{1276.2}\right)^2 + \left[\frac{790.5}{1289.4 \times \left(1 - \frac{437.9}{2030.8}\right)}\right] = \mathbf{0.90} \le 1.00$$ **OK**

@ Panel Point (12) (45):

The intersection of the webs with the top chord will provide lateral stability for the chord in the x-plane and the column stability factor C_p is assumed to be 1.0 at a panel point. Similarily the second order bending effect caused by the axial load and chord deflection is negligible and the amplification factor is assumed to be unity (Sec. 7.3.2 and Sec. 7.3.5.1). The bending and compression interaction equation and compression equation further reduce to:

$[1 - f_c/F_{cEx}] = 1$

thus:

$$\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{f_{bx}}{F_{bx}'}\right) \leq 1.00$$ *(Sec. 7.3.5.1)*

and:

$C_p = 1.0$

thus:

$F_c' = F_c*$ *(Sec. 7.3.2)*

Try 2 X 4 HF No.1:

$F_c = 1350$ psi *(Table 4A)*
$F_b = 975$ psi *(Table 4A)*

$$f_c = \frac{P_c}{A} = \frac{2298.83}{5.25} = 437.9 \text{ psi}$$

$$f_b = \frac{M_{pp}}{S_x} = \frac{2493.8}{3.06} = 814.3 \text{ psi}$$

$F_c' = F_c* = F_c(C_D)(C_M)(C_t)(C_F)(C_i) = 1350(1.15)(1.0)(1.0)(1.10)(1.0) = 1707.7$ psi

$F_b' = F_b* = F_b(C_D)(C_M)(C_t)(C_F)(C_{fu})(C_i)(K_m) = 975(1.15)(1.0)(1.0)(1.15)(1.0)(1.0)(1.0) = 1289.4$ psi

$$\text{CSI} = \left(\frac{437.9}{1707.7}\right)^2 + \left[\frac{814.3}{1289.4}\right] = \mathbf{0.70} \leq 1.00 \text{ OK}$$

4.) Bottom Chord Design (17) (56):

The bottom chord is also subject to combined axial and bending loads. The bottom chord generally carries a tensile load due to transfer of forces from the top chord. The bottom chord may also carry a bending load due to ceiling sheathing. The critical bottom chord member is (17) because it has the highest axial force and the largest mid-panel moment due to its length. The bottom chord will experience both axial tension and bending, therefore a combined stress analysis is required.

The L'/d ratio for the bottom chord (tension member) must be checked to ensure it does not exceed 80 in order to account for short-term compressive loads. Since the bottom chord is not continuously braced it needs to be checked for both buckling in the plane of the truss and buckling out of the plane of the truss. K, the effective buckling length adjustment factor for chords for buckling in the plane of the truss is now given by Kavanagh's equation (Sec. 7.2.1.1) in the *ANSI/TPI 1-2007*.

$L'/d \leq 80$ *(Sec. 7.3.6)*

Checking in-plane L'/d:

$L' = K(L_t)$

equations for K, N_a, N_b with given variables are idential to the Top Chord L'/d analysis shown above:

Try 2 X 4 HF No.1:

$L_t = 95.750$ in.
$L_a = 89.500$ in.
d = 3.5 in.
$E_t = E_a = 1500000$ psi *(Table 4A)*
$I_t = I_a = I_x = 5.36$ in^4
$N_a = (4 \times 1500000 \times 5.36/89.500)/(1500000 \times 5.36/95.750) = 4.3$
$N_b = 0.0$

$$K = \sqrt{\frac{(\pi^2 + 2(4.3))\ (\pi^2 + 2(0))}{(\pi^2 + 4(4.3))\ (\pi^2 + 4(0))}} = 0.826$$ (Sec. 7.2.1.1)

$$L'/d = \frac{K(L_t)}{d} = \frac{0.826(95.750)}{3.5} = 22.6 \leq 80 \text{ OK}$$

Checking out-of-plane L'/d:

$L' = L_u$ *(Sec. 7.3.5.3)*

given:

$L_u = 120.0$ in.
d = 1.5 in.

$$L'/d = \frac{L_u}{d} = \frac{120.0}{1.5} = 80.0 \leq 80 \text{ OK}$$

@ Mid-Panel (17) (56):

Interaction equation for combined bending and tension:

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} \leq 1.00$$ *(Sec. 7.3.4)*

with Net Compressive Stress Check:

$f_b - f_t \leq F_b'$ *(E7.3-6)*

where:

$f_b = M / S$
$F_b* = F_b(C_D)(C_M)(C_t)(C_F)(C_{fu})(C_i)(K_m)$

and:

$f_t = P_t / A$
$F_t' = F_t(C_D)(C_M)(C_t)(C_F)(C_i)$

given:

f_b = Actual bending stress in web member (psi)
M = Bending moment in web member (lbs.-in)
S = Section Modulus of web member (in.^{3})
F_b' = Adjusted ASD bending design value (psi)
F_b* = Adjusted ASD bending design value @ zero slenderness ratio (psi)
F_b = Reference ASD bending design value from NDS Table 4A (psi)
f_t = Actual axial tension stress in web member (psi)
P_t = Axial tension in web member (lbs.)
A = Cross-sectional area of web member (in.^{2})
F_t' = Adjusted ASD tension design value (psi)
F_t = Reference ASD tension design value from NDS Table 4A (psi)

and:

C_D = Load duration factor (dimensionless)
C_M = Wet service factor (dimensionless)
C_t = Temperature factor (dimensionless)
C_r = Repetitive member factor (dimensionless)
C_{fu} = Flat use factor (dimensionless)
C_i = Incising factor (dimensionless)
K_m = Bending capacity modification factor (dimensionless)

If the bottom chord's depth-to-thickness ratio does not exceed 5 to 1 and truss spacing is no more than 24 in. on center with bracing along full length of chord, as per ANSI/TPI 1-2007 (Sec. 7.3.3.4 and Sec. 7.3.3.5) lateral buckling is prevented, and F_b' shall be equal to F_b*:

$F_b' = F_b*$ *(Sec. 7.3.3.5)*

However, in the case where the compressive (top edge) side of the bottom chord is not fully laterally supported along its length then the member may buckle laterally in a manner similar to a slender column. Buckling takes place between points of lateral support, which in our case is specified as 10 ft. o/c.
The adjusted ASD bending design value is then calculated as:

$F_b' = F_b*(C_L)$ *(Sec. 7.3.3.1)*

where:

$$C_L = \left(\frac{1+\frac{F_{bE}}{F_b^*}}{1.9}\right) - \sqrt{\frac{\left(1+\frac{F_{bE}}{F_b^*}\right)^2}{3.61} - \left(\frac{\frac{F_{bE}}{F_b^*}}{0.95}\right)}$$

$$F_{bE} = \frac{1.2 \times E_{min}'}{{R_B}^2}$$

$$R_B = \sqrt{\frac{L_e \times d_1}{{d_2}^2}} < 50$$

and:

$L_e = 1.84L_u$ (when $L_u / d_1 > 14.3$)

$E'_{min} = E_{min}(C_T)(C_M)(C_t)(C_r)(C_i)$

given:

C_L = Beam stability factor (dimensionless)
F_{bE} = Euler-based critical buckling stress for bending members (psi)
E'_{min} = Adjusted modulus of elasticity for stability calculations (psi)
R_B = Slenderness ratio for a bending member (dimensionless)
d_1 = Beam depth (in.)
d_2 = Beam width (in.)
L_e = Effective unbraced length (in.)
L_u = Unbraced length of beam, distance between points of lateral support (in.)

given:

C_D = 1.15 *(Sec. 6.4.1)*
C_r = 1.10 (bending) *No Structural Panel Sheathing Attached (Sec. 6.4.2.1)*
C_r = 1.10 (tension & E_{min}) *(Sec. 6.4.2.1)*

Try 2 X 4 HF No.1:

F_t = 625 psi *(Table 4A)*
F_b = 975 psi *(Table 4A)*
E_{min} = 550000 psi *(Table 4A)*

$$f_t = \frac{P_t}{A} = \frac{2180.86}{5.25} = 415.4 \text{ psi}$$

$$f_b = \frac{M_{bc}}{S_x} = \frac{1649.3}{3.06} = 538.6 \text{ psi}$$

$F_t' = F_t(C_D)(C_M)(C_t)(C_r)(C_i)$ = 625(1.15)(1.0)(1.0)(1.10)(1.0) = 790.6 psi

$F_b^* = F_b(C_D)(C_M)(C_t)(C_r)(C_{fu})(C_i)(K_m)$ = 975(1.15)(1.0)(1.0)(1.10)(1.0)(1.0)(1.0) = 1233.4 psi

$$CSI = \frac{415.4}{790.6} + \frac{538.6}{1233.4} = \mathbf{0.96} \le 1.00$$ **OK**

Checking Net Compressive Stress:

C_T = 1.0 *Member in Tension, No Structural Panel Sheathing Attached (Sec. 6.4.5.1)*
L_u = 120.0 in.
d_1 = 3.5 in.
d_2 = 1.5 in.

