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$P_s = P_L + P_D = 25 + 35 = 60\ kips$

$P_u = 1.2P_D + 1.6P_L = 1.2(35) + 1.6(25) = 82\ kips$

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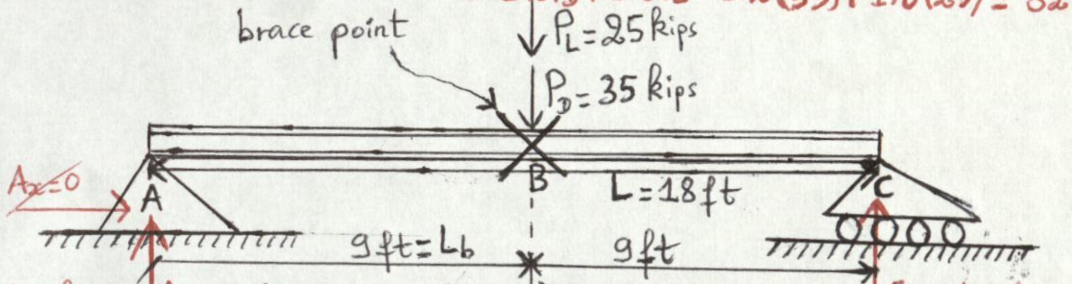
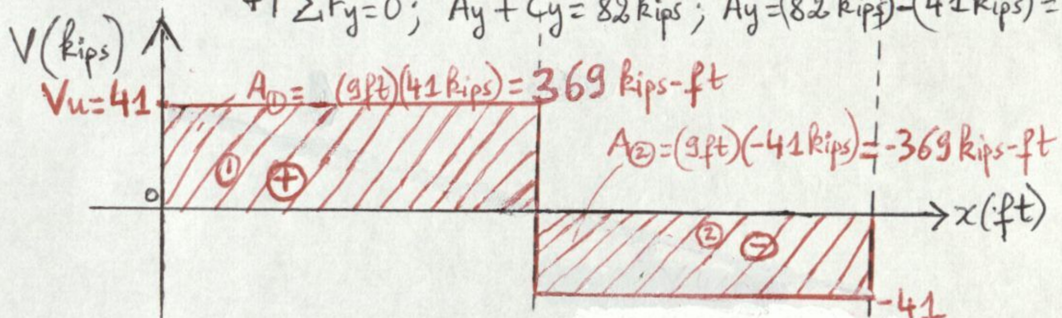
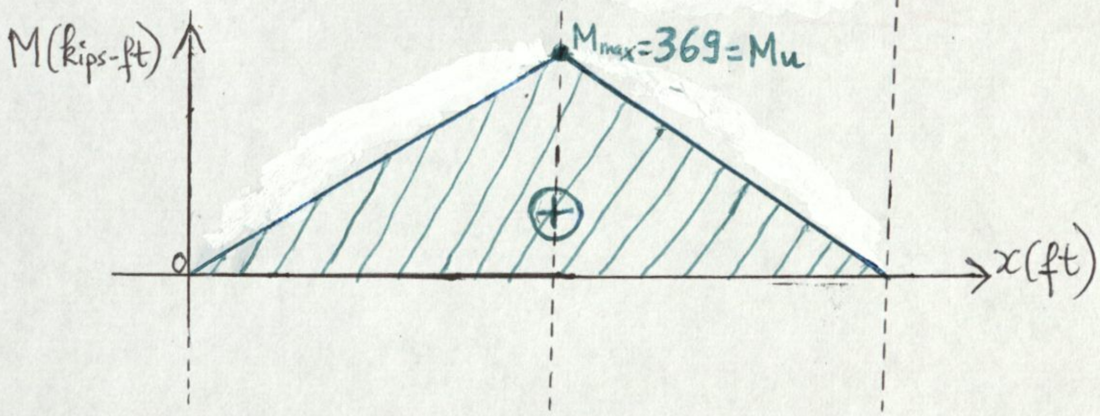


41 kips = A_y $\quad +\circlearrowleft \sum M_A = 0\ ;\ (-82\ kips)(9\ ft) + (C_y)(18\ ft) = 0$ $\quad C_y = 41\ kips$

