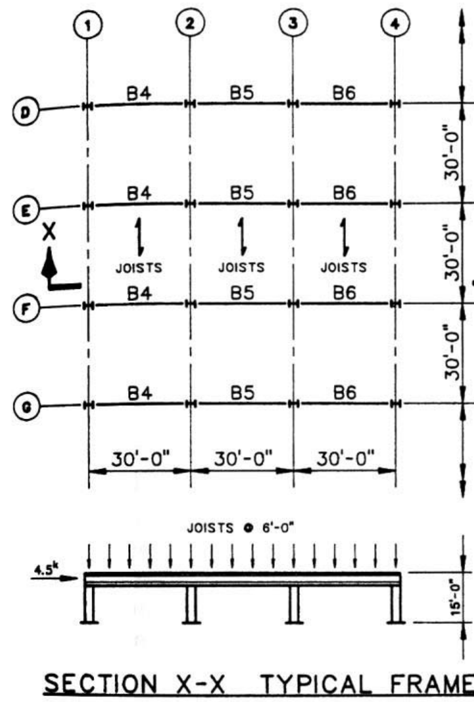
Case Study in Steel

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

Building description



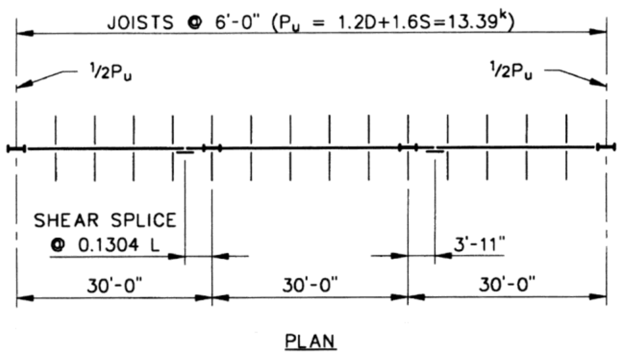
The building is a one-story steel structure, typical of an office building. The figure shows that it has three 30 ft. bays in the short direction and a large number of bays in the long direction. Some options for the structural system include fully restrained with rigid connections and fixed column bases, 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 c
for the beams and columns (F_y = 50
K series open web joists and roof de

Decking:

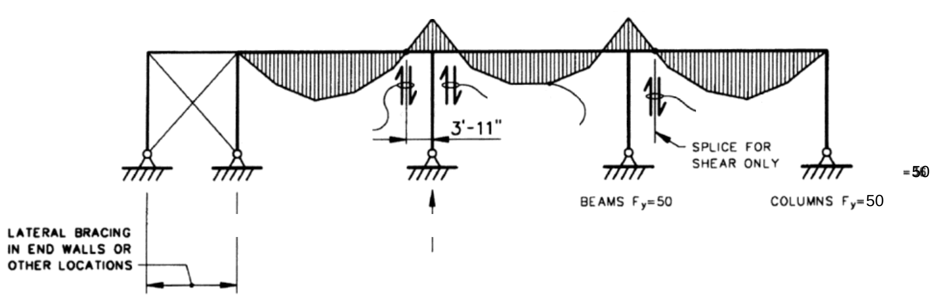


Figure 2.5(c) Type SF — cantilever-suspended span system, braced against sidesway

