ARCH 331. Assignment #9

Date: 10/24/13, *due 10/31/13* *Pass-fail work*

Problems: supplemental problems (9A, etc.) **and** from Onouye Chapter 9
*Notes: Problems marked with a * have been altered with respect to the problem stated in the text. Multiframe or other methods may be used for V & M diagrams and maximums.*

(9%) 9A) A joint similar to that in Figure 9a is formed with outer members of 1- inch nominal thickness (3/4-in. actual thickness) and 10d common wire nails. If the compression force to be transferred to the two side members having 5 nails each board side is 1200 lb, is the connection adequate?
(wood connection analysis)

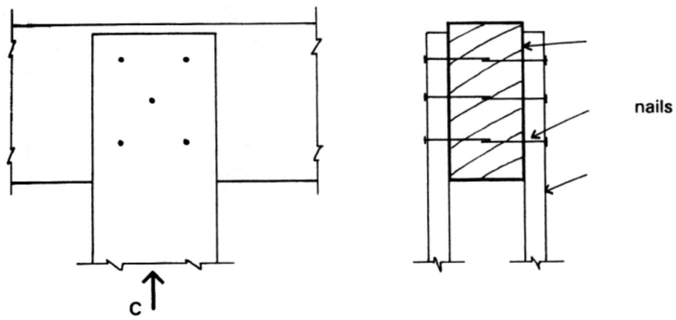


Figure 9a – nailed joint and side view

Partial answers to check with: F = 1050 lb

(9%) 9B) A truss heel joint similar to that in Figure 9b is made with gusset plates of ½-in. plywood and 8d nails. Find the tension force limit for the bottom chord *having 12 nails each plywood side. (wood connection analysis)*

Partial answers to check with:
F = 1560 lb.

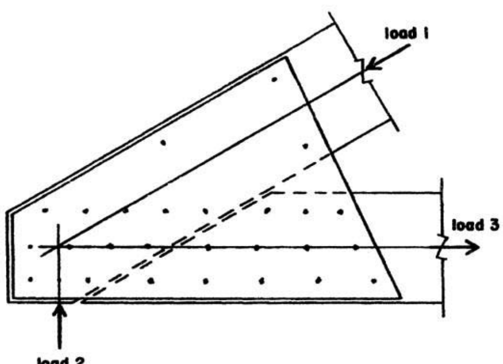


Figure 9b

TABLE 7.1 Lateral Load Capacity of Common Wire Nails (lb/nail)

Side Member Thickness, t_s (in.)	Nail Length, L (in.)	Nail Diameter, D (in.)	Pennyweight	Load per Nail for Douglas Fir-Larch G = 0.50, Z (lb)
Structural Plywood Side Members				
	2	0.113	6d	48
⅜	2½	0.131	8d	63
	3	0.148	10d	76
	2	0.113	6d	50
½	2½	0.131	8d	65
	3	0.148	10d	78
	3½	0.162	16d	92
	2	0.113	6d	58
¾	2½	0.131	8d	73
	3	0.148	10d	86
	3½	0.162	16d	100
Solid-Sawn Lumber Side Members				
	2½	0.131	8d	90
¾	3	0.148	10d	105
	3½	0.162	16d	121
	4	0.192	20d	138
	3	0.148	10d	118
	3½	0.162	16d	141
1½	4	0.192	20d	170
	4½	0.207	30d	186
	5	0.225	40d	205
	5½	0.244	50d	211

Source: Adapted from *National Design Specification for Wood Construction*, 2001 edition (Ref. 3), with permission of the publisher, American Forest & Paper Association.

MORE NEXT PAGE

(10%) 9C) A nominal 3 x 8 in redwood beam is to be supported by two 2 x 8 in. members acting as a spaced column. The minimum spacing and edge distances for the 5/8 inch bolts are shown. How many 5/8 in. bolts will be required to safely carry a load of 3200 lb? Use the chart provided.
(wood connection design)

Partial answer to check with: min n = 3.95.

