Standard Notations

(a) Rectangular Beams.

f_s = tensile unit stress in longitudinal reinforcement.

f_c = compressive unit stress in extreme fiber of concrete.

E_s = modulus of elasticity of steel.

E_c = modulus of elasticity of concrete.

$n = \frac{E_s}{E_c}$

M = bending moment, or moment of resistance in general.

A_s = effective cross sectional area of tension reinforcement.

b = width of beam.

d = effective depth, or depth from compression surface of beam to center of tension reinforcement.

k = ratio of depth of neutral axis to effective depth, d.

j = ratio of lever arm of resisting couple to depth, d.

jd = d − z = arm of resisting couple.

p = ratio of effective area of tension reinforcement to effective area of concrete in beam $= \frac{A_s}{bd}$

z = depth from compression surface of beam to resultant of compressive stresses.

(b) T-Beams.

b = width of flange.

b' = width of stem.

t = thickness of flange.

(c) Beams Reinforced for Compression.

A' = area of compressive steel.

p' = ratio of effective area of compression reinforcement to effective area of concrete in beam $= \frac{A'}{bd}$

f'_s = compressive unit stress in longitudinal reinforcement.

C = total compressive stress in concrete.

C' = total compressive stress in steel.

d' = depth from compression surface of beam to center of compression reinforcement.

z = depth from compression surface of beam to resultant of compressive stresses.

(d) Shear, Bond and Web Reinforcement.

V = total shear.

V' = external shear on any section after deducting that carried by the concrete.

v = shearing unit stress.

u = bond stress per unit of area of surface of bar.

o = perimeter of bar

Z_o = sum of perimeters of bars in one set.

a = spacing of web reinforcement bars, measured perpendicular to their direction.

s = spacing of web reinforcement bars, measured at the neutral axis and in the direction of the longitudinal axis of the beam.

A_v = total area of web reinforcement in tension within a distance, a, of the total area of all bars bent up in any one plane.

α = angle between web bars and longitudinal bars.

f_v = tensile unit stress in web reinforcement.

Design Formulas

(a) Flexure of Rectangular Reinforced Concrete Beams and Slabs

Computations of flexure in rectangular reinforced concrete beams and slabs shall be based on the following formulas:

(1) Reinforced for tension only.

Position of neutral axis,

$$k = \sqrt{2pn + (pn)^2} - pn.$$

Arm of resisting couple,

$$j = 1 - \frac{k}{3}.$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{2M}{jkbd^2} = \frac{2pf_s}{k}.$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s jd} = \frac{M}{pjbd^2}$$

Steel ratio for balanced reinforcement,

$$p = \tfrac{1}{2}\frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)}.$$

Note: For approximate computations, the following assumptions may be made:

$j = \frac{7}{8}$

$k = \frac{3}{8}$

$$A_s = \frac{M}{\frac{7}{8} f_s d}$$

$$f_c = \frac{6M}{bd^2}$$

(2) Reinforced for both tension and compression:

Position of neutral axis,

$$k = \sqrt{2n\left(p + p'\frac{d'}{d}\right) + n^2\left(p + p'\right)^2} - n\left(p + p'\right).$$

Position of resultant compression,

$$z = \frac{\frac{1}{3}k^3 d + 2p'nd'\left(k - \frac{d'}{d}\right)}{k^2 + 2p'n\left(k - \frac{d'}{d}\right)}.$$

Arm of resisting couple,

$jd = d - z.$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{6M}{bd^2\left[3k - k^2 + \frac{6p'n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]}$$

Tensile stress in longitudinal reinforcement,

$$f_s = \frac{M}{pjbd^2} = nfc\left(\frac{1-k}{k}\right)$$

Compressive stress in longitudinal reinforcement,

$$f'_s = nfc\left(\frac{k - \frac{d'}{d}}{k}\right)$$

(b) Flexure of Reinforced Concrete T-Beams;

Computations of flexure in reinforced concrete T-beams shall be based on the following formulas:

(a) Neutral axis in the flange:
Use the formulas for rectangular beams and slabs.

(b) Neutral axis below the flange:
The following formulas neglect the compression in the stem:

Position of neutral axis,

$$kd = \frac{2ndA_s + bt^2}{2nA_s + 2bt}.$$

Position of resultant compression,

$$z = \left(\frac{3kd - 2t}{2kd - t}\right)\frac{t}{3}.$$

Arm of resisting couple,

$$jd = d - z.$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{Mkd}{bt(kd - \frac{1}{2}t)jd} = \frac{f_s}{n}\left(\frac{k}{1-k}\right).$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s jd}$$

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem: they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$kd = \sqrt{\frac{2ndA_s + (b-b')t^2}{b'} + \left(\frac{nA_s + (b-b')t}{b'}\right)^2} - \frac{nA_s + (b-b')t}{b'}$$

Position of resultant compression,

$$z = \frac{(kdt^2 - \frac{2}{3}t^3)b + [(kd-t)^2(t + \frac{1}{3}(kd-t))]b'}{t(2kd-t)b + (kd-t)^2 b'}.$$

Arm of resisting couple,

$$jd = d - z.$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{2Mkd}{[(2kd-t)bt + (kd-t)^2 b']jd}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s jd}.$$

(c) Shear, Bond and Web Reinforcement

Diagonal tension and shear in reinforced concrete beams shall be calculated by the following formulas:

Shearing unit stress,

$$v = \frac{V}{bjd}.$$

Stress in vertical web reinforcement.

$$f'_v = \frac{V's}{A_v jd}.$$

When a series of web bars or bent-up longitudinal bars is used, the web reinforcement shall be designed in accordance with the formula:

$$A_v = \frac{V_s'}{f_v jd(\sin a) + \cos(a)}$$

When the web reinforcement consists of bars bent up in a single plane so as to reinforce all sections of the beam which require it, the bent-up bars shall be designed in accordance with the formula:

$$A_v = \frac{V'}{f_v \sin a}.$$

The bond between concrete and reinforcement bars in reinforced concrete beams and slabs shall be computed by the formula:

$$u = \frac{V}{jdZ_o}.$$

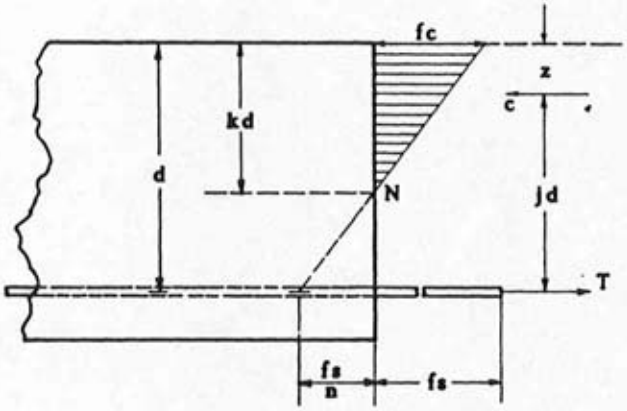
(For approximate results "j," in the above formulas, may be taken as ⅞.)

The value of "Z_0" in bundled bars should reflect only the outside surface of the bundle.

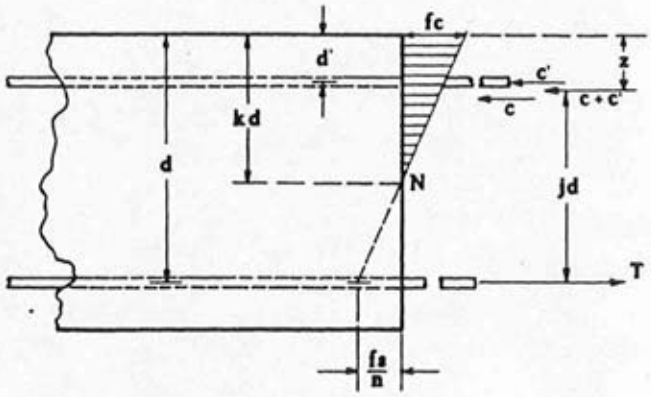
Z_0 2-bar bundle = Z_0 2 bars
Z_0 3-bar bundle = Z_0 2½ bars
Z_0 4-bar bundle = Z_0 3 bars

As regards shear and bond stress for tensile steel, the above formulas apply also to beams reinforced for compression.

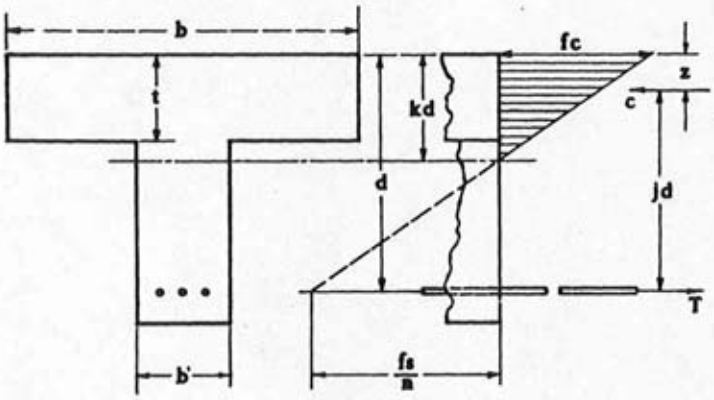
CONCRETE DESIGN



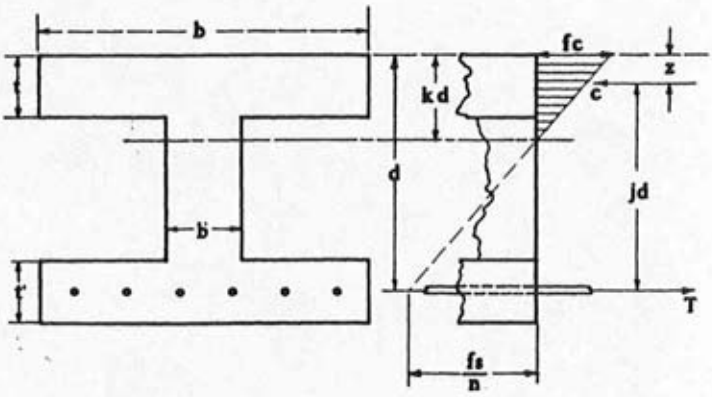
RECTANGULAR BEAM
WITHOUT COMPRESSIVE REINFORCEMENT



RECTANGULAR BEAM
WITH COMPRESSIVE REINFORCEMENT



T-BEAM



BOX GIRDER

RESISTING MOMENTS OF BEAMS AND SLABS FOR BALANCED DESIGN AND COMPRESSIVE REINFORCEMENT

NOTATION

f_c = 1,300 f_s = 24,000 psi

n = 10 for tensile reinforcement

n = 20 for compressive reinforcement

M_t = Total moment produced by external loads (ft-kips)

M = Resisting moment for balanced design (ft-kips)

M'_s = Resisting moment of compressive reinforcement (ft-kips)

A_s = Area of tensile reinforcement for balanced design (sq in)

A'_s = Area of compressive reinforcement for balanced design (sq in)

d = Effective depth of beam (inches)

d' = Embedment to center of gravity of compressive steel (inches)

b = width of girder (feet)

DESIGN CONSTANTS

j = 0.883 k = 0.351 K= 202 a=1.77

EXAMPLE I

Required:
Tensile and compressive reinforcement

Given:
A rectangular beam

d = 40" d' = 2" b = 1.5' M_t =600 ft-kips

Solution:

M for b of 1', d pf 40" = 323 ft-kips (from Table 5-11)

M for b of 1.5' = 1.5 x 323 = 484 ft-kips

M that must be provided by compressive reinforcement= 600-484=116 ft-kips

M' for d of 40", d' of 2" = 67 ft-kips per sq in (from Table 5-12,

A'_s required $= \frac{116}{67} = 1.73$ sq in

A_s for b of 1', d of 40" = 4.57 sq in (from Table 5-11)

A_s required $= \frac{600}{323} \times 4.57 = 8.50$ sq in

EXAMPLE II

Required:
Width of beam and tensile reinforcement

Given:
A rectangular beam

d = 40", d' = 2", M_t=600 ft-kips, A'_s =3.12 sq in

Solution:

M's for d of 40", d' of 2" = 67 ft-kips per sq in (from Table 5-12)

M'_s for 3.12 sq in = 3.12 x 67 = 209 ft-kips

M that must be provided by concrete=600-209=391 ft-kips

M for b of 1', d of 40"= 3.23 ft-kips (from Table 5-11)

Width of beam $= \frac{391}{323} = 1.21$ ft. say 1 ft 3 in

A_s for beam 1' wide, d of 40" = 4.57 sq in (from Table 5-11)

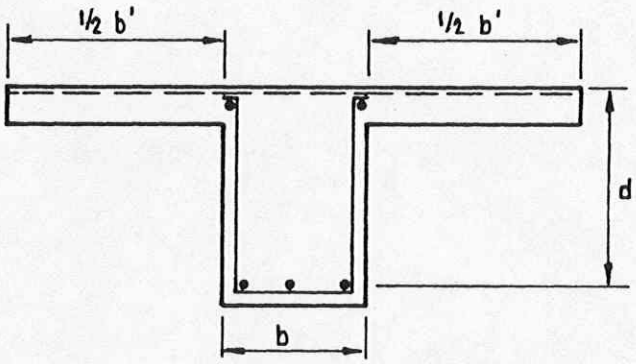
A_s required $= \frac{600}{323} \times 4.57 = 8.50$ sq in

T-BEAM VALUES
EXAMPLE III

Required:
Tensile and Compressive Reinforcement

Given: A T-Beam

d = 40" b = 12" b' = 66" t = 5.5" $M_t = 600^{ft\text{-}k}$



Solution:

M for b = 12", d = 40" = $323^{ft\text{-}k}$ (from Table 5-11)

M for b' = 12", d = 40 = $215^{ft\text{-}k}$ (from Table 5-10.1)

M for b' = 66, d = 40 = (66/12)x215 = $1183^{ft\text{-}k}$

M for T-Beam = $323^{ft\text{-}k} + 1183^{ft\text{-}k} = 1506^{ft\text{-}k}$

Since $1506^{ft\text{-}k} > 600^{ft\text{-}k}$, no compressive reinforcement required.

A_s for b = 12, d = 40 = 4.57□" (from Table 5-12)

A_s for T-Beam = (600/323) x 4.57 = 8.50□"

RESISTING MOMENTS FOR FLANGES OF BOX GIRDERS AND T-BEAMS (Foot-Kips)

f_c = 1,300 psi — Flange Width = 12"

DEPTH (inches)	FLANGE THICKNESS 5½"	6"	6½"	7"	7½"	8"	8½"	9"	9½"	10"	10½"	11"	11½"	12"
24	104	108	111	113	115	116	116							
26	118	123	127	130	133	135	136	136						
28	132	138	143	147	151	154	156	157	158	158				
30	146	153	159	165	169	173	176	178	180	181	181			
32	160	168	175	182	188	193	197	200	203	205	206	206		
34	174	183	192	199	206	212	217	222	225	228	230	232	233	233
36	187	198	208	217	225	232	238	244	248	252	255	258	260	261
38	201	213	224	234	244	252	259	266	271	276	281	284	287	289
40	215	229	241	252	262	272	280	288	295	301	306	311	314	317
42	230	244	257	270	281	292	302	310	318	325	332	337	342	346
44	244	259	274	288	300	312	323	333	342	350	357	364	370	375
46	258	275	290	305	319	332	344	355	365	375	383	391	398	404
48	272	290	307	323	338	352	366	378	389	400	409	418	426	433
50	286	305	324	341	357	373	387	400	413	425	435	445	454	463
52	300	321	340	359	376	393	409	423	437	450	462	473	483	492
54	314	336	357	377	396	413	430	446	461	475	488	500	511	522
56	328	352	374	395	415	434	452	469	485	500	514	528	540	552
58	343	367	390	413	434	454	473	492	509	525	541	555	569	582
60	357	382	407	430	453	475	495	514	533	551	567	583	598	612
62	371	398	424	449	472	495	517	537	557	576	594	611	627	642
64	385	413	440	467	491	515	538	560	581	601	620	638	655	672
66	399	429	457	485	511	536	560	583	605	627	647	666	684	702
68	414	444	474	503	530	556	582	606	630	652	674	694	714	732
70	428	460	491	521	549	577	604	629	654	678	700	722	743	762

RESISTING MOMENTS FOR FLANGES OF BOX GIRDERS AND T-BEAMS (Foot-Kips)

f_c = 1,300 psi — Flange Width = 12"

DEPTH (inches)	FLANGE THICKNESS 5½"	6"	6½"	7"	7½"	8"	8½"	9"	9½"	10"	10½"	11"	11½"	12"
72	442	475	507	539	569	598	625	652	678	703	727	750	772	793
74	456	491	524	557	588	618	647	675	703	729	754	778	801	823
76	470	506	541	575	607	639	669	699	727	754	781	806	830	854
78	485	522	558	593	627	659	691	722	751	780	807	834	860	884
80	499	537	575	611	646	680	713	745	776	805	834	862	889	915
82	513	553	591	629	665	700	735	768	800	831	861	890	918	945
84	527	568	608	647	685	721	757	791	824	857	888	918	948	976
86	542	584	625	665	704	742	778	814	849	882	915	946	977	1006
88	556	599	642	683	723	762	800	837	873	908	942	975	1006	1037
90	570	615	659	701	743	783	822	860	898	934	969	1003	1036	1068
92	584	630	675	719	762	804	844	884	922	959	996	1031	1065	1098
94	599	646	692	737	781	824	866	907	947	985	1023	1059	1095	1129
96	613	662	709	756	801	845	888	930	971	1011	1045	1087	1124	1156
98	627	677	726	774	820	866	910	953	995	1037	1077	1116	1154	1191
100	641	693	743	792	834	886	932	976	1020	1062	1104	1144	1183	1221
102	656	708	760	810	859	907	954	1000	1044	1088	1131	1172	1213	1252
104	670	724	776	828	879	928	976	1023	1069	1114	1158	1200	1242	1283
106	684	739	793	846	898	948	998	1046	1093	1140	1185	1229	1272	1314
108	698	755	810	864	917	969	1020	1069	1118	1165	1212	1257	1301	1345
110	713	770	827	882	937	990	1042	1093	1142	1191	1239	1285	1331	1375
112	727	786	844	900	956	1010	1064	1116	1167	1217	1266	1314	1361	1406
114	741	802	861	919	975	1031	1086	1139	1192	1243	1293	1342	1390	1437
116	756	817	877	937	995	1052	1108	1162	1216	1269	1320	1370	1420	1468
118	770	833	894	955	1014	1073	1130	1186	1241	1294	1347	1399	1449	1499
120	784	848	911	973	1034	1093	1152	1209	1265	1320	1374	1427	1479	1530