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

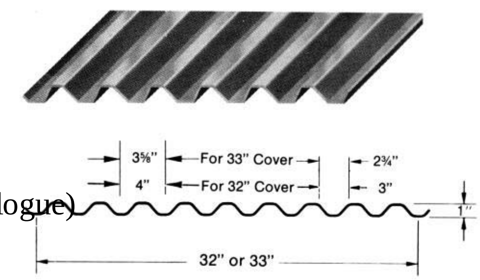
No. of Spans	Deck Type	Max. SDI Const. Span	Allowable Total (Dead + Live) Uniform Load (PSF) — Span (ft.-in.) C. to C. of Support: 2'-6	3'-0	3'-6	4'-0	4'-6	5'-0	5'-6	6'-0	6'-6	7'-0	7'-6
1	E26	2'-10	178	107	71	51	39	31	26	22	20	18	16
	E24	3'-5	249	148	97	68	51	40	32	27	24	21	19
	E22	3'-10	316	187	122	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} = (1.2×18lb/ft^2 + 1.6×30 lb/ft^2)(6 ft) + 1.2*(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	825	825	825	825	825	825														
	550	550	550	550	550	550	550														
19	771	825	825	825	825	825	825														
	494	523	523	523	523	523	523														
20	694	825	825	825	825	825	825	775	825	825	825	825	825	825							
	423	490	490	490	490	490	490	517	550	550	550	550	550	550							
21	630	759	825	825	825	825	825	702	825	825	825	825	825	825							
	364	426	460	460	460	460	460	453	520	520	520	520	520	520							
22	573	690	777	825	825	825	825	639	771	825	825	825	825	825	825	825	825	825	825	825	825
	316	370	414	438	438	438	438	393	461	490	490	490	490	490	548	548	548	548	548	548	548
23	523	630	709	774	825	825	825	583	703	793	825	825	825	825	777	825	825	825	825	825	825
	276	323	362	393	418	418	418	344	402	451	468	468	468	468	491	518	518	518	518	518	518
24	480	577	651	709	789	825	825	535	645	727	792	825	825	825	712	804	825	825	825	825	825
	242	284	318	345	382	396	396	302	353	396	430	448	448	448	431	483	495	495	495	495	495
25	441	532	600	652	727	825	825	493	594	669	729	811	825	825	657	739	805	825	825	825	825
	214	250	281	305	337	377	377	266	312	350	380	421	426	426	381	427	464	474	474	474	474
26	408	492	553	603	672	807	825	456	549	618	673	750	825	825	606	682	744	825	825	825	825
	190	222	249	271	299	354	361	236	277	310	337	373	405	405	338	379	411	454	454	454	454
27	378	454	513	558	622	747	825	421	508	573	624	694	825	825	561	633	688	768	825	825	825
	169	198	222	241	267	315	347	211	247	277	301	333	389	389	301	337	367	406	432	432	432
28	351	423	477	519	577	694	822	391	472	532	579	645	775	825	522	588	640	712	825	825	825
	151	177	199	216	239	282	331	189	221	248	269	298	353	375	270	302	328	364	413	413	413
29	327	394	444	483	538	646	766	364	439	495	540	601	723	825	486	547	597	664	798	825	825
	136	159	179	194	215	254	298	170	199	223	242	268	317	359	242	272	295	327	387	399	399
30	304	367	414	451	502	603	715	340	411	462	504	561	675	799	453	511	556	619	745	825	825
	123	144	161	175	194	229	269	153	179	201	218	242	286	336	219	245	266	295	349	385	385
31	285	343	387	421	469	564	669	318	384	433	471	525	631	748	424	478	520	580	697	825	825
	111	130	146	158	175	207	243	138	162	182	198	219	259	304	198	222	241	267	316	369	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: $w_D = 18\ \text{lb/ft}^2 \cdot 30\ \text{ft} + (8\ \text{lb/ft} \cdot 30\ \text{ft})/\ 6\ \text{ft} = 580\ \text{lb/ft}$

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

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

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

These DO NOT consider self weight of the beam.

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

$1.4D$ $\qquad w_u = 1.4(580\ \text{lb/ft}) = 812\ \text{lb/ft}$

$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$

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

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$

$w_u = 1.2(580\ \text{lb/ft}) + 1.6(900\ \text{lb/ft}) + 0.5(600\ \text{lb/ft}) = \underline{2436\ \text{lb/ft}}$

$1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$

$w_u = 1.2(580\ \text{lb/ft}) + 1.0(600\ \text{lb/ft}) + 0.5(900\ \text{lb/ft}) = 1746\ \text{lb/ft}$

