Load and Resistance Factor Design of Welded Box Section Trusses*

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INTRODUCTION

After the advent of Hollow Structural Sections (HSS) in Britain, experimental and theoretical studies on welded connections with square and round members took place at Sheffield University, leading to the design recommendations of Eastwood and Wood.[1,2] These were quickly implemented in Canada and publicized by Stelco[3] in the world's first HSS connections manual. Shortly thereafter they were available to U.S. engineers in an AISI Guide.[4] The more well known reference document in the U.S. for the design of tubular connections is Chapter 10 of AWS D1.1.[5] These AWS recommendations originally evolved from a background of practices and experience with fixed offshore steel platforms of welded tubular construction. The connection capacities therefore were expressed with much greater confidence for circular tubes than for box tubes (square or rectangular hollow sections).

During the 1970s and 1980s a large amount of experimental and theoretical research on connections between manufactured HSS has taken place in many countries, but almost exclusively outside the U.S. Much of it has been coordinated and synthesized by technical committees of CIDECT (Comité International pour le Développement et l'Etude de la Construction Tubulaire) and IIW (International Institute of Welding). An excellent appreciation of the behavior of welded connections in HSS trusses has evolved and comprehensive design recommendations have consequently been issued by IIW,[6] Kurobane,[7] Wardenier,[8] CIDECT[9,10,11] and Dutta and Wurker.[12] Very recently an international consensus has been obtained for LRFD design of statically-loaded, welded connections involving hollow section members in planar trusses.[13] These IIW recommendations are slightly different from the 1981 first edition[6] and other versions issued during the 1980s[8,9,10,12] but have already been partially or fully implemented in several countries. Of particular note is that they have also been adopted by Eurocode 3[14] which will ensure widespread acceptance. These recommendations, for T, Y, X, K and N-connections are given in Tables 1, 1a, 2, 2a, 3 and 3a, for circular and box tubes, and conform to the 1989 IIW recommendations[13] with some further minor improvements.

In the latest (13th) edition of AWS D1.1 in 1992 these IIW LRFD recommendations for box sections have been adopted, with only a few very minor alterations to expressions or resistances.[15] The AWS[5] design criteria for box connections are in an ultimate load format, with recommended resistance factors throughout. For Allowable Stress Design the allowable capacity is the ultimate capacity divided by a safety factor of 1.44 / ϕ. Tables 1, 2, and 3 give factored resistances for design to the AISC LRFD specification,[16] with the resistance factors already included, as also recommended for Eurocode 3. Previous cross checks performed in Canada by Packer, et al. for box K and N gap connections,[17] as well as circular T, Y, K and N gap and overlap connections,[18] have indicated that very similar calibration coefficients to those built into Tables 1, 2, and 3 would result for the Canadian Limit States Design steelwork specification.[19] The Canadian specification uses partial load factors of 1.25 and 1.50 for dead and live loads respectively, whereas the AISC LRFD specification uses 1.2 and 1.6 for the same combination of gravity loads. Thus these two specifications coincide for a live-to-dead load ratio of 2:1, and only differ in total factored loads by five percent at a live-to-dead load ratio of 5:1. Hence, the factored connection resistances given in Tables 1, 2, and 3 are sufficiently accurate for direct application to the AISC LRFD specification for structural steel buildings.[16]

The mathematical content of these tables may initially appear forbidding but a large number of load cases and connection geometries is covered, resulting in only a small portion of the tables being applicable for a particular connection. The symbols are defined in the list below and the application of the design rules is demonstrated by the box section truss design example. This truss is designed to the AISC LRFD specification[16] using cold-formed box sections (or HSS) conforming to ASTM A500 Grade C,[20] for which the minimum specified yield strength is 50 ksi (345 MPa). A more complete description of the behavior of HSS connections and trusses is available elsewhere (e.g., Refs. 8, 10, 21).

When designing HSS trusses one should bear in mind that:

1. Chords should have thick walls rather than thin walls.

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Table 1. Factored Resistance of Axially Loaded Welded Connections Between Circular Hollow Sections

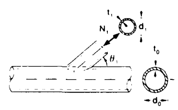
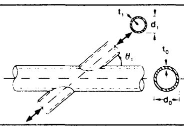
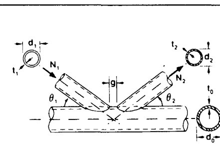
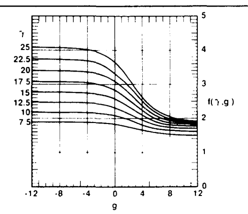
TYPE OF CONNECTION	FACTORED CONNECTION RESISTANCE (i=1,2)
T and Y Connections	Basis: CHORD PLASTIFICATION
	$N_1^* = \frac{f_{y0} \cdot t_0^2}{\sin\theta_1} (2.8 + 14\,2\beta^2) \cdot \gamma^{0\,2}\ f(n')$
X Connections	Basis: CHORD PLASTIFICATION
	$N_1^* = \frac{f_{y0} \cdot t_0^2}{\sin\theta_1} [\frac{5.2}{1 - 0\,81\beta}] \cdot f(n')$
K and N GAP or OVERLAP Connections	Basis: CHORD PLASTIFICATION
	$N_i^* = \frac{f_{y0} \cdot t_0^2}{\sin\theta_i} \cdot (1.8 + 10.2\frac{d_1}{d_0}) \cdot f(\gamma, g') \cdot f(n')$ $(i = 1, 2)$
GENERAL CHECK	Basis: PUNCHING SHEAR
Punching shear check for T, Y, X connections plus K, N, KT connections with gap.	$N_i^* = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \pi d_i \cdot \frac{1 + \sin\theta_i}{2\sin^2\theta_i}$
FUNCTIONS	
	$f(n') = 1\,0$ for $n' \geq 0$ (tension) $f(n') = 1 + 0\,3n' - 0.3n'^2$ for $n' < 0$ (compression) $f(\gamma, g') = \gamma^{0\,2}\ [1 + \frac{0.024\gamma^{1\,2}}{\exp(0\,5g' - 1\,33) + 1}]$

Table 1a. Range of Validity of Table 1

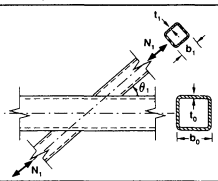
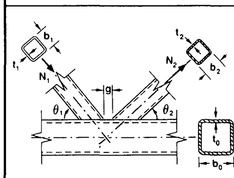
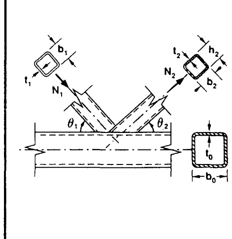
VALIDITY RANGES				
$0.2 < \frac{d_i}{d_0} \leq 1.0$	$\frac{d_i}{t_i} \leq 50$	$\frac{t_i}{t_j} \leq 1.0$ $-0.55 \leq \frac{e}{d_0} \leq 0.25$	$\gamma \leq 25$ $\gamma \leq 20$ (X-connections)	$O_v \geq 25\%$ $g \geq t_1 + t_2$