$C_y = 41\ kips$

$+\uparrow \sum F_y = 0\ ;\ A_y + C_y = 82\ kips\ ;\ A_y = (82\ kips) - (41\ kips) = 41\ kips$





Deflection (Floor members)

$$\Delta_{(L)} = \frac{L}{360} = \frac{(18\ \cancel{ft})\left(\frac{12\ in}{1\ \cancel{ft}}\right)}{360} = 0.6\ in$$

$$\Delta_{(D+L)} = \frac{L}{240} = \frac{(18\ ft)\left(\frac{12\ in}{1\ ft}\right)}{240} = 0.9\ in$$

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$$I_{req'd(L)} = \frac{P_L L^3}{28 E \Delta_{(L)}} = \frac{(25\,kips)\left[(18\,ft)\left(\frac{12\,in}{1\,ft}\right)\right]^3}{28\,(29{,}000\,ksi)(0.6\,in)} = 517.123\,in^4$$

$$I_{req'd(D+L)} = \frac{P_s L^3}{28 E \Delta_{(D+L)}} = \frac{(60\,kips)\left[(18\,ft)\left(\frac{12\,in}{1\,ft}\right)\right]^3}{28\,(29{,}000\,ksi)(0.9\,in)} = 827.4\,in^4 \rightarrow \text{governs}$$

Beam shape	I_x	$(L_b = 9\,ft)$ ϕM_n	$\Delta = \frac{P_s L^3}{28EI}$
W21×62	$1{,}330 > I_{req'd(D+L)} \rightarrow OK$	$492\,k\text{-}ft > M_u \rightarrow OK$	$0.658\,in < \Delta_{(D+L)} \rightarrow OK$
W24×55	$1{,}350 > I_{req'd(D+L)} \rightarrow OK$	$408\,k\text{-}ft > M_u \rightarrow OK$	$0.649\,in < \Delta_{(D+L)} \rightarrow OK$
W21×44	$843 > I_{req'd(D+L)} \rightarrow OK$	$290\,k\text{-}ft < M_u \rightarrow N.G$	$1.039\,in > \Delta_{(D+L)} \rightarrow NG$
W21×48	$959 > I_{req'd(D+L)} \rightarrow OK$	$355\,k\text{-}ft < M_u \rightarrow N.G$	$0.913\,in > \Delta_{(D+L)} \rightarrow N.G$
W21×55	$1{,}140 > I_{req'd(D+L)} \rightarrow OK$	$425\,k\text{-}ft > M_u \rightarrow OK$	$0.768\,in < \Delta_{(D+L)} \rightarrow OK$

W 24×55 properties: $d = 23.6\,in$; $F_y = 50\,ksi$
$t_w = 0.395\,in$; $W_{sw} = W_D = 55\,\frac{lbs}{ft} = 0.055\,klf$

Check shear

$$\frac{d}{t_w} = \frac{23.6\,in}{0.395\,in} = 59.75$$

$$2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{29{,}000\,ksi}{50\,ksi}} = 53.95 < \frac{d}{t_w} \rightarrow \phi_v = 0.9$$

$k_v = 5$ for W shapes, I shape beams

$$1.10\sqrt{\frac{k_v E}{F_y}} = 1.10\sqrt{\frac{(5)(29{,}000\,ksi)}{50\,ksi}} = 59.2 < \frac{d}{t_w}$$

$$C_v = \frac{1.10\sqrt{\frac{k_v E}{F_y}}}{\frac{h}{t_w}} = \frac{59.2}{59.75} = 0.9915$$

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$A_w = d\,t_w = (23.6\,in)(0.395\,in) = 9.322\,in^2$

$\phi_v V_n = (\phi_v)(0.6)(F_y)(A_w)(C_v)$

$= (0.9)(0.6)\left(50\,\frac{kips}{\cancel{in^2}}\right)(9.322\,\cancel{in^2})(0.9915)$