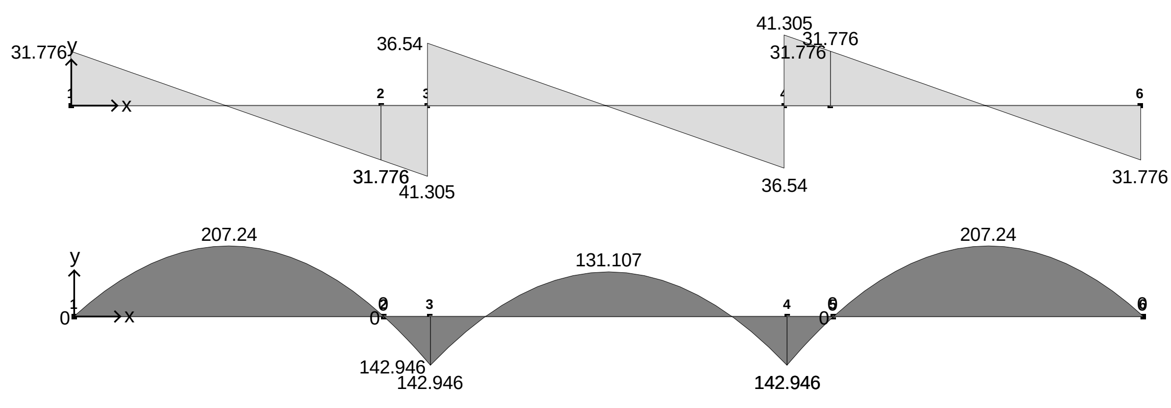
$1.2D + 1.0E + L + 0.25S$

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

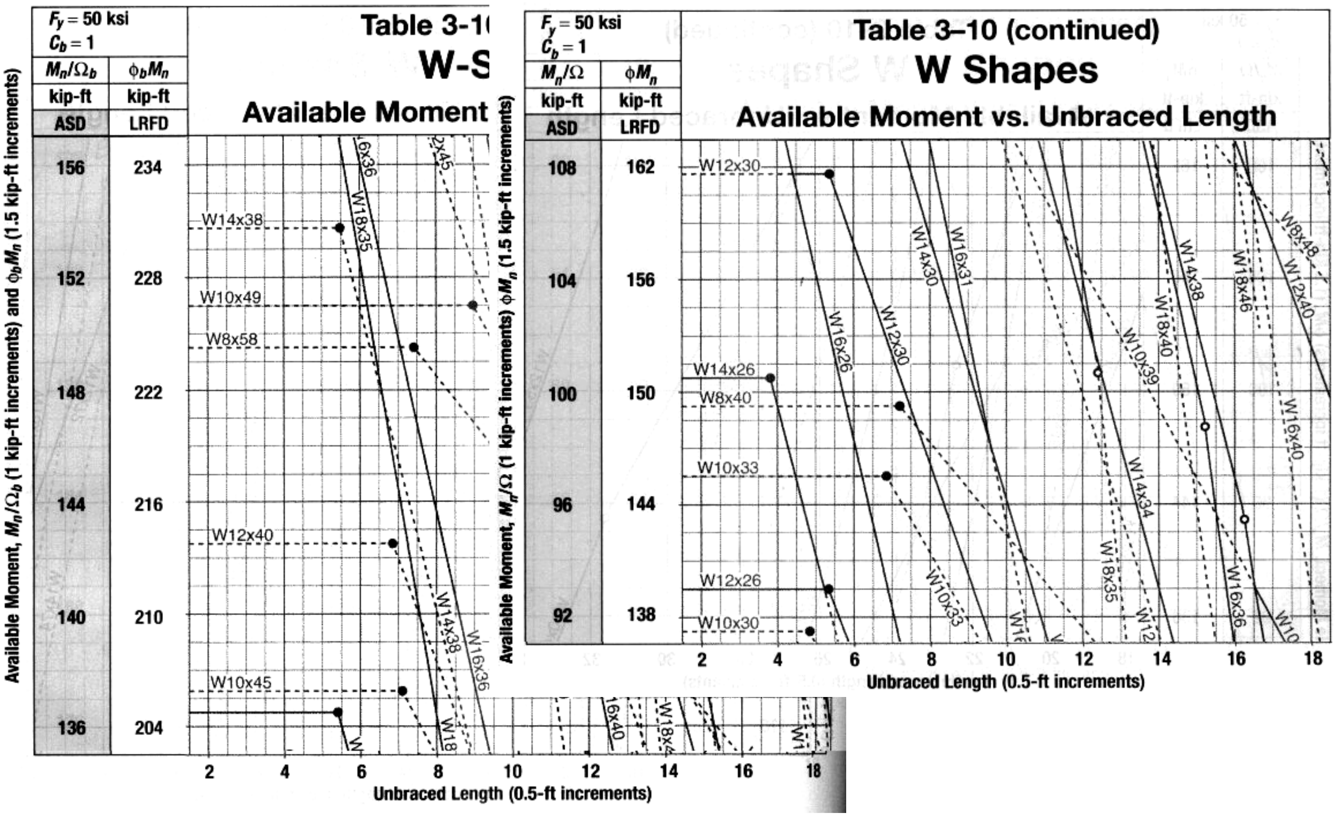
$0.9D + 1.0W$ $\qquad w_u = 0.9(580\ \text{lb/ft}) + 1.0(-600\ \text{lb/ft})$ *[uplift]* $= -78\ \text{lb/ft}$ (up)