TABLE OF UNIT VALUES

EFFECTIVE DEPTH d (inches)	M (ft - kips per ft.)	A_s per ft.	Values of M'_s (ft. - kips per sq. in.) for given values of d'					
			1½"	2"	2½"	3"	4"	5"
6	7	0.69	2.7					
7	10	0.80	4.4	1.9				
8	13	0.91	6.2	3.6				
9	16	1.03	8.1	5.3	2.8			
10	20	1.14	10.0	7.1	4.4	2.1		
11	24	1.26	12.0	8.9	6.2	3.7		
12	29	1.37	13.9	10.8	8.0	5.3		
13	34	1.48	15.9	12.7	9.8	7.1	2.3	
14	40	1.60	17.9	14.7	11.6	8.8	3.8	
15	45	1.71	19.9	16.6	13.5	10.6	5.5	
16	52	1.83	21.9	18.6	15.4	12.5	7.1	2.5
17	58	1.94	23.9	20.5	17.3	14.3	8.8	4.0
18	65	2.06	25.9	22.5	19.3	16.2	10.6	5.6
19	73	2.17	27.9	24.5	21.2	18.1	12.4	7.2
20	81	2.28	29.9	26.5	23.2	20.0	14.2	8.9
21	89	2.40	32.0	28.5	25.2	22.0	16.0	10.6
22	98	2.51	34.0	30.5	27.2	23.9	17.9	12.4
23	107	2.63	36.0	32.5	29.1	25.9	19.7	14.1
24	116	2.74	38.1	34.5	31.1	27.8	21.6	15.9
26	136	2.97	42.1	38.6	35.1	31.8	25.5	19.6
28	158	3.20	46.2	42.6	39.1	35.8	29.3	23.3
30	181	3.43	50.3	46.7	43.2	39.8	33.2	27.0
32	206	3.65	54.4	50.8	47.2	43.8	37.1	30.9
34	233	3.88	58.5	55.0	51.3	47.8	41.1	34.7
36	261	4.11	62.6	59.9	55.3	52.0	45.0	38.6
38	291	4.34	67.0	63.0	59.3	55.8	49.0	42.5
40	323	4.57	70.8	67.1	63.5	59.9	53.0	46.4
42	356	4.80	74.9	71.2	67.5	63.9	57.0	50.3
44	390	5.02	78.9	75.3	71.6	68.0	61.0	54.3
46	427	5.25	83.1	79.4	75.7	72.1	65.0	58.3
48	465	5.48	87.2	83.4	79.8	76.1	69.1	62.3
50	504	5.71	91.3	87.5	83.9	80.2	73.1	66.3
52	545	5.94	95.4	91.6	87.9	84.3	77.2	70.3
54	588	6.17	99.5	95.7	92.0	88.4	81.2	74.3

TABLE OF UNIT VALUES

EFFECTIVE DEPTH d (inches)	M (ft.-kips per ft.)	A_s per ft.	Values of M's (ft.-kips per sq. in.) for given values of d'					
			1½"	2"	2½"	3"	4"	5"
56	632	6.39	103.5	99.8	96.1	92.5	85.3	78.3
58	678	6.62	107.3	103.9	100.2	96.5	89.3	82.3
60	726	6.85	111.1	108.1	104.3	100.6	93.4	86.4
62	775	7.08	114.9	112.2	108.4	104.7	97.5	90.4
64	826	7.31	118.7	116.3	112.5	108.8	101.5	94.4
66	878	7.54	122.5	120.4	116.6	112.9	105.6	98.5
68	932	7.76	126.2	124.5	120.7	117.0	109.7	102.5
70	988	7.99	130.0	128.6	124.8	121.1	113.8	106.6
72	1045	8.22	133.8	132.7	128.9	125.2	117.8	110.6
74	1104	8.45	137.6	136.8	133.0	129.3	121.9	114.7
76	1165	8.68	141.4	140.6	137.1	133.4	126.0	118.8
78	1227	8.91	145.2	144.4	141.2	137.5	130.1	122.8
80	1290	9.14	148.9	148.1	145.3	141.6	134.2	126.9
82	1356	9.36	152.7	151.9	149.4	145.7	138.3	131.0
84	1423	9.59	156.5	155.7	153.5	149.8	142.3	135.1
86	1491	9.82	160.3	159.5	157.6	153.9	146.4	139.1
88	1561	10.05	164.1	163.3	161.8	158.0	150.5	143.2
90	1633	10.28	167.9	167.1	165.9	162.1	154.6	147.3
92	1707	10.51	171.7	170.8	170.0	166.2	158.7	151.4
94	1782	10.73	175.4	174.6	173.8	170.3	162.8	155.5
96	1858	10.96	179.2	178.4	177.6	174.4	166.9	159.4
98	1936	11.19	183.0	182.2	181.4	178.5	171.0	163.6
100	2016	11.42	186.8	186.0	185.2	182.6	175.1	167.7
102	2098	11.65	190.6	189.8	189.0	186.7	179.2	171.8
104	2181	11.88	194.3	193.5	192.8	190.8	183.3	175.9
106	2266	12.10	198.1	197.3	196.5	194.9	187.4	180.0
108	2352	12.33	201.9	201.1	200.3	199.0	191.5	184.1
110	2440	12.56	205.7	204.9	204.1	203.1	195.6	188.2
112	2529	12.79	209.5	208.7	207.9	207.1	199.7	192.3
114	2620	13.02	213.3	212.5	211.7	210.9	203.8	196.4
116	2713	13.25	217.0	216.3	215.5	214.7	207.9	200.4
118	2808	13.47	220.8	220.0	219.2	218.4	212.0	204.5
120	2903	13.70	224.6	223.8	223.0	222.2	216.1	208.6

Box Girders-Moments of Inertia and Weight Tables

The following tables were prepared to aid the designer in estimating the moments of inertia and weights of box girders.

The tables are based on an interior girder using thicknesses of slabs as shown and including the fillets. Intermediate values may be obtained by straight line interpolation.

Girder flares and diaphragms have been ignored in the preparation of this information.

If the moment of inertia of the entire superstructure is required, it can be obtained by assuming the exterior girder having an "I" of approximately 0.72 to 0.78 times the "I" of an interior girder.

BOX GIRDER – MOMENTS OF INERTIA (FT 4)
INTERIOR GIRDER, STEM = 8", FILLETS = 4"

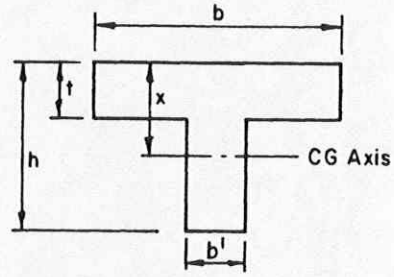
SLAB (IN)	TOP	6	6	6	6 1/8	6 1/4	6 1/4	6 1/4	6 3/8	6 1/2	6 5/8	6 3/4	7
	BOT	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 3/4	5 7/8
c-c GIRS (FT-IN)		5-9	6-0	6-3	6-6	6-9	7-0	7-3	7-6	7-9	8-0	8-3	8-6
DEPTH (FT-IN)	3-6	13.9	14.4	15.0	15.6	16.2	16.8	17.4	18.0	18.7	19.3	20.3	21.2
	3-9	16.4	17.0	17.7	18.4	19.2	19.8	20.5	21.3	22.0	22.8	23.9	25.0
	4-0	19.2	19.9	20.7	21.5	22.4	23.2	23.9	24.8	25.7	26.6	27.9	29.2
	4-3	22.2	23.0	23.9	24.9	25.9	26.8	27.6	28.7	29.7	30.7	32.3	33.8
	4-6	25.4	26.4	27.4	28.5	29.7	30.7	31.6	32.8	34.0	35.2	36.9	38.7
	4-9	28.9	30.0	31.1	32.4	33.7	34.8	35.9	37.3	38.6	40.0	42.0	43.9
	5-0	32.7	33.9	35.2	36.6	38.1	39.3	40.6	42.1	43.6	45.1	47.3	49.5
	5-3	36.7	38.1	39.5	41.1	42.7	44.1	45.5	47.2	48.8	50.5	53.1	55.5
	5-6	41.0	42.6	44.1	45.9	47.7	49.2	50.7	52.6	54.5	56.3	59.2	61.9
	5-9	45.6	47.3	48.9	50.9	52.9	54.6	56.3	58.4	60.4	62.5	65.6	68.7
	6-0	50.5	52.3	54.1	56.3	58.5	60.4	62.2	64.5	66.7	69.0	72.4	75.8
	6-3	55.6	57.6	59.6	62.0	64.4	66.4	68.4	70.9	73.4	75.9	79.7	83.4
	6-6	61.0	63.2	65.4	68.0	70.6	72.8	75.0	77.7	80.4	83.1	87.3	91.3
	6-9	66.7	69.1	71.5	74.3	77.1	79.5	81.9	84.8	87.8	90.8	95.2	99.6
	7-0	72.8	75.3	77.9	80.9	84.0	86.6	89.2	92.4	95.5	98.8	103.6	108.4
	7-3	79.1	81.8	84.6	87.9	91.2	94.0	96.8	100.2	103.7	107.1	112.4	117.6
	7-6	85.7	88.7	91.6	95.2	98.8	101.8	104.8	108.5	112.2	115.9	121.6	127.1
	7-9	92.7	95.8	99.0	102.8	106.7	109.9	113.1	117.1	121.1	125.1	131.2	137.2
	8-0	99.9	103.3	106.7	110.8	114.9	118.4	121.8	126.1	130.3	134.6	141.2	147.6
	8-3	107.5	111.2	114.8	119.1	123.5	127.2	130.9	135.4	140.0	144.6	151.6	158.5
	8-6	115.5	119.3	123.2	127.8	132.5	136.5	140.4	145.2	150.1	155.0	162.4	169.8
	8-9	123.7	127.8	131.9	136.9	141.9	146.1	150.2	155.4	160.6	165.8	173.7	181.6
	9-0	132.3	136.7	141.0	146.3	151.6	156.0	160.5	165.9	171.4	177.0	185.4	193.8
	9-3	141.3	145.9	150.5	156.1	161.7	166.4	171.1	176.9	182.7	188.6	197.6	206.5
	9-6	150.6	155.5	160.3	166.2	172.2	177.2	182.1	188.3	194.5	200.7	210.2	219.6

BOX GIRDER – GIRDER WEIGHT (K/ft)
INTERIOR GIRDER, STEM = 8", FILLETS = 4"

SLAB (IN)	TOP	6	6	6	6 1/8	6 1/4	6 1/4	6 1/4	6 3/8	6 1/2	6 5/8	6 3/4	7
	BOT	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 1/2	5 3/4	5 7/8
c-c GIRS (FT-IN)		5-9	6-0	6-3	6-6	6-9	7-0	7-3	7-6	7-9	8-0	8-3	8-6
DEPTH (FT-IN)	3-6	1.11	1.15	1.19	1.23	1.28	1.31	1.35	1.40	1.45	1.49	1.57	1.64
	3-9	1.14	1.17	1.21	1.26	1.30	1.34	1.38	1.42	1.47	1.52	1.59	1.67
	4-0	1.16	1.20	1.24	1.28	1.33	1.36	1.40	1.45	1.50	1.54	1.62	1.69
	4-3	1.19	1.22	1.26	1.31	1.35	1.39	1.43	1.47	1.52	1.57	1.64	1.72
	4-6	1.21	1.25	1.29	1.33	1.38	1.41	1.45	1.50	1.55	1.59	1.67	1.74
	4-9	1.24	1.27	1.31	1.36	1.40	1.44	1.48	1.52	1.57	1.62	1.69	1.77
	5-0	1.26	1.30	1.34	1.38	1.43	1.46	1.50	1.55	1.60	1.64	1.72	1.79
	5-3	1.29	1.32	1.36	1.41	1.45	1.49	1.53	1.57	1.62	1.67	1.74	1.82
	5-6	1.31	1.35	1.39	1.43	1.48	1.51	1.55	1.60	1.65	1.69	1.77	1.84
	5-9	1.34	1.37	1.41	1.46	1.50	1.54	1.58	1.62	1.67	1.72	1.79	1.87
	6-0	1.36	1.40	1.44	1.48	1.53	1.56	1.60	1.65	1.70	1.74	1.82	1.89
	6-3	1.39	1.42	1.46	1.51	1.55	1.59	1.63	1.67	1.72	1.77	1.84	1.92
	6-6	1.41	1.45	1.49	1.53	1.58	1.61	1.65	1.70	1.75	1.79	1.87	1.94
	6-9	1.44	1.47	1.51	1.56	1.60	1.64	1.68	1.72	1.77	1.82	1.89	1.97
	7-0	1.46	1.50	1.54	1.58	1.63	1.66	1.70	1.75	1.80	1.84	1.92	1.99
	7-3	1.49	1.52	1.56	1.61	1.65	1.69	1.73	1.77	1.82	1.87	1.94	2.02
	7-6	1.51	1.55	1.59	1.63	1.68	1.71	1.75	1.80	1.85	1.89	1.97	2.04
	7-9	1.54	1.58	1.61	1.66	1.70	1.74	1.78	1.82	1.87	1.92	1.99	2.07
	8-0	1.56	1.60	1.64	1.68	1.73	1.76	1.80	1.85	1.90	1.94	2.02	2.09
	8-3	1.59	1.63	1.66	1.71	1.75	1.79	1.83	1.87	1.92	1.97	2.04	2.12
	8-6	1.61	1.65	1.69	1.73	1.78	1.81	1.85	1.90	1.95	1.99	2.07	2.14
	8-9	1.64	1.68	1.71	1.76	1.80	1.84	1.88	1.92	1.97	2.02	2.09	2.17
	9-0	1.66	1.70	1.74	1.78	1.83	1.86	1.90	1.95	2.00	2.04	2.12	2.19
	9-3	1.69	1.73	1.76	1.81	1.85	1.89	1.93	1.97	2.02	2.07	2.14	2.22
	9-6	1.71	1.75	1.79	1.83	1.88	1.91	1.95	2.00	2.05	2.09	2.17	2.24

Moments of Inertia For T-Beams

Given below are three tables showing moments of inertia for reinforced concrete T-beams to be used in determining the elastic qualities of members for structural design purposes.



I = Moment of inertia about the C.G. axis

$I = 1/12\ bh^3\ C$

t/h	VALUES OF C											t/h
	b'/b											
	.10	.12	.14	.16	.18	.20	.22	.24	.26	.28	.30	
.05	.184	.207	.230	.250	.271	.290	.310	.328	.347	.366	.385	.05
.06	.193	.217	.240	.262	.282	.303	.322	.342	.360	.379	.398	.06
.07	.201	.226	.249	.272	.294	.314	.334	.353	.372	.391	.410	.07
.08	.207	.233	.258	.281	.302	.324	.344	.363	.383	.402	.420	.08
.09	.212	.240	.264	.288	.310	.332	.354	.373	.392	.412	.430	.09
.10	.216	.245	.270	.295	.318	.340	.361	.380	.401	.420	.439	.10
.11	.219	.249	.275	.300	.324	.346	.368	.389	.408	.428	.447	.11
.12	.222	.252	.280	.305	.329	.352	.374	.395	.416	.434	.454	.12
.13	.224	.255	.283	.309	.333	.357	.379	.401	.421	.441	.462	.13
.14	.226	.257	.286	.313	.337	.361	.384	.406	.427	.447	.466	.14
.15	.228	.259	.289	.316	.343	.365	.388	.410	.432	.452	.471	.15
.16	.229	.260	.290	.318	.344	.368	.392	.414	.436	.456	.476	.16
.17	.229	.262	.292	.320	.347	.371	.395	.418	.440	.460	.480	.17
.18	.230	.262	.293	.321	.348	.374	.398	.420	.442	.463	.483	.18
.19	.231	.264	.294	.323	.350	.375	.400	.423	.444	.466	.486	.19
.20	.231	.264	.295	.324	.351	.377	.401	.425	.447	.468	.489	.20
.21	.231	.264	.296	.325	.353	.378	.403	.427	.449	.470	.491	.21
.22	.231	.264	.296	.325	.354	.379	.404	.428	.451	.472	.493	.22
.23	.231	.265	.296	.326	.354	.380	.405	.429	.452	.474	.495	.23
.24	.231	.265	.296	.326	.354	.381	.406	.430	.453	.475	.496	.24
.25	.231	.265	.296	.326	.354	.381	.406	.430	.453	.476	.497	.25
.26	.231	.265	.296	.326	.355	.381	.407	.431	.454	.477	.498	.26
.27	.231	.265	.296	.326	.355	.382	.407	.431	.454	.478	.499	.27
.28	.231	.265	.296	.326	.355	.382	.408	.432	.455	.478	.499	.28
.29	.231	.265	.296	.326	.355	.382	.408	.432	.455	.479	.500	.29
.30	.232	.265	.296	.326	.355	.382	.408	.432	.456	.479	.500	.30

T-BEAM MOMENT OF INERTIA

t/h	VALUES OF C FOR I = 1/12 bh³ (C)											t/h
	b'/b											
	.30	.32	.34	.36	.38	.40	.42	.44	.46	.48	.50	
.05	.385	.403	.422	.440	.457	.475	.494	.511	.529	.547	.563	.05
.06	.398	.415	.434	.452	.470	.487	.505	.524	.540	.558	.574	.06
.07	.410	.428	.446	.463	.481	.499	.518	.534	.550	.568	.585	.07
.08	.420	.438	.456	.473	.491	.509	.526	.544	.561	.577	.594	.08
.09	.430	.448	.466	.484	.500	.518	.536	.553	.570	.586	.603	.09
.10	.439	.457	.475	.492	.508	.528	.544	.561	.578	.595	.611	.10
.11	.447	.466	.483	.500	.518	.535	.552	.569	.585	.602	.618	.11
.12	.454	.473	.490	.508	.526	.543	.560	.576	.592	.609	.625	.12
.13	.462	.479	.498	.515	.532	.549	.566	.583	.599	.615	.631	.13
.14	.466	.484	.502	.521	.539	.555	.572	.589	.605	.620	.637	.14
.15	.471	.489	.508	.526	.544	.562	.578	.595	.612	.626	.643	.15
.16	.476	.495	.513	.532	.548	.566	.583	.602	.616	.632	.648	.16
.17	.480	.499	.518	.536	.554	.570	.587	.604	.620	.637	.653	.17
.18	.483	.502	.521	.540	.557	.575	.592	.608	.624	.642	.656	.18
.19	.486	.506	.525	.544	.561	.579	.596	.613	.629	.645	.660	.19
.20	.489	.509	.528	.547	.565	.582	.600	.616	.632	.649	.664	.20
.21	.491	.511	.531	.550	.568	.585	.603	.619	.635	.652	.667	.21
.22	.493	.513	.533	.552	.570	.588	.605	.622	.638	.654	.670	.22
.23	.495	.515	.535	.554	.572	.590	.607	.624	.641	.657	.672	.23
.24	.496	.517	.536	.556	.574	.592	.609	.626	.643	.659	.675	.24
.25	.497	.518	.538	.557	.576	.594	.611	.628	.645	.661	.677	.25
.26	.498	.519	.539	.558	.577	.595	.612	.631	.646	.662	.678	.26
.27	.499	.520	.540	.559	.579	.596	.614	.632	.647	.664	.679	.27
.28	.499	.520	.541	.560	.580	.597	.615	.632	.648	.665	.680	.28
.29	.500	.521	.542	.561	.581	.598	.616	.633	.650	.666	.682	.29
.30	.500	.521	.543	.562	.581	.599	.616	.634	.651	.667	.683	.30