$\phi_v V_n = 249.6 = 250\,kips > V_u = 41\,kips \rightarrow$ <u>OK</u>

close

From AISC Table 3-2, W24x55 $\rightarrow \phi_v V_{nx} = 251\,kips$

<u>Beam choice:</u> W24x55 or W21x55

Table 1-1 (continued)
W-Shapes
Properties

W24-W21

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
370	2.51	14.2	13400	957	11.1	1130	1160	170	3.27	267	3.92	25.3	201	186000
335	2.73	15.6	11900	864	11.0	1020	1030	152	3.23	238	3.86	25.0	152	161000
306	2.94	17.1	10700	789	10.9	922	919	137	3.20	214	3.81	24.8	117	142000
279	3.18	18.6	9600	718	10.8	835	823	124	3.17	193	3.76	24.6	90.5	125000
250	3.49	20.7	8490	644	10.7	744	724	110	3.14	171	3.71	24.4	66.6	108000
229	3.79	22.5	7650	588	10.7	675	651	99.4	3.11	154	3.67	24.3	51.3	96100
207	4.14	24.8	6820	531	10.6	606	578	88.8	3.08	137	3.62	24.1	38.3	84100
192	4.43	26.6	6260	491	10.5	559	530	81.8	3.07	126	3.60	24.0	30.8	76300
176	4.81	28.7	5680	450	10.5	511	479	74.3	3.04	115	3.57	23.9	23.9	68400
162	5.31	30.6	5170	414	10.4	468	443	68.4	3.05	105	3.57	23.8	18.5	62600
146	5.92	33.2	4580	371	10.3	418	391	60.5	3.01	93.2	3.53	23.6	13.4	54600
131	6.70	35.6	4020	329	10.2	370	340	53.0	2.97	81.5	3.49	23.5	9.50	47100
117	7.53	39.2	3540	291	10.1	327	297	46.5	2.94	71.4	3.46	23.5	6.72	40800
104	8.50	43.1	3100	258	10.1	289	259	40.7	2.91	62.4	3.42	23.4	4.72	35200
103	4.59	39.2	3000	245	10.0	280	119	26.5	1.99	41.5	2.40	23.5	7.07	16600
94	5.18	41.9	2700	222	9.87	254	109	24.0	1.98	37.5	2.40	23.4	5.26	15000
84	5.86	45.9	2370	196	9.79	224	94.4	20.9	1.95	32.6	2.37	23.3	3.70	12800
76	6.61	49.0	2100	176	9.69	200	82.5	18.4	1.92	28.6	2.33	23.2	2.68	11100
68	7.66	52.0	1830	154	9.55	177	70.4	15.7	1.87	24.5	2.30	23.1	1.87	9430
62	5.97	50.1	1550	131	9.23	153	34.5	9.80	1.38	15.7	1.75	23.1	1.71	4620
W24x 55	6.94	54.6	1350	114	9.11	134	29.1	8.30	1.34	13.3	1.72	23.1	1.18	3870
201	3.86	20.6	5310	461	9.47	530	542	86.1	3.02	133	3.55	21.4	40.9	62000
182	4.22	22.6	4730	417	9.40	476	483	77.2	3.00	119	3.51	21.2	30.7	54400
166	4.57	25.0	4280	380	9.36	432	435	70.0	2.99	108	3.48	21.1	23.6	48500
147	5.44	26.1	3630	329	9.17	373	376	60.1	2.95	92.6	3.46	21.0	15.4	41100
132	6.01	28.9	3220	295	9.12	333	333	53.5	2.93	82.3	3.43	20.8	11.3	36000
122	6.45	31.3	2960	273	9.09	307	305	49.2	2.92	75.6	3.40	20.7	8.98	32700
111	7.05	34.1	2670	249	9.05	279	274	44.5	2.90	68.2	3.37	20.6	6.83	29200
101	7.68	37.5	2420	227	9.02	253	248	40.3	2.89	61.7	3.35	20.6	5.21	26200