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

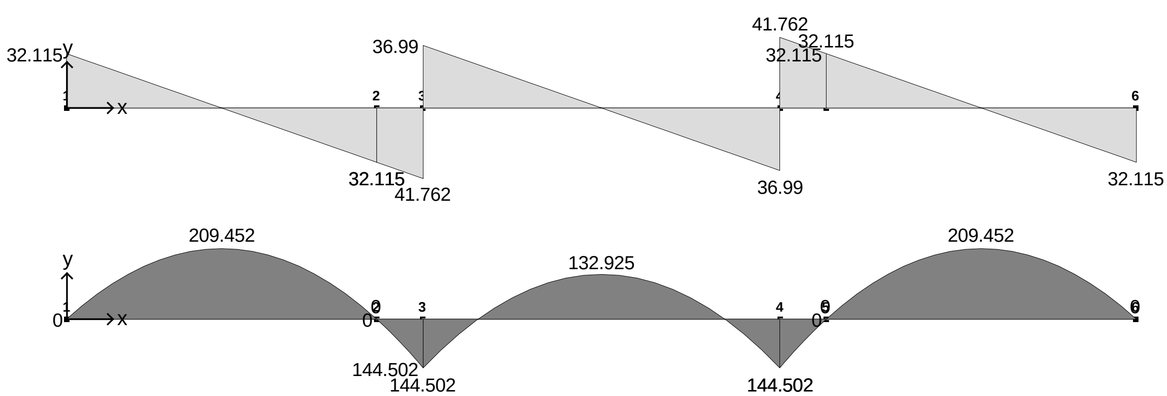
For the largest load case, the shear & bending moment diagrams are:



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 W16x26 (M_u = 147.5 k-ft) for the interior beam, the self weight can be included in the total weight. The diagrams change to:



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

Exterior V_u = 31.12 k $\le$ 1.0(0.6)(50 ksi)(17.1 in.)(0.3 in.) = 153.9 k OK

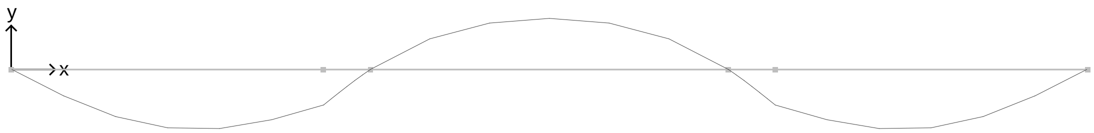
W18x35: d = 17.7 in., t_w = 0.3 in., I_x = 510 in.^{4}

Interior V_u = 41.76 k $\le$ 1.0(0.6)(50 ksi)(15.7 in.)(0.25 in.) = 117.75 k OK

W16x26: d = 15.7 in., t_w = 0.25 in., I_x = 301 in.^{4}

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

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



Maximum Δ_{total} = 3.20 in. in end spans and 1.87 in. at midspan

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

We need an I about (3.20in./1.5in.)(510 in.^{4}) = 1088 in.^{4} for the ends, and similarly, about 375.2 in^4 for the mid section.

Maximum Δ_{live} = 2.55 in. in end spans and 2.48 in. at midspan

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

We need an I about (2.55in./1.0in.)(510 in.^{4}) = 1300.5 in.^{4} for the ends, and similarly about 746.5 in^4 for the mid section.

Live load governs.