Connection efficiency for the compression web member = $N_1^*/(A_1 f_{y1})$ must be limited to the values given below, due to the possibility of premature local buckling of the compression web member.

d_1/t_1 Limits up to which the connection factored resistance (or efficiency) is not decreased.		Compression Web Member Efficiency Limit = $N_1^*/(A_1 f_{y1})$					
For yield strengths up to:	No reduction up to d_1/t_1 limit of:	For f_{y1} up to:	For d_1/t_1 of: 30	35	40	45	50
f_{y1} = 235 MPa (34 ksi)	$d_1/t_1 \leq 43$	235 MPa	1.0	1.0	1.0	0.98	0.93
f_{y2} = 275 MPa (40 ksi)	$d_1/t_1 \leq 37$	275 MPa	1.0	1.0	0 96	0.88	0.86
f_{y3} = 355 MPa (52 ksi)	$d_1/t_1 \leq 28$	355 MPa	0.98	0.88	0.85	0.78	0.76

2. Web members should have thin walls rather than thick walls.
3. The web member should be fairly wide relative to the chord member, but still able to sit on the "flat" face of the chord section, if possible. The outside corner radius of a North American coldformed box section can be taken as $2t_i$.[22]
4. Gap connections (for K and N situations) are preferred

Table 2. Factored Resistance of Axially Loaded Welded Connections Between Square or Circular Web Members and a Square Chord Section

TYPE OF CONNECTION	FACTORED CONNECTION RESISTANCE (i=1,2)
T-, Y- and X-Connections	$\beta \leq 0.85$ Basis: CHORD FACE YIELDING
	$N_1^* = \frac{f_{y0} \cdot t_0^2}{(1-\beta)\sin\theta_1} \cdot [\frac{2\beta}{\sin\theta_1} + 4(1-\beta)^{0.5}] \cdot f(n)$
K- and N-GAP Connections	$\beta \leq 1.0$ Basis: CHORD FACE PLASTIFICATION
	$N_i^* = 8.9 \cdot \frac{f_{y0} \cdot t_0^2}{\sin\theta_i} \cdot [\frac{b_1 + b_2}{2b_0}] \cdot \gamma^{0.5} \cdot f(n)$ $(i = 1, 2)$
K- and N-OVERLAP Connections[1]	$25\% \leq O_v < 50\%$ Basis: EFFECTIVE WIDTH
	$N_i^* = f_{yi} \cdot t_i[(\frac{O_v}{50})(2h_i - 4t_i) + b_e + b_{e(ov)}]$
	$50\% \leq O_v < 80\%$ Basis: EFFECTIVE WIDTH
	$N_i^* = f_{yi} \cdot t_i[2h_i - 4t_i + b_e + b_{e(ov)}]$
	$O_v \geq 80\%$ Basis: EFFECTIVE WIDTH
	$N_i^* = f_{yi} \cdot t_i[2h_i - 4t_i + b_i + b_{e(ov)}]$
CIRCULAR BRACES	Multiply formulae by $\pi/4$ and replace $b_{1,2}$ and $h_{1,2}$ by $d_{1,2}$
FUNCTIONS	
$f(n) = 1.0$ for $n \geq 0$ (tension)	$f(n) = 1.3 + \frac{0.4}{\beta} \cdot n$ for $n < 0$ (compression) But ≤ 1.0
$b_e = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot b_i$ $\leq b_i$	$b_{e(ov)} = \frac{10}{b_j/t_j} \cdot \frac{f_{yj} \cdot t_j}{f_{yi} \cdot t_i} \cdot b_i$ $\leq b_i$

Note[1]: Effective width computations need only be done for the *overlapping* web member. However the efficiency, (the factored connection resistance divided by the full yield capacity of the web member), of the *overlapped* web member is not to be taken higher than that of the *overlapping* web member.

to overlap connections because the members are easier to prepare, fit and weld.

NOTATION

- A_i — Cross-sectional area of member i ($i = 0,1,2,3$).
- A_V — Effective shear area of the chord (See Table 3).
- b_e — Effective width of a web member (See Tables 2 and 3).
- $b_{e(ov)}$ — Effective width for *overlapping* web member connected to *overlapped* web member. (See Tables 2 and 3).
- b_{ep} — Effective punching shear width (See Table 3).
- b_i — External width of square or rectangular box section member i (90° to plane of truss). ($i = 0,1,2,3$).
- d_i — External diameter of circular hollow section for member i ($i = 0,1,2,3$).

Table 2a.
Range of Validity of Table 2

TYPE OF CONNECTION	CONNECTION PARAMETERS (i = 1 or 2, j = overlapped brace)						
	b_i/b_0	b_i/t_i compression	b_i/t_i tension	b_0/t_0	$(b_1+b_2)/2b_i$ b_i/b_j t_i/t_j	gap/overlap	eccentricity
T, Y, X	$0.25 \leq \beta \leq 0.85^a$	$\leq 1.25\sqrt{\frac{E}{f_{y1}}}$	≤ 35	$10^a \leq \frac{b_0}{t_0} \leq 35$			
K, N GAP	$\geq 0.1 + 0.01\frac{b_0}{t_0}$ $\beta \geq 0.35$	≤ 35		$15^a \leq \frac{b_0}{t_0} \leq 35$	$b_i \geq 0.77\,\frac{b_1+b_2}{2}^a$	$0.5(1-\beta) \leq \frac{g}{b_0} \leq 1.5(1-\beta)$ $g \geq t_1 + t_2$	$-0.55 \leq \frac{e}{h_0} \leq 0.25$
K, N OVERLAP	≥ 0.25	$\leq 1.1\sqrt{\frac{E}{f_{y1}}}$		$\frac{b_0}{t_0} \leq 40$	$\frac{t_i}{t_j} \leq 1.0$ $\frac{b_i}{b_j} \geq 0.75$	$25\% \leq O_v \leq 100\%$	
for CIRCULAR BRACES (Web Members)	$0.4 \leq \frac{d_i}{b_0} \leq 0.8$	$\frac{d_i}{t_i} \leq 1.5\sqrt{\frac{E}{f_{y1}}}$	$\frac{d_2}{t_2} \leq 50$	Limitations as above for $d_i = b_i$			