T-BEAM MOMENT OF INERTIA

t/h	VALUES OF C FOR I = 1/12 bh³ (C)										t/h
	b'/b										
	.10	.20	.30	.40	.50	.60	.70	.80	.90	1.00	
.05	.184	.290	.385	.475	.563	.653	.740	.826	.913	1.00	.05
.06	.193	.303	.398	.487	.574	.661	.748	.831	.914	1.00	.06
.07	.201	.314	.410	.499	.585	.670	.755	.836	.914	1.00	.07
.08	.207	.324	.420	.509	.594	.677	.760	.840	.918	1.00	.08
.09	.212	.332	.430	.518	.603	.684	.765	.843	.921	1.00	.09
.10	.216	.340	.439	.528	.611	.691	.770	.847	.924	1.00	.10
.11	.219	.346	.447	.535	.618	.698	.775	.850	.926	1.00	.11
.12	.222	.352	.454	.543	.625	.704	.780	.852	.928	1.00	.12
.13	.224	.357	.462	.549	.631	.709	.784	.856	.929	1.00	.13
.14	.226	.361	.466	.555	.637	.715	.789	.859	.931	1.00	.14
.15	.228	.365	.471	.562	.643	.720	.793	.863	.932	1.00	.15
.16	.229	.368	.476	.566	.648	.724	.797	.866	.934	1.00	.16
.17	.229	.371	.480	.570	.653	.728	.800	.868	.935	1.00	.17
.18	.230	.374	.483	.575	.656	.732	.803	.870	.936	1.00	.18
.19	.231	.375	.486	.579	.660	.736	.806	.873	.937	1.00	.19
.20	.231	.377	.489	.582	.664	.739	.809	.875	.938	1.00	.20
.21	.231	.378	.491	.585	.667	.742	.811	.877	.939	1.00	.21
.22	.231	.379	.493	.588	.670	.744	.813	.878	.940	1.00	.22
.23	.231	.380	.495	.590	.672	.747	.815	.880	.941	1.00	.23
.24	.231	.381	.496	.592	.675	.749	.818	.881	.942	1.00	.24
.25	.231	.381	.497	.594	.677	.751	.820	.882	.943	1.00	.25
.26	.231	.381	.498	.595	.678	.753	.821	.883	.944	1.00	.26
.27	.231	.382	.499	.596	.679	.754	.822	.884	.944	1.00	.27
.28	.231	.382	.499	.597	.680	.756	.823	.886	.945	1.00	.28
.29	.231	.382	.500	.598	.682	.757	.824	.887	.945	1.00	.29
.30	.232	.382	.500	.599	.683	.758	.826	.888	.946	1.00	.30
.31	.232	.382	.501	.599	.684	.759	.826	.888	.946	1.00	.31
.32	.233	.382	.501	.599	.684	.759	.827	.889	.946	1.00	.32
.33	.234	.382	.501	.600	.685	.760	.827	.889	.946	1.00	.33
.34	.234	.382	.501	.600	.685	.760	.828	.890	.947	1.00	.34
.35	.235	.382	.501	.600	.686	.761	.829	.890	.947	1.00	.35
.36	.236	.383	.501	.600	.686	.761	.829	.890	.947	1.00	.36
.37	.238	.383	.501	.600	.686	.761	.829	.891	.947	1.00	.37
.38	.239	.383	.502	.600	.686	.762	.829	.891	.947	1.00	.38
.39	.241	.384	.502	.600	.686	.762	.830	.891	.948	1.00	.39
.40	.242	.384	.503	.600	.686	.762	.830	.891	.948	1.00	.40
.41	.245	.385	.503	.601	.686	.762	.830	.891	.948	1.00	.41
.42	.247	.386	.503	.601	.686	.762	.830	.891	.948	1.00	.42
.43	.250	.387	.503	.601	.686	.762	.830	.892	.948	1.00	.43
.44	.252	.388	.503	.601	.686	.762	.830	.892	.948	1.00	.44
.45	.255	.390	.503	.601	.686	.762	.830	.892	.948	1.00	.45
.46	.259	.392	.504	.602	.687	.762	.830	.892	.948	1.00	.46
.47	.262	.394	.505	.602	.687	.762	.830	.892	.948	1.00	.47
.48	.266	.396	.507	.603	.687	.762	.830	.892	.948	1.00	.48
.49	.270	.398	.508	.603	.687	.762	.830	.892	.948	1.00	.49
.50	.274	.400	.509	.604	.688	.762	.830	.892	.948	1.00	.50

T-Beams - Moments of Inertia and Weight Tables

The following tables were prepared to aid the designer in estimating the moments of inertia and weights of T-Beams.

The tables are based on an interior girder using thicknesses of slabs as shown and including the fillets. Intermediate values may be obtained by straight line interpolation.

Girder flares and diaphragms have been ignored in the preparation of this information.

T – BEAM MOMENTS OF INERTIA (ft^4)
Interior Girder Stem = 11" Fillets = 4"

SLAB (in)		6	6	6	6	6-1/8	6-1/4	6-1/4	6-1/4	6-3/8	6-1/2	6-5/8	6-3/4	7	7-1/8	7-1/4	7-1/4
c-c GIRS (ft-in)		5-9	6-0	6-3	6-6	6-9	7-0	7-3	7-6	7-9	8-0	8-3	8-6	8-9	9-0	9-3	9-6
DEPTH (ft-in)	1-6	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
	1-9	0.8	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	1.0	1.0	1.0	1.0
	2-0	1.2	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4
	2-3	1.8	1.8	1.8	1.8	1.9	1.9	1.9	1.9	1.9	1.9	2.0	2.0	2.0	2.0	2.0	2.0
	2-6	2.4	2.5	2.5	2.5	2.5	2.6	2.6	2.6	2.6	2.7	2.7	2.7	2.7	2.7	2.8	2.8
	2-9	3.2	3.2	3.3	3.3	3.4	3.4	3.4	3.5	3.5	3.5	3.6	3.6	3.6	3.7	3.7	3.7
	3-0	4.1	4.2	4.2	4.3	4.3	4.4	4.4	4.5	4.5	4.6	4.6	4.7	4.7	4.7	4.8	4.8
	3-3	5.2	5.3	5.4	5.4	5.5	5.6	5.6	5.7	5.7	5.8	5.9	5.9	6.0	6.0	6.1	6.1
	3-6	6.5	6.5	6.6	6.7	6.8	6.9	7.0	7.0	7.1	7.2	7.3	7.3	7.4	7.5	7.5	7.6
	3-9	7.9	8.0	8.1	8.2	8.3	8.4	8.5	8.6	8.7	8.8	8.9	9.0	9.1	9.2	9.2	9.3
	4-0	9.5	9.6	9.7	9.8	10.0	10.1	10.3	10.4	10.5	10.6	10.7	10.8	11.0	11.1	11.2	11.2
	4-3	11.2	11.4	11.5	11.7	11.9	12.1	12.2	12.3	12.5	12.6	12.8	12.9	13.1	13.2	13.3	13.4
	4-6	13.2	13.4	13.6	13.8	14.0	14.2	14.4	14.5	14.7	14.9	15.1	15.2	15.4	15.6	15.7	15.9
	4-9	15.4	15.6	15.8	16.0	16.3	16.5	16.7	16.9	17.1	17.4	17.6	17.8	18.0	18.2	18.4	18.5
	5-0	17.7	18.0	18.2	18.5	18.8	19.1	19.3	19.6	19.8	20.1	20.4	20.6	20.9	21.1	21.3	21.5
	5-3	20.3	20.6	20.9	21.2	21.6	21.9	22.2	22.4	22.8	23.1	23.4	23.7	24.0	24.3	24.5	24.7
	5-6	23.2	23.5	23.8	24.2	24.6	25.0	25.3	25.6	25.9	26.3	26.7	27.0	27.4	27.7	28.0	28.3
	5-9	26.2	26.6	27.0	27.4	27.8	28.3	28.6	29.0	29.4	29.8	30.2	30.6	31.1	31.5	31.8	32.1
	6-0	29.5	30.0	30.4	30.8	31.3	31.9	32.2	32.6	33.1	33.6	34.1	34.5	35.1	35.5	35.9	36.2
	6-3	33.1	33.5	34.0	34.5	35.1	35.7	36.1	36.6	37.1	37.7	38.2	38.7	39.3	39.8	40.3	40.6
	6-6	36.8	37.4	37.9	38.5	39.1	39.8	40.3	40.8	41.4	42.0	42.6	43.2	43.9	44.5	45.0	45.4

T – BEAM GIRDER WEIGHTS (K/ft.)
Interior Girder Stem = 11" Fillets = 4"

SLAB (in)		6	6	6	6	6-1/8	6-1/4	6-1/4	6-1/4	6-3/8	6-1/2	6-5/8	6-3/4	7	7-1/8	7-1/4	7-1/4
c-c GIRS (ft-in)		5-9	6-0	6-3	6-6	6-9	7-0	7-3	7-6	7-9	8-0	8-3	8-6	8-9	9-0	9-3	9-6
DEPTH (ft-in)	1-6	0.59	0.60	0.62	0.64	0.67	0.70	0.72	0.74	0.77	0.80	0.83	0.86	0.91	0.94	0.98	1.00
	1-9	0.62	0.64	0.66	0.68	0.70	0.73	0.75	0.77	0.80	0.83	0.86	0.90	0.94	0.98	1.01	1.04
	2-0	0.65	0.67	0.69	0.71	0.74	0.77	0.79	0.81	0.84	0.87	0.90	0.93	0.98	1.01	1.05	1.07
	2-3	0.69	0.71	0.73	0.74	0.77	0.80	0.82	0.84	0.87	0.90	0.93	0.97	1.01	1.05	1.08	1.10
	2-6	0.72	0.74	0.76	0.78	0.81	0.84	0.86	0.87	0.91	0.94	0.97	1.00	1.05	1.08	1.12	1.14
	2-9	0.76	0.78	0.79	0.81	0.84	0.87	0.89	0.91	0.94	0.97	1.00	1.03	1.08	1.11	1.15	1.17
	3-0	0.79	0.81	0.83	0.85	0.88	0.90	0.92	0.94	0.97	1.00	1.04	1.07	1.11	1.15	1.18	1.21
	3-3	0.83	0.84	0.86	0.88	0.91	0.94	0.96	0.98	1.01	1.04	1.07	1.10	1.15	1.18	1.22	1.24
	3-6	0.86	0.88	0.90	0.92	0.94	0.97	0.99	1.01	1.04	1.07	1.11	1.14	1.18	1.22	1.25	1.28
	3-9	0.89	0.91	0.93	0.95	0.98	1.01	1.03	1.05	1.08	1.11	1.14	1.17	1.22	1.25	1.29	1.31
	4-0	0.93	0.95	0.97	0.99	1.01	1.04	1.06	1.08	1.11	1.14	1.17	1.21	1.25	1.29	1.32	1.34
	4-3	0.96	0.98	1.00	1.02	1.05	1.08	1.10	1.12	1.15	1.18	1.21	1.24	1.29	1.32	1.36	1.38
	4-6	1.00	1.02	1.04	1.05	1.08	1.11	1.13	1.15	1.18	1.21	1.24	1.28	1.32	1.36	1.39	1.41
	4-9	1.03	1.05	1.07	1.09	1.12	1.15	1.16	1.18	1.21	1.25	1.28	1.31	1.36	1.39	1.43	1.45
	5-0	1.07	1.09	1.10	1.12	1.15	1.18	1.20	1.22	1.25	1.28	1.31	1.34	1.39	1.42	1.46	1.48
	5-3	1.10	1.12	1.14	1.16	1.19	1.21	1.23	1.25	1.28	1.31	1.35	1.38	1.42	1.46	1.49	1.52
	5-6	1.14	1.15	1.17	1.19	1.22	1.25	1.27	1.29	1.32	1.35	1.38	1.41	1.46	1.49	1.53	1.55
	5-9	1.17	1.19	1.21	1.23	1.25	1.28	1.30	1.32	1.35	1.38	1.41	1.45	1.49	1.53	1.56	1.59
	6-0	1.20	1.22	1.24	1.26	1.29	1.32	1.34	1.36	1.39	1.42	1.45	1.48	1.53	1.56	1.60	1.62
	6-3	1.24	1.26	1.28	1.29	1.32	1.35	1.37	1.39	1.42	1.45	1.48	1.52	1.56	1.60	1.63	1.65
	6-6	1.27	1.29	1.31	1.33	1.36	1.39	1.41	1.42	1.46	1.49	1.52	1.55	1.60	1.63	1.67	1.69

T-Beams - Moments of Inertia and Weight Tables

T – BEAM MOMENTS OF INERTIA (ft^4)
Interior Girder Stem = 13" Fillets = 4"

SLAB (in)	6	6	6	6	6	6-1/8	6-1/4	6-1/4	6-1/4	6-3/8	6-1/2	6-5/8	6-3/4	7	7-1/8	7-1/4
c-c GIRR (ft-in)	5-9	6-0	6-3	6-6	6-9	7-0	7-3	7-6	7-9	8-0	8-3	8-6	8-9	9-0	9-3	9-6
DEPTH (ft-in) 1-6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
1-9	0.9	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1
2-0	1.4	1.4	1.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.6	1.6
2-3	2.0	2.0	2.0	2.1	2.1	2.1	2.1	2.2	2.2	2.2	2.2	2.2	2.3	2.3	2.3	2.3
2-6	2.7	2.8	2.8	2.8	2.9	2.9	2.9	3.0	3.0	3.0	3.0	3.1	3.1	3.1	3.1	3.2
2-9	3.6	3.6	3.7	3.7	3.8	3.8	3.9	3.9	4.0	4.0	4.0	4.1	4.1	4.1	4.2	4.2
3-0	4.6	4.7	4.8	4.8	4.9	4.9	5.0	5.1	5.1	5.2	5.2	5.3	5.3	5.4	5.4	5.4
3-3	5.8	5.9	6.0	6.1	6.1	6.2	6.3	6.4	6.4	6.5	6.6	6.7	6.7	6.8	6.8	6.9
3-6	7.2	7.3	7.4	7.5	7.6	7.7	7.8	7.9	8.0	8.1	8.2	8.3	8.3	8.4	8.5	8.6
3-9	8.8	8.9	9.0	9.1	9.3	9.4	9.5	9.6	9.7	9.9	10.0	10.1	10.2	10.3	10.4	10.5
4-0	10.5	10.7	10.8	11.0	11.1	11.3	11.5	11.6	11.7	11.9	12.0	12.2	12.3	12.5	12.6	12.7
4-3	12.5	12.7	12.9	13.1	13.2	13.4	13.6	13.8	13.9	14.1	14.3	14.5	14.6	14.8	15.0	15.1
4-6	14.7	14.9	15.1	15.3	15.5	15.8	16.0	16.2	16.4	16.6	16.8	17.1	17.3	17.5	17.7	17.9
4-9	17.1	17.4	17.6	17.9	18.1	18.4	18.7	18.9	19.1	19.4	19.6	19.9	20.1	20.4	20.6	20.9
5-0	19.8	20.1	20.3	20.6	20.9	21.2	21.6	21.8	22.1	22.4	22.7	23.0	23.3	23.6	23.9	24.2
5-3	22.6	23.0	23.3	23.6	24.0	24.4	24.8	25.0	25.3	25.7	26.1	26.4	26.7	27.2	27.5	27.8
5-6	25.6	26.2	26.5	26.9	27.3	27.7	28.2	28.5	28.9	29.3	29.7	30.1	30.5	31.0	31.3	31.7
5-9	29.2	29.6	30.0	30.5	30.9	31.4	31.9	32.3	32.7	33.2	33.6	34.1	34.6	35.1	35.5	36.0
6-0	32.8	33.3	33.8	34.3	34.7	35.3	35.9	36.4	36.8	37.3	37.9	38.4	38.9	39.6	40.1	40.6
6-3	36.8	37.3	37.9	38.4	38.9	39.6	40.3	40.7	41.2	41.8	42.5	43.1	43.7	44.4	45.0	45.5
6-6	41.0	41.6	42.2	42.8	43.4	44.1	44.9	45.4	45.9	46.7	47.4	48.0	48.7	49.5	50.2	50.8

T – BEAM GIRDER WEIGHTS (K/ft.)
Interior Girder Stem = 13" Fillets = 4"

SLAB (in)	6	6	6	6	6	6-1/8	6-1/4	6-1/4	6-1/4	6-3/8	6-1/2	6-5/8	6-3/4	7	7-1/8	7-1/4
c-c GIRS (ft-in)	5-9	6-0	6-3	6-6	6-9	7-0	7-3	7-6	7-9	8-0	8-3	8-6	8-9	9-0	9-3	9-6
DEPTH (ft-in) 1-6	0.61	0.63	0.65	0.67	0.69	0.71	0.74	0.76	0.78	0.81	0.84	0.87	0.91	0.95	0.99	1.02
1-9	0.65	0.67	0.69	0.71	0.73	0.75	0.78	0.80	0.82	0.85	0.88	0.92	0.95	0.99	1.03	1.06
2-0	0.69	0.71	0.73	0.75	0.77	0.79	0.82	0.84	0.86	0.89	0.92	0.96	0.99	1.03	1.07	1.10
2-3	0.73	0.75	0.77	0.79	0.81	0.84	0.86	0.88	0.90	0.93	0.96	1.00	1.03	1.07	1.11	1.15
2-6	0.77	0.79	0.81	0.83	0.85	0.88	0.90	0.92	0.94	0.97	1.01	1.04	1.07	1.12	1.15	1.19
2-9	0.81	0.83	0.85	0.87	0.89	0.92	0.95	0.96	0.98	1.01	1.05	1.08	1.11	1.16	1.19	1.23
3-0	0.85	0.87	0.89	0.91	0.93	0.96	0.99	1.01	1.02	1.06	1.09	1.12	1.15	1.20	1.23	1.27
3-3	0.89	0.91	0.93	0.95	0.97	1.00	1.03	1.05	1.07	1.10	1.13	1.16	1.19	1.24	1.27	1.31
3-6	0.94	0.95	0.97	0.99	1.01	1.04	1.07	1.09	1.11	1.14	1.17	1.20	1.23	1.28	1.31	1.35
3-9	0.98	0.99	1.01	1.03	1.05	1.08	1.11	1.13	1.15	1.18	1.21	1.24	1.27	1.32	1.35	1.39
4-0	1.02	1.04	1.05	1.07	1.09	1.12	1.15	1.17	1.19	1.22	1.25	1.28	1.31	1.36	1.39	1.43
4-3	1.06	1.08	1.09	1.11	1.13	1.16	1.19	1.21	1.23	1.26	1.29	1.32	1.35	1.40	1.43	1.47
4-6	1.10	1.12	1.14	1.15	1.17	1.20	1.23	1.25	1.27	1.30	1.33	1.36	1.39	1.44	1.48	1.51
4-9	1.14	1.16	1.18	1.19	1.21	1.24	1.27	1.29	1.31	1.34	1.37	1.40	1.44	1.48	1.52	1.55
5-0	1.18	1.20	1.22	1.24	1.25	1.28	1.31	1.33	1.35	1.38	1.41	1.44	1.48	1.52	1.55	1.59
5-3	1.22	1.24	1.26	1.28	1.29	1.32	1.35	1.37	1.39	1.42	1.45	1.48	1.52	1.56	1.60	1.63
5-6	1.26	1.28	1.30	1.32	1.34	1.36	1.39	1.41	1.43	1.46	1.49	1.52	1.56	1.60	1.64	1.67
5-9	1.30	1.32	1.34	1.36	1.38	1.40	1.43	1.45	1.47	1.50	1.53	1.57	1.60	1.64	1.68	1.71
6-0	1.34	1.36	1.38	1.40	1.42	1.44	1.47	1.49	1.51	1.54	1.57	1.61	1.64	1.68	1.72	1.75
6-3	1.38	1.40	1.42	1.44	1.46	1.49	1.51	1.53	1.55	1.58	1.61	1.65	1.68	1.72	1.76	1.80
6-6	1.42	1.44	1.46	1.48	1.50	1.53	1.55	1.57	1.59	1.62	1.66	1.69	1.72	1.77	1.80	1.84

Charts For Resisting Moments-Box Girders

Given below are charts for determination of resisting moments for interior and exterior girders of box girder sections for effective depths from 30" to 90".

The graphs can be used for determining the resisting moment, value of "jd" and "A_s" required for the design moment for various effective depths, "d", with slabs of 5½", 6", 6¼", 6½", 6¾" and 7". The graphs are based on f_s=20,000 psi, n=10, and f_c as shown

It should be noted that in accordance with AASHO Specifications the effective flange width shall not exceed:

1. One-fourth the span length of the beam. (For girders with flange on one side only, use one-twelfth the span length.)
2. The distance center to center of beams.
3. Twelve times the least thickness of slab plus the width of the girder stem.

Since (3) usually governs, the graphs have been made on this basis. As long as the girder spacing is equal to or greater than that noted for any specified slab depth, the graphs can be used without modification. For girder spacings of 6'-6" or greater for any slab depth, a simple ratio will give approximate values sufficiently close for practical purposes. (Within 3%.)

For example: Slab thickness=7"; d=52.5"; f_c=1200 psi; girder spacing=6'-6". From Graph for 7" slab, M_R= 2700 ft. kips for 7'-8" girder spacing. Therefore, for 6'-6" girder spacing,

$$M_R = \frac{6.5}{7.67} \times 2700 = 2280 \text{ ft. kips.}$$

The value of "jd" is plotted as a constant for all concrete stresses for any given "d". A maximum error is on the conservative side.