The W24x55 is the most economical out of the sections for the ends, shown with bold type in the group, with I_x = 1330 in.^{4}

The W21x44 is the most economical out of the sections for the ends, shown with bold type in the group, with I_x = 843 in.^{4}

Now, Δ_{live} = 0.7 in., which is less than allowable (by a bit).

We could probably go with the next most economical (because we have software to do the analysis) with a W21x55 and W18x40 which results in Δ_{live} = 0.96 in.!

Z_x – US (in.^{3})	I_x – US (in.^{4})	Section	(
144	**1330**	**W21X62**	
139	881	W14X82	
133	**1350**	**W24X55**	
132	1070	W18X65	
131	740	W12X87	
130	954	W16X67	
129	623	W10X100	
129	1170	W21X57	
126	**1140**	**W21X55**	
126	795	W14X74	
123	984	W18X60	
118	662	W12X79	
115	722	W14X68	
113	534	W10X88	
112	**890**	**W18X55**	
110	**984**	**W21X50**	
108	597	W12X72	
107	**959**	**W21X48**	
105	758	W16X57	
102	640	W14X61	
100	800	W18X50	
96.8	455	W10X77	
95.5	533	W12X65	
95.4	**843**	**W21X44**	
91.7	659	W16X50	
90.6	712	W18X46	
86.5	541	W14X53	
86.4	475	W12X58	
85.2	394	W10X68	
82.1	586	W16X45	
78.4	**612**	**W18X40**	
78.1	484	W14X48	
77.3	425	W12X53	
74.4	341	W10X60	

Columns:

The load in the interior columns: P_u = 79 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 = 33 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 ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length KL (ft) with respect to least radius of gyration r_y: 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				

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

$$(33k)(8.0in/2)(\frac{1ft}{12in}) = = 11.0^{\text{ k-ft.}}$$

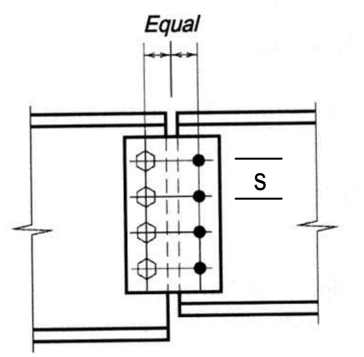
The capacity of a W8x31 with an unbraced length of 15 ft (from another beam chart) = 114 $^{\text{k-ft}}$.

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

$$\frac{33k}{230k} = 0.14 < 0.2: \qquad \frac{33k}{2(230k)} + \left(\frac{11.0^{k-ft}}{114^{k-ft}}\right) = 0.168 \le 1.0$$

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

Beam Shear Splice Connection:

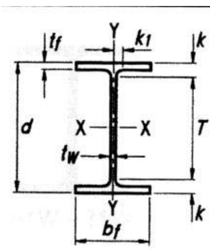


For this all-bolted single-plate shear splice, R_u = 33 k

W21x55: d = 20.8 in., t_w = 0.375 in.

W18x40: d = 17.9 in., t_w = 0.315 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 W18x40, T = 15.5 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: $: R_u \le \phi_v R_n \quad \phi = 0.75,\ R_n = F_n A_b$, where F_n = 54 ksi

$$33k \le n(0.75)(54ksi)[\frac{\pi(0.75in)^2}{4}]$$

so $n \ge 1.84$. Use 2 bolts (1@3 in. + 2@1.25 ≈ 5.5 in. < 15.5 in.)

bearing for 2 rows of bolts: depends on thickness of thinnest web (t=0.315 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

$$33k \le 2^{bolts}[0.75(1.5)(1.75in)(0.315in)(65ksi) = 80.6 \text{ k}$$
$$\le 2^{bolts}[0.75(3)(0.75in)(0.315in)(65ksi) = 69.1 \text{ k OK}$$

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.

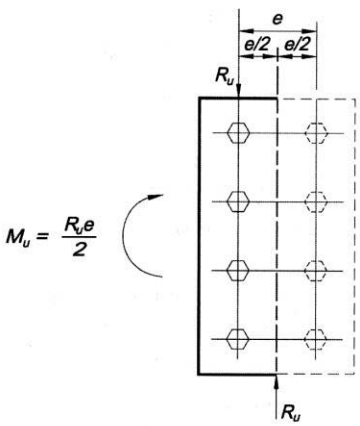


Fig. 10–22. Eccentricity in a symmetrical shear splice.