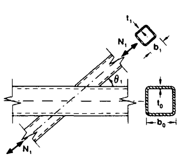
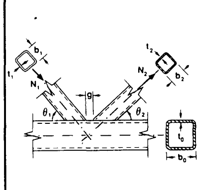
Note: [a] Outside this range of validity other failure criteria may be governing; e.g., punching shear, effective width, side wall failure, chord shear or local buckling. If these particular limits of validity are violated the connection may still be checked as one having a rectangular chord using Table 3, provided the limits of validity in Table 3a are still met.

e — Noding eccentricity for a connection—positive being towards the outside of the truss.
E — Modulus of elasticity.
f_i — Axial stress in member i (i = 0,1,2,3).
f_k — Web buckling stress.
f_{yi} — Yield strength of member i (i = 0,1,2,3).
$f(n), f(n')$ — Functions which incorporate the chord stress in the connection strength equations.
f_O — Maximum applied axial stress in chord, (or maximum stress due to axial force and bending moment where moment is taken into account).
f_{Op} — Additional stress in chord, other than that required to maintain equilibrium with web member forces.
g — Gap between the web members (ignoring welds) of a K, N, or KT-connection, at the face of the chord.
g' — Gap divided by chord wall thickness, $g' = g / t_O$.
h_i — External depth of square or rectangular box section member i (in plane of truss). (i = 0,1,2,3).
i — Subscript to denote member of connection; i = 0 designates chord; i = 1 refers in general to a web member for T, Y, and X-connections, or it refers to the compression web for K, N, and KT-connections; i = 2 refers to the tension web for K, N, and KT-connections, i = 3 refers to the vertical for KT-connections; $i = i$ refers to the *overlapping* web member for K and N-type overlap connections.
j — Subscript to denote the *overlapped* web member for K and N-type overlap connections.
KL — Effective length.
L — Length.
P — Applied load.
n — $\frac{f_o}{f_{yo}} = \frac{N_o}{A_o f_{yo}} + \frac{M_o}{S_o f_{yo}}$
n' — $\frac{f_{op}}{f_{yo}} = \frac{N_{op}}{A_o f_{yo}} + \frac{M_o}{S_o f_{yo}}$
N_i — Axial force applied to member i (i = 0,1,2,3).
N_i^* — Connection factored resistance, expressed as an axial force in member i.
$N^*_{O\,(in\ gap)}$ — Reduced axial load resistance, due to shear, in the cross-section of the chord at the gap.
N_{Op} — Axial prestressing force in the chord; i.e., load in the chord not necessary for the equilibrium of the web members' horizontal components.
O_v — Overlap, $O_v = q / p \times 100$ percent.
p — Length of projected contact area between *overlapping* web member and chord without presence of the *overlapped* web member.
q — Length of overlap between web members of a K or N-connection at the chord face.
S_i — Elastic modulus of member i (i = 0,1,2,3).
t_i — Thickness of hollow section member i (i = 0,1,2,3).
V — Applied shear force.
V_p — Shear yield capacity of a section (See Table 3).
α — Non-dimensional factor for the effectiveness of the chord flange in shear.
β — Width or diameter ratio between web member(s) and chord.

$$\beta = \frac{d_1}{d_o}, \frac{d_1}{b_o}, \frac{b_1}{b_o}\ (T,Y,X)$$

$$\beta = \frac{d_1+d_2}{2d_o}, \frac{d_1+d_2}{2b_o}, \frac{b_1+b_2+h_1+h_2}{4b_o}\ (K,N)$$

Table 3.
Factored Resistance of Axially Loaded Welded Connections Between Rectangular, Square, or Circular Web Members and a Rectangular Chord Section

<table>
<tr><th>TYPE OF CONNECTION</th><th colspan="2">FACTORED CONNECTION RESISTANCE (i=1,2)</th></tr>
<tr><td>T-, Y- and X-Connections</td><td colspan="2">$\beta \leq 0.85$ Basis: CHORD FACE YIELDING</td></tr>
<tr><td rowspan="8"></td><td colspan="2">$N_1^* = \frac{f_{y0}\ t_0^2}{(1-\beta)\sin\theta_1} \cdot [\frac{2\eta}{\sin\theta_1} + 4(1-\beta)^{0\,5}] \cdot f(n)$</td></tr>
<tr><td>$\beta = 1.0$ CHORD SIDE WALL FAILURE</td><td rowspan="2">for $0\,85 < \beta \leq 1\,0$ use linear interpolation of chord face yielding and chord side wall criteria</td></tr>
<tr><td>$N_1^* = \frac{f_k \cdot t_0}{\sin\theta_1} \cdot [\frac{2h_1}{\sin\theta_1} + 10t_0]$</td></tr>
<tr><td colspan="2">$\beta > 0.85$ Basis: EFFECTIVE WIDTH</td></tr>
<tr><td colspan="2">$N_1^* = f_{y1} \cdot t_1[2h_1 - 4t_1 + 2b_e]$</td></tr>
<tr><td colspan="2">$0.85 \leq \beta \leq 1 - 1/\gamma$ Basis: PUNCHING SHEAR</td></tr>
<tr><td colspan="2">$N_1^* = \frac{f_{y0} \cdot t_0}{\sqrt{3}\sin\theta_1}\ [\frac{2h_1}{\sin\theta_1} + 2b_{e_p}]$</td></tr>
<tr><td colspan="2"></td></tr>
<tr><td>K- and N-GAP Connections</td><td colspan="2">Basis: CHORD FACE PLASTIFICATION</td></tr>
<tr><td rowspan="8"></td><td colspan="2">$N_i^* = 8\ 9 \cdot \frac{f_{y0} \cdot t_0^2}{\sin\theta_i} \cdot [\frac{b_1 + b_2 + h_1 + h_2}{4b_0}] \cdot \gamma^{0\,5}\ \ f(n) \quad (i = 1, 2)$</td></tr>
<tr><td colspan="2">Basis: CHORD SHEAR</td></tr>
<tr><td colspan="2">$N_i^* = \frac{f_{y0} \cdot A_V}{\sqrt{3} \cdot \sin\theta_i}$ Also, $N^*_{0\,(in\ gap)} \leq (A_0 - A_V)f_{y0} + A_V \cdot f_{y0}[1 - (V/V_p)^2]^{0\,5}$</td></tr>
<tr><td colspan="2">Basis: EFFECTIVE WIDTH</td></tr>
<tr><td colspan="2">$N_i^* = f_{yi} \cdot t_i[2h_i - 4t_i + b_i + b_e]$</td></tr>
<tr><td colspan="2">$\beta \leq 1 - 1/\gamma$ Basis: PUNCHING SHEAR</td></tr>
<tr><td colspan="2">$N_i^* = \frac{f_{y0} \cdot t_0}{\sqrt{3}\sin\theta_i}\ [\frac{2h_i}{\sin\theta_i} + b_i + b_{e_p}]$</td></tr>
<tr><td colspan="2"></td></tr>
<tr><td>K- and N-OVERLAP Connections</td><td colspan="2">SIMILAR TO CONNECTIONS OF SQUARE HOLLOW SECTIONS (Table 2)</td></tr>
<tr><td>CIRCULAR BRACES</td><td colspan="2">Additional check for chord shear if $h_0/b_0 < 1.0$</td></tr>
<tr><td colspan="3">FUNCTIONS</td></tr>
<tr><td colspan="3">tension: $f_k = f_{y0}$ compression: $f_k = f_{kn}$ (T-, Y-connections) $f_k = 0.8\sin\theta_1 \cdot f_{kn}$ (X-connections)
f_{kn} = buckling stress according to national steelwork specification, using a column slenderness ratio (KL/r) of $3.46\,(h_0/t_0 - 2)(1/\sin\theta_1)^{0\,5}$</td></tr>
<tr><td>$f(n) = 1.0$ for $n \geq 0$ (tension)
$f(n) = 1\ 3 + \frac{0.4}{\beta} \cdot n$ for $n < 0$ but ≤ 1.0 (compression)</td><td colspan="2">$V_p = \frac{f_{y0} \cdot A_V}{\sqrt{3}}$ $\alpha = (\frac{1}{1 + \frac{4g^2}{3t_0^2}})^{0\,5}$
For square and rectangular webs, $A_V = (2h_0 + \alpha \cdot b_0) \cdot t_0$
For circular webs, $A_V = 2h_0\ \ t_0$</td></tr>
<tr><td>$b_e = \frac{10}{b_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i}\ b_i \leq b_i$</td><td>$b_{e_p} = \frac{10}{b_0/t_0} \cdot b_i \leq b_i$</td><td>$b_{e(ov)} = \frac{10}{b_j/t_j} \cdot \frac{f_{yj}\ \ t_j}{f_{yi}\ \ t_i}\ b_i \leq b_i$</td></tr>
<tr><td colspan="3">Note[1]: For X-connections with angles $\theta < 90°$, the chord side walls must be checked for shear</td></tr>
</table>