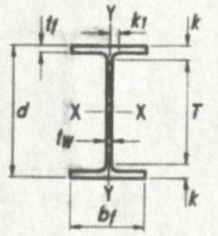

Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Distance Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W24×370[h]	109	28.0	28	1.52	1 1/2	3/4	13.7	13 5/8	2.72	2 3/4	3.22	3 5/8	1 9/16	20 3/4	5 1/2
×335[h]	98.3	27.5	27 1/2	1.38	1 3/8	11/16	13.5	13 1/2	2.48	2 1/2	2.98	3 3/8	1 1/2	↓	↓
×306[h]	89.7	27.1	27 1/8	1.26	1 1/4	5/8	13.4	13 3/8	2.28	2 1/4	2.78	3 3/16	1 7/16		
x279[h]	81.9	26.7	26 3/4	1.16	1 3/16	5/8	13.3	13 1/4	2.09	2 1/16	2.59	3	1 7/16		
×250	73.5	26.3	26 3/8	1.04	1 1/16	9/16	13.2	13 1/8	1.89	1 7/8	2.39	2 13/16	1 3/8		
×229	67.2	26.0	26	0.960	15/16	1/2	13.1	13 1/8	1.73	1 3/4	2.23	2 5/8	1 5/16		
×207	60.7	25.7	25 3/4	0.870	7/8	7/16	13.0	13	1.57	1 9/16	2.07	2 1/2	1 1/4		
×192	56.5	25.5	25 1/2	0.810	13/16	7/16	13.0	13	1.46	1 7/16	1.96	2 3/8	1 1/4		
×176	51.7	25.2	25 1/4	0.750	3/4	3/8	12.9	12 7/8	1.34	1 5/16	1.84	2 1/4	1 3/16		
×162	47.8	25.0	25	0.705	11/16	3/8	13.0	13	1.22	1 1/4	1.72	2 1/8	1 3/16		
×146	43.0	24.7	24 3/4	0.650	5/8	5/16	12.9	12 7/8	1.09	1 1/16	1.59	2	1 1/8		
×131	38.6	24.5	24 1/2	0.605	5/8	5/16	12.9	12 7/8	0.960	15/16	1.46	1 7/8	1 1/8		
×117[c]	34.4	24.3	24 1/4	0.550	9/16	5/16	12.8	12 3/4	0.850	7/8	1.35	1 3/4	1 1/8		
×104[c]	30.7	24.1	24	0.500	1/2	1/4	12.8	12 3/4	0.750	3/4	1.25	1 5/8	1 1/16	↓	↓
W24×103[c]	30.3	24.5	24 1/2	0.550	9/16	5/16	9.00	9	0.980	1	1.48	1 7/8	1 1/8	20 3/4	5 1/2
×94[c]	27.7	24.3	24 1/4	0.515	1/2	1/4	9.07	9 1/8	0.875	7/8	1.38	1 3/4	1 1/16	↓	↓
×84[c]	24.7	24.1	24 1/8	0.470	1/2	1/4	9.02	9	0.770	3/4	1.27	1 11/16	1 1/16		
×76[c]	22.4	23.9	23 7/8	0.440	7/16	1/4	8.99	9	0.680	11/16	1.18	1 9/16	1 1/16		
×68[c]	20.1	23.7	23 3/4	0.415	7/16	1/4	8.97	9	0.585	9/16	1.09	1 1/2	1 1/16	↓	↓
Choice → W24×62[c]	18.2	23.7	23 3/4	0.430	7/16	1/4	7.04	7	0.590	9/16	1.09	1 1/2	1 1/16	20 3/4	3 1/2[g]
×55[c,v]	16.2	23.6	23 5/8	0.395	3/8	3/16	7.01	7	0.505	1/2	1.01	1 7/16	1	20 3/4	3 1/2[g]
W21×201	59.3	23.0	23	0.910	15/16	1/2	12.6	12 5/8	1.63	1 5/8	2.13	2 1/2	1 5/16	18	5 1/2
×182	53.6	22.7	22 3/4	0.830	13/16	7/16	12.5	12 1/2	1.48	1 1/2	1.98	2 3/8	1 1/4	↓	↓
×166	48.8	22.5	22 1/2	0.750	3/4	3/8	12.4	12 3/8	1.36	1 3/8	1.86	2 1/4	1 3/16		
×147	43.2	22.1	22	0.720	3/4	3/8	12.5	12 1/2	1.15	1 1/8	1.65	2	1 3/16		
×132	38.8	21.8	21 7/8	0.650	5/8	5/16	12.4	12 1/2	1.04	1 1/16	1.54	1 15/16	1 1/8		
×122	35.9	21.7	21 5/8	0.600	5/8	5/16	12.4	12 3/8	0.960	15/16	1.46	1 13/16	1 1/8		
×111	32.6	21.5	21 1/2	0.550	9/16	5/16	12.3	12 3/8	0.875	7/8	1.38	1 3/4	1 1/8		
×101[c]	29.8	21.4	21 3/8	0.500	1/2	1/4	12.3	12 1/4	0.800	13/16	1.30	1 11/16	1 1/16	↓	↓

