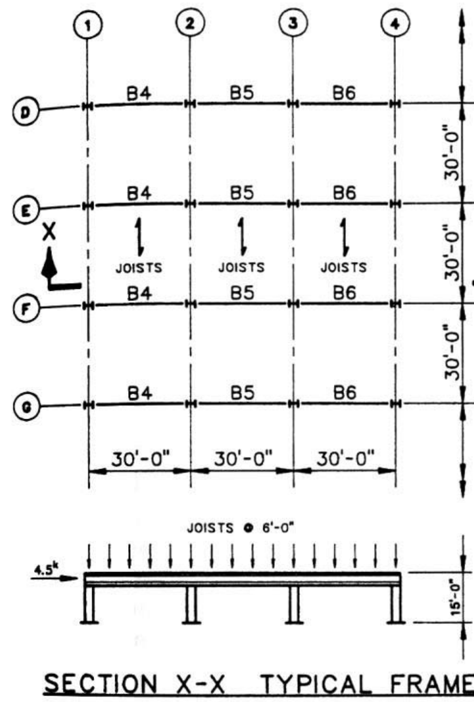
Case Study in Steel

adapted from Structural Design Guide, Hoffman, Gouwens, Gustafson & Rice., 2nd ed.

Building description

The building is a one-story steel structure, typical of an office building. The figure shows that it has three 30 ft. bays in the short direction and a large number of bays in the long direction. Some options for the structural system include fully restrained with rigid connections and fixed column bases, simple framing with "pinned" connections and column bases requiring bracing against sideway, and simple framing with continuous beams and shear connections, pinned column bases and bracing against sidesway. This last situation is the one we'll evaluate as shown in Figure 2.5(c).

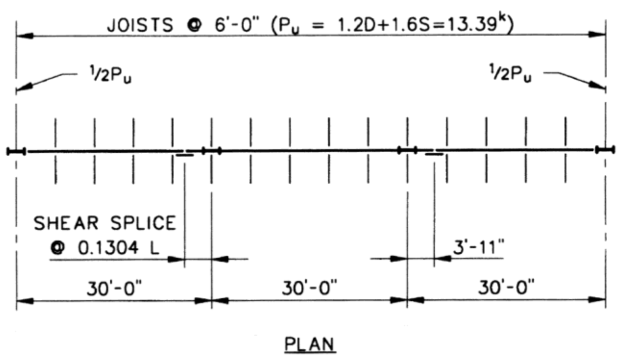


Loads

Live Loads:
Snow on Roof: 30 lb/ft^2 (1.44 kPa)

Wind: 20 lb/ft^2 (0.96 kPa)

Dead Loads:
Roofing: 8 lb/ft^2 (0.38 kPa)
Estimated decking: 3 lb/ft^2 (0.14 l
Ceiling: 7 lb/ft^2 (0.34 kPa)
Total: 18 lb/ft^2 (0.86 kPa)



Materials

A36 steel for the connection angles a
(F_y = 36 ksi, F_u = 58 ksi) and A992 s
for the beams and columns (F_y = 50
K series open web joists and roof de

Decking:

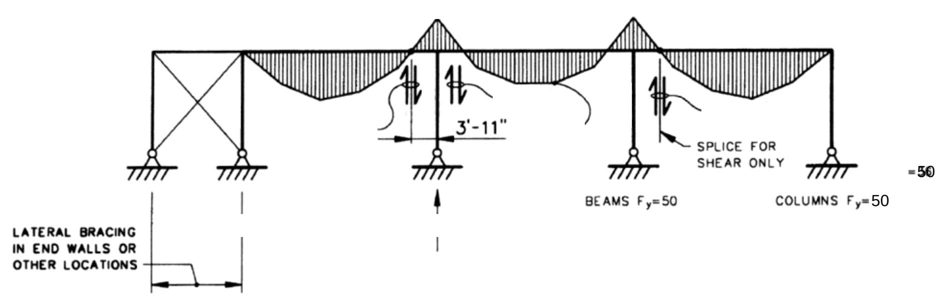


Figure 2.5(c) Type SF — cantilever-suspended span system, braced against sidesway

Decking selection is typically allowable stress design. Tables will give allowable total uniform load (taking self weight into account) based on stresses and deflection criteria for typical spans and how many spans are supported. The table (and description) for a Vulcraft 1.0 E deck is provided.

Areas in gray are governed by live load roof deflection.

The total load with snow and roofing = 30 psf + 8 psf = 38 psf.