The following examples demonstrate the use of the graphs:

Design:

Given a box girder with effective depth d = 52.5"; girder spacing 7'-6"; top slab = 6"; and allowable f_c= 1200 psi. The design DL+LL+I moment for the interior girder = 2000 ft. kips, and for the exterior girders = 1600 ft. kips. Required to determine if f_c is within the allowable, and the A_s required.

Since the girder spacing is greater than the 6'-8" minimum for a 6" slab the graphs can be used without modification. For d= 52.5" and f_c = 1200 psi, the maximum moment that can be applied to the interior girder= 2160 ft. kips. Therefore, f_c is less than 1200 psi. For "d" = 52.5" the value of "jd" is 49". Therefore,

$$A_s = \frac{0.6M}{jd} = \frac{.6 \times 2000}{49} = 24.5 \text{ sq. in.}$$

for the interior girder.

For the exterior girder with d = 52.5" and f_c =1200 psi, the maximum moment that can be applied is 1280 ft. kips. Therefore, compressive steel must be used since the moment applied on the exterior girder is 1600 ft. kips. Value of jd = 48.5".

$$\text{Tensile } A_s = \frac{.6 \times 1600}{48.5} = 19.8 \text{ sq. in.}$$

(See Article 6-9 of Vol. I, BP&DM) By using Table on page 5-12 of Vol. III BP&DM (Assume d'= 2½) A'_s = $\frac{1600-1280}{66.5}$= 4.8□".

INVESTIGATION:

Given a box girder with effective depth= 62.5", slab =6" and girder spacing=7'-3", allowable f_c =1200 psi.

Interior girder:

Furnished A_s=25.0 sq. in. M for DL+LL+I=2700 ft. kips

Exterior girder:

Furnished A =17.2 sq. in. M for DL+LL+I=1600 ft. kips

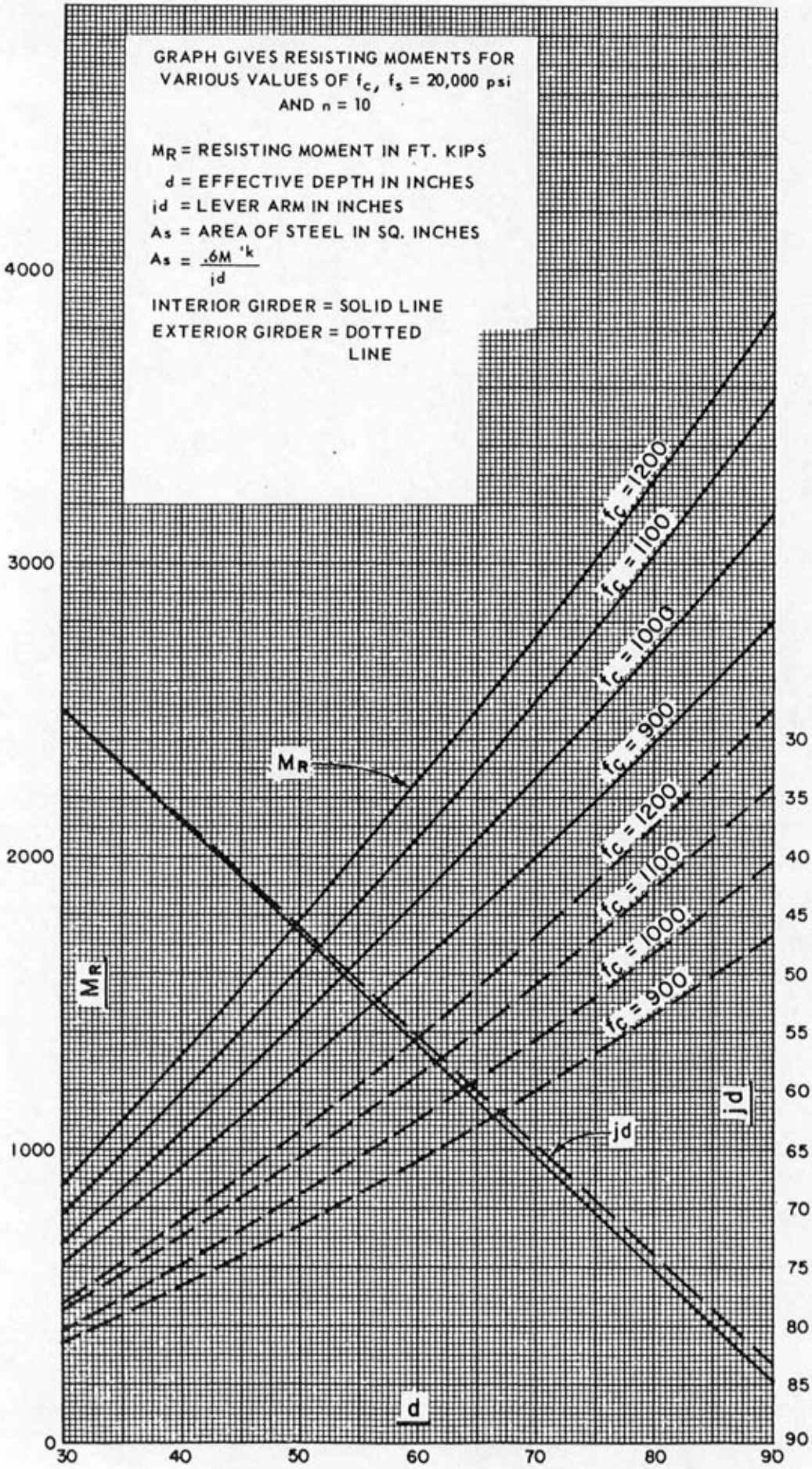
From the graph it can be seen that the interior and exterior girder stresses will result in f_c less than 1200 psi provided that f_s =20,000 psi or less.

For Interior Girder required $A_s = \frac{.6 \times 2700}{58.5} = 27.5$ sq. in.

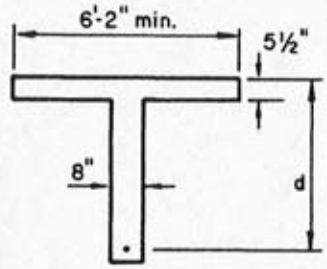
For Exterior Girder required $A = \frac{.6 \times 1600}{58} = 16.5$ sq. in.

By inspection since the furnished A_s for the interior girder is less than the required A_s, the f_s is greater than 20,000 psi. Therefore, steel must be added to than 20,000 psi. Therefore, steel must be added to equal or exceed that required as calculated to bring f_s equal to or less than 20,000 psi.

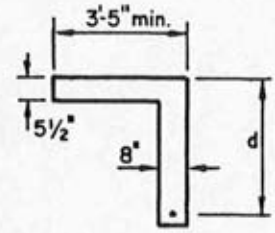
For the exterior girder, A_s furnished is greater than that required. Therefore, the f_s is less than 20,000 psi. As there is less than one #11 bar difference, this steel should be left as is.



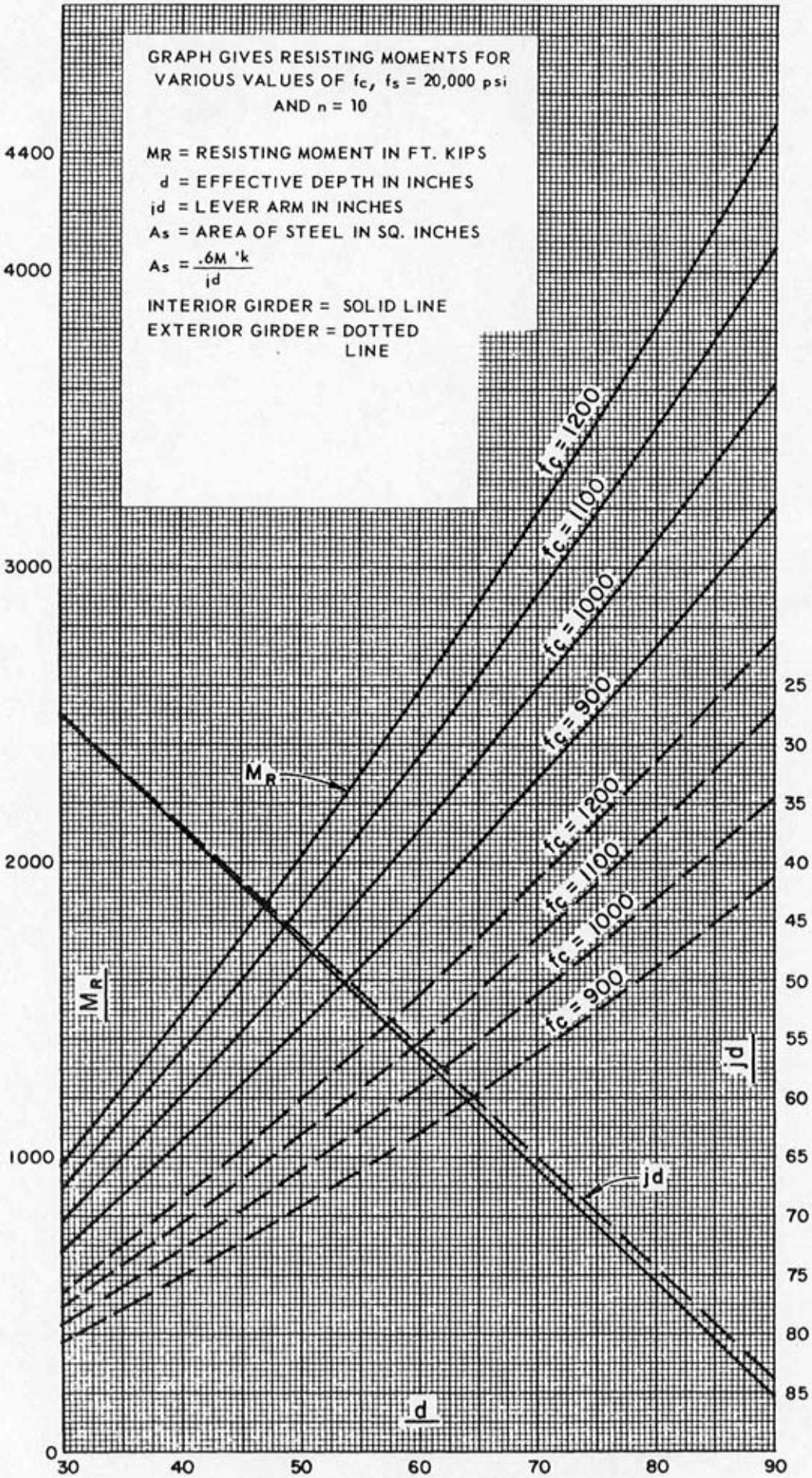
SLAB = 5½"



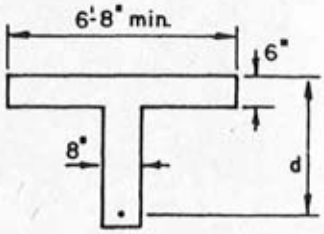
INTERIOR GIRDER



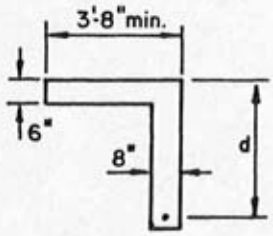
EXTERIOR GIRDER



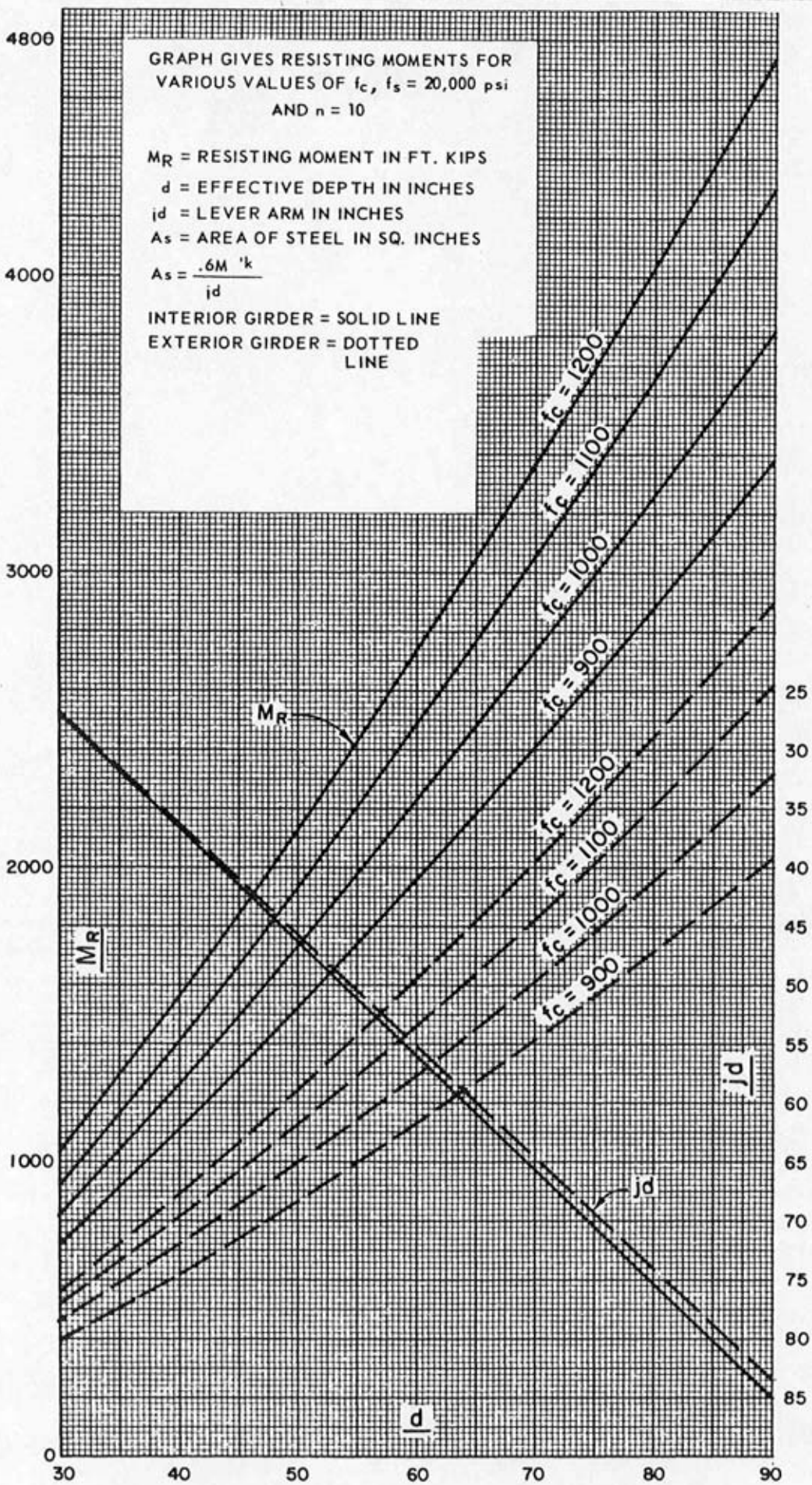
SLAB = 6"



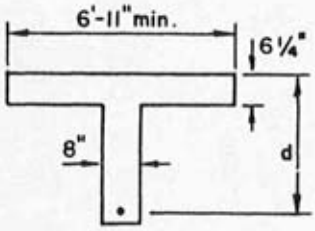
INTERIOR GIRDER



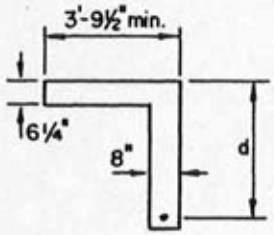
EXTERIOR GIRDER



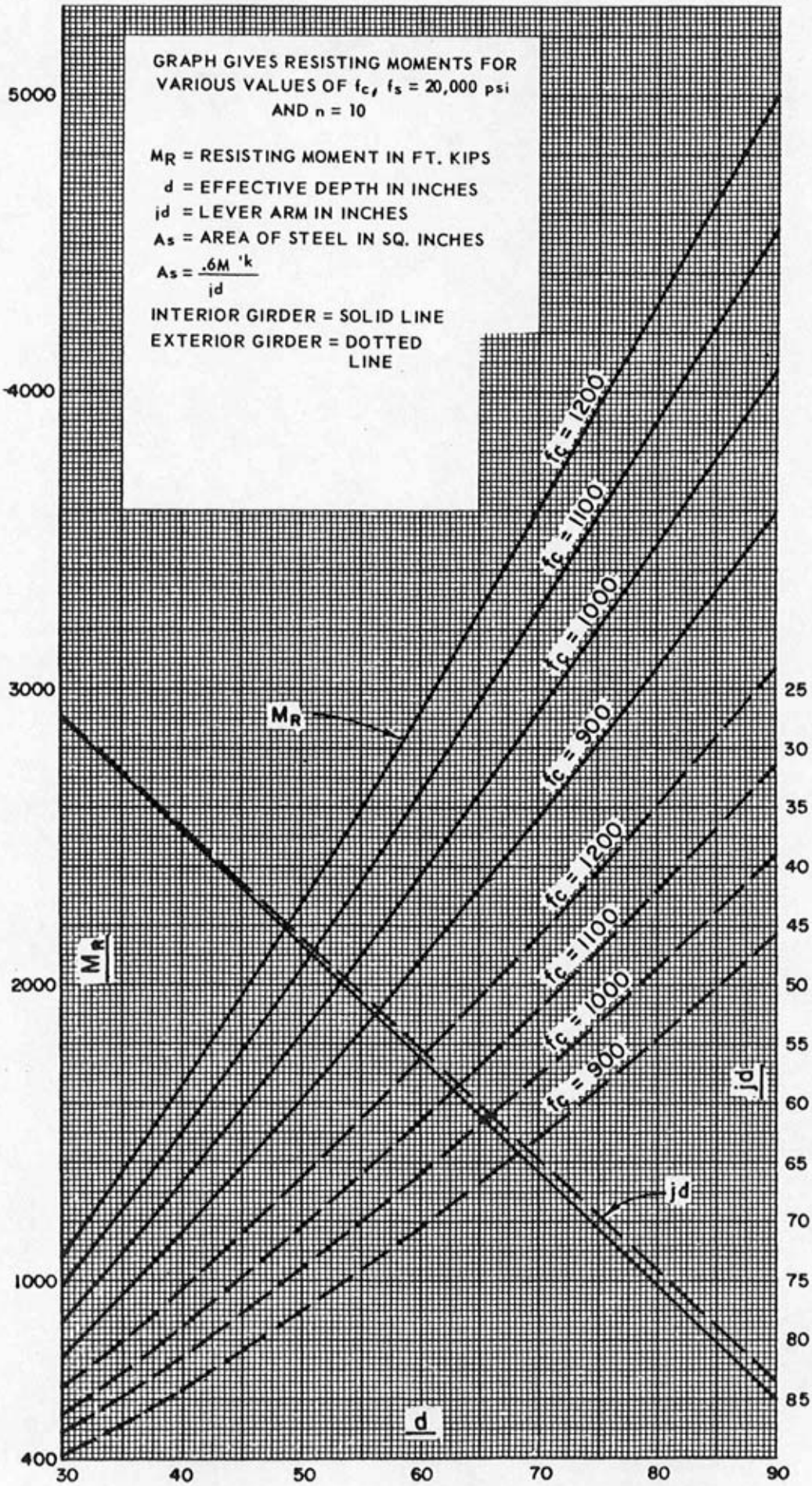
SLAB = 6¼"