[c] Shape is slender for compression with F_y = 50 ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi.

The design shear strength is defined as

$$\boxed{\phi_v V_n = \phi_v 0.6 F_y A_w C_v,} \tag{6-19}$$

where

F_y = Yield stress,
$\boxed{A_w = \text{Area of the web} = dt_w,}$
C_v = Web shear coefficient (see below), and
ϕ_v = 0.9 or 1.0 (see below).

Since the shear stress is concentrated in the beam web, localized buckling of the web needs to be checked. A web slenderness limit for local web buckling if there are I-shaped members is defined as

$$\frac{h}{t_w} \le 2.24\sqrt{\frac{E}{F_y}}. \tag{6-20}$$

When this limit is satisfied, local web buckling does not occur and $C_v = 1.0$ and $\phi_v = 1.0$. Most I-shaped members meet the criteria in equation (6-20), except for the following shapes for $F_y = 50$ ksi: W12 × 14, W16 × 26, W24 × 55, W30 × 90, W33 × 118, W36 × 135, W40 × 149, and W44 × 230.

In Part 1 of the *AISCM*, shapes that do not meet the web slenderness criteria are marked with a superscript *v*.

For the I-shaped members listed above and for all other doubly and singly symmetric shapes and channels (excluding round HSS), $\phi_v = 0.9$ and the web shear coefficient, C_v, is as follows:

For $\dfrac{h}{t_w} \le 1.10\sqrt{\dfrac{k_v E}{F_y}}$, $C_v = 1.0$.

For $1.10\sqrt{\dfrac{k_v E}{F_y}} < \dfrac{h}{t_w} \le 1.37\sqrt{\dfrac{k_v E}{F_y}}$,

$$\boxed{C_v = \frac{1.10\sqrt{\dfrac{k_v E}{F_y}}}{\dfrac{h}{t_w}}.} \tag{6-21}$$

For $\dfrac{h}{t_w} > 1.37\sqrt{\dfrac{k_v E}{F_y}}$,

$$C_v = \frac{1.51 E k_v}{\left(\dfrac{h}{t_w}\right)^2 F_y}, \tag{6-22}$$

where $k_v = 5$ for unstiffened webs with $h/t_w < 260$, except that $k_v = 1.2$ for the stem of T-shapes.

For all steel shapes, $C_v = 1.0$, except for the following for $F_y = 50$ ksi: M10 × 7.5, M10 × 8, M12 × 10, M12 × 10.8, M12 × 11.8, M12.5 × 11.6, and M12.5 × 12.4.

Table 6-3 Values of C_b for simple-span beams

Load Description	Lateral Bracing	C_b
Concentrated load at midspan	None	1.32 — *a. concentrated load at midspan, no lateral bracing*
	At load point	1.67 = C_b — *b. concentrated load at midspan, lateral brace at midspan*
Concentrated load at ⅓ points	None	1.14 — *c. concentrated load at ⅓ points, no lateral bracing*
	At all load points	1.67 1.0 1.67 — *d. concentrated load at ⅓ points, lateral bracing at ⅓ points*
Concentrated load at ¼ points	None	1.14 — *e. concentrated load at ¼ points, no lateral bracing*
	At all load points	1.67 1.11 1.67 — *f. concentrated load at ¼ points, lateral bracing at ¼ points*
Uniformly loaded	None	1.14 — *g. uniformly loaded, no lateral bracing*
	At midspan	1.30 — *h. uniformly loaded, lateral brace at midspan*
	At ⅓ points	1.45 1.01 1.45 — *i. uniformly loaded, lateral bracing at ⅓ points*
	At ¼ points	1.52 1.06 1.52 — *j. uniformly loaded, lateral bracing at ¼ points*

Adapted from Table 3-1 of the *AISCM*.