E'_{min} = 550000(1.0)(1.0)(1.0)(1.10)(1.0) = 605000 psi

L_e = 1.84(120.0) = 220.8 in.

$$R_B = \sqrt{\frac{220.8 \times 3.5}{1.5^2}} = 18.5 < 50$$

$$F_{bE} = \frac{1.2 \times 605000}{18.5^2} = 2113.7 \text{ psi}$$

$$C_L = \left(\frac{1+\frac{2113.7}{1233.4}}{1.9}\right) - \sqrt{\frac{\left(1+\frac{2113.7}{1233.4}\right)^2}{3.61} - \left(\frac{\frac{2113.7}{1233.4}}{0.95}\right)} = 0.94$$

F_b' = 1233.4(0.94) = 1162.4 psi

$f_b - f_t$ = 538.6 - 415.4 = 123.2 psi ≤ 1162.4 psi **OK**

Bearing Check at Heel

Bearing perpendicular to the grain of the wood occurs at the contact area where the truss rest on the top plates of the wall framing. The compressive stress is given by the reactions and the net bearing area and should not exceed the design values given by the following equations:

$f_{c\perp} \le F_{c\perp}'$ *(Sec. 7.3.8.1)*

$f_{c\perp} \le E' \times (d_2/d_1)^2/20$ *(Sec. 7.3.8.1)*

where:

$f_{c\perp} = R / A_b$
$F_{c\perp}' = F_{c\perp}(C_M)(C_t)(C_i)(C_b)(C_{plate})$
$E' = E(C_M)(C_t)(C_i)$

given:

$f_{c\perp}$ = Actual compressive bearing stress in member (psi)
$F_{c\perp}$ = Reference ASD compression perpendicular to grain design value from NDS Table 4A (psi)
$F_{c\perp}$ = Adjusted ASD compression perpendicular to grain design value (psi)
E = Modulus of elasticity of chord member from NDS Table 4A (psi)
E' = Adjusted modulus of elasticity of chord member (psi)
R = Reaction force transferred through bearing area (lbs.)
A_b = Net bearing area (in.^{2})
d_1 = Heel height as shown in Fig. 7.3-6 of ANSI/TPI 1-2007 (in.)
d_2 = Thickness of chord member(s) at bearing (in.)

C_M = Wet service factor (dimensionless)
C_t = Temperature factor (dimensionless)
C_i = Incising factor (dimensionless)
C_b = Bearing area factor (dimensionless)
C_{plate} = Bearing plate increase factor (dimensionless)

Checking Bearing at Heel with 2 X 4 HF No.1 Bottom Chord:

$F_{c\perp}$ = 405 psi *(Table 4A)*
E = 1500000 psi *(Table 4A)*
d_1 = 3.261 in. *(Sec. 7.3.8.1.2)*
A_b = 5.25 in.
C_b = 1.0 *(Sec. 7.3.8.2)*
C_{plate} = 1.0 *(Sec. 7.3.8.3)*

$F_{c\perp}' = F_{c\perp}(C_M)(C_t)(C_i)(C_b)(C_{plate})$ = 405(1.0)(1.0)(1.0)(1.0)(1.0) = 405.0 psi

$$f_{c\perp} = \frac{R}{A_b} = \frac{1081.84}{5.25} = 206.1 \text{ psi} \le 405.0 \text{ psi} \quad (CSI = 0.51)$$ **OK**

$E' = E(C_M)(C_t)(C_i)$ = 1500000(1.0)(1.0)(1.0) = 1500000 psi

206.1 psi ≤ $E' \times (d_2/d_1)^2/20$ = 1500000 x $(1.5/3.261)^2/20$ = 15865.5 psi (CSI = 0.01) **OK**

Metal Connector Plate Joint Design

Given the calculated axial forces and moments acting at the joints of a truss it is possible to design the connector plates at these joints such that they meet the requirements of chapter 8 of the *ANSI/TPI 1-2007* for lateral resistance, tensile and shear strengths, moment capacity for both steel section and lateral resistance and net section for axial tension and compression. All joint design procedures presented are for trusses with web members that are cut to bear on a surface, and not for round-ended or squared-ended webs.

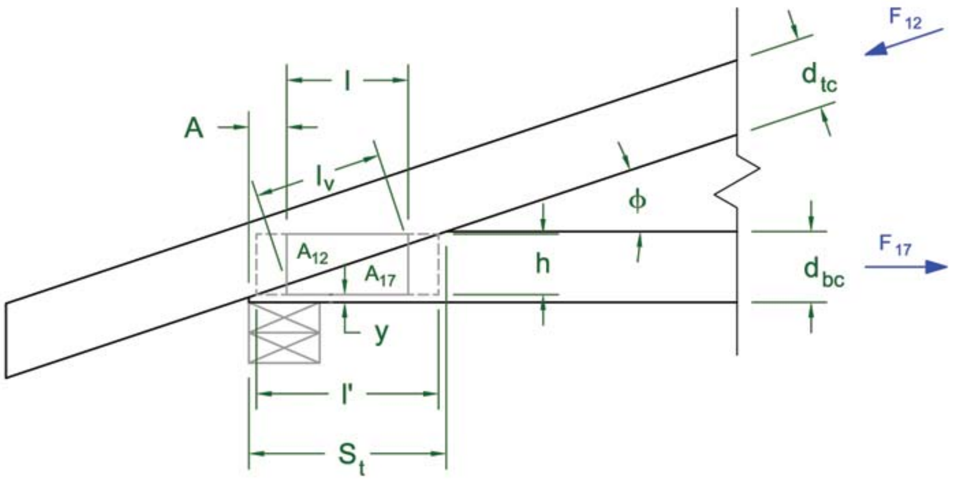
For trusses with spans exceeding 16 ft. in length, the minimum axial design force for any member to be used when designing metal connector plates shall be not less than 375 lbs. as per ANSI/TPI 1-2007 (Sec. 8.2).

Common practice in the truss industry is to show a joint stress index (JSI) for each joint, which effectively quantifies the stress level of a plate contact area and all applicable design forces acting at the joint. It is computed by taking the ratio of applied force to the allowable design force with a JSI computed for each design check and the maximum JSI controlling.

The heel joint, peak joint, top chord web joint, bottom chord web joint and bottom chord splice joint are considered below:

1.) Joint (1) (5) Heel Joint: (JSI=0.98)

Assume symetrical placement of heel plate centered on scarf line. For simplicity we only consider a single, symetric heel plate in which its height does not exceed the depth of the bottom chord. Given the definitions, calculations and corresponding figure below we establish the geometry of the heel joint:



Joint (1) (5) Heel Joint

where:

h = Actual height of heel plate (in.)
l = Actual length of heel plate (in.)
l' = Maximum shear length of a plate at a given height (in.)
l_v = Length along the scarf line through the plate (in.)
d_{tc} = Actual top chord depth (in.)
d_{bc} = Actual bottom chord depth (in.)
y = Clearance between the bottom of the plate and the bottom of the bottom chord (in.)
A = Distance from heel plate to butt of truss (in.)
S_t = Total scarf (in.)
buttcut = Vertical cut at the outside edge of bottom chord (in.)

Try MT20 3.0 x 5.0 plate:

h = 3.0 in.
l = 5.0 in.

$$l' = \frac{h}{\tan\Phi} = \frac{3.0}{\tan(18.43)} = 9.0 \text{ in.}$$

$$S_t = \frac{d_{bc} - \text{buttcut}}{\tan\Phi} = \frac{3.5 - 0.25}{\tan(18.43)} = 9.750 \text{ in.}$$

for l' < l: $$l_v = h \times \sqrt{1 + \left(\frac{1}{\tan\phi}\right)^2}$$

for l' ≥ l: $$l_v = l \times \frac{\sqrt{1 + \left(\frac{1}{\tan\phi}\right)^2}}{\frac{1}{\tan\phi}} = 5 \times \frac{\sqrt{1 + \left(\frac{1}{\tan(18.43)}\right)^2}}{\frac{1}{\tan(18.43)}} = 5.27 \text{ in.}$$

$$A = \frac{S_t - l}{2} = \frac{9.750 - 5.0}{2} = 2.375 \text{ in.}$$

$$y = \frac{d_{bc} + \text{buttcut} - h}{2} = \frac{3.5 + 0.25 - 3}{2} = 0.375 \text{ in.}$$

To allow for moment effects, Metal Connector Plates at heel joints shall be designed to have sufficient capability to withstand the direct axial force of the top and bottom chords using lateral resistance design values multiplied by the following reduction factor, H_R:

$H_R = 0.85 - 0.05(12 \times \tan\Phi - 2.0)$ *(Sec. 8.3.2.2)*

where:

$0.65 \leq H_R \leq 0.85$

For a 4/12 top chord pitch:

$H_R = 0.85 - 0.05[12 \times \tan(18.43) - 2.0] = 0.750$

A.) Checking A17: Bottom Chord in tension at heel joint:

i.) Lateral Resistance:

$P_t = F_{17} = 2180.86$ lbs.

Mitek MT20 Plate with HF:
$V_{LRAA} = 188$ psi/plate

Direction of Loading:
α = 0.0 degrees (angle between load direction and slot direction)
θ = 0.0 degrees (angle between load direction and grain direction)

$\therefore V_{LR} = V_{LRAA} = 188.0$ psi/plate

$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R) = 188.0(1.15)(1.0)(1.0)(0.750) = 162.1$ psi/plate

$$A_{17(req)} = \frac{P_t}{2(V_{LR}')} = \frac{2180.9}{2 \times 162.1} = 6.72 \text{ in.}^2$$

$$A_{17} = \frac{hl}{2} = \frac{3.0 \times 5.0}{2} = 7.50 \text{ in.}^2 > 6.72 \text{ in.}^2 \quad (\text{JSI} = 0.90) \quad \textbf{OK}$$

ii.) Tension:

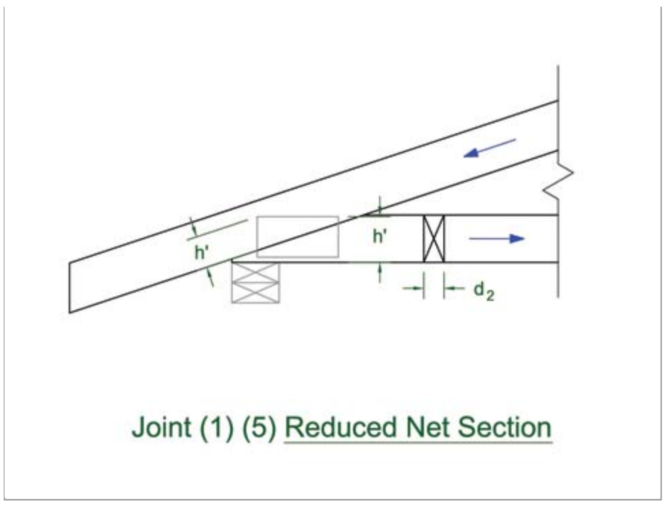
Mitek MT20 Plate:
V_t @ 0° = 857.0 lbs/in/pair

$$W_{p(req)} = \frac{P_t}{V_t} = \frac{2180.9}{857.0} = 2.54 \text{ in.}$$

$W_p = l_v = 5.27$ in. > 2.54 in. (JSI = 0.48) **OK**

iii.) Net Section (H'):

At all joints, members shall have Metal Connetor Plates sized or positioned so that the allowable axial tension stress, F_t', of any wood member, or the allowable axial compressive stress, F_c*, of any wood member at any joint without wood-to-wood bearing in the diretion of the axial force, is not exceeded on the reduced net section, h' times d_2. Typical reduced net sections for a heel joint shown below:



$f_{tnet} \le F_t'$

$$h' = h + y - \tan(\Phi)(\frac{l' - l}{2}) = 3 + 0.375 - \tan(18.43)(\frac{9-5}{2}) = 2.708 \text{ in.}$$

$$f_{tnet} = \frac{P_t}{h' \times d_2} = \frac{2180.9}{2.708 \times 1.5} = 536.8 \text{ psi} \le 790.6 \text{ psi} \quad \text{(JSI = 0.68)} \quad \textbf{OK}$$

A.) Checking A12: Top Chord in compression at heel joint:

i.) Lateral Resistance:

$P_c = F_{12} = 2298.83$ lbs.