VERTICAL LOADS FOR TYPE 1.0E

No. of Spans	Deck Type	Max. SDI Const. Span	Allowable Total (Dead + Live) Uniform Load (PSF) — Span (ft.-in.) C. to C. of Support: 2'-6	3'-0	3'-6	4'-0	4'-6	5'-0	5'-6	6'-0	6'-6	7'-0	7'-6
1	E26	2'-10	178	107	71	51	39	31	26	22	20	18	16
	E24	3'-5	249	148	97	68	51	40	32	27	24	21	19
	E22	3'-10	316	187	122	85	63	48	39	32	27	24	21
	E20	4'-2	379	224	145	100	73	56	45	37	31	27	24
2	E26	3'-4	273	189	139	107	81	62	49	40	34	29	25
	E24	4'-0	396	275	202	153	111	83	65	52	43	37	32
	E22	4'-6	515	357	263	190	137	102	79	63	52	44	37
	E20	5'-0	634	440	323	227	162	121	94	74	61	51	43
3	E26	3'-4	310	198	128	89	66	51	40	33	28	25	22
	E24	4'-0	469	276	177	122	89	67	53	43	36	31	27
	E22	4'-6	588	344	221	151	109	82	64	52	43	36	31
	E20	5'-0	707	413	264	180	129	97	75	60	50	42	36

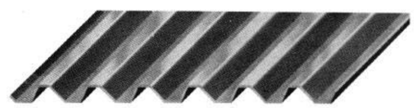
Notes: 1. Load tables are calculated using sectional properties based on the steel design thickness shown in the Steel Deck Institute (SDI) Design Manual.

2. Loads shown in the shaded areas are governed by the live load deflection not in excess of 1/240 of the span. A dead load of 10 PSF has been included.

1.0 E

Maximum Sheet Length 42'-0

Extra Charge for Lengths Under 6'-0

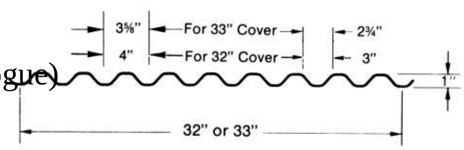


Open Web Joists:

Open web joist selection is either based on allowable stress design or LRFD resistance for flexure (*not for deflection*). The total factored distributed load for joists at 6 ft on center will be:

$w_{total} = (1.2\times 18 lb/ft^2 + 1.6\times 30\ lb/ft^2)(6\ ft) + 1.2(\textit{8 lb/ft estimated})$

$= 427.2$ lb/ft (with $1.2D + 1.6(L, or\ L_r, or\ S, or\ R)$ by catalogue)



$w_{live} = 30\ lb/ft^2(6\ ft) = 180$ lb/ft

LRFD

STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, K-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown in Pounds per Linear Foot (plf)

Joist Designation	18K3	18K4	18K5	18K6	18K7	18K9	18K10	20K3	20K4	20K5	20K6	20K7	20K9	20K10	22K4	22K5	22K6	22K7	22K9	22K10	22K11
Depth (In.)	18	18	18	18	18	18	18	20	20	20	20	20	20	20	22	22	22	22	22	22	22
Approx. Wt. (lbs./ft.)	6.6	7.2	7.7	8.5	9	10.2	11.7	6.7	7.6	8.2	8.9	9.3	10.8	12.2	8	8.8	9.2	9.7	11.3	12.6	13.8
Span (ft.) ↓																					
18	825 550	825 550	825 550	825 550	825 550	825 550	825 550														
19	771 494	825 523	825 523	825 523	825 523	825 523	825 523														
20	694 423	825 490	825 490	825 490	825 490	825 490	825 490	775 517	825 550	825 550	825 550	825 550	825 550	825 550							
21	630 364	759 426	825 460	825 460	825 460	825 460	825 460	702 453	825 520	825 520	825 520	825 520	825 520	825 520							
22	573 316	690 370	777 414	825 438	825 438	825 438	825 438	639 393	771 461	825 490	825 490	825 490	825 490	825 490	825 548	825 548	825 548	825 548	825 548	825 548	825 548
23	523 276	630 323	709 362	774 393	825 418	825 418	825 418	583 344	703 402	793 451	825 468	825 468	825 468	825 468	777 491	825 518	825 518	825 518	825 518	825 518	825 518
24	480 242	577 284	651 318	709 345	789 382	825 396	825 396	535 302	645 353	727 396	792 430	825 448	825 448	825 448	712 431	804 483	825 495	825 495	825 495	825 495	825 495
25	441 214	532 250	600 281	652 305	727 337	825 377	825 377	493 266	594 312	669 350	729 380	811 421	825 426	825 426	657 381	739 427	805 464	825 474	825 474	825 474	825 474
26	408 190	492 222	553 249	603 271	672 299	807 354	825 361	456 236	549 277	618 310	673 337	750 373	825 405	825 405	606 338	682 379	744 411	825 454	825 454	825 454	825 454
27	378 169	454 198	513 222	558 241	622 267	747 315	825 347	421 211	508 247	573 277	624 301	694 333	825 389	825 389	561 301	633 337	688 367	768 406	825 432	825 432	825 432
28	351 151	423 177	477 199	519 216	577 239	694 282	822 331	391 189	472 221	532 248	579 269	645 298	775 353	825 375	522 270	588 302	640 328	712 364	825 413	825 413	825 413
29	327 136	394 159	444 179	483 194	538 215	646 254	766 298	364 170	439 199	495 223	540 242	601 268	723 317	825 359	486 242	547 272	597 295	664 327	798 387	825 399	825 399
30	304 123	367 144	414 161	451 175	502 194	603 229	715 269	340 153	411 179	462 201	504 218	561 242	675 286	799 336	453 219	511 245	556 266	619 295	745 349	825 385	825 385
31	285 111	343 130	387 146	421 158	469 175	564 207	669 243	318 138	384 162	433 182	471 198	525 219	631 259	748 304	424 198	478 222	520 241	580 267	697 316	825 369	825 369