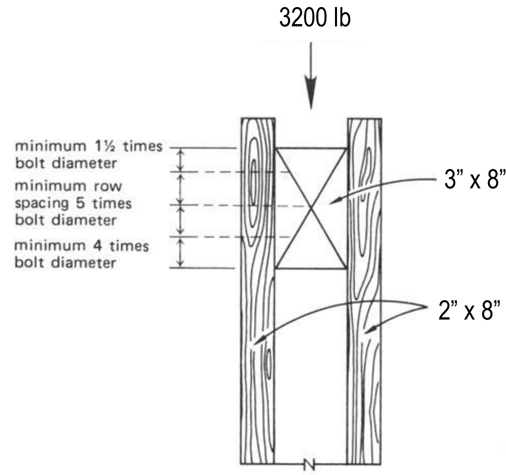


Table 7.1
Holding Power of Bolts

p = Safe loads parallel to grain in pounds
q = Safe loads perpendicular to grain in pounds

Length of Bolt in Main Wood Member[3] (in Inches)		DIAMETER OF BOLT (IN INCHES)								
		3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 1/2
1½	Single p	325	470	590	710	830	945			
	Shear q	185	215	245	270	300	325			
	Double p	650	940	1180	1420	1660	1890			
	Shear q	370	430	490	540	600	650			
2½	Single p		630	910	1155	1370	1575			
	Shear q		360	405	450	495	540			
	Double p	710	1260	1820	2310	2740	3150			
	Shear q	620	720	810	900	990	1080			
3½	Single p			990	1400	1790	2135	2455	2740	3305
	Shear q			565	630	695	760	825	895	1020
	Double p	710	1270	1980	2800	3580	4270	4910	5480	6610
	Shear q	640	980	1130	1260	1390	1520	1650	1780	2040
	Single p					1950	2535	3190	3820	4975

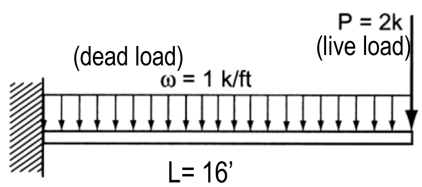
*(30%)*****9.1.21** Assuming **A992** steel, select the most economical ~~W8~~ section. Check the shear stress and determine the deflection at the free end. Assume the length is fully braced.

F_b = ~~22 ksi~~ ***(unified** ASD design and deflection)*

F_v = ~~14.5 ksi~~ $F_y = 50$ ksi

$E = 30 \times 10^3$ ksi

$\Delta_{LL} = L/260$ and $\Delta_{LL+DL} = L/200$



Problem 9.1.21

Partial answers to check with:
(for final section) $Z \geq 66.4\ in^3$, $A_{web} \geq 0.928\ in^2$, $I \geq 674.7\ in^3$

(12%) 9D) For the beam of problem 9.1.21, design the most economical beam for <u>plastic</u> flexure only *(Z_x)* for the dead and live load shown. Make certain to include self weight. The material has the following properties: $F_y = 50$ ksi, E = 30,000 ksi, $\phi_b = 0.9$. *(LRFD stress design)*

Partial answer to check with: $Z_x \geq 54.6\ in.^3$

MORE NEXT PAGE

(30%) 9E) For the beam of problem 9.1.21, use the LRFD design method and the following available moment diagram to select the most economical beam with an unbraced length of 7.75 ft and the dead and live load shown. Assume F_{yw} = 50 ksi, and ϕ_b = 0.9. The (unfactored) live load deflection and total load deflections are identical to those in the allowable stress design of problem 9.1.21.

(LRFD stress design)

Partial answer to check with: M_u = 204.8 k-ft, V_u = 22.4 k, (when the final section has been chosen, it must have: $I_{req'd} \geq 675.8$ in^{4,} $\phi M_n \geq 211.7$ k-ft. $\phi V_n \geq 23.9$ k.)