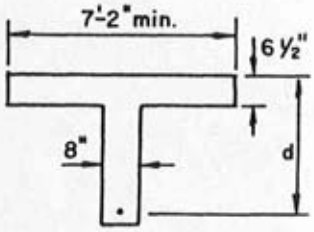
INTERIOR GIRDER



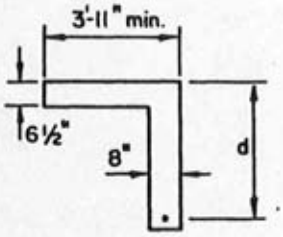
EXTERIOR GIRDER



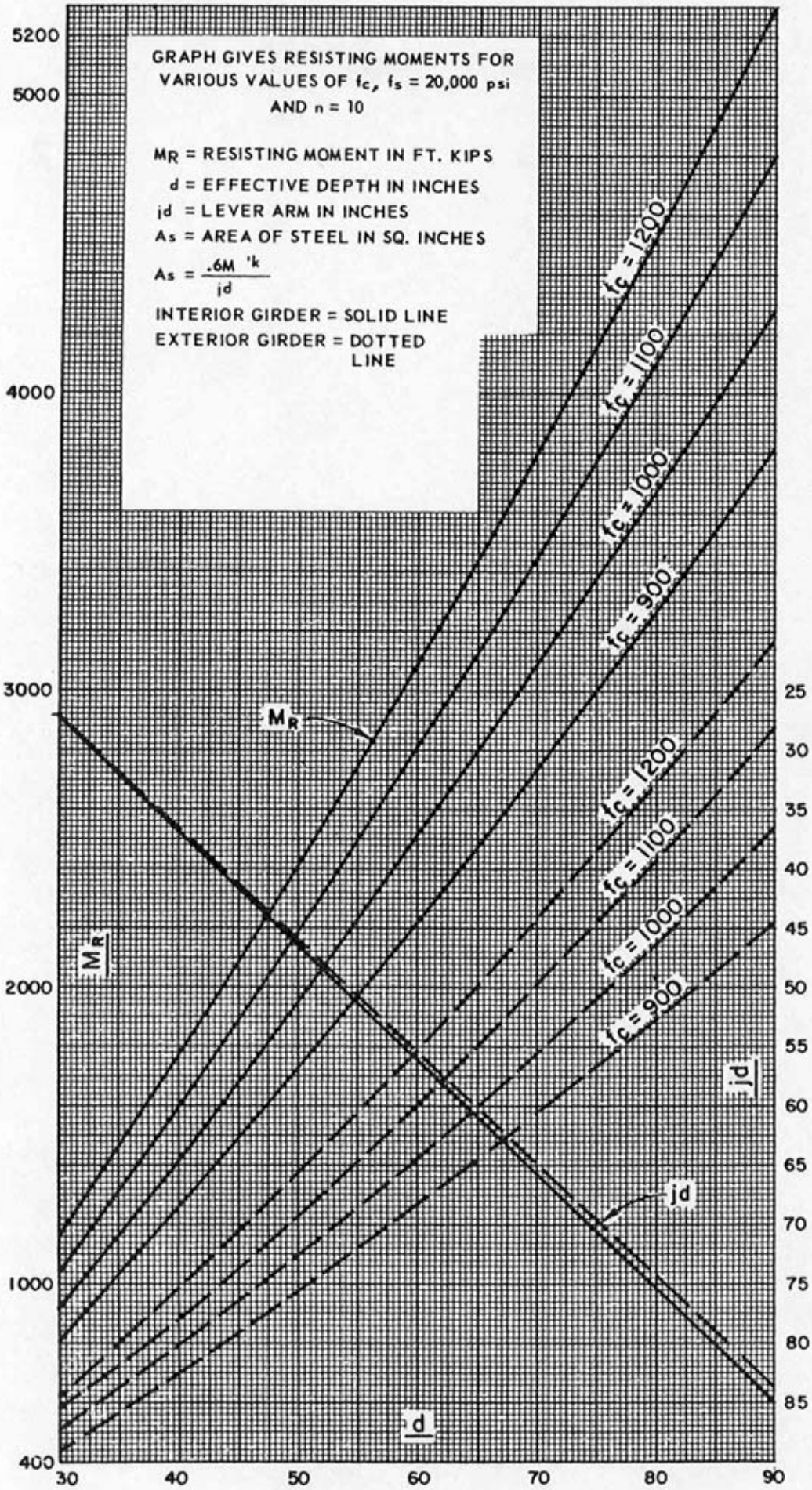
SLAB = 6 1/2"



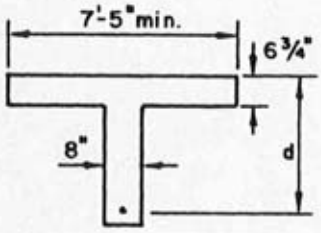
INTERIOR GIRDER



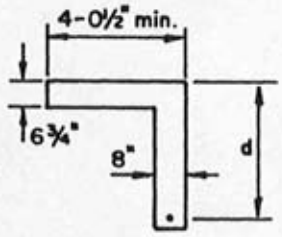
EXTERIOR GIRDER



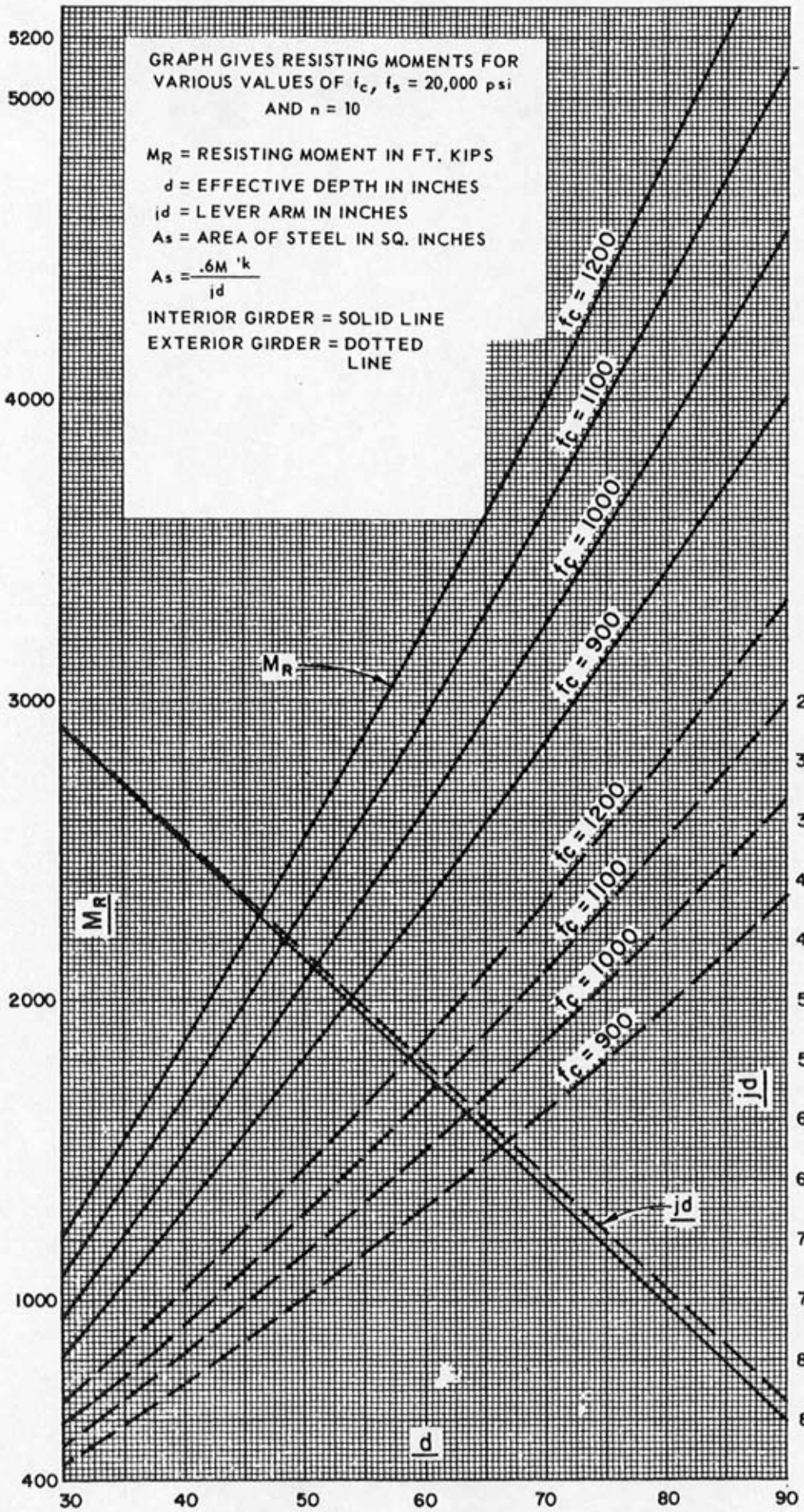
SLAB = 6¾"



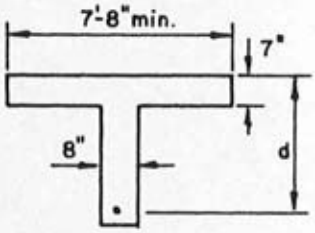
INTERIOR GIRDER



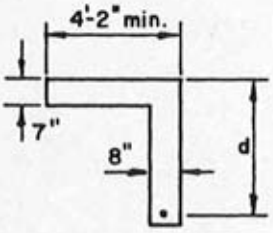
EXTERIOR GIRDER



SLAB = 7"



INTERIOR GIRDER



EXTERIOR GIRDER

Column Design Charts

Notations:

d = least lateral dimension of column.
f'_c = compressive stength of concrete.
f_y = yield stress of reinforcement.
M = design bending moment at section due to ultimate loads.
P = design axial load at section due to ultimate loads.

Design Specifications:

See Bridge Design Manual Volume 1, Article 6–11

Stresses:

f'_c = 3,250 psi
f_y = 60,000 psi

Max Column Length = 10d

Use of Charts:

Enter the charts with an ultimate axial design load and an ultimate design bending moment and determine the reinforcement required.

Example 1: Given - 5'–6" octagonal column
P = 7,500 kips
M = 5,500 ft-kips
by interpolation Use 24-#18

Example 2: Given - 6' round column
P = 3,250 kips
M = 12,250 ft-kips
by interpolation Use 24-#18

Spiral Spacing:

The spacing of spirals depends on the strength of concrete and clearance to the spiral. For f'_c of 3250 psi and 2" clear to the spiral, #4 at 3–1/2" is sufficient for all column diameters. If a stronger concrete or a greater clearance is needed (as for corrosion protection in high cloride or marine environment), the spiral must be calculated from the following formula:

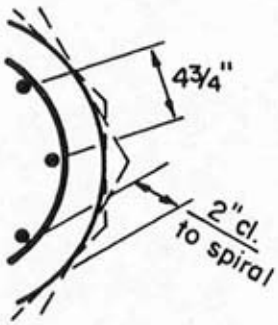
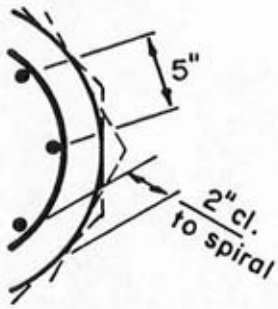
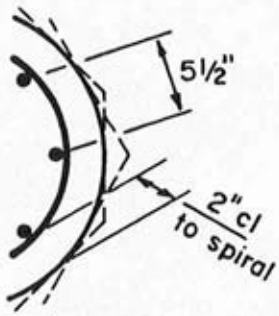
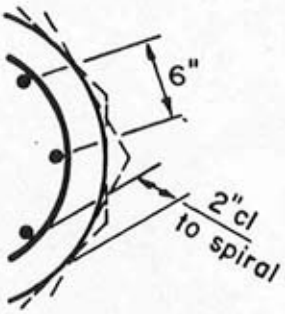
$$P' = 0.45 \quad \frac{Ag}{Ac} - 1 \quad \frac{f'_c}{f'_{sp}}$$

f'_{sp} = 60,000 psi
Ag = gross column area
Ac = area of column core

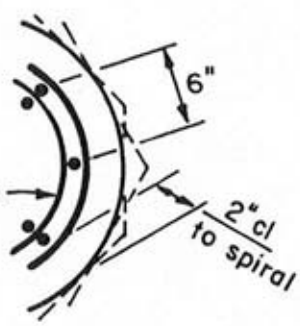
P' = ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals).

Maximum clear spacing for spirals is 3".

COLUMN BAR ARRANGEMENT
SINGLE RING OF MAIN REINFORCEMENT

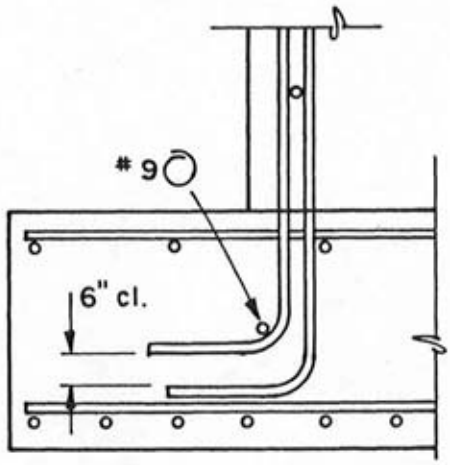
Minimum Bar Spacing	Maximum Number of Bars (c-c bars)				
	Column Diameter				
	4'	5'-6"	6'	7'	8'
#10 Bars 	27	39	43	51	59
#11 Bars 	25	37	41	48	56
#14 Bars 	23	33	37	44	51
#18 Bars 	21	30	34	40	46

COLUMN BAR ARRANGEMENT
TWO RINGS OF MAIN REINFORCEMENT

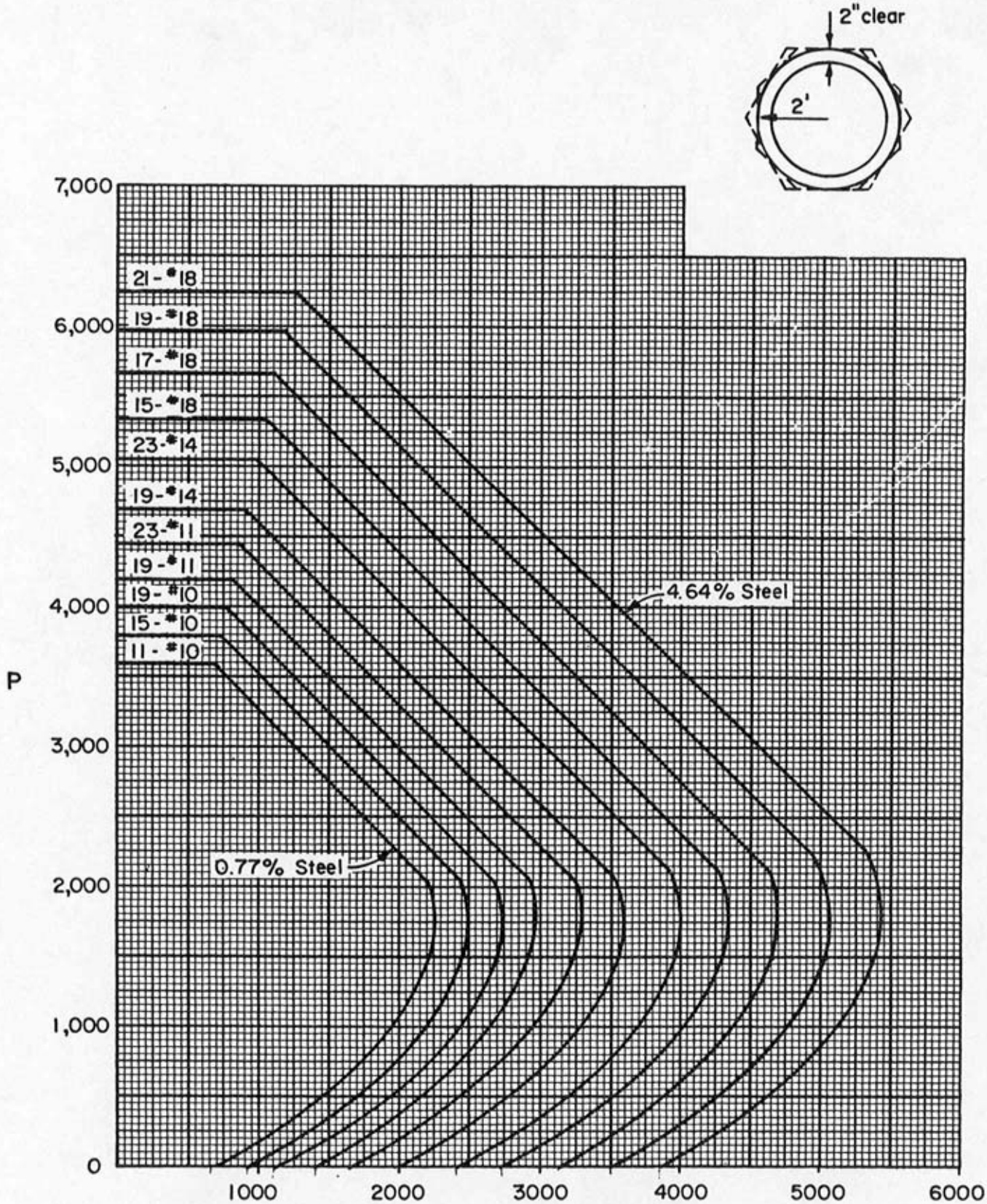
Minimum Bar Spacing	Maximum Number of Bars (c-c bars)				
	Column Diameter				
	4'	5'-6"	6'	7'	8'
#18 Bars 		40	48	62	84

The number of bars in the inner ring shall be a convenient fraction of the number of bars in the outer ring, so the reinforcement can be bundled and symetrically placed.