Mitek MT20 Plate with HF:
$V_{LRAA} = 188$ psi/plate
$V_{LREA} = 159$ psi/plate
$V_{LRAE} = 133$ psi/plate
$V_{LREE} = 141$ psi/plate

Direction of Loading:
$\alpha = 18.43$ degrees (angle between load direction and slot direction)
$\theta = 0.0$ degrees (angle between load direction and grain direction)

$$V_{LRA\theta} = \frac{V_{LRAA} \times V_{LRAE}}{(V_{LRAA} \times \sin^2\theta) + (V_{LRAE} \times \cos^2\theta)} = \frac{188 \times 133}{(188 \times \sin^2 0.0) + (133 \times \cos^2 0.0)} = 188.0 \text{ psi/plate}$$

$$V_{LRE\theta} = \frac{V_{LREA} \times V_{LREE}}{(V_{LREA} \times \sin^2\theta) + (V_{LREE} \times \cos^2\theta)} = \frac{159 \times 141}{(159 \times \sin^2 0.0) + (141 \times \cos^2 0.0)} = 159.0 \text{ psi/plate}$$

$$V_{LR} = \frac{((90-\alpha) \times V_{LRA\theta}) + (\alpha \times V_{LRE\theta})}{90} = \frac{((90-18.43) \times 188.0) + (18.43 \times 159.0)}{90} = 182.1 \text{ psi/plate}$$

$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R) = 182.1(1.15)(1.0)(1.0)(0.750) = 157.0$ psi/plate

$$A_{12(req)} = \frac{P_c}{2(V_{LR}')} = \frac{2298.8}{2 \times 157.0} = 7.32 \text{ in.}^2$$

$$A_{12} = \frac{hl}{2} = \frac{3.0 \times 5.0}{2} = 7.50 \text{ in.}^2 > 7.32 \text{ in.}^2 \quad \text{(JSI = 0.98)} \quad \textbf{OK}$$

ii.) Shear:

Mitek MT20 Plate:
V_s @ 0° = 604 lbs/in/pair
V_s @ 30° = 876 lbs/in/pair
V_s @ 60° = 970 lbs/in/pair

Direction of Loading:
$\alpha_s = 18.43$ degrees (angle between force parallel to joint shear plane and slot direction)

Linear interpolation for allowable shear design value at angle of load with respect to plate slot direction:

$$V_{s\ @\ 18.43} = V_{s0} + (V_{s1} - V_{s0}) \frac{\alpha_s - \alpha_o}{\alpha_1 - \alpha_o} = 604 + (876 - 604) \frac{18.43 - 0}{30 - 0} = 771.1 \text{ lbs/in/pair}$$

$$W_{p(req)} = \frac{P_s}{V_s} = \frac{2298.8}{771.1} = 2.98 \text{ in.}$$

$W_p = l_v = 5.27$ in. > 2.98 in. (JSI = 0.57) **OK**

iii.) Net Section (H'):

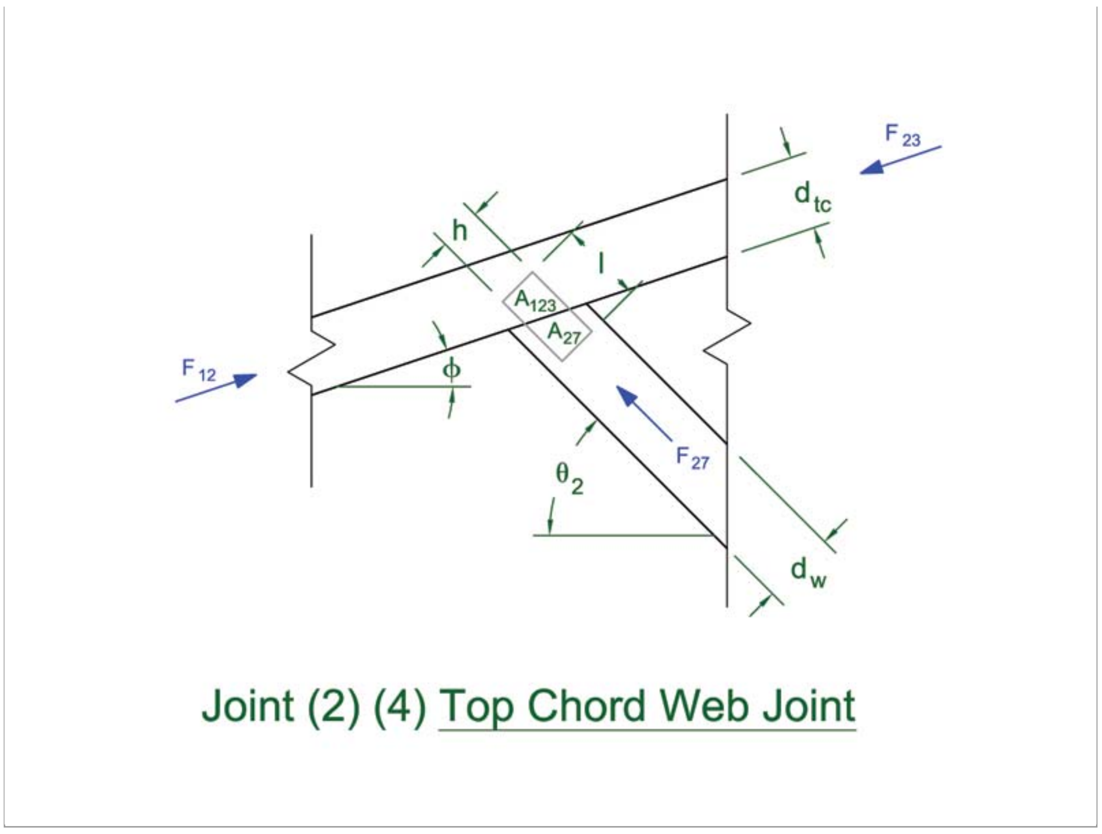
$f_{cnet} \le F_c^*$

$h' = \cos\Phi \times [h - 0.5 \times \tan\Phi \times (l'-l)] = \cos(18.43) \times [3.0 - 0.5 \times \tan(18.43)(9.0-5.0)] = 2.214$ in.

$$f_{cnet} = \frac{P_c}{h' \times d_2} = \frac{2298.8}{2.214 \times 1.5} = 692.3 \text{ psi} \le 1707.7 \text{ psi} \quad \text{(JSI = 0.41)} \quad \textbf{OK}$$

2.) Joint (2) (4) Top Chord Web Joint: (JSI=0.32)

Assume symetrical placement of plate centered on joint with plate oriented parallel to web member. Given the definitions, calculations and corresponding figure below we establish the geometry of the top chord web joint:



where:

h = Height of plate (in.)
l = Length of plate (in.)
θ_2 = Angle of web member (27) with the horizontal (deg.)
Φ = Angle of the top chord member (12) with the horizontal (deg.)
d_{tc} = Actual top chord depth (in.)
d_w = Actual web depth (in.)

Try MT20 2.0 x 4.0 plate:

h = 2.0 in.
l = 4.0 in.

A.) Checking A27: Compression web framed into top chord at top chord web joint:

i.) Lateral Resistance:

Metal connector plates resisting member compressive forces shall be sized to provide lateral resistance equal to the vectorial sum of the reduced component forces(s) normal to the wood member interface and 100 percent of the component force(s) parallel to the wood member interface.

$$P_c' = \sqrt{(P_{iP})^2 + (P_{iN} \times C_R)^2} \quad \text{(Sec. 8.3.3.3)}$$

where:

P_c' = Resultant compressive force used for determination of minimum required metal plate contact area (lbs)
C_R = Reduction factor for cmopression force component across the joint interface for metal connector plate design: $0 \leq C_R \leq 1.0$ (lbs)
P_{iN} = Compression force component of the wood member under investigation normal to the wood member interface (lbs)
P_{iP} = Compression force component of the wood member under investigation parallel to the wood member interface (lbs)

given:

$C_R = 0.5$
$P_i = F_{27} = 399.53$ lbs.
$P_{iN} = P_i\sin(\Phi+\theta_2) = 399.5 \times \sin(18.43 + 45.00) = 357.4$ lbs.
$P_{iP} = P_i\cos(\Phi+\theta_2) = 399.5 \times \cos(18.43 + 45.00) = 178.7$ lbs.

$\theta = \Phi + \theta_2 - \text{atan}(P_{iN} \times C_R/P_{iP}) = 18.43 + 45.00 - \text{atan}(357.4 \times 0.5/178.7) = 18.4$ deg.
(angle between resultant compressive force and web member)

$P_c' = \sqrt{}(P_{iP})^2 + (P_{iN} \times C_R)^2 = \sqrt{}(178.7)^2 + (357.4 \times 0.5)^2 = 252.7$ lbs. @ 18.4 deg.

The minimum axial design force for any wood member to be used when designing metal connector plates on trusses with overall lengths exceeding 16 ft. shall not be less than 375 lbs.