MORE NEXT PAGE (Available Moment Diagrams)

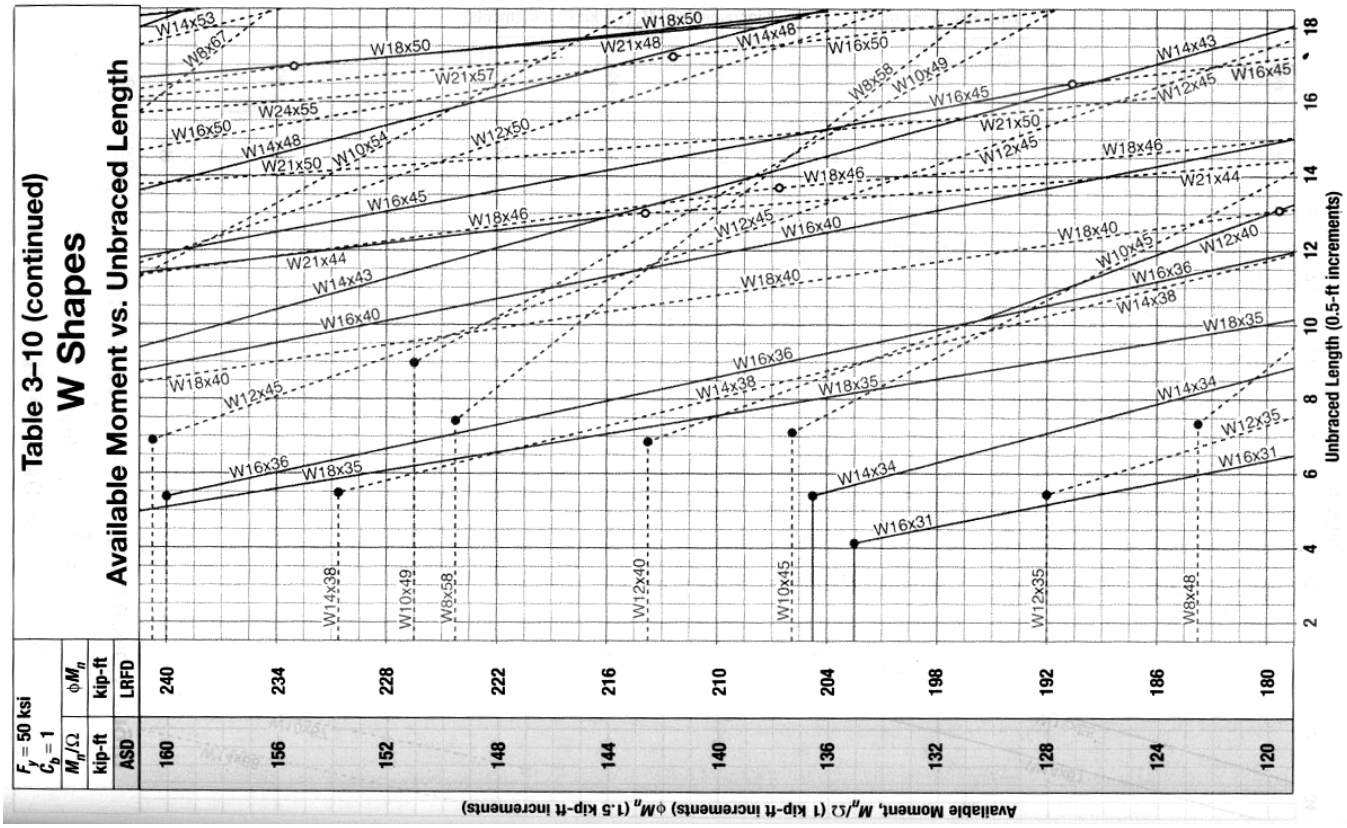


Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length
Fy = 50 ksi
Cb = 1
Mn/Ω kip-ft ASD
φMn kip-ft LRFD
Unbraced Length (0.5-ft increments)
Available Moment, Mn/Ω (1 kip-ft increments) φMn (1.5 kip-ft increments)