Deflection will limit the selection, and the most lightweight choice is the 22K4 which weighs approximately 8 lb/ft. Special provisions for bridging are required for the shaded area lengths and sections.

Continuous Beams:

LRFD design is required for the remaining structural steel for the combinations of load involving Dead, Snow and Wind. The bracing must be designed to resist the lateral wind load.

The load values are:

for D: $w_D = 18\ lb/ft^2 \cdot 30\ ft + (8\ lb/ft \cdot 30\ ft)/\ 6\ ft = 580\ lb/ft$

for S: $w_S = 30\ lb/ft^2 \cdot 30\ ft = 900\ lb/ft$

for W: $w_W = 20\ lb/ft^2 \cdot 30\ ft = 600\ lb/ft$ (up or down)

and laterally $V = 600\ lb/ft(15ft/2) = 4500\ lb$

These DO NOT consider self weight of the beam.

The applicable combinations for the tributary width of 30 ft. are:

1.4*D* $w_u = 1.4(580\ lb/ft) = 812\ lb/ft$

1.2*D* + 1.6*L* + 0.5(*L*$_r$ *or S or R*)

$w_u = 1.2(580\ lb/ft) + 0.5(900\ lb/ft) = 1146\ lb/ft$

1.2D + 1.6(L_r *or S or R) + (L or 0.8W)*

$w_u = 1.2(580\ lb/ft) + 1.6(900\ lb/ft) + 0.8(600\ lb/ft) = \underline{2616\ lb/ft}$