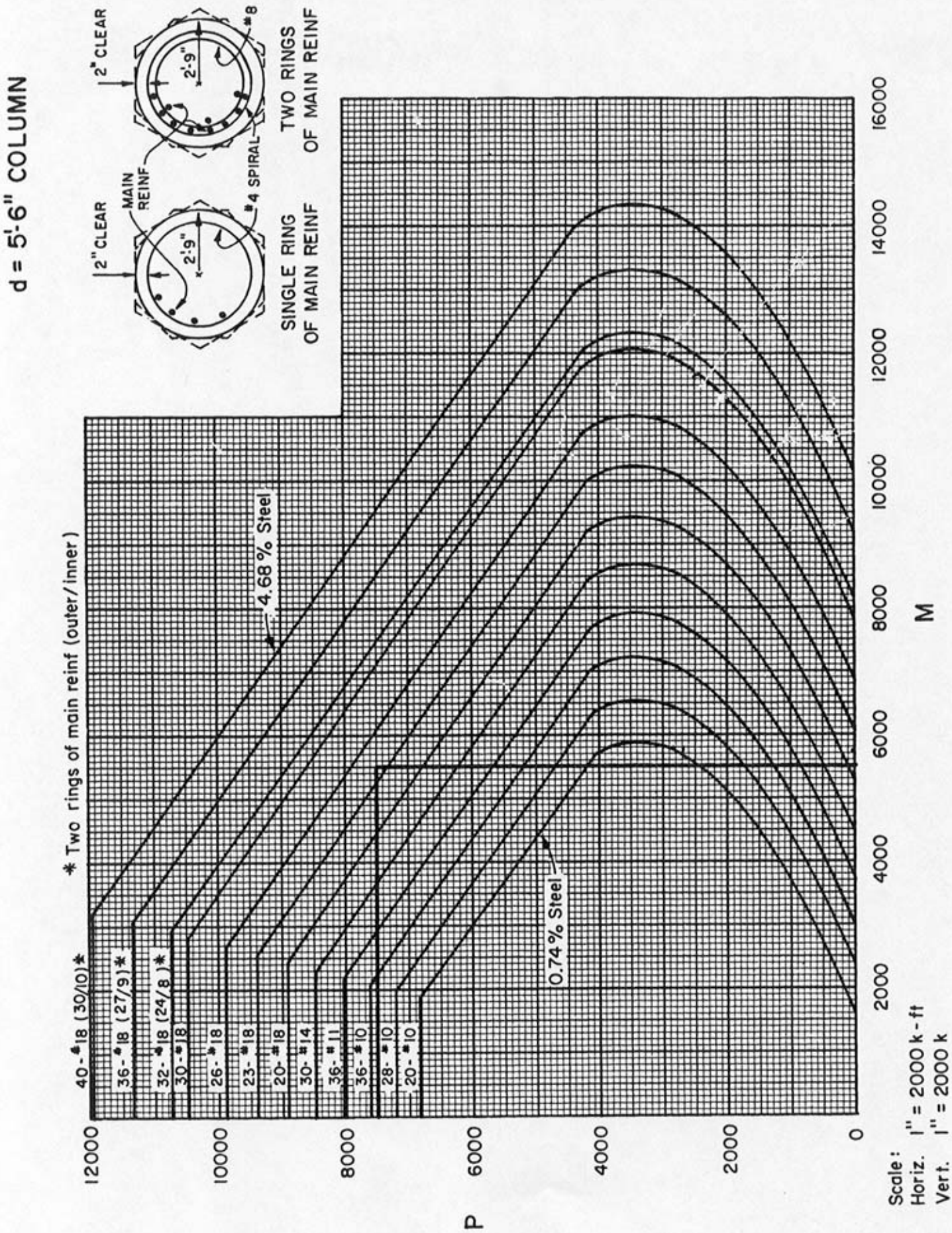
Whenever the footing depth is sufficient to provide adequate bond length, straight bars shall be used for the inner ring of reinforcement. When footing depth is not sufficient to provide adequate bond length, hooked bars shall be used and detailed on the plans as shown below:

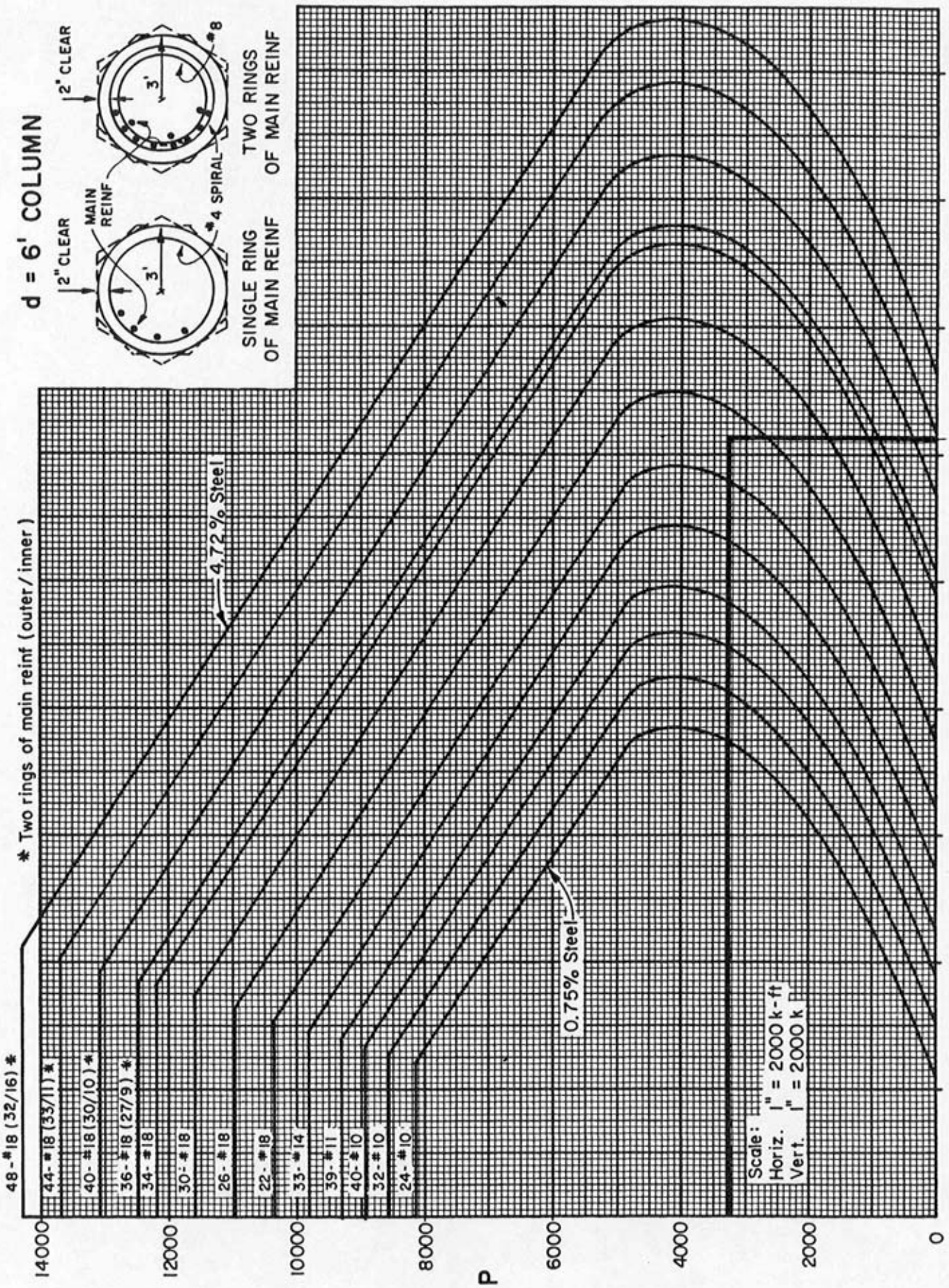


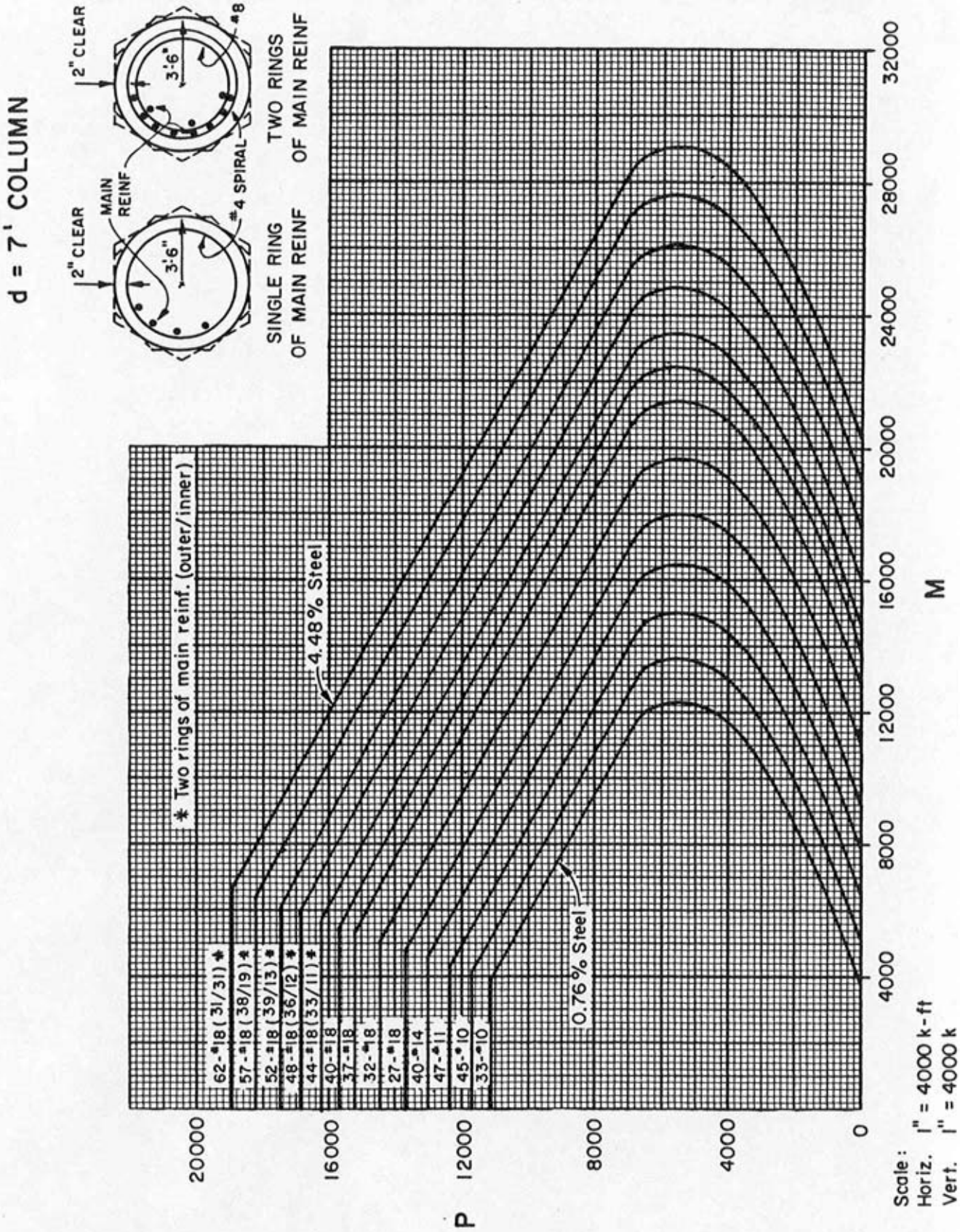
d = 4' COLUMN

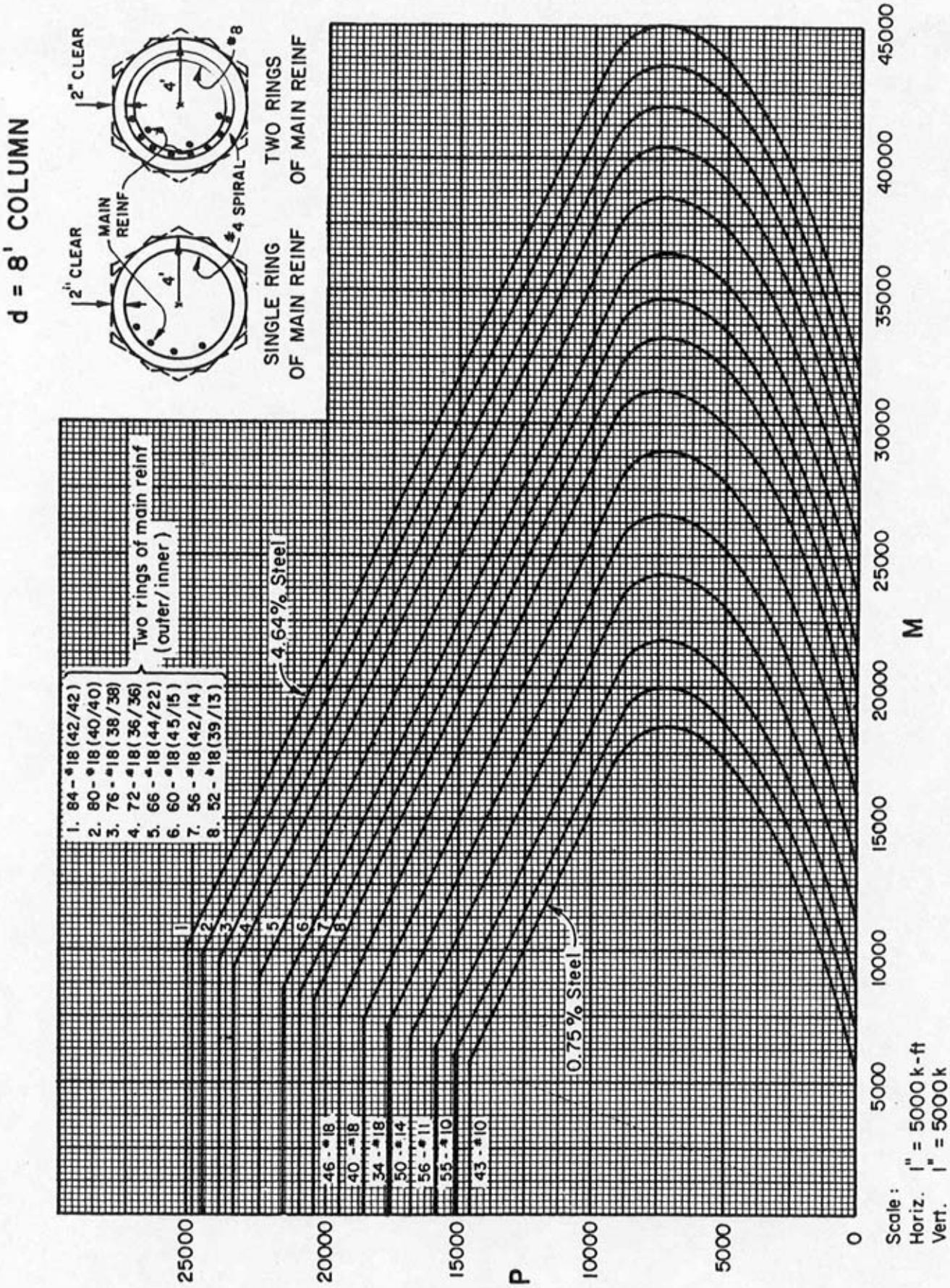


Scale:
Horiz. 1" = 1000 k-ft
Vert. 1" = 1000 k


d = 5'-6" COLUMN
2" CLEAR
MAIN REINF
#4 SPIRAL
2'-9"
SINGLE RING OF MAIN REINF
2" CLEAR
#8
2'-9"
TWO RINGS OF MAIN REINF
* Two rings of main reinf (outer/inner)
40-#18 (30/10)*
36-#18 (27/9)*
32-#18 (24/8)*
30-#18
26-#18
23-#18
20-#18
30-#14
36-#11
36-#10
28-#10
20-#10
4.68% Steel
0.74% Steel
P
M
12000
10000
8000
6000
4000
2000
0
2000
4000
6000
8000
10000
12000
14000
16000
Scale:
Horiz. 1" = 2000 k-ft
Vert. 1" = 2000 k


d = 6' COLUMN
2" CLEAR
MAIN REINF
3'
#4 SPIRAL
SINGLE RING OF MAIN REINF
2" CLEAR
3'
#8
TWO RINGS OF MAIN REINF
* Two rings of main reinf (outer/inner)
48-#18 (32/16) *
44-#18 (33/11) *
40-#18 (30/10) *
36-#18 (27/9) *
34-#18
30-#18
26-#18
22-#18
33-#14
39-#11
40-#10
32-#10
24-#10
4.72% Steel
0.75% Steel
Scale: 1" = 2000 k-ft
Horiz. 1" = 2000 k
Vert.
P
M
0
2000
4000
6000
8000
10000
12000
14000
16000
18000


d = 7' COLUMN
2" CLEAR
MAIN REINF
3'-6"
#4 SPIRAL
SINGLE RING OF MAIN REINF
2" CLEAR
3'-6"
#8
TWO RINGS OF MAIN REINF
* Two rings of main reinf. (outer/inner)
62-#8 (31/31) *
57-#8 (38/19) *
52-#8 (39/13) *
48-#8 (36/12) *
44-#8 (33/11) *
40-#8
37-#8
32-#8
27-#8
40-#4
47-#11
45-#10
33-#10
4.48% Steel
0.76% Steel
P
20000
16000
12000
8000
4000
0
M
4000
8000
12000
16000
20000
24000
28000
32000
Scale: 1" = 4000 k-ft
Horiz. 1" = 4000 k
Vert.


d = 8' COLUMN
2" CLEAR
MAIN REINF
#4 SPIRAL
#8
4'
SINGLE RING OF MAIN REINF
TWO RINGS OF MAIN REINF
1. 84 - #18 (42/42)
2. 80 - #18 (40/40)
3. 76 - #18 (38/38)
4. 72 - #18 (36/36)
5. 66 - #18 (44/22)
6. 60 - #18 (45/15)
7. 56 - #18 (42/14)
8. 52 - #18 (39/13)
Two rings of main reinf (outer/inner)
4.64% Steel
0.75% Steel
46 - #18
40 - #18
34 - #18
50 - #14
56 - #11
55 - #10
43 - #10
P
M
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
Scale:
Horiz. 1" = 5000 k-ft
Vert. 1" = 5000 k

SHEAR MODIFICATION FOR SKEWED CONCRETE GIRDERS

General

In nonskewed bridges the shear load from a span is distributed uniformly into a support by assuming each girder carries an equal portion. In a skewed bridge, the load tends to distribute to the supports in a direction normal to the support. This causes a greater portion of the load to be concentrated at the obtuse corners of the span and less at the acute corners.

The following graph was developed to provide adjustment factors for applied shears calculated without considering skew effects. The graph is based largely on the research report "Skew Parameter Studies, Volumes 1 & 2," dated October 1976, and authored by Ray Davis and Mark Wallace.

For curved bridges having large skews (> 45°), the designer should consider a more exact analysis such as STRUDL or CELL computer programs which also consider torsion.

Chart Use

Calculate the applied shear in accordance with *Bridge Design Specifications*. Assume the total shear to be distributed equally to all girders. Next, modify the applied shear at the support by multiplying it by the chart value. Let the design shear vary linearly to 1.0 × applied shear at the midspan for *all* spans regardless of end condition.

For bridges with less than 5 girders, the interior girders need not be modified.

Girder flare lengths, stem thickness and stirrup spacing in all girders should be adjusted to be logical and as repetitive as possible.

See the following examples of shear modification to a reinforced and prestressed box girder bridge.