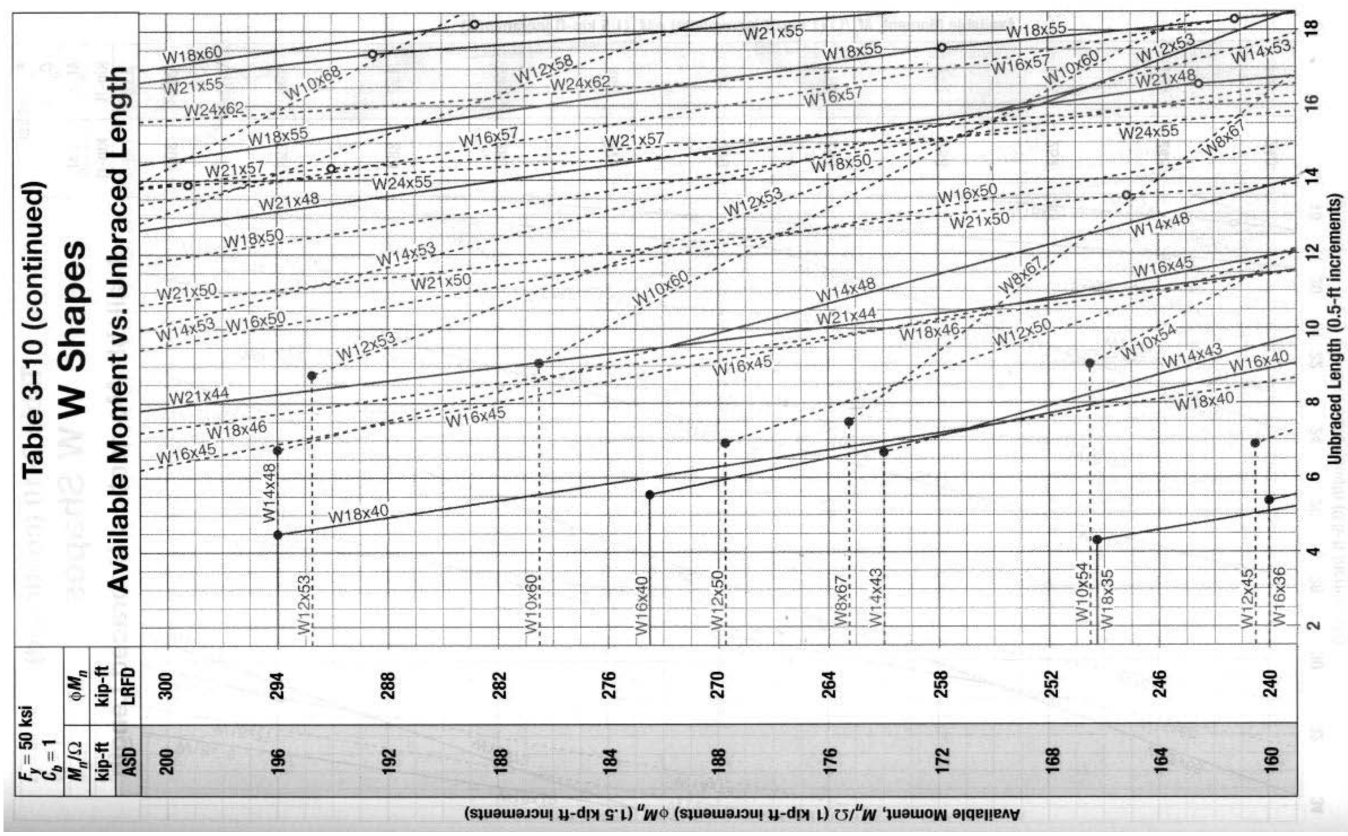


Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length
Fy = 50 ksi
Cb = 1
Mn/Ω kip-ft ASD
φMn kip-ft LRFD
Unbraced Length (0.5-ft increments)
Available Moment, Mn/Ω (1 kip-ft increments) φMn (1.5 kip-ft increments)