Use: P_c' = 375 lbs. *(Sec. 8.2)*

Mitek MT20 Plate with HF:
V_{LRAA} = 188 psi/plate
V_{LREA} = 159 psi/plate
V_{LRAE} = 133 psi/plate
V_{LREE} = 141 psi/plate

Direction of Loading:
α = 18.4 degrees (angle between load direction and slot direction)
θ = 18.4 degrees (angle between load direction and grain direction)

$$V_{LRA\theta} = \frac{V_{LRAA} \times V_{LRAE}}{(V_{LRAA} \times \sin^2\theta) + (V_{LRAE} \times \cos^2\theta)} = \frac{188 \times 133}{(188 \times \sin^2 18.4) + (133 \times \cos^2 18.4)} = 180.5 \text{ psi/plate}$$

$$V_{LRE\theta} = \frac{V_{LREA} \times V_{LREE}}{(V_{LREA} \times \sin^2\theta) + (V_{LREE} \times \cos^2\theta)} = \frac{159 \times 141}{(159 \times \sin^2 18.4) + (141 \times \cos^2 18.4)} = 157.0 \text{ psi/plate}$$

$$V_{LR} = \frac{((90-\alpha) \times V_{LRA\theta}) + (\alpha \times V_{LRE\theta})}{90} = \frac{((90-18.4) \times 180.5) + (18.4 \times 157.0)}{90} = 175.7 \text{ psi/plate}$$

$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R) = 175.7(1.15)(1.0)(1.0)(1.0) = 202.1$ psi/plate

$$A_{27(req)} = \frac{P_c'}{2(V_{LR}')} = \frac{375.0}{2 \times 202.1} = 0.93 \text{ in.}^2$$

$$A_{27} = \frac{hl}{2} = \frac{2.0 \times 4.0}{2} = 4.00 \text{ in.}^2 > 0.93 \text{ in.}^2 \text{ (JSI = 0.23)} \textbf{ OK}$$

ii.) Shear:

$P_s = P_{iP} = 178.7$ lbs.

Mitek MT20 Plate:
V_s @ 0° = 604 lbs/in/pair
V_s @ 30° = 876 lbs/in/pair
V_s @ 60° = 970 lbs/in/pair
V_s @ 90° = 686 lbs/in/pair
V_s @ 120° = 498 lbs/in/pair
V_s @ 150° = 592 lbs/in/pair

Direction of Loading:
α_s = 116.6 degrees (angle between force parallel to joint shear plane and slot direction)

Linear interpolation for allowable shear design value at angle of load with respect to plate slot direction:

$$V_{s\ @\ 116.6} = V_{s0} + (V_{s1} - V_{s0}) \frac{\alpha_s - \alpha_o}{\alpha_1 - \alpha_o} = 686 + (498 - 686) \frac{116.6 - 90}{120 - 90} = 519.5 \text{ lbs/in/pair}$$

$$W_{p(req)} = \frac{P_s}{V_s} = \frac{178.7}{519.5} = 0.34 \text{ in.}$$

$W_p = h/\sin(\Phi + \theta_2) = 2.0/\sin(18.43 + 45.00) = 2.24$ in. > 0.34 in. (JSI = 0.15) **OK**

A.) Checking A123: Top Chord with compression web attached at joint:

i.) Lateral Resistance:

$P_c = F_{12} - F_{23} = 2298.83 - 2021.83 = 277.0$ lbs.

The minimum axial design force for any wood member to be used when designing metal connector plates on trusses with overall lengths exceeding 16 ft. shall not be less than 375 lbs.

Use: P_c' = 375 lbs. *(Sec. 8.2)*

Mitek MT20 Plate with HF:
V_{LRAA} = 188 psi/plate
V_{LREA} = 159 psi/plate
V_{LRAE} = 133 psi/plate
V_{LREE} = 141 psi/plate

Direction of Loading:
α = 63.4 degrees (angle between load direction and slot direction)
θ = 0.0 degrees (angle between load direction and grain direction)

$$V_{LRA\theta} = \frac{V_{LRAA} \times V_{LRAE}}{(V_{LRAA} \times \sin^2\theta) + (V_{LRAE} \times \cos^2\theta)} = \frac{188 \times 133}{(188 \times \sin^2 0.0) + (133 \times \cos^2 0.0)} = 188.0 \text{ psi/plate}$$

$$V_{LRE\theta} = \frac{V_{LREA} \times V_{LREE}}{(V_{LREA} \times \sin^2\theta) + (V_{LREE} \times \cos^2\theta)} = \frac{159 \times 141}{(159 \times \sin^2 0.0) + (141 \times \cos^2 0.0)} = 159.0 \text{ psi/plate}$$

$$V_{LR} = \frac{((90-\alpha) \times V_{LRA\theta}) + (\alpha \times V_{LRE\theta})}{90} = \frac{((90-63.4) \times 188.0) + (63.4 \times 159.0)}{90} = 167.6 \text{ psi/plate}$$

$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R) = 167.6(1.15)(1.0)(1.0)(1.0) = 192.7$ psi/plate

$$A_{123(req)} = \frac{P_c}{2(V_{LR}')} = \frac{375.0}{2 \times 192.7} = 0.97 \text{ in.}^2$$

$$A_{123} = \frac{hl}{2} = \frac{2.0 \times 4.0}{2} = 4.00 \text{ in.}^2 > 0.97 \text{ in.}^2 \text{ (JSI = 0.24)} \textbf{ OK}$$

ii.) Shear:

$P_s = P_c = 375.0$ lbs.

Mitek MT20 Plate:
V_s @ 0° = 604 lbs/in/pair
V_s @ 30° = 876 lbs/in/pair
V_s @ 60° = 970 lbs/in/pair
V_s @ 90° = 686 lbs/in/pair
V_s @ 120° = 498 lbs/in/pair
V_s @ 150° = 592 lbs/in/pair

Direction of Loading:
α_s = 116.6 degrees (angle between force parallel to joint shear plane and slot direction)

Linear interpolation for allowable shear design value at angle of load with respect to plate slot direction:

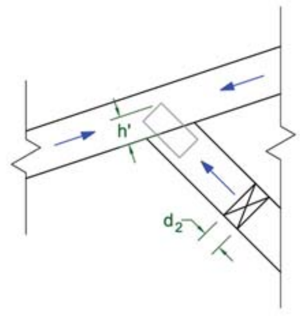
$$V_{s\ @\ 116.6} = V_{s0} + (V_{s1} - V_{s0}) \frac{\alpha_s - \alpha_o}{\alpha_1 - \alpha_o} = 686 + (498 - 686) \frac{116.6 - 90}{120 - 90} = 519.5 \text{ lbs/in/pair}$$

$$W_{p(req)} = \frac{P_s}{V_s} = \frac{375.0}{519.5} = 0.72 \text{ in.}$$

$W_p = h/\sin(\Phi + \theta_2) = 2.0/\sin(18.43 + 45.00) = 2.24$ in. > 0.72 in. (JSI = 0.32) **OK**

iii.) Net Section (H'):

Typical reduced net sections for a top chord web joint shown below:



Joint (2) (4) Reduced Net Section

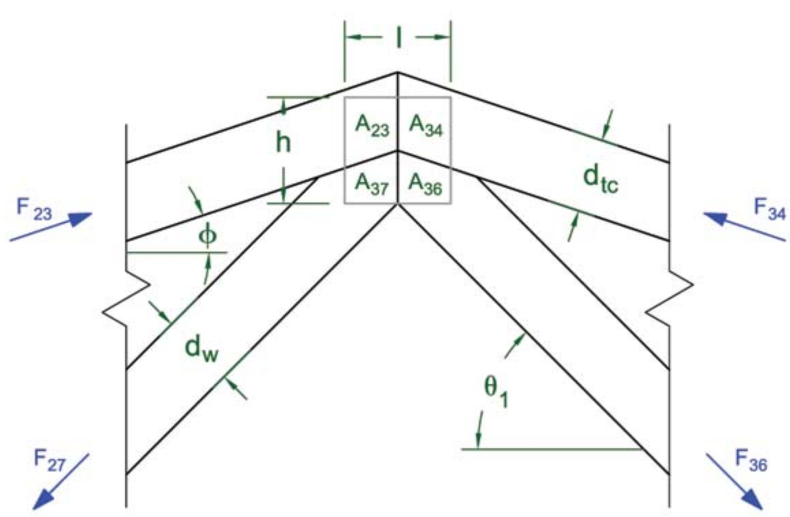
$f_{cnet} \le F_c{*}$

$h' = 0.5l\sin(\Phi+\theta_2) + 0.5h\sin[90 - (\Phi+\theta_2)] = 0.5 \times 4.0 \times \sin(18.43 + 45.00) + 0.5 \times 2.0 \times \sin[90 - (18.43 + 45.00)] = 2.236$ in.

$$f_{cnet} = \frac{P_c}{h' \times d_2} = \frac{375.0}{2.236 \times 1.5} = 111.8 \text{ psi} \le 1707.7 \text{ psi}$$ (JSI = 0.07) **OK**

3.) Joint (3) Peak Joint: (JSI=0.79)

Assume symetrical placement of plate centered on joint with plate oriented parallel to the horizontal. Given the definitions, calculations and corresponding figure below we establish the geometry of the peak joint:



Joint (3) Peak Joint

where:

h = Height of plate (in.)
l = Length of plate (in.)
θ_1 = Angle of web member (37) with the horizontal (deg.)
Φ = Angle of the top chord member (23) with the horizontal (deg.)
d_{tc} = Actual top chord depth (in.)
d_w = Actual web depth (in.)

Try MT20 3.0 x 4.0 plate:

h = 3.0 in.
l = 4.0 in.