1.2D + 1.6W + L + 0.5(L_r *or S or R)*

$w_u = 1.2(580\ lb/ft) + 1.6(600\ lb/ft) + 0.5(900\ lb/ft) = 2106\ lb/ft$

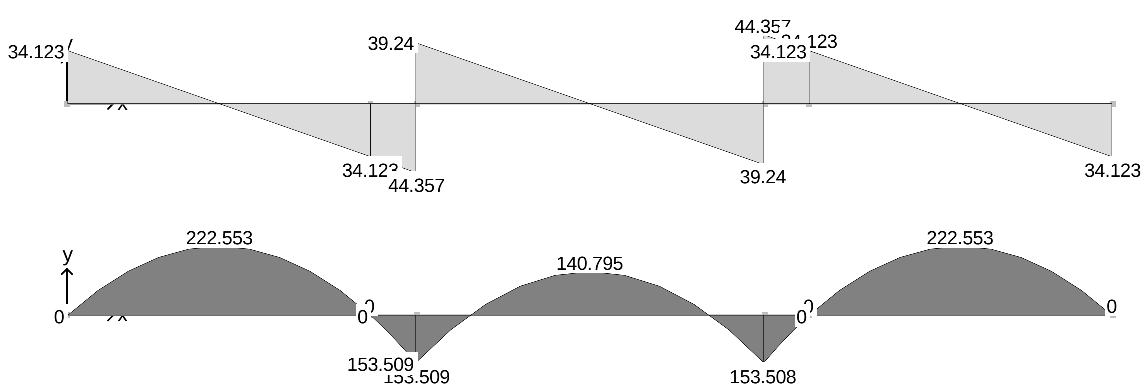
1.2D + 1.0E + L + 0.25S

$w_u = 1.2(580\ lb/ft) + 0.25(900\ lb/ft) = 921\ lb/ft$

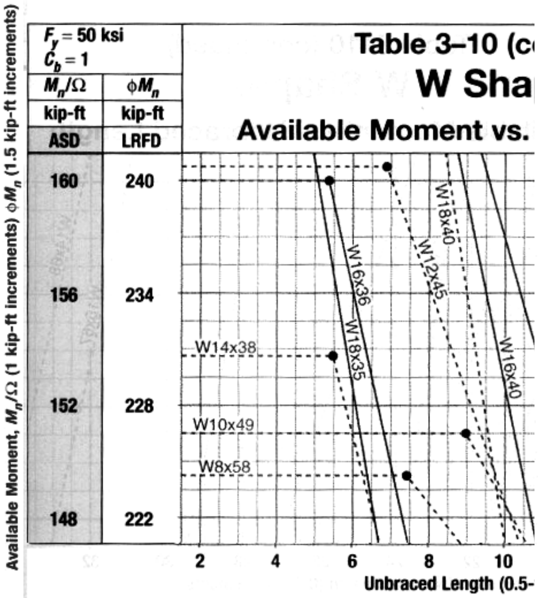
0.9D + 1.6W +1.6H $w_u = 0.9(580\ lb/ft) + 1.6(-600\ lb/ft)$ *[uplift]* = - 438 lb/ft (up)

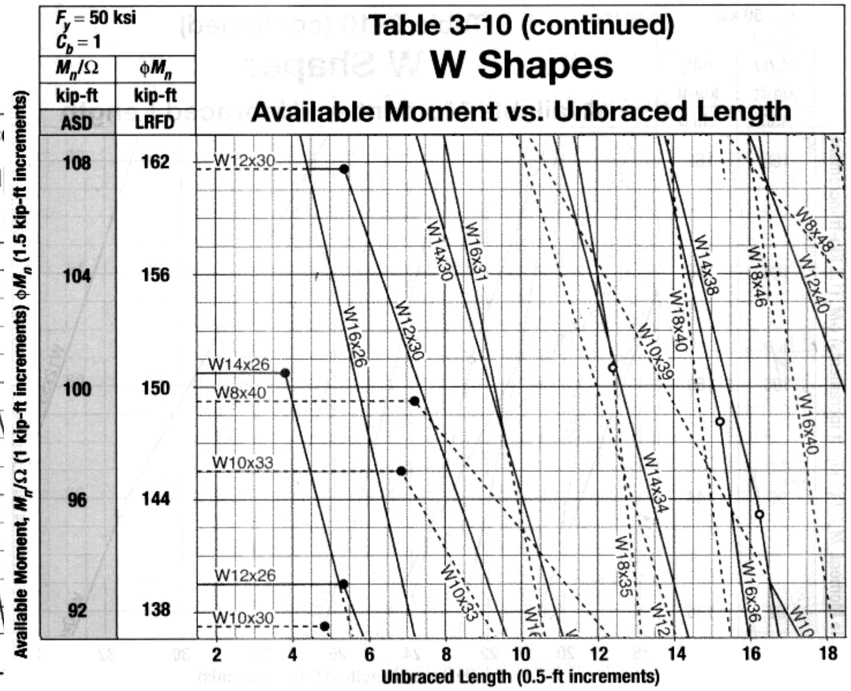
L, R, L_r *, E & H don't exist for our case.*

For the largest load case, the shear & bending moment diagrams are:

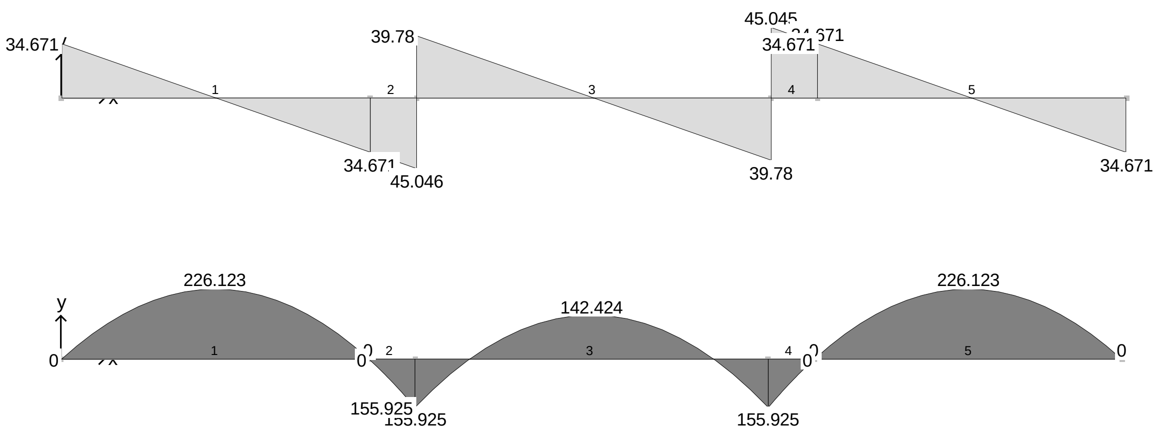


For the beams, we know that the maximum unbraced length is 6 ft. For the middle 6 feet of the end span, the moment is nearly uniform, so $C_b = 1$ is acceptable ($C_b = 1.08$ for constant moment). For the interior span, C_b is nearly 1 as well.





Choosing a W18x35 (M_u = 229 k-ft) for the end beams, and a W12x30 (M_u = 158 k-ft) for the interior beam, the self weight can be included in the total weight. The diagrams change to:



Check beam shear: $V_u \le \phi_v V_n = 1.0(0.6F_{yw}A_w)$

Exterior V_u = 34.67 k $\le$ 1.0(0.6)(50 ksi)(17.1 in.)(0.3 in.) = 153.9 k OK

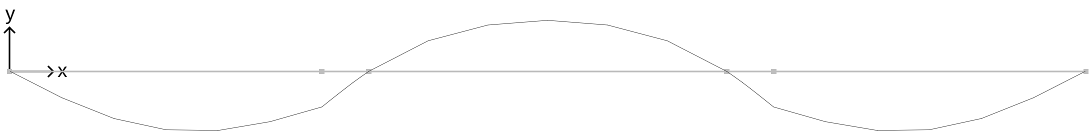
W18x35: d = 17.7 in., t_w = 0.3 in., I_x = 510 in.^{4}

Interior V_u = 45.05 k $\le$ 1.0(0.6)(50 ksi)(12.3 in.)(0.26 in.) = 95.94 k OK

W12x30: d = 12.3 in., t_w = 0.26 in., I_x = 238 in.^{4}

Check deflection (NO LOAD FACTORS) for total and live load (gravity and snow).

Exterior Beam: worst deflection is from no live load on the center span:



Maximum Δ_{total} = 2.20 in.

Is $\Delta_{total} \leq L/240$ = 360 in./240 = 1.5 in.? NO GOOD

We need an I ≥ (2.20in./1.5in.)(510 in.^{4}) = 748 in.^{4}

Maximum Δ_{live} = 1.86 in.

Is $\Delta_{live} \leq L/360$ = 360 in./360 = 1.0 in.? NO GOOD

We need an I ≥ (1.86in./1.0in.)(510 in.^{4}) = 949 in.^{4}

The W21x48 looks promising, but it has a note that it exceeds the compact limit for flexure.

Choose a W21 x 50 (I_x = 984 in.^{4}) (because the W21x48 would require extra work!)

Now, Δ_{live} = 1.07 in., which is reasonable close.

Table 3–2 (continued)
W Shapes
Selection by Z_x

F_y = 50 ksi — Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF	BF	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
	in.^{3}	kip-ft ASD	kip-ft LRFD	kip-ft ASD	kip-ft LRFD	kips ASD	kips LRFD	ft	ft	in.^{4}	kips ASD	kips LRFD
W21×55	**126**	**314**	**473**	**192**	**289**	**10.8**	**16.3**	**6.11**	**17.4**	**1140**	**156**	**234**
W14×74	126	314	473	196	294	5.34	8.03	8.76	31.0	795	128	191
W18×60	123	307	461	189	284	9.64	14.5	5.93	18.2	984	151	227
W12×79	119	297	446	187	281	3.77	5.67	10.8	39.9	662	116	175
W14×68	115	287	431	180	270	5.20	7.81	8.69	29.3	722	117	175
W10×88	113	282	424	172	259	2.63	3.95	9.29	51.1	534	131	197
W18×55	**112**	**279**	**420**	**172**	**258**	**9.26**	**13.9**	**5.90**	**17.5**	**890**	**141**	**212**
W21×50	**110**	**274**	**413**	**165**	**248**	**12.2**	**18.3**	**4.59**	**13.6**	**984**	**158**	**237**
W12×72	108	269	405	170	256	3.72	5.59	10.7	37.4	597	105	158
W21×48^f	**107**	**265**	**398**	**162**	**244**	**9.78**	**14.7**	**6.09**	**16.6**	**959**	**144**	**217**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.96	7.46	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.69	13.1	5.83	17.0	800	128	192
W10×77	97.6	244	366	150	225	2.59	3.90	9.18	45.2	455	112	169
W12×65^f	96.8	237	356	154	231	3.60	5.41	11.9	35.1	533	94.5	142

ASD	LRFD
Ω_b = 1.67 Ω_v = 1.50	ϕ_b = 0.90 ϕ_v = 1.00

^{f} Shape exceeds compact limit for flexure with F_y = 50 ksi.