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

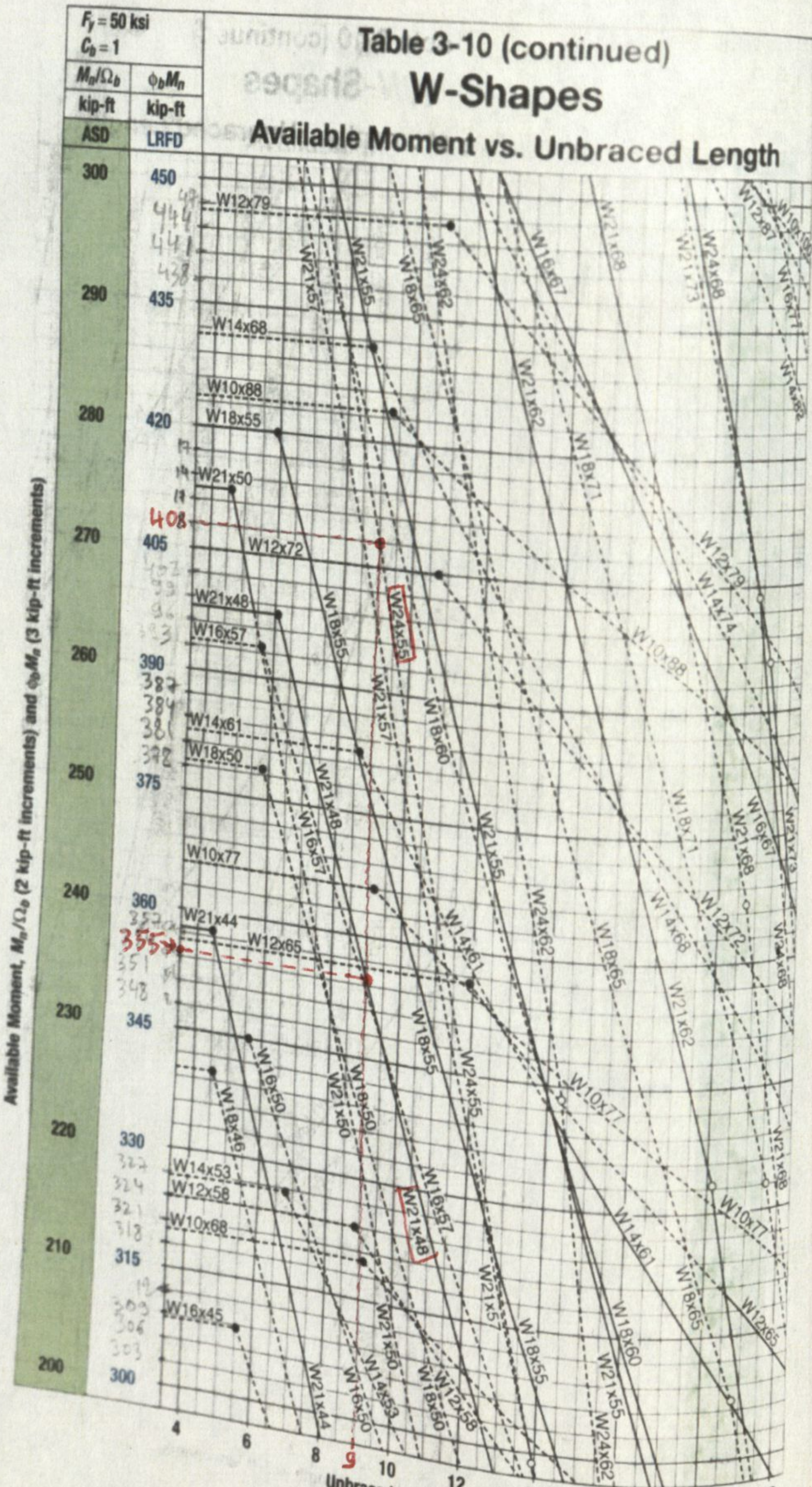

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