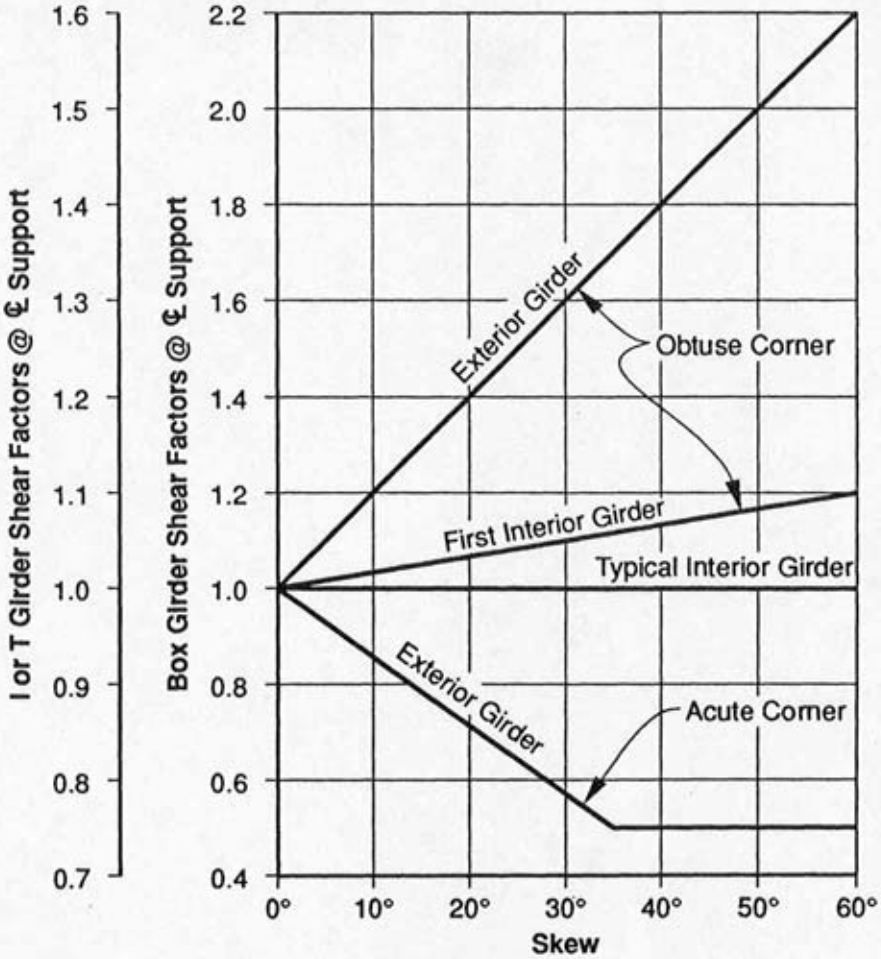
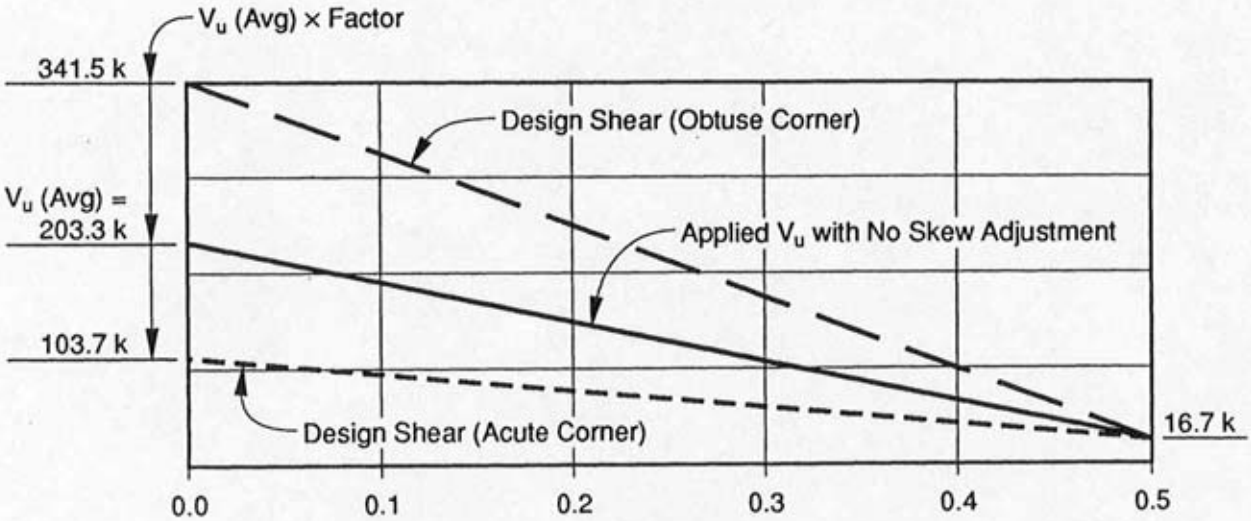


Chart No. 1



Note: Adjustment for interior girder not shown

34° Skew Adjustment Sketch for Example Problem No. 1

Shear Magnification Example No. 1

Reinforced Concrete Box Girder. Fully Continuous 78' Span.

$f'_c = 3250$ psi
Skew = 34°

d = 4' = 48"
6 girders

Face of cap @ 2.5' along girder from centerline bent.
V_u (D + L + I) = 1220 k @ centerline bent.
= 100 k @ centerline span.

V_u (avg.) = 1220/6 = 203.3 k/girder @ centerline bent.
= 100/6 = 16.7 k/girder @ centerline span.

Skew factors @ centerline bent from Chart No. 1:

Obtuse corner, exterior girder = 1.68
Obtuse corner, first interior girder = 1.11
Acute corner, exterior girder = 0.51

Calculate Design V_u (see 34° skew adjustment sketch)

Factor ×	V_u (avg) =	V_u (@ centerline bent)	V_u (@ centerline span)	V_u (@ d from face)
1.68	203.3	341.5	16.7	287.4
1.11	203.3	225.7	16.7	190.9
1.00	203.3	203.3	16.7	172.2 (typ.int. girder)
0.51	203.3	103.7	16.7	89.2

Calculate b_w, stirrups

V_u (@d)	V_u/dG	b_w (req.)	b_w (des.)	SF_c	SF_s	#5 Stirrup Spacing	
						Req.	Use
287.4	5.99	12.4	12*	1.16	4.83	@ 5	@ 5
190.9	3.98	8.2	8	0.78	3.20	@ 8	@ 8
172.2	3.59	7.4	8	0.78	2.81	@ 9	@ 8
89.2	1.86	3.8	8	0.78	1.08	@ 24	@ 24**

* Usually 10" minimum @ exterior girder. ** See maximum stirrup spacing.

Maximum Stirrup Spacing

24" or d / 2 (BDS 8.19.3) unless $V_u / dG > \phi \times 6 \times \sqrt{f'c} \times b_w$ (BDS 8.16.6.1 and 8.16.6.3.8)
$= 0.2907\, b_w$ for 3250 psi concrete
$0.2907\, b_w = 0.2907 \times 8 = 2.33 > 1.86$ ok
Maximum spacing = 48 / 2 = 24"

Check midspan spacing

$V_u/d = 16.7/48 = 0.35 < SF_c = 0.78$ ∴ Stirrups not required by analysis. Use #5 @ 24.
For #5 @ 24, $SF_s = 1.098 > 1.08$ required at acute corner.
Use #5 @ 24 from midspan to support @ acute corner.

Exterior Girder Flare Dimensions @ Obtuse Corner

V_u @ face of bent = 320.7 k
$b_w = V_u/(d \times 0.4846) = 320.7/(48 \times 0.4846) = 13.8''$ say 14"

Assume 16' long flare.
b_w @ d from face = 12.5" (for 16' flare) > 12.4" (required) ok

Use 14" × 16' flare, exterior girder, obtuse corner only.

References

Bridge Design Practice, Tables 17, 18 and 19 (pp. 2-244, 245, & 249), dated November 1981.

Shear Magnification Example No. 2

Prestressed Concrete Box Girder. Simple/Continuous 150' span.

$f'_c = 4000$ psi
Skew = 30°

Structure Depth = 6'
7 girders

Face of abutment @ 1.5' along girder from centerline abutment.

Skew factors @ centerline abutment from Chart No. 1:

Obtuse corner, exterior girder = 1.60
Obtuse corner, first interior girder = 1.10
Acute corner, exterior girder: No adjustment suggested

Design: (Exterior girder)

	Centerline Abutment	0.1	0.2	0.3	0.4	0.5
Skew Factor	1.60	1.48	1.36	1.24	1.12	1.00
Calculated V_u*	3632	2675	1904	1166	494	1212
Calculated V_c*	2780	2637	1313	659	527	811
A_v required**	1.82	0.87	0.77	0.47	minimum	0.27
b' required***	18.0	8.6	7.7	4.6	—	2.6

* From BDS output or other method.

$$** \; A_v = \left(in^2 / ft\right) = \frac{(\text{Skew Factor})\left(\frac{V_u}{\varnothing}\right) - V_c}{60(d)(\text{No. of girders})} = \frac{(\text{Skew Factor})(1.11)(V_u) - V_c}{48(D)(\text{No. of girders})}$$

where d = (0.8)(D) and Ø = 0.90

See Chart No. 3 for selecting size and spacing of stirrups.

$$*** \text{Min } b' = \frac{625A_v}{\sqrt{f'_c}}$$ (or from attached Chart 2)

where the expression for b' is derived from the following expressions:

$$V_s = \frac{A_v f_{sy} d}{s}$$ and maximum $V_s = 8\sqrt{f'_c}\, b'd$ (BDS Article 9.20.3.1)

Exterior Girder Flare Dimensions @ Obtuse Corner

b′ required at face of diaphragm = 18.0 – (1.5/15) (18.0 – 8.6) = 17.1". Use 18".

Assume 12' long flare.

b′ required at end of flare = 18.0 – (13.5/15) (18.0 – 8.6) = 9.5" < 12" OK.

Use 18" × 12' long flare.

Check Maximum Spacing (From Chart 2)

At diaphragm face:	For A_v = 1.82 and b′ = 18"	→	Reduce by ½ (½ × 24" maximum = 12")
At midspan:	For A_v = 0.27 and b′ = 12"	→	24" maximum okay

Notes

1) The above examples are supplied to make the designer aware of considerations, specifications and available design aids when designing for girder shear in skewed bridges. Normally a less involved process for actual designs would be acceptable because some of the data calculated or tabulated for the examples is known by inspection.

2) For prestressed structures, other design methods are available. One more detailed method is a computer program called "PSHEAR." See page 5-39 for PSHEAR instructions.

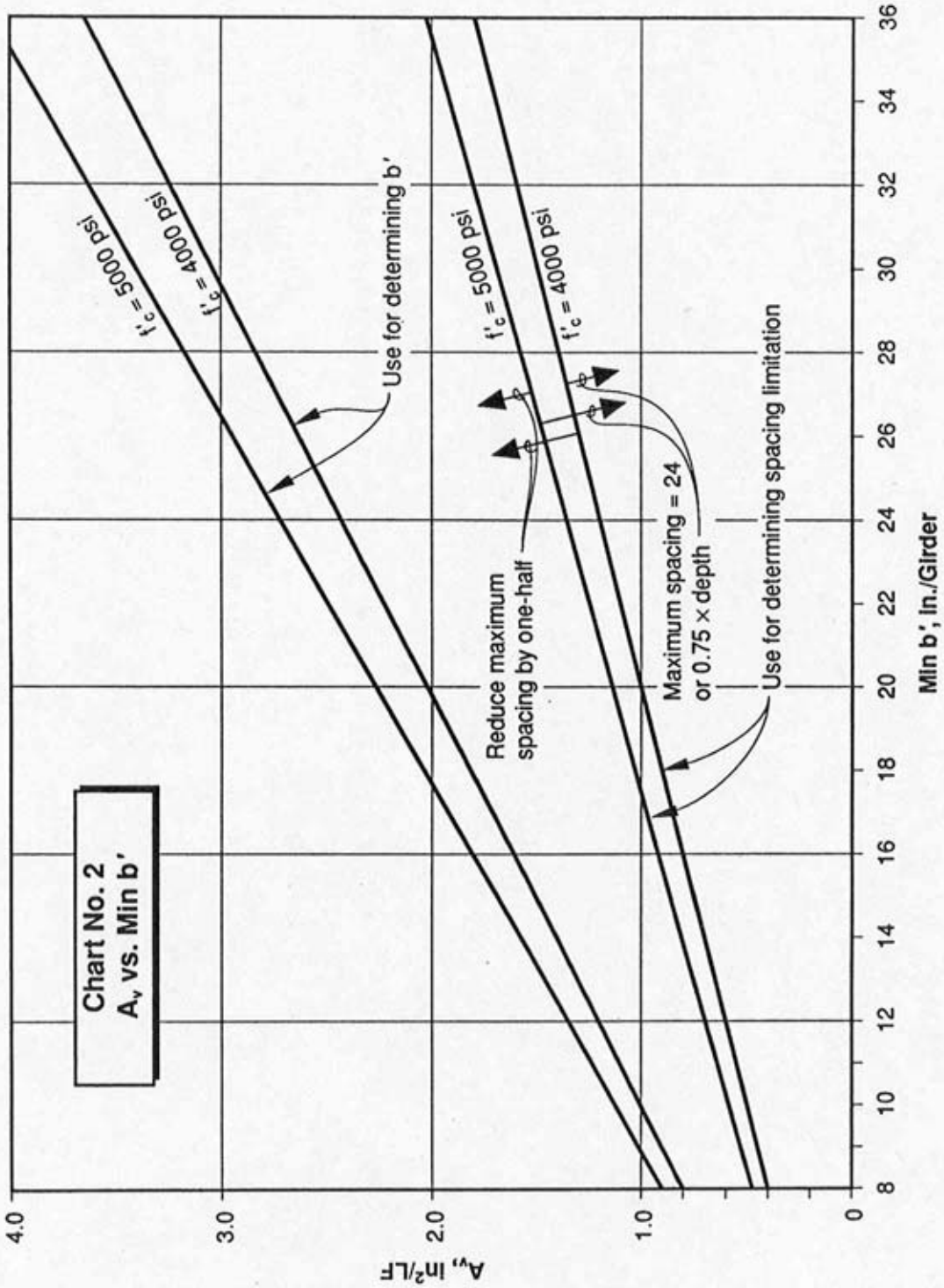


Chart No. 2
A_v vs. Min b'
f'_c = 5000 psi
f'_c = 4000 psi
Use for determining b'
f'_c = 5000 psi
f'_c = 4000 psi
Reduce maximum spacing by one-half
Maximum spacing = 24 or 0.75 × depth
Use for determining spacing limitation
A_v, in²/LF
4.0
3.0
2.0
1.0
0
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
Min b', in./Girder

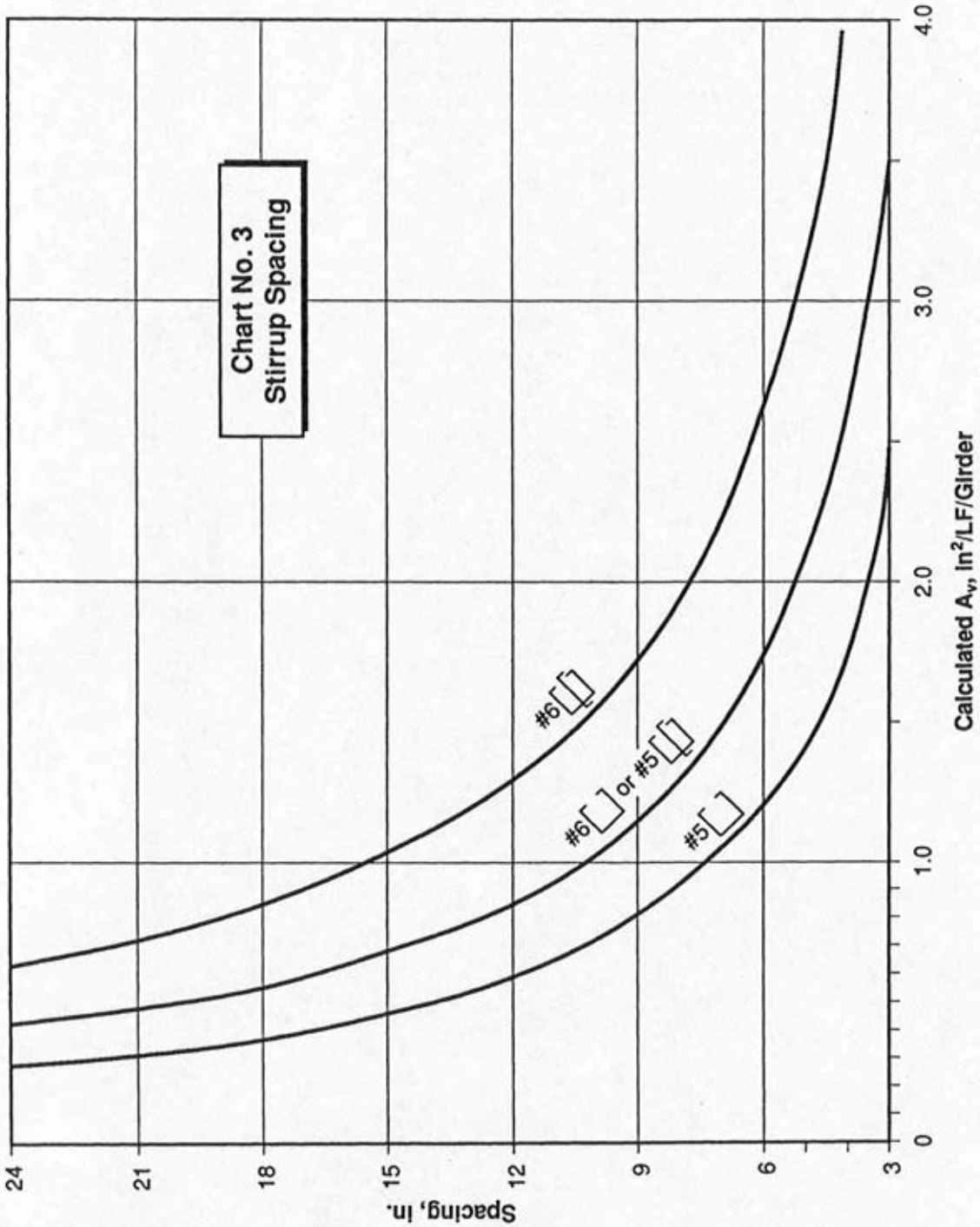


Chart No. 3
Stirrup Spacing
#6
#6 or #5
#5
Spacing, in.
24
21
18
15
12
9
6
3
0
1.0
2.0
3.0
4.0
Calculated A_v, in^2/LF/Girder

STEPS FOR ACCESSING AND RUNNING PSHEAR

The program can be found in the Bridge Computer Library.

1. Go through the general work account to the main bridge menu.

2. Type: Run, PSHEAR

3. Instruction and Definitions:

   a. Instructions and variable definitions can be listed if you are unfamiliar with the program by answering "yes" to the instructional subroutine. Strike PF3 to exit instructional subroutine.

   b. Answering "no" sends you directly to the main program after supplying an appropriate file name. Have your input file data, obtained from BDS output, ready to enter into the program. After entering the data onto the CRT screen blank form, type "file" to execute the data file and PSHEAR1 output will appear on screen.

4. Review results and record the desired output data.

5. PF3 to exit and to continue.

6. The program now asks if you would like to analyze another section.

   a. If you answer "yes", your original data files will be displayed. Modify the input data as needed for the next 10th pt. and file the data again as in Step 3b.

   b. Review results as in Step 4. Proceed to Step 5. Printouts of output can be obtained. If you answer "no", a print option screen will be displayed. After making your selection, the program then returns you to the main bridge menu. Return to Step 2 to continue or log off.

Inverted T-Caps

Inverted T-cap bents should be designed so that the falsework can be removed before the girders are placed. If, for unusual circumstances, it is necessary to leave the bent falsework in place until the superstructure is completed, suitable notes shall be placed on the plans requiring falsework to be designed to support the entire superstructure load and not to be removed until deck (or top of cap) concrete attains a specified strength.

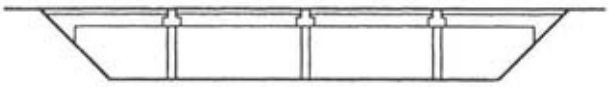
In addition to the forces which are ordinarily used for design, end of girder and seat details are subjected to other forces caused by construction irregularities, skews, deflections, impact during construction, and changes in length caused by creep and shrinkage. These factors must be considered in the design process. Several instances of cracking in seats of inverted T-caps, used for supporting precast girders, have primarily been due to:

Girder rotation
Edge loading
Plastic prestress shortening (creep) between bents
Insufficient reinforcing steel
Poor arrangement of reinforcing steel

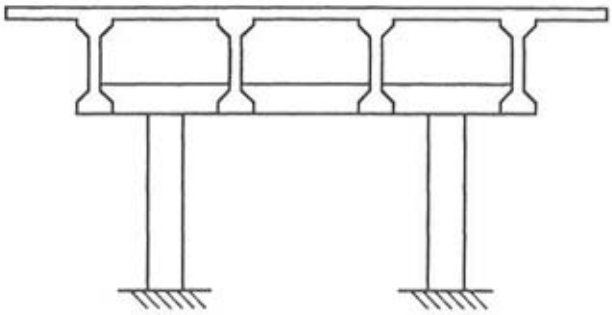
Memo to Designers 7-1 gives bearing pad recommendations which will help prevent spalling of the girder ends and seat edges. Sufficient prestressing steel must be placed in the girders, and sufficient reinforcement placed continuously across bent caps to satisfy tensile stresses caused by girder plastic prestress shortening between adjacent supports not having expansion joints.

Refer to the following pages for analysis and design instructions and examples.

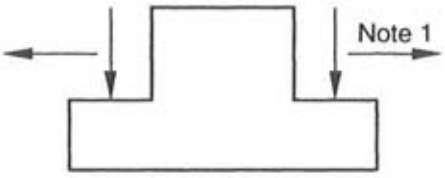
Following are some illustrations which will help visualize the design procedure and complexity for inverted T-Caps.