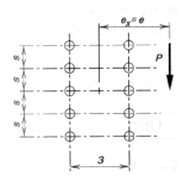
Table 7-7
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

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:

$$C_{min} = \frac{33k}{0.75(54ksi)(0.75in)^2 \pi / 4} = 1.84 \quad \text{(which we found as } n\text{)}$$

C off the table is 2.54 bolts which is more than the minimum of 1.84 (which is why we have 2). OK. (*The available strength with ϕr_n found in Table 7-1 is* 2.54x17.9k = 45.5k)

If the plate is 3/8 in. thick x 8 in. wide x 5.5 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 > 33 k OK

Check *flexure of the plate:*

<u>design moment</u>: $M_u = \frac{R_u e}{2} = \frac{33k \times 4in}{2}$ = 66.0 k-in

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

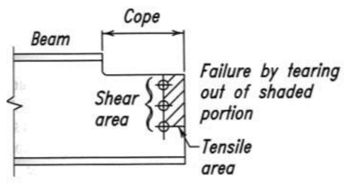
$0.9(36ksi)\left[\frac{0.375in(5.5in)^2}{6}\right]$ = 61.25 k-in > 66.0 k-in NOT OK

<u>with 6 in. tall</u>, $\phi M_n = 72.9$ k-in

<u>rupture</u> $\phi M_n = \phi F_u S_{net}$ $\phi = 0.75$

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

$0.75(58ksi)(1.74in^3)$ = 75.7 k-in > 66.0 k-in OK



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

(1.00)[0.6(36 ksi)(6 in.)(0.375 in.)] = 48.6 k > 33 k OK

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

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 > 33 k OK

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

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

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)*

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

33 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*.

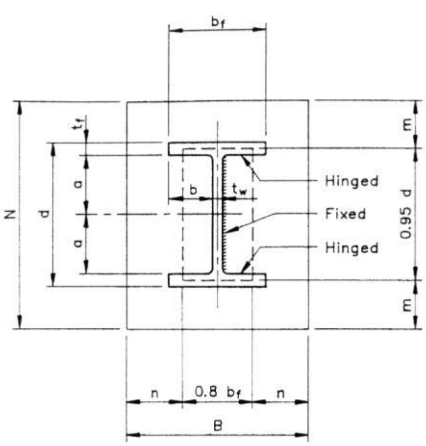


Figure 5.6. Column base plate dimensions

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 79 k.

minimum thickness: $t_{min} = l\sqrt{\dfrac{2P_u}{0.9F_y BN}}$

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:

$$X = \frac{4db_f}{(d+b_f)^2}\cdot\frac{P_u}{\phi_c P_p} = \frac{4db_f}{(d+b_f)^2}\cdot\frac{P_u}{\phi_c(0.85f'_c)BN} = \frac{4\cdot 8.0in\cdot 8.0in}{(8.0in+8.0in)^2}\cdot\frac{79k}{0.6(0.85\cdot 3ksi)12in\cdot 12in}$$

$$= 0.359$$

$$\lambda = \frac{2\sqrt{X}}{(1+\sqrt{1-X})} \le 1 \qquad = \frac{2\sqrt{0.359}}{(1+\sqrt{1-0.359})} = 0.666$$ so $\lambda n'$ =(0.666)(2.0 in.) = 1.33 in.

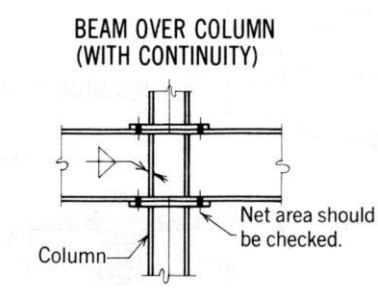
therefor: l = 2.8 in.:

$$t_p = l\sqrt{\frac{2P_u}{0.9F_y BN}} = (2.8in)\sqrt{\frac{2\cdot 79k}{0.9(36ksi)(12in)(12in)}} = 0.515 \text{ in.}$$

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.