Interior Beam: worst deflection is from load on all spans:

Maximum Δ_{total} (at midspan) = 1.31 in.

Is $\Delta_{total} \leq L/240$ = 360 in./240 = 1.5 in.? OK

Maximum Δ_{live} (at midspan) = 0.94 in.

Is $\Delta_{live} \leq L/360$ = 360 in./360 = 1.0 in.? OK

Columns:

The load in the interior columns: P_u = 85 k (sum of the shears). This column will see minimal eccentricity from the difference in shear and half the column depth as the moment arm.

The load in the exterior columns: P_u = 35 k. These columns will see some eccentricity from the beam shear connections. We can determine this by using half the column depth as the eccentricity distance.

The effective length of the columns is 15 ft (no intermediate bracing). Table 4-1 shows design strength in kips for W8 shapes (the smallest). The lightest section at 15 feet has a capacity of 230 k; much greater than what we need even with eccentricity.

Table 4-1 (continued)
Available Strength in Axial Compression, kips
F_y = 50 ksi W Shapes W8

Shape	W8×			
Wt/ft	35		31	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD
0	308	463	273	411
6	281	423	249	374
7	272	409	241	362
8	262	394	232	348
9	251	377	222	333
10	239	359	211	317
11	226	340	200	301
12	213	321	189	283
13	200	301	177	266
14	187	281	165	248
15	174	261	153	230
16	160	241	141	212
17	147	221	130	195
18	135	203	118	178
19	123	184	108	162
20	111	166	97.2	146
22	91.5	138	80.3	121
24	76.9	116	67.5	101
26	65.5	98.5	57.5	86.5
28	56.5	84.9	49.6	74.5
30	49.2	74.0	43.2	64.9
32	43.3	65.0	38.0	57.1
34				

Effective length KL (ft) with respect to least radius of gyration r_y

The exterior column connection moment (unmagnified) when the W8x31 depth = 8.0in

$$(35k)(8.0in/2)(\frac{1ft}{12in}) = \ = 11.7 \text{ k-ft.}$$

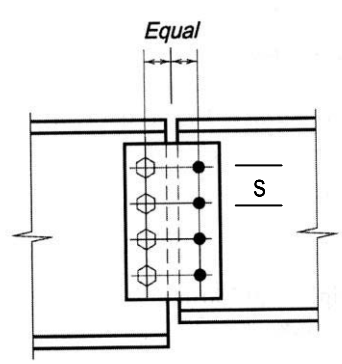
The capacity of a W8x31 with an unbraced length of 15 ft (from another beam chart) = 114 $^{\text{k-ft}}$.

For $\frac{P_r}{P_c} < 0.2$: $$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \le 1.0$$

$$\frac{35k}{230k} = 0.15 < 0.2: \qquad \frac{35k}{2(230k)} + \left(\frac{11.7^{k-ft}}{114^{k-ft}}\right) = 0.179 \le 1.0$$

so OK for eccentric loading of the beam-column (but we knew that).

Beam Shear Splice Connection:

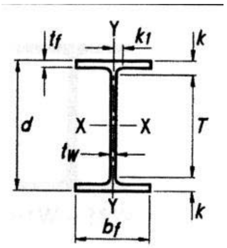


For this all-bolted single-plate shear splice, R_u = 35 k

W21x50: d = 20.8 in., t_w = 0.38 in.

W12x30: d = 12.3 in., t_w = 0.26 in.

The plate material is A36 with F_y = 36 ksi and F_u = 58 ksi. We need to check that we can fit a plate within the fillets and provide enough distance from the last holes to the edge.



For the W12x30, T = 10.125 in., which limits the plate height.

For a plate, s (hole spacing) = 3" and minimum edge distance is 1¼".