$$\beta = \frac{d_1{+}d_2{+}d_3}{3d_O}, \frac{d_1{+}d_2{+}d_3}{3b_O}, \frac{b_1{+}b_2{+}b_3{+}h_1{+}h_2{+}h_3}{6b_O} \quad (KT)$$

ϕ Resistance factor.

γ Half width to thickness ratio of the chord,
$$\gamma = \frac{d_o}{2t_o} \text{ or } \frac{b_o}{2t_o}.$$

η Web member depth to chord width ratio, $\eta = h_i / b_O$.

θ_i Included angle between web member i ($i = 1,2,3$) and the chord.

Note: When mechanical or geometric properties of members listed in the Notation are used in LRFD design equations, the nominal or specified values are to be used.

TRUSS DESIGN PROCEDURE

In summary, the design of a box section (HSS) truss should be approached in the following way to obtain an efficient and economical structure.

1. Determine the truss layout, span, depth, panel lengths, truss spacing, and bracing by the usual methods, but keep the number of connections to a minimum.
2. Determine loads at connections and on members; simplify these to equivalent loads at the panel points.
3. Determine axial forces in all members, assuming pinned joints and that member centerlines are all noding.
4. Determine chord member sizes considering axial loading, corrosion protection, and tube wall slenderness. (Usual width to thickness ratios are 15 to 25). An effective length factor of $K = 0.9$ can be used for the design of the compression chord.[11]
5. Determine web member sizes based on axial loading, preferably with thickness smaller than the chord thickness. The effective length factor for the web members can initially be assumed to be 0.75.[11]
6. Standardize the web members to a few selected dimensions, (perhaps even two), to minimize the number of section sizes for the structure. Consider availability of all sections when making member selections. For aesthetic reasons, a constant outside member width may be preferred for all web members, with wall thickness varying; but this will require special quality control procedures in the fabrication shop.
7. Layout the connections, trying gap joints first. Check that the connection geometry and member dimensions satisfy the validity ranges for the dimensional parameters given in Tables 2a and 3a, with particular attention to the eccentricity limits. Consider the fabrication procedure when deciding on a connection layout.
8. Check the connection factored resistances using equations given in Tables 2 and 3.
9. If the connection resistances are not adequate, modify the connection layout (for example, overlap rather than gap), or modify the web or chord members as appropriate, and recheck the connection capacities. Generally only a few connections will need checking.
10. Check the effect of primary moments on the design of the chords. For example, use the proper load positions, (rather than equivalent panel point loading), and determine the bending moments in the chords by assuming either: (a) pinned joints everywhere, or (b) continuous chords with pinended web members. For the compression chord, also determine the bending moments produced by any noding eccentricities, by using either of the above analysis assumptions. Then check that the factored resistance of all chord members is still adequate, under the influence of both axial loads and primary bending moments.
11. Check truss deflections at the specified (unfactored) load level, using the proper load positions.
12. Design the welds.

TRUSS DESIGN EXAMPLE

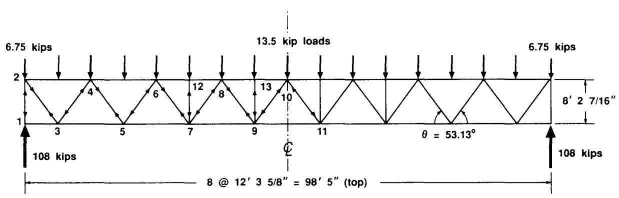
Figure 1 shows the truss and factored loads along with member axial forces, determined by a pin-jointed analysis. The top