A.) Checking A23: Peak joint in compression and flexure (bending moment):

i.) Lateral Resistance:

Metal connector plates resisting member compressive forces shall be sized to provide lateral resistance equal to the vectorial sum of the reduced component forces(s) normal to the wood member interface and 100 percent of the component force(s) parallel to the wood member interface.

$$P_c' = \sqrt{(P_{iP})^2 + (P_{iN} \times C_R)^2}$$ (Sec. 8.3.3.3)

where:

P_c' = Resultant compressive force used for determination of minimum required metal plate contact area (lbs)
C_R = Reduction factor for cmopression force component across the joint interface for metal connector plate design: $0 \le C_R \le 1.0$ (lbs)
P_{iN} = Compression force component of the wood member under investigation normal to the wood member interface (lbs)
P_{iP} = Compression force component of the wood member under investigation parallel to the wood member interface (lbs)

given:

$C_R = 0.5$
$P_i = F_{23} = 2021.83$ lbs.
$P_{iN} = P_i\cos(\Phi) = 2021.8 \times \cos(18.43) = 1918.1$ lbs.
$P_{iP} = P_i\sin(\Phi) = 2021.8 \times \sin(18.43) = 639.4$ lbs.

$\theta = \text{atan}(P_{iP}/P_{iN} \times C_R) - \Phi = \text{atan}(639.4/1918.1 \times 0.5) - 18.43 = 15.3$ deg.
(angle between resultant compressive force and web member)

$P_c' = \sqrt{}(P_{iP})^2 + (P_{iN} \times C_R)^2 = \sqrt{}(639.4)^2 + (1918.1 \times 0.5)^2 = 1152.6$ lbs. @ 15.3 deg.

Mitek MT20 Plate with HF:
V_{LRAA} = 188 psi/plate
V_{LREA} = 159 psi/plate
V_{LRAE} = 133 psi/plate
V_{LREE} = 141 psi/plate

Direction of Loading:
α = 33.7 degrees (angle between load direction and slot direction)
θ = 15.3 degrees (angle between load direction and grain direction)

$$V_{LRA\theta} = \frac{V_{LRAA} \times V_{LRAE}}{(V_{LRAA} \times \sin^2\theta) + (V_{LRAE} \times \cos^2\theta)} = \frac{188 \times 133}{(188 \times \sin^2 15.3) + (133 \times \cos^2 15.3)} = 182.8 \text{ psi/plate}$$

$$V_{LRE\theta} = \frac{V_{LREA} \times V_{LREE}}{(V_{LREA} \times \sin^2\theta) + (V_{LREE} \times \cos^2\theta)} = \frac{159 \times 141}{(159 \times \sin^2 15.3) + (141 \times \cos^2 15.3)} = 157.6 \text{ psi/plate}$$

$$V_{LR} = \frac{((90-\alpha) \times V_{LRA\theta}) + (\alpha \times V_{LRE\theta})}{90} = \frac{((90-33.7) \times 182.8) + (33.7 \times 157.6)}{90} = 173.3 \text{ psi/plate}$$

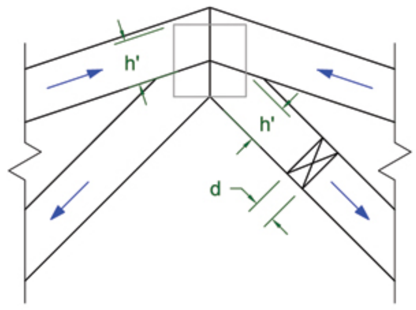
$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R) = 173.3(1.15)(1.0)(1.0)(1.0) = 199.4$ psi/plate

$$A_{23(req)} = \frac{P_c'}{2(V_{LR}')} = \frac{1152.6}{2 \times 199.4} = 2.89 \text{ in.}^2$$

$$A_{23} = \frac{hl}{4} + \frac{l^2\tan(\Phi)}{8} = \frac{3.0 \times 4.0}{4} + \frac{4.0^2\tan(18.43)}{8} = 3.67 \text{ in.}^2 > 2.89 \text{ in.}^2$$ (JSI = 0.79) **OK**

iii.) Net Section (H'):

Typical reduced net sections for a peak joint shown below:



Joint (3) Reduced Net Section

$f_{cnet} \le F_c{}^*$

Conservatively neglect any wood-to-wood bearing where the top chords meet at the peak:

$P_c = F_{23} = 2021.83$ lbs.

$h' = \cos(\Phi)[0.5l\tan(\Phi) + 0.5h] = \cos(18.43)[0.5 \times 4.0 \times \tan(18.43) + 0.5 \times 3.0] = 2.055$ in.

$$f_{cnet} = \frac{P_c}{h' \times d_2} = \frac{2021.8}{2.055 \times 1.5} = 655.8 \text{ psi} \le 1707.7 \text{ psi (JSI = 0.38) } \textbf{OK}$$

A.) Checking A37: Tension web framed into peak joint:

i.) Lateral Resistance:

$P_t = F_{37} = 626.50$ lbs.

Mitek MT20 Plate with HF:
$V_{LRAA} = 188$ psi/plate
$V_{LREA} = 159$ psi/plate
$V_{LRAE} = 133$ psi/plate
$V_{LREE} = 141$ psi/plate

Direction of Loading:
$\alpha = 45.0$ degrees (angle between load direction and slot direction)
$\theta = 0.0$ degrees (angle between load direction and grain direction)

$$V_{LRA\theta} = \frac{V_{LRAA} \times V_{LRAE}}{(V_{LRAA} \times \sin^2\theta) + (V_{LRAE} \times \cos^2\theta)} = \frac{188 \times 133}{(188 \times \sin^2 0.0) + (133 \times \cos^2 0.0)} = 188.0 \text{ psi/plate}$$

$$V_{LRE\theta} = \frac{V_{LREA} \times V_{LREE}}{(V_{LREA} \times \sin^2\theta) + (V_{LREE} \times \cos^2\theta)} = \frac{159 \times 141}{(159 \times \sin^2 0.0) + (141 \times \cos^2 0.0)} = 159.0 \text{ psi/plate}$$

$$V_{LR} = \frac{((90\text{-}\alpha) \times V_{LRA\theta}) + (\alpha \times V_{LRE\theta})}{90} = \frac{((90\text{-}45.0) \times 188.0) + (45.0 \times 159.0)}{90} = 173.5 \text{ psi/plate}$$

$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R) = 173.5(1.15)(1.0)(1.0)(1.0) = 199.5$ psi/plate

$$A_{37(req)} = \frac{P_t}{2(V_{LR}')} = \frac{626.5}{2 \times 199.5} = 1.57 \text{ in.}^2$$

$$A_{37} = \frac{hl}{4} - \frac{l^2\tan(\Phi)}{8} = \frac{3.0 \times 4.0}{4} - \frac{4.0^2\tan(18.43)}{8} = 2.33 \text{ in.}^2 > 1.57 \text{ in.}^2 \text{ (JSI = 0.67) } \textbf{OK}$$

ii.) Tension:

Mitek MT20 Plate:
V_t @ 0° = 857 lbs/in/pair
V_t @ 90° = 854 lbs/in/pair

Direction of Loading:
$\alpha_t = 45.0$ degrees (angle between axial tensile force and slot direction)

Linear interpolation for allowable tensile design value at angle of load with respect to plate slot direction:

$$V_{t\,@\,45.0} = \frac{((90\text{-}\alpha) \times V_{t\,@\,0}) + (\alpha \times V_{t\,@\,90})}{90} = \frac{((90\text{-}45.0) \times 857) + (45.0 \times 854)}{90} = 855.5 \text{ lbs/in/pair}$$

$$W_{p(req)} = \frac{P_t}{V_t} = \frac{626.5}{855.5} = 0.73 \text{ in.}$$

$$W_p = \frac{h}{2} + \frac{l}{2\cos(\Phi)} = \frac{3.0}{2} + \frac{4.0}{2\cos(18.43)} = 3.61 \text{ in.} > 0.73 \text{ in. (JSI = 0.20) } \textbf{OK}$$

iii.) Net Section (H'):

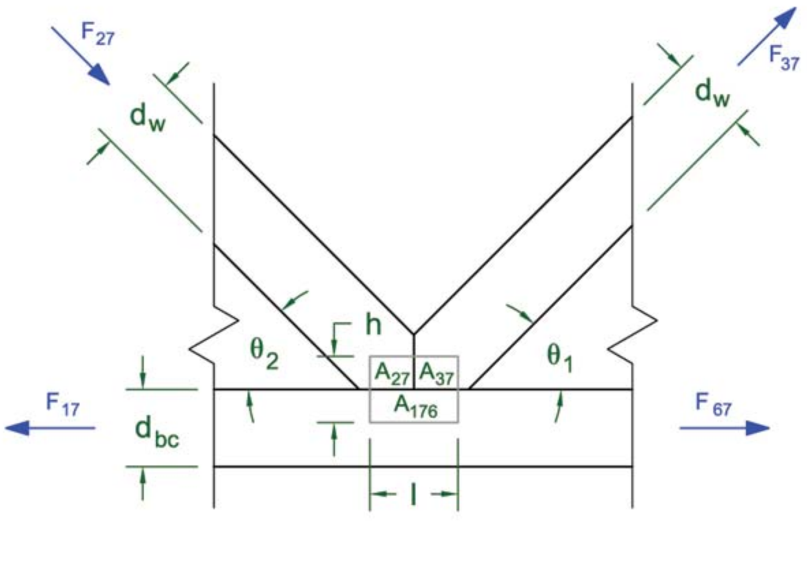
$f_{tnet} \le F_t'$

$$h' = \frac{h\cos(\Theta_1)}{2} + \frac{l\sin(\Theta_1 - \Phi)}{2\cos(\Phi)} = \frac{3.0\cos(45.00)}{2} + \frac{4.0\sin(45.00 - 18.43)}{2\cos(18.43)} = 2.00 \text{ in.}$$

$$f_{tnet} = \frac{P_t}{h' \times d_2} = \frac{626.5}{2.00 \times 1.5} = 208.5 \text{ psi} \le 460.0 \text{ psi (JSI = 0.45) } \textbf{OK}$$

4.) Joint (6) (7) Bottom Chord Web Joint: (JSI=0.70)

Assume symetrical placement of plate centered on joint with plate oriented parallel to the horizontal. Given the definitions, calculations and corresponding figure below we establish the geometry of the bottom chord web joint:



Joint (6) (7) Bottom Chord Web Joint

where:

h = Height of plate (in.)
l = Length of plate (in.)
θ_1 = Angle of web member (37) with the horizontal (deg.)
θ_2 = Angle of web member (27) with the horizontal (deg.)
d_{bc} = Actual bottom chord depth (in.)
d_w = Actual web depth (in.)