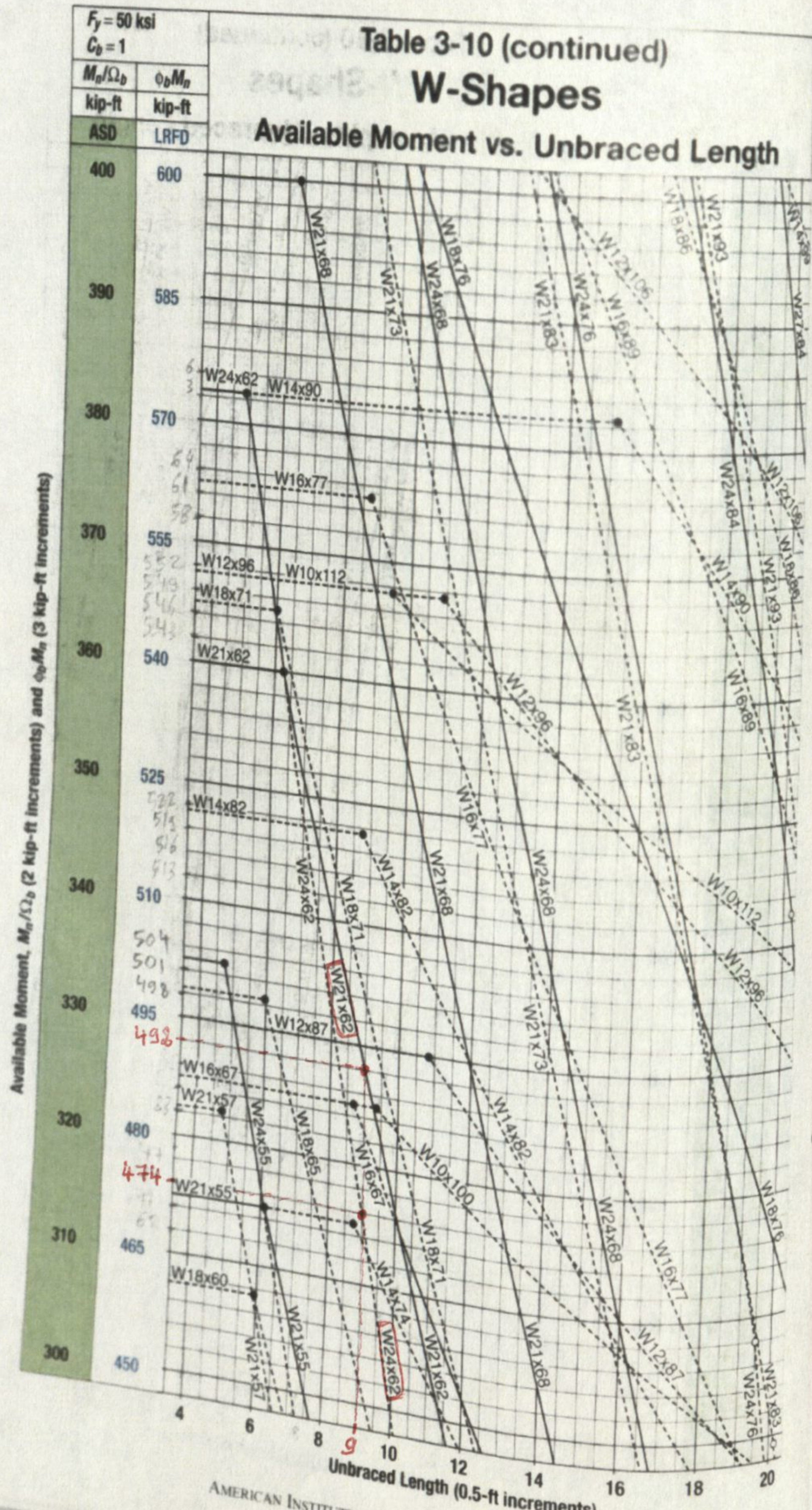

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length