Table 3a.
Range of Validity of Table 3

TYPE OF CONNECTION	CONNECTION PARAMETERS (i = 1 or 2, j = overlapped brace)						
	b_i/b_0 h_i/b_0	$b_i/t_i, h_i/t_i, d_i/t_i$ compression	tension	h_i/b_i	b_0/t_0 h_0/t_0	gap/overlap b_i/b_j t_i/t_j	eccentricity
T, Y, X	≥ 0.25	$\leq 1.25\sqrt{\frac{E}{f_{y1}}}$			≤ 35		
K, N GAP	$\geq 0.1 + 0.01\frac{b_0}{t_0}$ $\beta \geq 0.35$	≤ 35	≤ 35	$0.5 \leq \frac{h_i}{b_i} \leq 2$	≤ 35	$0.5(1-\beta) \leq \frac{g}{b_0} \leq 1.5(1-\beta)$ $g \geq t_1 + t_2$	$-0.55 \leq \frac{e}{h_0} < 0.25$
K, N OVERLAP	≥ 0.25	$\leq 1.1\sqrt{\frac{E}{f_{y1}}}$			≤ 40	$25\% \leq O_v \leq 100\%$ $\frac{t_i}{t_j} \leq 1.0,$ $\frac{b_i}{b_j} \geq 0.75$	
for CIRCULAR BRACES (Web Members)	$0.4 \leq \frac{d_i}{b_0} \leq 0.8$	$\leq 1.5\sqrt{\frac{E}{f_{y1}}}$	≤ 50	Limitations as above for $d_i = b_i$			

(compression) chord is considered to be laterally supported at each purlin position. It will be noticed that this is a modified Warren truss with vertical members in the central region. Such a configuration can work well because the verticals provide support for purlins where the top chord has large axial loads. Towards the ends, where axial forces are less, bending loads from purlins can be accepted by a chord section approximately the same size as that used at the center. This can reduce the number of different size sections to procure, and it eliminates several verticals along with their connections, at the expense of more pounds of material in the chord. Such a trade-off can be advantageous.

Time will be saved when selecting member sizes by keeping in mind the basic constraints, or "limits of validity" of various dimensional parameters, which must be met for connections. (See Tables 2a and 3a.) Also, it can be expedient to pick sections taking account of the efficiency of the connections right from the beginning.



Member	Type	Force	Member	Type	Force
2-4	Top chord	-76.1 kips	2-3	Diagonal	118 kips
4-6	Top chord	-198 kips	3-4	Diagonal	-118 kips
6-8	Top chord	-279 kips	4-5	Diagonal	84.4 kips
8-10	Top chord	-319 kips	5-6	Diagonal	- 84.4 kips
1-3	Bottom chord	0 kips	6-7	Diagonal	59.2 kips
3-5	Bottom chord	142 kips	7-8	Diagonal	- 42.2 kips
5-7	Bottom chord	243 kips	8-9	Diagonal	25.3 kips
7-9	Bottom chord	304 kips	9-10	Diagonal	- 8.4 kips
9-11	Bottom chord	324 kips	7-12	Vertical	- 13.5 kips
			9-13	Vertical	- 13.5 kips

Fig. 1. Warren truss showing applied loads and resulting member forces. (Loads between panel points are distributed to the panel points to determine the bar forces.)

Preliminary Member Selections

Central Top Chord: Load in length 8-10 = −319 kips.

$KL = 0.9$ (73.8 in.) = 66.4 in.

Try 7 × 7 × ⅜ in.

Design compressive strength ($\phi_c P_n$) = −389 kips **o.k.**

(It can be shown later that 7 × 7 × 5⁄16 in. ($\phi_c P_n$ = −330 kips) would be too thin.)

Outer Top Chord: Load in length 4-6 = −192 kips.

$KL = 0.9$ (147.6 in.) = 132.8 in.

Knowing that the applied forces, (axial load plus bending), are to be subsequently combined for checking the member as a beam-column, select a generous size at this stage.

Try 7 × 7 × ⅜ in.

Design compressive strength ($\phi_c P_n$) = −340 kips **o.k.**

(Again, it can be shown later that a 7 × 7 × 5⁄16-in. ($\phi_c P_n$ = −289 kips) would be too thin.)

Central Bottom Chord: Load in member 9-11 is 324 kips.

A_{min} = 324 / (0.9 × 50) = 7.20 in.^{2}

Try 6 × 6 × ⅜-in., A = 8.08 in.^{2}

Outer Bottom Chord: Load in member 5-7 is 243 kips.

A_{min} = 243 / (0.9 × 50) = 5.40 in.^{2}

Try 6 × 6 × 5⁄16-in., A = 6.86 in.^{2}

(Once again, it can be shown later that a 6 × 6 × ¼-in. (A = 5.59 in.^{2}) would be too thin.)

Tension Diagonal at end of truss: Load in member 2-3 is 118 kips.

$A_{min} = 118 / (0.9 \times 50) = 2.62$ in.^{2}

Try $4 \times 4 \times 3/16$-in., $A = 2.77$ in.^{2}

Compression Diagonal at end of truss: Load in member 3-4 is −118 kips.

$KL = 0.75$ (123 in.) = 92.3 in.

Try $5 \times 5 \times 3/16$-in. Design compressive strength ($\phi_c P_n$) = −127 kips.

Compression Diagonal at middle of truss: Load in member 9-10 is −8.4 kips.

$KL = 92.3$ in.

Try $2\frac{1}{2} \times 2\frac{1}{2} \times 1/8$-in. Design compressive strength ($\phi_c P_n$) = −25 kips **o.k.**

This is probably about the smallest size one would want to use in a truss this large, especially if one reflects on the member width relative to the truss depth. Although the AISC LRFD Manual of Steel Construction[22] does not show that this HSS size is available, it is available from some tube manufacturers (e.g., Welded Tube Company of America).

All Verticals: Load is −13.5 kips.

$KL = 0.75$ (98.4 in.) = 73.8 in.

Try $2\frac{1}{2} \times 2\frac{1}{2} \times 1/8$-in. Design compressive strength ($\phi_c P_n$) = −32 kips **o.k.**

Tension Diagonal near truss center: Load in member 8-9 is 25.3 kips.

$A_{min} = 25.3 / (0.9 \times 50) = 0.56$ in.^{2}

Try $2\frac{1}{2} \times 2\frac{1}{2} \times 1/8$-in., $A = 1.15$ in.^{2}

Compression Diagonal 7-8: Load in member is −42.2 kips.

$KL = 92.3$ in.

Try $3 \times 3 \times 3/16$-in. Design compressive strength ($\phi_c P_n$) = −53 kips

Diagonals 4-5 and 5-6: Loads are 84.4 kips and −84.4 kips

Use same sections as for members 2-3 and 3-4 to avoid a multitude of different sizes. (It will also be found later that diagonal 5-6 needs this size of section, even though the load is less than for diagonal 3-4).