For ¾ in. diameter A325-N bolts and standard holes without a concern for deformation of the holes, the capacity per bolt is:

shear: $: R_u \le \phi_v R_n$ $\phi = 0.75$, $R_n = F_n A_b$, where F_n = 54 ksi

$$35k \le n(0.75)(54ksi)[\frac{\pi(0.75in)^2}{4}]$$

so n ≥ 1.96. Use 2 bolts (1@3 in. + 2@1.25 ≈ 5.5 in. < 10.125 in.)

bearing for 2 rows of bolts: depends on thickness of thinnest web (t=0.26 in.) and the connected material

$$R_u \le \phi R_n \quad \phi = 0.75,\ R_n = 1.5L_c tF_u \le 3.0dtF_u$$

L_c = 1.75 in. from the vertical edge of the beam to the edge of a hole

$$35k \le 2^{bolts}[0.75(1.5)(1.75in)(0.26in)(65ksi) = 38.0 \text{ k}$$
$$\le 2^{bolts}[0.75(3)(0.75in)(0.26in)(65ksi) = 57.0 \text{ k OK}$$

If the spacing between the holes across the splice is 4 in., the eccentricity, e_x,is 2 inches. We need to find C, which represents the number of bolts that are effective in resisting the eccentric shear force.

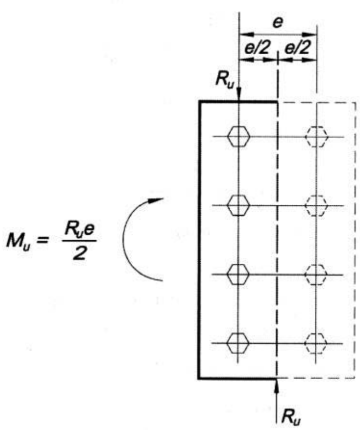


Fig. 10–22. Eccentricity in a symmetrical shear splice.

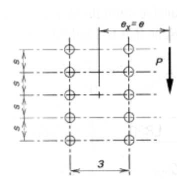
Table 7-7
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$R_n = C \times r_n$
or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi r_n}$	$C_{min} = \frac{\Omega P_a}{r_n}$

where
P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
	2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0
	3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5
	4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	5	0.45	1.42	2.59	4.21	6.01	8.00	10.1	12.2	14.4	16.5	18.7	20.8
	6	0.39	1.22	2.25	3.69	5.32	7.17	9.16	11.2	13.4	15.5	17.7	19.8
	7	0.35	1.08	1.99	3.27	4.74	6.46	8.33	10.3	12.4	14.5	16.7	18.8
	8	0.31	0.96	1.78	2.93	4.27	5.86	7.60	9.50	11.5	13.6	15.7	17.8
	9	0.28	0.86	1.60	2.65	3.87	5.34	6.97	8.75	10.7	12.7	14.7	16.8
	10	0.26	0.78	1.46	2.42	3.53	4.90	6.42	8.10	9.91	11.8	13.8	15.9
3	12	0.22	0.66	1.24	2.06	3.01	4.19	5.51	7.01	8.63	10.4	12.2	14.2

r_n is the nominal shear per bolt:

$$C_{min} = \frac{35k}{0.75(54ksi)(0.75in)^2 \pi/4} = 1.95 \quad \text{(which we found as } n\text{)}$$

C off the table is 2.54 bolts which is more than the minimum of 1.95 (which is why we have 2). OK.

If the plate is 3/8 in. thick x 8 in. wide x 9 in. tall, check *bolt bearing on plate:*

$\phi R_n = 2.4dtF_u$ (per bolt)

2 bolts[2.4(0.75 in.)(0.375 in.)(58 ksi) = 78.3 k > 35 k OK

Check *flexure of the plate:*

design moment: $M_u = \frac{R_u e}{2} = \frac{35k \times 4in}{2}$ = 70.0 k-in

yielding capacity: $\phi M_n = \phi F_y S_x \quad \phi = 0.9$ (5.5 in. tall section, 3/8 in. thick)

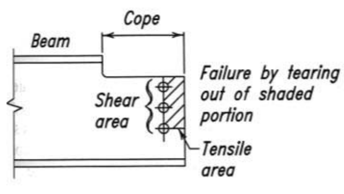
$0.9(36ksi)\left[\frac{0.375in(5.5in)^2}{6}\right]$ = 61.25 k-in > 70.0 k-in NOT OK

with 6 in. tall, $\phi M_n = 72.9$ k-in

rupture $\phi M_n = \phi F_u S_{net} \quad \phi = 0.75$

$S_{net} = {}^{I_{net}}\!/_{c}$ and can be looked up or calculated = 1.74 in^3

$0.75(58ksi)(1.74in^3)$ = 75.7 k-in > 70.0 k-in OK



Check *shear yielding of the plate:* $R_u \le \phi R_n \quad \phi = 1.00 \quad R_n = 0.6 F_y A_g$

(1.00)[0.6(36 ksi)(6 in.)(0.375 in.)] = 48.6 k > 35 k OK

Check *shear rupture of the plate:* $R_u \le \phi R_n \quad \phi = 0.75 \quad R_n = 0.6 F_u A_{nv}$

for ¾" diameter bolts, the effective hole width is (0.75 + 1/8) = 0.875 in.:

(0.75)[0.6(58 ksi)(6 in. – 2 x 0.875 in.)(0.375 in.)] = 41.6 k > 35 k OK

Check *block shear rupture of the plate:* $R_u \le \phi R_n \quad \phi = 0.75$

$$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$$

with U_{bs} = 0.5 when the tensile stress is non-uniform. (The tensile stress switches direction across the splice.) *(and assuming 2 in. of width to the center of the bolt hole)*

$$R_n = 0.60(58ksi)(0.375in)[1.5in + 3in - 1.5^{holes}(0.875)] +$$
$$0.5(58ksi)(0.375in)(2in - {}^{0.875in}\!/_{2}) = 58.6.9k$$
$$\le 0.6(36ksi)(0.375in)(1.5in + 3in) + 0.5(36ksi)(0.375in)(2in - {}^{0.875in}\!/_{2}) = 47.0k$$

35 k < 0.75(47.0 k) = 35.2 k OK

Column Base Plate:

Column base plates are designed for bearing on the concrete (concrete capacity) and plastic hinge development from flexure because the column "punches" down the plate and it could bend upward near the edges of the column (shown as *0.8b$_f$* and *0.95d*). The plate dimensions are B and N. The concrete has a compressive strength, *f'_c = 3 ksi.*

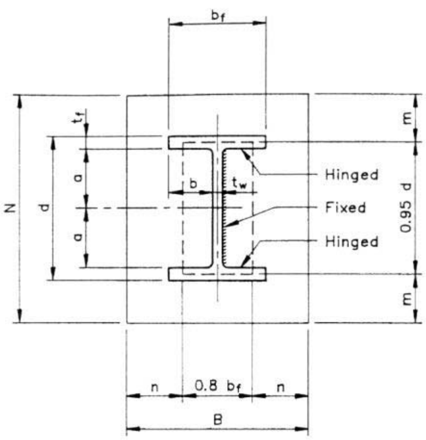


Figure 5.6. Column base plate dimensions

For W8 x 31: d = 8.0 in., b_f = 8.0 in., and if we provide width to put in bolt holes, we could use a 12 in. by12 in. plate (allowing about 2 inches each side). We will look at the interior column load of 85 k.

minimum thickness: $t_{min} = l\sqrt{\frac{2P_u}{0.9F_yBN}}$

where l is the larger of m, n and $\lambda n'$

$m = (N - 0.95d)/2$ = (12 in. – 0.95 x 8.0 in.)/2 = 2.2 in.

$n = (B - 0.8b_f)/2$ = (12 in. – 0.8 x 8.0 in.)/2 = 2.8 in.

$$n' = \frac{\sqrt{db_f}}{4} = \frac{\sqrt{8.0in \cdot 8.0in}}{4} = 2.0 \text{ in.}$$

λ is derived from a term X which takes the bounding area of the column, the perimeter, the axial force, and the concrete compressive strength into account:

$$X = \frac{4db_f}{(d+b_f)^2}\cdot\frac{P_u}{\phi_c P_p} = \frac{4db_f}{(d+b_f)^2}\cdot\frac{P_u}{\phi_c(0.85f_c')BN} = \frac{4\cdot 8.0in\cdot 8.0in}{(8.0in+8.0in)^2}\cdot\frac{85k}{0.6(0.85\cdot 3ksi)12in\cdot 12in}$$

$$= 0.386$$

$$\lambda = \frac{2\sqrt{X}}{(1+\sqrt{1-X})} \le 1 \qquad = \frac{2\sqrt{0.386}}{(1+\sqrt{1-0.386})} = 0.697 \text{ so } \lambda n' = (0.697)(2.0 \text{ in.}) = 1.39 \text{ in.}$$

$$t_p = l\sqrt{\frac{2P_u}{0.9F_yBN}} = (2.8in)\sqrt{\frac{2\cdot 85k}{0.9(36ksi)(12in)(12in)}} = 0.534 \text{ in.}$$

Use a 9/16 in. thick plate.

The anchor bolts must also be able to resist lateral shear. There also is friction between the steel and concrete to help. The International Building Code provided specifications for minimum edge distances and anchorage.

Continuous Beam Over Interior Column:

The design for this connection will involve a bearing plate at the top of the column, with a minimum number of bolts through the beam flanges to the plate. Because there will be high local compression, stiffener plates for the web will need to be added (refer to a plate girder design). Flexure with a reduced cross section area of the flanges should be checked.