Try MT20 3.0 x 3.0 plate:

h = 3.0 in.
l = 3.0 in.

A.) Checking A27: Compression web framed into bottom chord. Assume no force transfer between web members (27) (37) at A_{27} A_{37} interface:

i.) Lateral Resistance:

Metal connector plates resisting member compressive forces shall be sized to provide lateral resistance equal to the vectorial sum of the reduced component forces(s) normal to the wood member interface and 100 percent of the component force(s) parallel to the wood member interface.

$$P_c' = \sqrt{(P_{iP})^2 + (P_{iN} \times C_R)^2}$$ (Sec. 8.3.3.3)

where:

P_c' = Resultant compressive force used for determination of minimum required metal plate contact area (lbs)
C_R = Reduction factor for cmopression force component across the joint interface for metal connector plate design: $0 \le C_R \le 1.0$ (lbs)
P_{iN} = Compression force component of the wood member under investigation normal to the wood member interface (lbs)
P_{iP} = Compression force component of the wood member under investigation parallel to the wood member interface (lbs)

given:

$C_R = 0.5$ ($P_{aN} = P_{iN}$)
$P_i = F_{27}$ = 399.53 lbs.
$P_{iN} = P_i\sin(\theta_2)$ = 399.5 x sin(45.00) = 282.5 lbs.
$P_{iP} = P_i\cos(\theta_2)$ = 399.5 x cos(45.00) = 282.5 lbs.

$\theta = \theta_2$ - atan(P_{iN} x C_R/P_{iP}) = 45.00 - atan(282.5 x 0.5/282.5) = 18.4 deg.
(angle between resultant compressive force and web member)

$P_c' = \sqrt{(P_{iP})^2 + (P_{iN} \times C_R)^2} = \sqrt{(282.5)^2 + (282.5 \times 0.5)^2}$ = 315.9 lbs. @ 18.4 deg.

The minimum axial design force for any wood member to be used when designing metal connector plates on trusses with overall lengths exceeding 16 ft. shall not be less than 375 lbs.

Use: P_c' = 375 lbs. *(Sec. 8.2)*

Mitek MT20 Plate with HF:
V_{LRAA} = 188 psi/plate
V_{LREA} = 159 psi/plate
V_{LRAE} = 133 psi/plate
V_{LREE} = 141 psi/plate

Direction of Loading:
α = 26.6 degrees (angle between load direction and slot direction)
θ = 18.4 degrees (angle between load direction and grain direction)

$$V_{LRA\theta} = \frac{V_{LRAA} \times V_{LRAE}}{(V_{LRAA} \times \sin^2\theta) + (V_{LRAE} \times \cos^2\theta)} = \frac{188 \times 133}{(188 \times \sin^2 18.4) + (133 \times \cos^2 18.4)} = 180.5 \text{ psi/plate}$$

$$V_{LRE\theta} = \frac{V_{LREA} \times V_{LREE}}{(V_{LREA} \times \sin^2\theta) + (V_{LREE} \times \cos^2\theta)} = \frac{159 \times 141}{(159 \times \sin^2 18.4) + (141 \times \cos^2 18.4)} = 157.0 \text{ psi/plate}$$

$$V_{LR} = \frac{((90-\alpha) \times V_{LRA\theta}) + (\alpha \times V_{LRE\theta})}{90} = \frac{((90-26.6) \times 180.5) + (26.6 \times 157.0)}{90} = 173.6 \text{ psi/plate}$$

$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R)$ = 173.6(1.15)(1.0)(1.0)(1.0) = 199.6 psi/plate

$$A_{27(req)} = \frac{P_c'}{2(V_{LR}')} = \frac{375.0}{2 \times 199.6} = 0.94 \text{ in.}^2$$

$$A_{27} = \frac{hl}{4} = \frac{3.0x3.0}{4} = 2.25 \text{ in.}^2 > 0.94 \text{ in.}^2$$ (JSI = 0.42) **OK**

B.) Checking A37: Tension web framed into bottom chord. Assume no force transfer between web members (27) (37) at A_{27} A_{37} interface:

i.) Lateral Resistance:

$P_t = F_{37}$ = 626.50 lbs.

Mitek MT20 Plate with HF:
V_{LRAA} = 188 psi/plate
V_{LREA} = 159 psi/plate
V_{LRAE} = 133 psi/plate
V_{LREE} = 141 psi/plate

Direction of Loading:
α = 45.0 degrees (angle between load direction and slot direction)
θ = 0.0 degrees (angle between load direction and grain direction)

$$V_{LRA\theta} = \frac{V_{LRAA} \times V_{LRAE}}{(V_{LRAA} \times \sin^2\theta) + (V_{LRAE} \times \cos^2\theta)} = \frac{188 \times 133}{(188 \times \sin^2 0.0) + (133 \times \cos^2 0.0)} = 188.0 \text{ psi/plate}$$

$$V_{LRE\theta} = \frac{V_{LREA} \times V_{LREE}}{(V_{LREA} \times \sin^2\theta) + (V_{LREE} \times \cos^2\theta)} = \frac{159 \times 141}{(159 \times \sin^2 0.0) + (141 \times \cos^2 0.0)} = 159.0 \text{ psi/plate}$$

$$V_{LR} = \frac{((90-\alpha) \times V_{LRA\theta}) + (\alpha \times V_{LRE\theta})}{90} = \frac{((90-45.0) \times 188.0) + (45.0 \times 159.0)}{90} = 173.5 \text{ psi/plate}$$

$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R)$ = 173.5(1.15)(1.0)(1.0)(1.0) = 199.5 psi/plate

$$A_{37(req)} = \frac{P_t}{2(V_{LR}')} = \frac{626.5}{2 \times 199.5} = 1.57 \text{ in.}^2$$

$$A_{37} = \frac{hl}{4} = \frac{3.0x3.0}{4} = 2.25 \text{ in.}^2 > 1.57 \text{ in.}^2$$ (JSI = 0.70) **OK**

ii.) Tension:

Mitek MT20 Plate:
V_t @ 0° = 857 lbs/in/pair
V_t @ 90° = 854 lbs/in/pair

Direction of Loading:
α_t = 45.0 degrees (angle between axial tensile force and slot direction)

Linear interpolation for allowable tensile design value at angle of load with respect to plate slot direction:

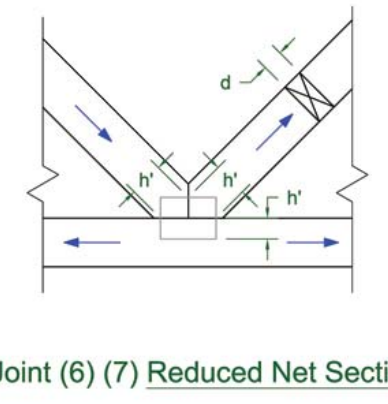
$$V_{t\ @\ 45.0} = \frac{((90-\alpha) \times V_{t\ @\ 0}) + (\alpha \times V_{t\ @\ 90})}{90} = \frac{((90-45.0) \times 857) + (45.0 \times 854)}{90} = 855.5 \text{ lbs/in/pair}$$

$$W_{p(req)} = \frac{P_t}{V_t} = \frac{626.5}{855.5} = 0.73 \text{ in.}$$

$$W_p = \frac{h}{2} + \frac{l}{2} = \frac{3.0}{2} + \frac{3.0}{2} = 3.00 \text{ in.} > 0.73 \text{ in.}$$ (JSI = 0.24) **OK**

iii.) Net Section (H'):

Typical reduced net sections for a bottom chord web joint shown below:



$f_{tnet} \le F_t'$

$$h' = \frac{h\cos(\Theta_1)}{2} + \frac{l\sin(\Theta_1)}{2} = \frac{3.0\cos(45.00)}{2} + \frac{3.0\sin(45.00)}{2} = 2.12 \text{ in.}$$

$$f_{tnet} = \frac{P_t}{h' \times d_2} = \frac{626.5}{2.12 \times 1.5} = 196.9 \text{ psi} \le 460.0 \text{ psi} \quad (\text{JSI} = 0.43) \ \textbf{OK}$$

C.) Checking A176: Bottom Chord with tension and compression web attached at joint:

i.) Lateral Resistance:

$P_t = F_{17} - F_{67} = 2180.86 - 1490.86 = 690.0$ lbs.

Mitek MT20 Plate with HF:
$V_{LRAA} = 188$ psi/plate

Direction of Loading:
$\alpha = 0.0$ degrees (angle between load direction and slot direction)
$\theta = 0.0$ degrees (angle between load direction and grain direction)

$\therefore V_{LR} = V_{LRAA} = 188.0$ psi/plate

$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R) = 188.0(1.15)(1.0)(1.0)(1.0) = 216.2$ psi/plate

$$A_{176(req)} = \frac{P_t}{2(V_{LR}')} = \frac{690.0}{2 \times 216.2} = 1.60 \text{ in.}^2$$

$$A_{176} = \frac{hl}{2} = \frac{3.0 \times 3.0}{2} = 4.50 \text{ in.}^2 > 1.60 \text{ in.}^2 \quad (\text{JSI} = 0.35) \ \textbf{OK}$$

ii.) Shear:

$P_s = P_t = 690.0$ lbs.