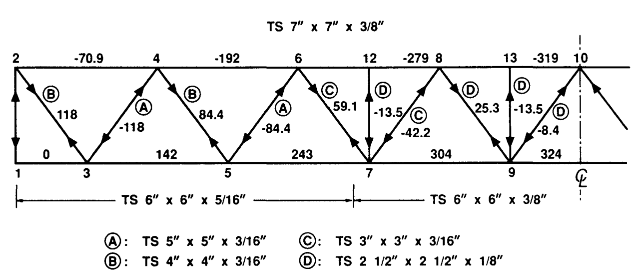


Fig. 2. Preliminary selection of truss members with member loads in kips.

Tension Diagonal 6-7: Load in member is 59.1 kips

$A_{min} = 59.1 / (0.9 \times 50) = 1.31$ in.^{2}

Try $3 \times 3 \times 3/16$-in., $A = 2.02$ in.^{2}

This section may appear oversize at first but the parametric requirement for a gap connection at Panel Point No. 6, that the width of the smaller web member be at least $0.77\,(b_1 + b_2) / 2 = 0.77\,(5 + 3) / 2 = 3.1$ in., is already being stretched.

The truss with the trial members is shown in Fig. 2.

Resistance of Gap K Connections

Panel Points No. 3 and No. 5:

Is a *gap* connection feasible?

$h_o / b_o = 1.0$, $\theta = 53.1°$, $\beta = (4 + 5) / (2 \times 6) = 0.75$

Minimum gap allowed = $0.5b_o(1 - \beta) = 0.75$ in.,

or $t_1 + t_2 = 0.375$ in. (Table 2a)

Therefore, $g_{min} = 0.75$ in., so use $g = 0.75$ in.

$$e = [(\sin\theta_1 \sin\theta_2) / \sin(\theta_1 + \theta_2)][(h_1 / 2\sin\theta_1) + (h_2 / 2\sin\theta_2) + g] - (h_o / 2)$$

$$= [(0.8 \times 0.8) / 0.96][(5 / 1.6) + (4 / 1.6) + 0.75] - (6 / 2)$$

$$= 1.25 \text{ in.}$$

Therefore, $e / h_o = 0.21 \leq 0.25$ **o.k.** (Table 2a)

Confirm validity of dimensional parameters (See Table 2a).

$\beta = 0.75 \geq 0.35$ **o.k.**

$b_i = 4$ in. or 5 in. $\geq 0.77\,[(b_1 + b_2) / 2] = 3.5$ in. **o.k.**

$b_2 / b_o = 4 / 6 = 0.67 \geq 0.01\,(b_o / t_o) + 0.1$
$= 0.01\,(6 / (5 / 16)) + 0.1 = 0.29$ **o.k.**

$b_1 / t_1 = 26.7 \leq 1.25\sqrt{E / f_{y1}} = 1.25\sqrt{29{,}000 / 50}$
$= 30.1$ **o.k.**

$b_2 / t_2 = 4 / (3 / 16) = 21.3 \leq 35$ **o.k.**

$b_o / t_o = 6 / (5 / 16) = 19.2 \geq 15$
and ≤ 35 **o.k.**

Determine connection resistance of compression diagonal ($i = 1$)

$N_1^* = 8.9 f_{yo}\, t_o^2\, [(b_1 + b_2) / 2b_o] \gamma^{0.5} f(n) / \sin\theta_1$ (Table 2)

$f(n) = 1.0$ for a tension chord (Table 2)

$N_1^* = 8.9(50)(5 / 16)^2[(4 + 5) / (2 \times 6)]9.6^{0.5} / 0.8$

$= 126$ kips

$\geq N_1 = 118$ kips **o.k.**

Determine connection resistance of tension diagonal ($i = 2$)

From Table 2 one can see that $N_1^*\sin\theta_1 = N_2^*\sin\theta_2$, which in this case means that $N_1^* = N_2^*$, therefore $N_2^* = 126$ kips $\geq N_2 = 118$ kips **o.k.**

Hence, Panel Points 3 and 5 are acceptable. Note that the connection resistances would have been insufficient if the chord had a ¼-in. thickness rather than the 5⁄16-in. selected.

Panel Point No. 4

A gap connection can again be shown to be feasible, and a review of the dimensional parameters shows that they are valid.

The purlin load midway between Panel Points 2 and 4, as well as midway between Panel Points 4 and 6, will create bending moments within members 2-4 and 4-6 as well as at Panel Point 4. An approximation of the chord bending moment at Panel Point 4 can be obtained by considering the portion of the top chord from the end of the truss to Panel Point 6 to be a two-span continuous beam. The moment at Panel Point 4, the interior support, would then be $0.188PL = 0.188(13.5)(147.6) = 375$ kip-in. The forces acting on Panel Point 4 are shown in Fig. 3.

Determine $f(n)$ for Panel Point No. 4

$f(n) = 1.3 + (0.4 / \beta)n$, but $\not> 1.0$ (Table 2)

$\beta = 0.643$

$n = N_o / A_o f_{yo} + M_o / S_o f_{yo}$, by definition (see Notation)

$= -192 / (9.58 \times 50) + (-375) / (19.6 \times 50) = -0.784$

$f(n) = 1.3 + (0.4 / 0.643)(-0.784) = 0.813$

Note that only the primary moment due to transverse loading on the chord, (or purlin loading), was used for M_o during this *connection* check. The primary bending moment due to nodal eccentricity was not included because the eccentricity e is within specified limits *for the connection* (i.e., $-0.55 \leq e / h_o \leq 0.25$ in Table 2a).

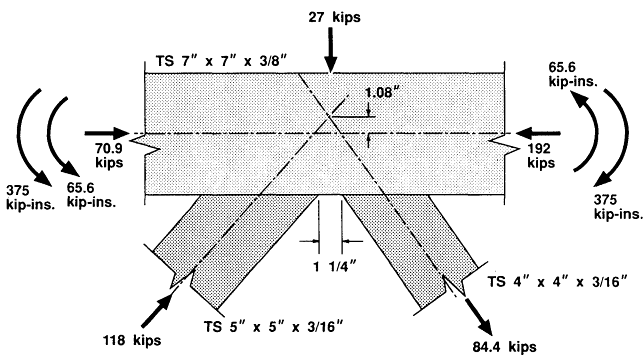


Fig. 3. Panel Point No. 4.

Determine connection resistance of compression diagonal ($i = 1$)

$N_1^* = 8.9 f_{yo} t_o^2 [(b_1 + b_2) / 2b_o]\gamma^{0.5} f(n) / \sin\theta_1$ (Table 2)

$= 8.9(50)(3/8)^2[(5 + 4) / (2 \times 7)](9.33)^{0.5}(0.813) / 0.8$

$= 125$ kips

$\geq N_1 = 118$ kips **o.k.**

Determine connection resistance of tension diagonal ($i = 2$)

$N_2^*\sin\theta_2 = N_1^*\sin\theta_1$ (Table 2)

so $N_2^* = 125$ kips $\geq N_2 = 84.4$ kips **o.k.**

Hence, Panel Point No. 4 is acceptable. Note that the connection resistance of the compression diagonal would have been insufficient if the chord had a 5⁄16-in. thickness rather than the ⅜-in. selected.

Panel Point No. 6

Panel Point 6 is similar to Panel Point 4 but has a smaller tension web member. The validity of the dimensional parameters checks out o.k. and a gap connection is again feasible. At Panel Point 4, in order to conservatively estimate the primary bending moment in the chord due to the purlin loads on either side of the panel point, the chord was considered to be a two-span continuous beam from Panel Point 2 to Panel Point 6. By considering the chord as a continuous member (past Panel Points 4 and 6) which is pin-connected to the web members, a conservative estimate of the chord bending moment at Panel Point 6 generated by the purlin loads (P) would be $0.125PL$, or $0.125(13.5)(147.6) = 249$ kip-in.

Determine $f(n)$ for Panel Point No. 6.

$\beta = (5 + 3) / (2 \times 7) = 0.571$

$n = N_o / A_o f_{yo} + M_o / S_o f_{yo}$, by definition (See Notation)

$= -279 / (9.58 \times 50) + (-249) / (19.6 \times 50) = -0.837$

$f(n) = 1.3 + (0.4 / 0.571)(-0.837) = 0.714$

Note that the primary moment due to purlin loads away from the panel points (M_o) has a negative influence on the connection strength in this example, by lowering the value of the term $f(n)$. One could perform the entire truss design by assuming that all joints were pinned, in which case $M_o = 0$ and the connection resistance would thereby be slightly higher. However, this penalizes the compression chord member severely when it is designed as a beam-column, since the maximum member moment increases considerably to the single-span beam value (0.25PL).

Determine connection resistance of compression diagonal (i = 1)

$$N_1^* = 8.9 f_{yo} t_o^2 [(b_1 + b_2) / 2b_o] \gamma^{0.5} f(n) / \sin\theta_1 \qquad \text{(Table 2)}$$

$$= 8.9(50)(3/8)^2[(5 + 3) / (2 \times 7)](9.33)^{0.5}(0.714) / 0.8$$

$$= 97.5 \text{ kips}$$

$$\geq N_1 = 84.4 \text{ kips} \quad \textbf{o.k.}$$

Note again that the connection resistance of the compression diagonal would be inadequate with a thinner chord.

Determine connection resistance of tension diagonal (i = 2)

As discussed previously,

$N_2^* = N_1^*$ for this truss = 97.5 kips $\geq N_2$ = 59.1 kips

Hence, Panel Point No. 6 is acceptable.

Panel Point No. 8—another K-connection—can likewise be shown to be acceptable.

Resistance of Top Chord as a Beam-Column between Panel Points No. 2 and No. 6

Determine nodal eccentricity at Panel Point No. 4

Minimum gap allowed = $0.5b_o(1 - \beta)$ = 1.25 in., or

$t_1 + t_2$ = 0.375 in. (Table 2a)

Therefore, g_{min} = 1.25 in., so use g = 1.25 in. (See Fig. 3)

$$e = [(\sin\theta_1 \sin\theta_2) / \sin(\theta_1 + \theta_2)][(h_1 / 2\sin\theta_1) + (h_2 / 2\sin\theta_2) + g] - (h_o / 2)$$

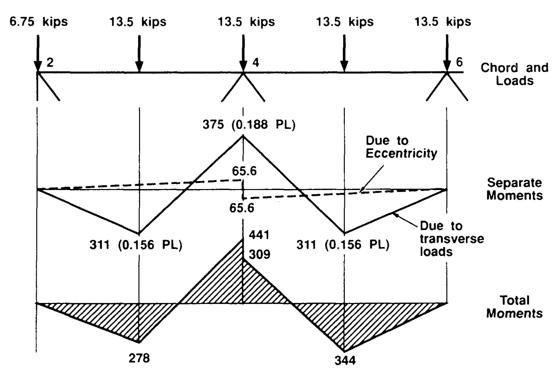


Fig. 4. Simplified bending moments in chord for manual design as a beam-column. (Moments plotted on tension side of chord and in kip-in.)

$$= [(0.8 \times 0.8) / 0.96][(5 / 1.6) + (4 / 1.6) + 1.25] - (7 / 2) = 1.08 \text{ in.}$$

(Hence e / h_o = 0.15 ≤ 0.25 which is acceptable)

Determine bending moments in the chord at Panel Point 4.

Moment from noding eccentricity
= e [(118 + 84.4) kips] cosθ = 131 kip-ins.
or 65.6 kip-in. each side of the panel point.

Moment from purlin loads is considered to be 0.188PL at the connection = 375 kip-in., as discussed previously.

For the design of the compression chord, both the moment due to transverse loading (purlin loads) and the moment due to noding eccentricity must be taken into account. Thus, Fig. 4 shows the moment combinations existing at Panel Point 4 which are:

375 + 65.6 = 441 kip-in., for chord length 2-4

and 375 − 65.6 = 309 kip-in., for chord length 4-6.

Determine bending moments under the purlins.

Use of the same model that was employed for the chord moment at Panel Point 4, (chord being two continuous spans over the panel point), gives a conservative value of 0.156PL = 0.156(13.5)(147.6) = 311 kip-in. Figure 4 shows the total midspan moments under the purlins, which are:

311 − 65.6 / 2 = 278 kip-in., for chord length 2-4

and 311 + 65.6 / 2 = 344 kip-in., for chord length 4-6.