Mitek MT20 Plate:
V_s @ 0° = 604 lbs/in/pair

Direction of Loading:
$\alpha_s = 0.0$ degrees (angle between force parallel to joint shear plane and slot direction)

$V_s = V_{s\ @\ 0.0} = 604.0$ lbs/in/pair

$$W_{p(req)} = \frac{P_s}{V_s} = \frac{690.0}{604.0} = 1.14 \text{ in.}$$

$W_p = l = 3.00$ in. > 1.14 in. (JSI = 0.38) **OK**

iii.) Net Section (H'):

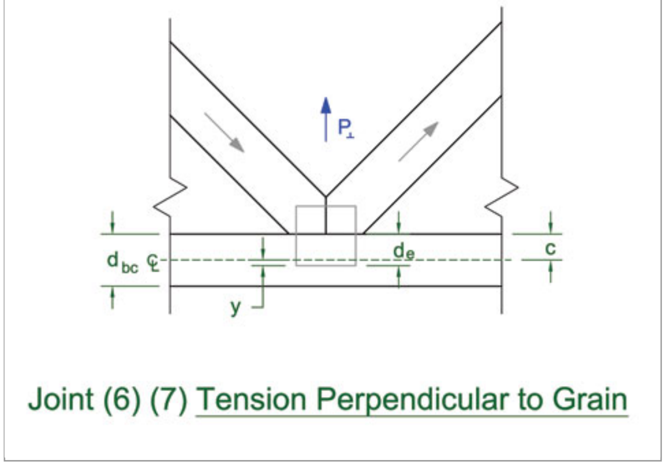
$f_{tnet} \le F_t'$

$h' = h/2 = 3.0/2 = 1.500$ in.

$$f_{tnet} = \frac{P_t}{h' \times d_2} = \frac{690.0}{1.500 \times 1.5} = 306.7 \text{ psi} \le 790.6 \text{ psi} \quad (\text{JSI} = 0.39) \ \textbf{OK}$$

iv.) Tension Perpendicular to Grain:

Any joint with connector plates in which the net force component perpendicular to the member induces tension perpendicular to the grain , shall require a Metal Connector Plate that extends past the centerline of the member a minimum distance, y, when $P_\perp$ exceeds 800 lbs.

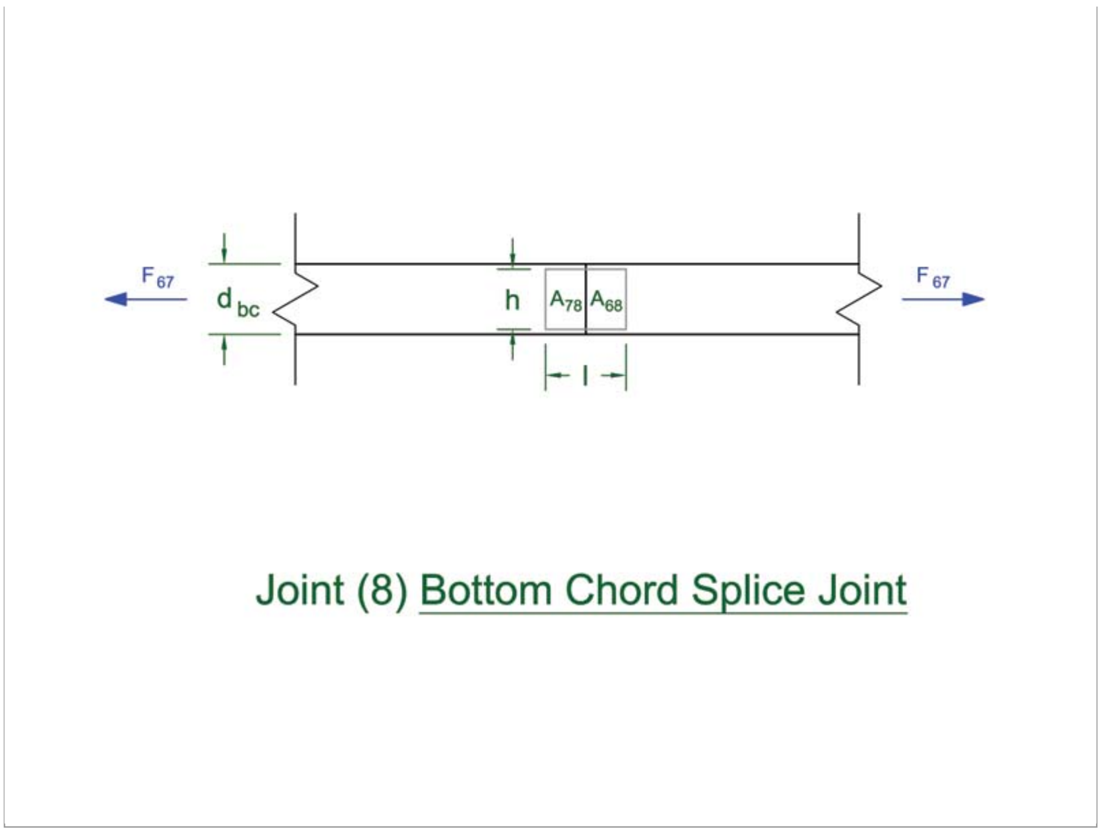


$P_\perp = F_{37}\sin(\theta_1) - F_{27}\sin(\theta_2) = 626.50 \times \sin(45.00) - 399.53 \times \sin(45.00) = 160.5$ lbs. < 800 lbs. **OK**

$P_\perp$ is less than 800 lbs, the requirements of Section 7.5.3.3 (Design for Tension Perpendicular to Grain) do not apply.

5.) Joint (8) Bottom Chord Splice Joint: (JSI=0.58)

Assume symetrical placement of plate centered on joint with plate oriented parallel to the horizontal. Given the definitions, calculations and corresponding figure below we establish the geometry of the bottom chord splice joint:



where:

h = Height of plate (in.)
l = Length of plate (in.)
d_{tc} = Actual bottom chord depth (in.)

Try MT20 3.0 x 4.0 plate:

h = 3.0 in.
l = 4.0 in.

A.) Checking A78: Bottom Chord Splice joint in flexure and tension:

i.) Lateral Resistance:

$P_t = F_{67}$ = 1490.86 lbs.

Mitek MT20 Plate with HF:
V_{LRAA} = 188 psi/plate

Direction of Loading:
α = 0.0 degrees (angle between load direction and slot direction)
θ = 0.0 degrees (angle between load direction and grain direction)

∴ $V_{LR} = V_{LRAA}$ = 188.0 psi/plate

$V_{LR}' = V_{LR}(C_D)(C_M)(C_q)(H_R)$ = 188.0(1.15)(1.0)(1.0)(1.0) = 216.2 psi/plate

$$A_{78(req)} = \frac{P_t}{2(V_{LR}')} = \frac{1490.9}{2 \times 216.2} = 3.45 \text{ in.}^2$$

$$A_{78} = \frac{hl}{2} = \frac{3.0 \times 4.0}{2} = 6.00 \text{ in.}^2 > 3.45 \text{ in.}^2 \text{ (JSI = 0.57)}$$ **OK**

ii.) Tension:

P_t = 1490.9 lbs.

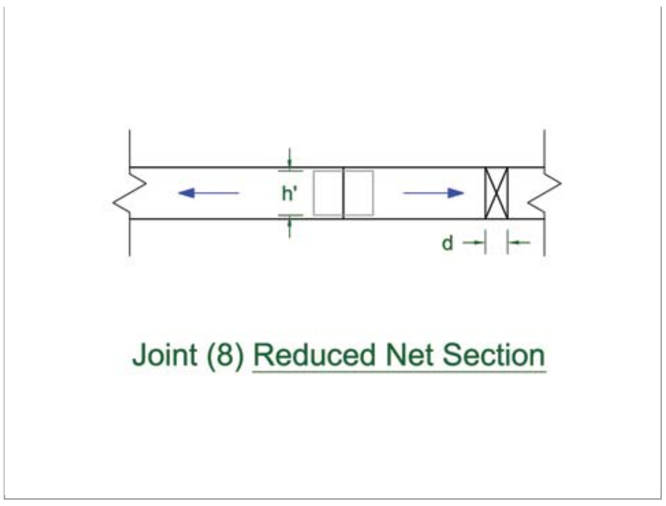
Mitek MT20 Plate:
V_t @ 0° = 857.0 lbs/in/pair

$$W_{p(req)} = \frac{P_t}{V_t} = \frac{1490.9}{857.0} = 1.74 \text{ in.}$$

W_p = h = 3.00 in. > 1.74 in. (JSI = 0.58) **OK**

iii.) Net Section (H'):

Typical reduced net sections for a bottom chord splice joint shown below:



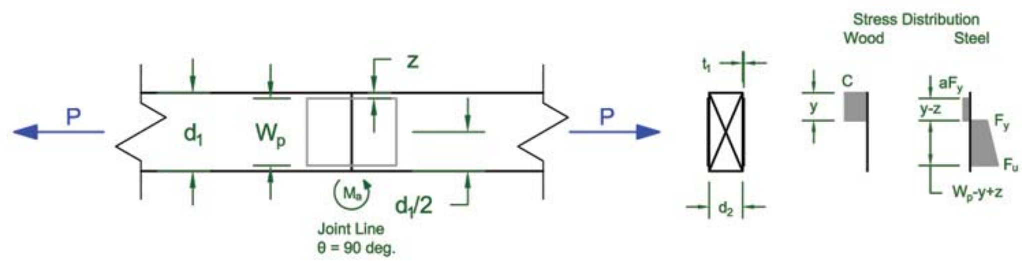
$f_{tnet} \leq F_t'$

h' = h = 3.00 in.

$$f_{tnet} = \frac{P_t}{h' \times d_2} = \frac{1490.9}{3.00 \times 1.5} = 331.3 \text{ psi} \leq 790.6 \text{ psi (JSI = 0.42)}$$ **OK**

iv.) Steel Section for Effect of Moment:

The moment applied to a metal connector plate used in chord splices shall not exceed the moment capacity as defined below:



Joint (8) Steel Section for Effect of Moment

$$M_a = \frac{C_m}{2.5}\left[\frac{T_1(W_p+y+z-d_1)}{2}+\frac{T_2(4W_p+2y+4z-3d_1)}{6}+\frac{C_s(d_1-y-z)}{2}+\frac{C_w(d_1-y)}{2}\right]$$ (Sec. 8.7.1)

where:

C_m = For splices designed using member forces resulting from a structural analysis that produces moment without consideration of interaction of axial compression and transverse deflection on moment (P-delta effect) and when the joint carrying moment is simultaneously subject to a compression force. For all other situations use unity (dimensionless)
x = Distance between splice and neareast panel point (in.)
L = Length of panel in which splice is located (in.)
f_c F_{cEx} = Compressive stresses per Section 7.3.5.1 for wood members adjacent to splice (psi)
y = Distance to neutral axis from wood edge with moment-induced compression stress (in.)
M_a = Maximum allowable moment in plane parallel to metal connector plate surface acting in direction causing compression on wood edge used to reference plate location (see variable z below). M_a shall be considered to equal zero when this equation produces a value less than zero (in.-lbs.)
T_1 = Net steel tension force (rectangular distribution block, with a constant stress of F_y) (lbs.)
T_2 = Net steel tension force (triangular distribution block, with stress extending from zero to F_u-F_y) (lbs.)
C_s = Net steel compression force (rectangular distribution block, with a constant stess of aF_y) (lbs.)
C_w = Net wood compression force (rectangular distribution block, with a constant stess of C) (lbs.)
R_t = Plate tensile effectiveness ratio for direction perpendicular to joint (dimensionless)
C = Allowable wood compression stress for the wood members adjacent to the joint line (psi)
θ = Angle between the joint line and the length of the wood member (deg.)
W_p = Plate dimension parallel to joint line, not to exceed W_p' as specified in Sec. 8.4.3.1 and 8.4.3.2 (in.)
z = Distance from compression edge of lumber to compression edge of plate (in.)
F_y = Tensile yield strength of metal connector plate (psi)
F_u = Tensile ultimate strength of metal connector plate (psi)
P = Axial force applied to joint: positive if tension, negative if compression (lbs.)
t_1 = Plate steel design thickness (in.)
d_1 = Wood cross section dimensions in the plane of the truss, measured along the joint cut line. For differing chord size, d_1 is the dimension of the joint cut line for the smaller chord (in.)
d_2 = Wood cross section dimension perpendicular to the plane of the truss (in.)
a = Factor applied to net steel compressive stress to account for the post-buckling strength of the plates (dimensionless)
F_c^* = Adjusted allowable compressive design value parallel to grain (psi)
$F_{c\perp}$ = Reference allowable compressive design value perpendicular to grain from NDS Table 4A (psi)

and:

$C_m = 1 - (x/L)(f_c/F_{cEx})$

$$y = \frac{t_1 R_1\,[F_y(1.8z+W_p)+F_u(W_p+z)]-2P}{d_2 C + t_1 R_t(1.8F_y+F_u)}$$

$T_1 = 2t_1R_tF_y(W_p - y + z)$

$T_2 = t_1R_t(F_u - F_y)(W_p - y + z)$

$C_s = 0.8t_1R_tF_y(y - z)$

$C_w = yd_2C$

$C = F_{c\perp}(1.7F_c^*)/(F_{c\perp}\sin^2\theta + 1.7F_c^*\cos^2\theta)$

given:

C_m = 1.0 *Joint in Tension*
a = 0.4 (Design of Metal Plate Connected Wood Truss Joints for Moment)
R_t = 0.49 (MT20 plate in tension at 0 deg.)
W_p = h = 3.0 in.
z = $(d_{bc}$ - h)/2 = (3.5 - 3.0)/2 = 0.25 in.

For MT20 plate: ASTM A653 SS, GRADE 40 STEEL
F_y = 40000 psi
F_u = 55000 psi
$t_1 = (t-t_c)/.95$ = (0.0356 - 0.001)/.95 = 0.0364 in.

P = F_{67} = 1490.9 lbs.
d_1 = 3.5 in.
d_2 = 1.5 in.

$F_c^* = F_c(C_D)(C_M)(C_t)(C_F)(C_i)$ = 1350(1.15)(1)(1)(1.1)(1) = 1707.7 psi
$F_{c\perp}$ = 405 psi

then:

$C = F_{c\perp}(1.7F_c^*)/(F_{c\perp}\sin^2\theta + 1.7F_c^*\cos^2\theta) = 405(1.7 \times 1707.7)/[405 \sin^2(90.0) + 1.7 \times 1707.7 \cos^2(90.0)]$ = 2903.2 psi

$$y = \frac{t_1 R_1\,[F_y(1.8z+W_p)+F_u(W_p+z)]-2P}{d_2 C + t_1 R_t(1.8F_y+F_u)} = \frac{0.0364\times 0.49\,[40000(1.8\times 0.25+3.0)+55000(3.0+0.25)]-2\times 1490.9}{1.5\times 2903.2+0.0364\times 0.49(1.8\times 40000+55000)}$$ = 0.40 in.

$T_1 = 2t_1R_tF_y(W_p - y + z)$ = 20.0364 x 0.49 x 40000(3.0 - 0.40 + 0.25) = 4064.1 lbs.

$T_2 = t_1R_t(F_u - F_y)(W_p - y + z)$ = 0.0364 x 0.49(55000 - 40000)(3.0 - 0.40 + 0.25) = 762.0 lbs.

$C_s = 0.8t_1R_tF_y(y - z)$ = 0.8 x 0.0364 x 0.49 x 40000(0.40 - 0.25) = 87.6 lbs.

$C_w = yd_2C$ = 0.40 x 1.5 x 2903.2 = 1756.7 lbs.

$$M_a = \frac{1.0}{2.5}\left[\frac{4064.1(3.0+0.40+0.25-3.5)}{2}+\frac{762.0(4\times 3.0+2\times 0.40+4\times 0.25-3\times 3.5)}{6}+\frac{87.6(3.5-0.40-0.25)}{2}+\frac{1756.7(3.5-0.40)}{2}\right]$$ = 1431 in-lbs.

The lowest combined moments and tensile loads on the bottom chord is found at the center panel of the bottom chord between joints (6) and (7). We conservatively assume the splice is placed at the centerline of truss and the applied moment is the max. mid-panel moment as shown in the matrix analysis bending moment diagram in the previous section. To further minimize moments applied to this splice joint it is recommended that the splice occur at **123.58 inches** as measured from the butt of truss.

M_{mp} (67) = 348 in-lbs. ≤ 1431 in-lbs. (JSI = 0.24) **OK**

Deflection Calculations

VIRTUAL WORK METHOD: $\Delta = \Sigma \dfrac{P_n\, u_n\, L_n}{E_n\, A_n}$

where:

Δ = Deflection of truss (in.)
P = Axial force load in a truss member caused by design loads (lbs.)
u = Axial force in a truss member caused by a unit (virtual) load (dimensionless)
L = Length of the truss member (in.)
A = Cross-sectional area of the truss member (in.^{2})
E = Modulus of elasticity of truss member material (psi)
n = Member number (ie. 12, 23, 67...)

n	Description	E_n	A_n	L_n	P_n (LL)	P_n (DL)	u_n	Δ_n (LL)	Δ_n (DL)
12	Top Chord 2 X 4 HF No.1	1500000 psi	5.25 $in.^2$	81.033 in.	-1390.91 lbs.	-907.92 lbs.	-2.072	0.030 in.	0.019 in.
23	Top Chord 2 X 4 HF No.1	1500000 psi	5.25 $in.^2$	70.756 in.	-1177.04 lbs.	-844.80 lbs.	0.000	0.000 in.	0.000 in.
17	Bottom Chord 2 X 4 HF No.1	1500000 psi	5.25 $in.^2$	99.250 in.	1319.53 lbs.	861.33 lbs.	1.966	0.033 in.	0.021 in.
67	Bottom Chord 2 X 4 HF No.1	1500000 psi	5.25 $in.^2$	89.500 in.	900.00 lbs.	590.86 lbs.	1.034	0.011 in.	0.007 in.
37	Web 2 X 4 HF Stud	1200000 psi	5.25 $in.^2$	65.624 in.	317.69 lbs.	308.81 lbs.	1.367	0.005 in.	0.004 in.
27	Web 2 X 4 HF Stud	1200000 psi	5.25 $in.^2$	34.019 in.	-308.48 lbs.	-91.05 lbs.	0.000	0.000 in.	0.000 in.
34	Top Chord 2 X 4 HF No.1	1500000 psi	5.25 $in.^2$	70.756 in.	-1177.04 lbs.	-844.80 lbs.	-1.090	0.012 in.	0.008 in.
45	Top Chord 2 X 4 HF No.1	1500000 psi	5.25 $in.^2$	81.033 in.	-1390.91 lbs.	-907.92 lbs.	-1.090	0.016 in.	0.010 in.
56	Bottom Chord 2 X 4 HF No.1	1500000 psi	5.25 $in.^2$	99.250 in.	1319.53 lbs.	861.33 lbs.	1.034	0.017 in.	0.011 in.
46	Web 2 X 4 HF Stud	1200000 psi	5.25 $in.^2$	34.019 in.	-308.48 lbs.	-91.05 lbs.	0.000	0.000 in.	0.000 in.
36	Web 2 X 4 HF Stud	1200000 psi	5.25 $in.^2$	65.624 in.	317.69 lbs.	308.81 lbs.	0.000	0.000 in.	0.000 in.

K_{cr} = 1.5 (Creep Factor for seasoned lumber used in dry service conditions as per ANSI/TPI 1-2007 *Sec. 7.6.1*)

Δ_{LL} = **0.122 in.** $(L/d)_{LL}$ = **2365.0**
Δ_{DL} = 0.082 in. $(L/d)_{DL}$ = 3524.1
$\Delta_{TL} = 1.5\Delta_{DL} + \Delta_{LL}$ = **0.244 in.** $(L/d)_{TL}$ = **1178.6**

*Note: Scalable Vector Graphic output of Truss Designer is not viewable with Internet Explorer Browsers. Recommended browsers are *Google Chrome* or *Mozilla Firefox*.

*Disclaimer: The truss designs produced herein are for initial design and estimating purposes only. The calculations and drawings presented do not constitute a fully engineered truss design. The truss manufacturer will calculate final loads, metal plate sizing, member sizing, webs and chord deflections based on local climatic and/or seismic conditions. Wood truss construction drawings shall be prepared by a registered and licensed engineer as per *IRC 2012 Sec. R802.10.2* and designed according to the minimum requirements of *ANSI/TPI 1-2007*. The truss designs and calculations provided by this online tool are for educational and illustrative purposes only. Medeek Design assumes no liability or loss for any designs presented and does not guarantee fitness for use.