Member 2-4 Resistance as a Beam-Column

This is governed by Chapter H of the AISC LRFD specification,[16] formulas H1-1a and H1-1b, using KL = 0.9 (147.6 in.) (AISC LRFD Commentary)

$P_u / (\phi_c P_n)$ = required compressive strength / compressive resistance (AISC LRFD E2)

$$= -70.9 / -340.2 = 0.21$$

Therefore, check that

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}}\right) \leq 1.0 \qquad \text{Eq. H1-1a}$$

where M_{ux} is the "required flexural strength," (determined from AISC LRFD Eqs. H1-2, H1-3 and H1-4) and $\phi_b M_{nx}$ is the flexural resistance of the member (AISC LRFD Section F1.7)

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}}\right) = 0.21 + \frac{8}{9}\left(\frac{1.0 \times 441}{0.90 \times 1{,}175}\right)$$

$$= 0.21 + 0.37 = 0.58 \leq 1.0 \quad \textbf{o.k.}$$

Member 4-6 Resistance as a Beam-Column

$$P_u \,/\, (\phi_c P_n) = -192 \,/\, -340.2 = 0.56$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}}\right) = 0.56 + \frac{8}{9}\left(\frac{1.003 \times 344}{0.90 \times 1{,}175}\right)$$

$$= 0.56 + 0.30 = 0.86 \le 1.0 \quad \textbf{o.k.}$$

Resistance of X Connections

Panel Point No. 13 is categorized as an X connection because load is transferred through the chord member. It is assumed that the width of the load application point on the chord is no less than the width of the web member on the underside of the chord (2.5 in.) $\beta = 0.357$, so Table 2 is applicable. Note that if $\beta > 0.85$ one could use Table 3, even though the chord member is not "rectangular."

$$f(n) = 1.3 + (0.4 \,/\, \beta)n, \text{ but} \ngtr 1.0 \qquad \text{(Table 2)}$$

$$= 1.3 + (0.4 \,/\, 0.36)(-319 \,/\, (9.58 \times 50)) = 0.56$$

$$N_1^* = \frac{f_{yO}\, t_O^2}{(1-\beta)\sin\theta_1}\left[\frac{2\beta}{\sin\theta_1} + 4(1-\beta)^{0.5}\right] f(n) \qquad \text{(Table2)}$$

$$= (50 \times 0.375^2 / \, 0.643)[0.714 + 4(0.643)^{0.5}](0.56)$$

$$= 24.0 \text{ kips}$$

$$\ge N_1 = 13.5 \text{ kips} \quad \textbf{o.k.}$$

Panel Point No. 10 is also an X connection, transferring the same force through the chord as in the example above (13.5 kips). The two web members framing into the chord at Panel Point 10 are both 2½×2½×⅛-in. (as above), but the load will be dispersed over a greater length along the underside of the chord, effectively increasing the dimension h_1 of the web member on one side of the chord and also the connection resistance. This connection will hence be adequate.

Resistance of KT Overlap Connections

Panel Point No. 7 is a KT connection as shown in Fig. 5. It can be quickly seen that a gap connection is not feasible within the nodal eccentricity limits, so an overlap connection will be necessary. Overlap the vertical member onto the diagonals, with 25 percent of the vertical member width sitting on each diagonal.

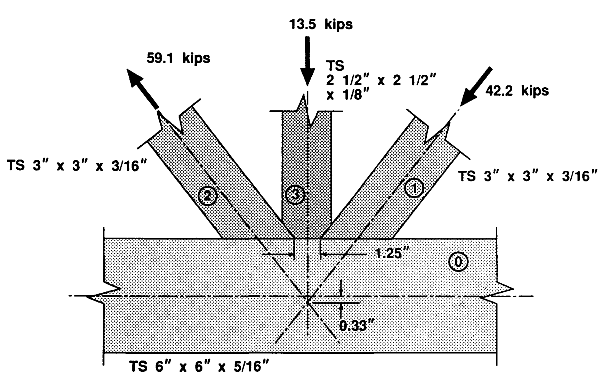


Fig. 5. KT Connection at Panel Point No. 7.

Confirm validity of parameters

$b_3 \,/\, b_O = 2.5 \,/\, 6$
$= 0.42 \ge 0.25$ **o.k.** (Table 2a for overlap connections)

$b_3 \,/\, b_j = 2.5 \,/\, 3 = 0.83 \ge 0.75$ **o.k.**

$t_3 \,/\, t_j = 0.125 \,/\, 0.1875 = 0.67 \le 1.0$ **o.k.**

$b_1 \,/\, t_1 = 3 \,/\, 0.1875 = 16 \le 1.1\sqrt{E \,/\, f_{y1}} = 26.5$ **o.k.**

$b_3 \,/\, t_3 = 2.5 \,/\, 0.125 = 20 \le 1.1\sqrt{E \,/\, f_{y3}} = 26.5$ **o.k.**

$b_2 \,/\, t_2 = 3 \,/\, 0.1875 = 16 \le 35$ **o.k.**

$b_o \,/\, t_o = 6 \,/\, 0.375 = 16 \le 40$ **o.k.**

25 percent ≤ overlap ≤ 100 percent **o.k.**

$-0.55 \le e \,/\, h_o = 0.05 \le 0.25$ **o.k.**

$N_3^* = f_{y3} t_3 [(O_v \,/\, 50)(2h_3 - 4t_3) + b_e + b_{e(ov)}]$

$b_e = 0$ because neither "flange" of the vertical lands on the chord, but there will be two $b_{e(ov)}$ terms.

$b_{e(ov)} = [10 \,/\, (b_j \,/\, t_j)](t_j \,/\, t_3)b_3$
$= [10 \,/\, (3 \,/\, 0.1875)](0.1875 \,/\, 0.125)2.5 = 2.34$

$N_3^* = (50)(0.125)[0.5(5 - 0.5) + 2.34 + 2.34] = 43.3$ kips
$\ge N_3 = 13.5$ kips

Now confirm that the connection efficiency of the *overlapped* members does not exceed that of the *overlapping* member. (See note at bottom of Table 2 for overlap connections.)

Efficiency of the vertical is $N_3^* \,/\, (A_3 f_{y3})$
$= 43.3 \,/\, (1.15 \times 50) = 0.75$

Hence, efficiency of diagonal cannot exceed 0.75.

So, $N_2^* = 0.75(2.02)(50) = 76.1$ kips $\ge N_2$
$= 59.2$ kips **o.k.**

Therefore, Panel Point 7 is acceptable.

Panel Point No. 9 is another KT connection, similar to Panel Point No. 7. A repeat of the previous calculations for the members of this panel point also shows that this connection is acceptable.

SUMMARY

The design of a welded planar truss has been demonstrated using cold-formed Hollow Structural Section, or box section, members complying with ASTM A500 Grade C. Effective lengths for compression members are in accordance with the latest CIDECT recommendations and member design is performed to the AISC LRFD specification. Specific connection checks are illustrated for gap K connections, X connections and KT (overlap) connections, in accordance with the most recent IIW/CIDECT recommendations (1990). These recommendations, for box sections, have also been adopted in the U.S. in 1992 by AWS D1.1. It is demonstrated that in order to avoid reinforcement of the connections, the structural designer must perform connection design at the same time that member selection is made, as the connections frequently control member dimensions.

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