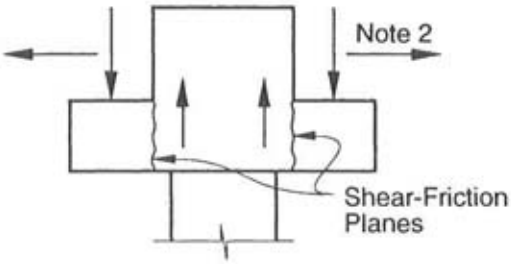
Bridge Elevation



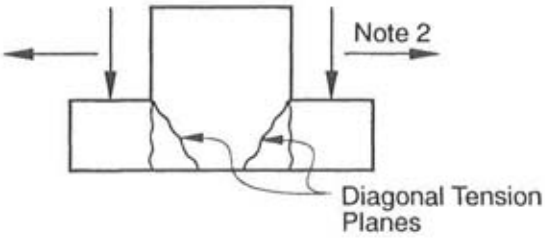
Typical Section



Loaded Section



Corbel Design At Column

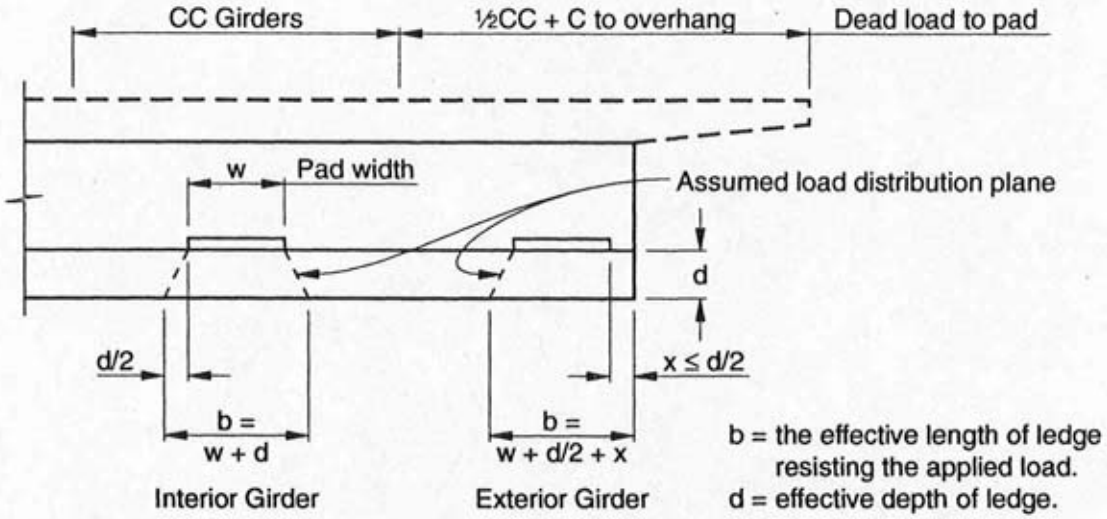


Corbel Design Between Columns

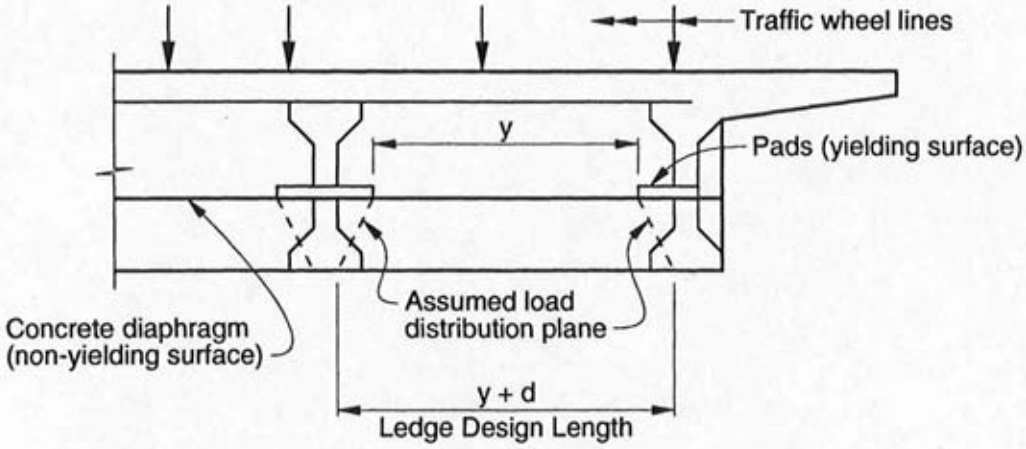
General Load Applications
(Sections through Cap)

Notes: 1. Horizontal plastic prestress shortening (creep) and thermal loads to be resisted by continuous reinforcement in deck.
2. Minimum tensile loads required by specifications.

Ledge Design Length



Dead Load Application
(Longitudinal View of Ledge)



Live Load Application
(Longitudinal View of Ledge)

A. Design Strategy

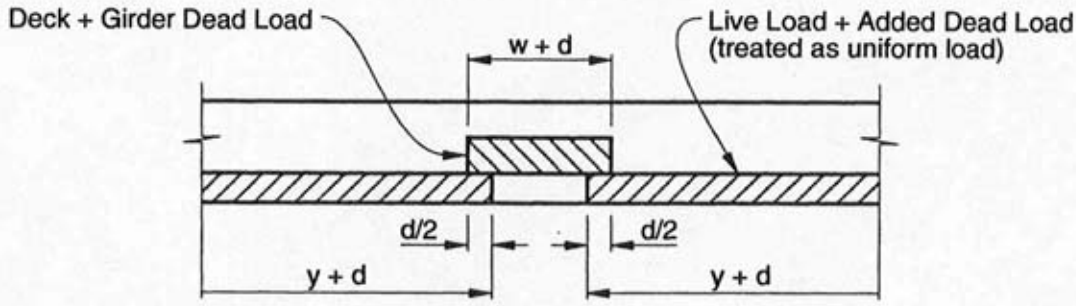
Dead load of girders and deck is transmitted directly to portion of ledge under girders through the pad, assuming diaphragm concrete is placed with deck concrete.

Live load and added dead load are transmitted through the deck and the girders to the end diaphragms into ledge.

1. Corbel Design

a. Under girder : $D_{(Deck + Girder)} + (L + I + D_{Added})$ (Minor Portion)

b. Between Girders : $(L + I + D_{Added})$ (Major Portion)



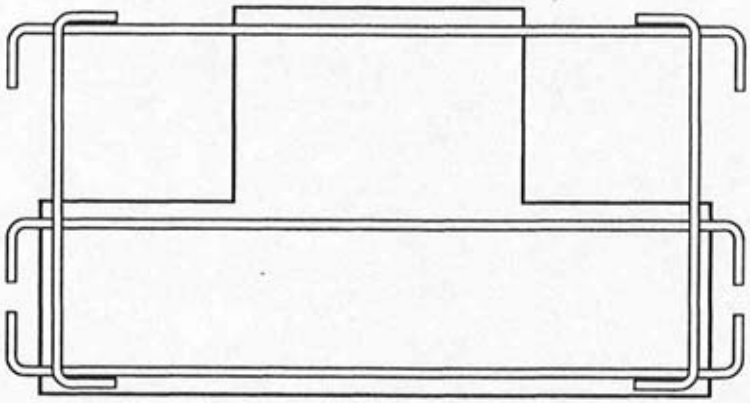
Typical Ledge Loading at Interior Girder

Note: Live load and added dead load distribution to ledge within width "w + d" should be assumed distributed uniformly across "w + d" for design purposes.

2. Bent Cap Design

Design should be similar to conventional bent caps (i.e., girders and wheel lines treated as concentrated loads). The inverted T-Section should be used for the shape of the design member, and all flexural and shear reinforcement should be fully contained within the section. One exception is that the top hooks of stirrups may extend into the deck slab.

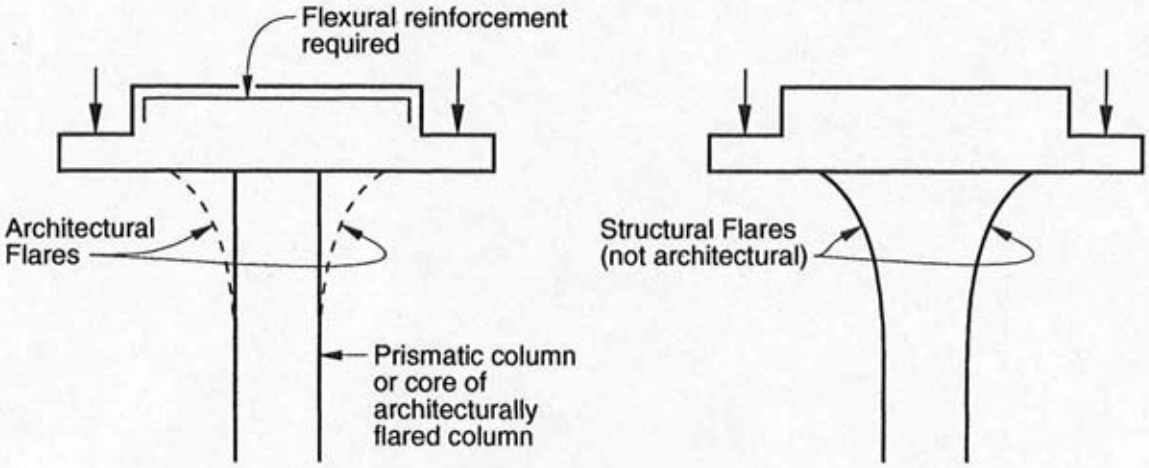
It is recommended that other nominal or tensile reinforcement be extended from the horizontal and vertical ledge faces between fixed girder ends to enhance continuity.



Typical where spans are continuous across bent cap.

Section of Cap between Girders

Designers must address flexural problems in the cross-sectional direction if the inverted-T becomes relatively wide (see illustrations below). Normally the cap is slightly wider than the column with only the ledges extending noticeably beyond the column face. The designer must be sure that the support is stable under all temporary construction stages.

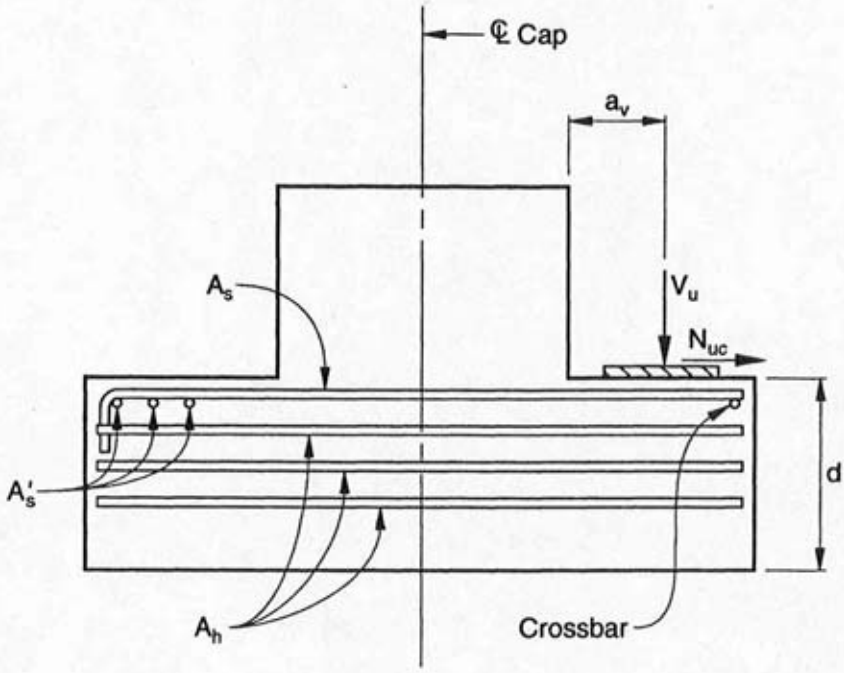


Examples of Non-Typical Inverted T-Caps

B. Design Commentary

Lower cap projections which support girders must meet the criteria for corbels. Corbel design limits and criteria are presented in *Bridge Design Specifications*, Article 8.16.6.8. The following criteria are to be considered:

1. The corbel criteria is suitable without modifications at columns which provide a compression reaction below the resisting shear-friction plane. An additional calculation for diagonal shear reinforcement is required at locations between columns if the column is inset more than normal from the shear-friction plane, or if a non-structural column flare, which could be lost in a seismic event, exists. Article 8.16.6.2.3, "Shear in Tension Members", should be used to satisfy diagonal shear.



Nomenclature Sketch

2. Use vertical and horizontal loads of:

   $V_u = \quad 1.30\,[DL + \frac{5}{3}(LL + I)_{HS}]$ *or*

   $1.30\,[DL + (LL + I)_P]$ — *Avoid widely spaced girders*

   N_{uc} = shear force as per *Memo to Designers* 7-1 for expansion ends. In no case shall N_{uc} be less than 0.2 V_u (ACI 11.9.4) at both expansion and fixed ends.

3. Check for the effect of the appropriate loads acting with the girder on the area below the girder. Determine the width of seat "b". For interior girders, "b" equals the width of the bearing pad plus the depth "d" of the corbel. For exterior girders, "b" equals the bearing pad plus one-half the depth "d" of the corbel plus edge distance to end of cap, not to exceed d/2. The seat reinforcement must be placed within the width of the seat, "b".

4. Compute A_s for both exterior and interior girders, and for ledge between girders. On either fixed ends or expansion ends which require additional pads between the girders, a load distribution scheme must be determined by the designer consistent with the construction sequence. The ledge must be reinforced accordingly.

5. Secondary tension bars shall be uniformly distributed in the upper two-thirds of the effective depth "d". They shall be placed parallel to the tension reinforcement "A_s" and have a cross-sectional area "A_h" not less than $0.5(A_s - A_n)$. See *Bridge Design Specifications*, Article 8.16.6.8.

6. Longitudinal corbel distribution bars, A'_s, shall be centered under all exterior bearing pads. Minimum area should be $A_s/2$. Uniformly space bars and extend them "d" beyond the seat width "b".

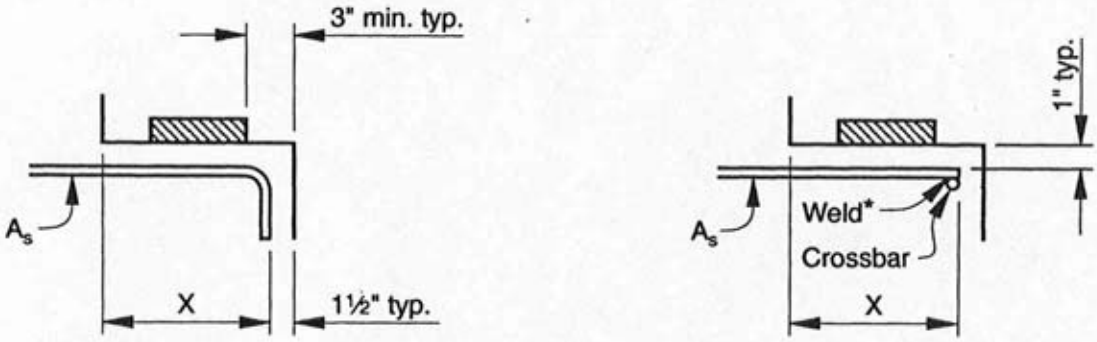
7. Keep pad a minimum of 3 inches from the edge of corbel to prevent high edge loadings.

8. Reinforcing steel at the edges of bearing seats may need specially detailed hooks to accommodate intersecting bars because of tight clearances.

9. A_s bar size should be chosen to allow required extension and development in the confined area. Crossbars welded to the ends of straight tension reinforcement (A_s) is an alternative when the radius of the hook bend is too large relative to ledger size. Size of crossbars should be that of the tension reinforcement. The following table shows allowable lengths for minimum "X" (see illustrations) based on hooked top bars without enclosure and $f'_c = 3{,}250$ psi.

Bar Size	Minimum "X"
#4	1'-3"
#5	1'-5"
#6	1'-9"
#7	2'-6"

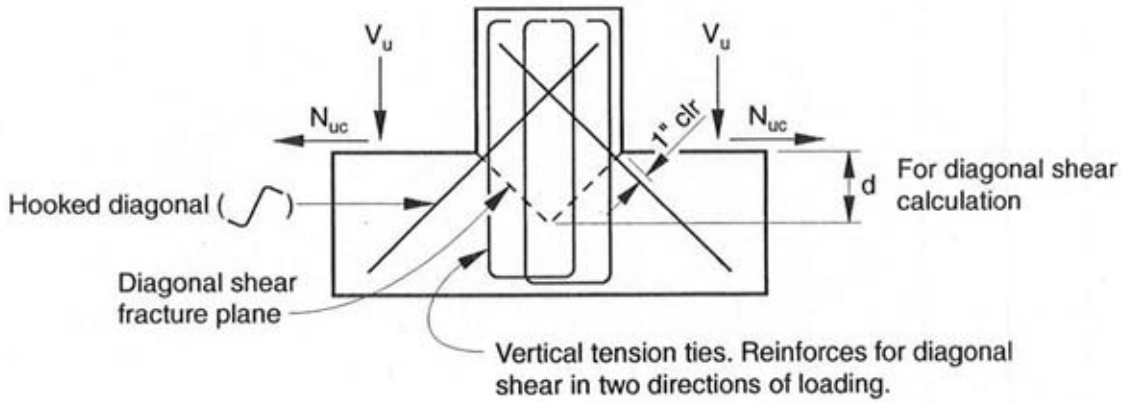
Note: It is not reasonable to use bars larger than #7 because the ledge extension would become excessively large. Closer girder spacing, deeper ledge section, or higher strength concrete are three methods to reduce the bar size.



*In accordance with Figure 11.9.6 in ACI 318-83 Commentary

Hook/Crossbar Illustrations

10. Check diagonal tension reinforcement requirements for loading on the beam ledge. Use *Bridge Design Specifications*, Articles 8.16.6.2.3 (Shear in Tension Members) and 8.16.6.3 (Shear Strength Provided by Shear Reinforcement).



Any combination of vertical and diagonal bars may be used to satisfy the condition. Diagonal bar areas must be corrected for the angle of the bar to an effective area.

These shear bars are not in addition to the cap shear stirrups from a bent analysis. The corbel loads used to satisfy the diagonal shear are the same loads used to analyze the bent. The analysis requiring the greatest area of reinforcement per unit length of bent cap should be used.

C. Details

Sufficient plan details must be provided to show all reinforcement patterns and for all stages of construction. The details must clearly identify corbel and bent cap reinforcement at columns and between columns, for loads at pads and for loads in between pads. Care must be taken to assure that the corbel steel can be placed amongst the column bars and spiral. Bridge skews complicate the layering and interweaving of bars. Special attention by the designer is required to avoid conflicts.

Sections need to be shown for constructing the inverted T, and also for a final condition with girders in place and diaphragm concrete cast around the girder ends.

D. Design Example

Following is a design example using the foregoing criteria. The example should be considered a guide, and not a standard solution for all inverted T-Caps. Major widenings should be designed with a T-Cap independent from the existing cap using the foregoing criteria. Strip widenings requiring an existing cap extension should use existing reinforcement details, but improved to meet the foregoing design considerations

The latest OSD policies on bent cap joint shear are not considered in this example. The designer is responsible for performing a joint shear analysis, and provide supplemental reinforcement, as required, to satisfy load demands from the analysis.

Inverted "T" Bent Cap Design Example

I. Design Considerations

A. Flange/Ledge Design

(1) – flange punching shear at girder bearings

(2) – primary tension reinforcement

(3) – secondary tension reinforcement

(4) – corbel distribution reinforcement

(5) – diagonal tension

B. Overall Bent Cap Design

The inverted "T" bent cap should be designed for the following conditions.

- Max moment and associated shear and torsion
- Max shear and associated moment and torsion
- Max torsion and associated shear and moment

These items will not be addressed in this example.

II. Design Procedure and Example Problem: Inverted 'T' Bent Cap – Ledge Design

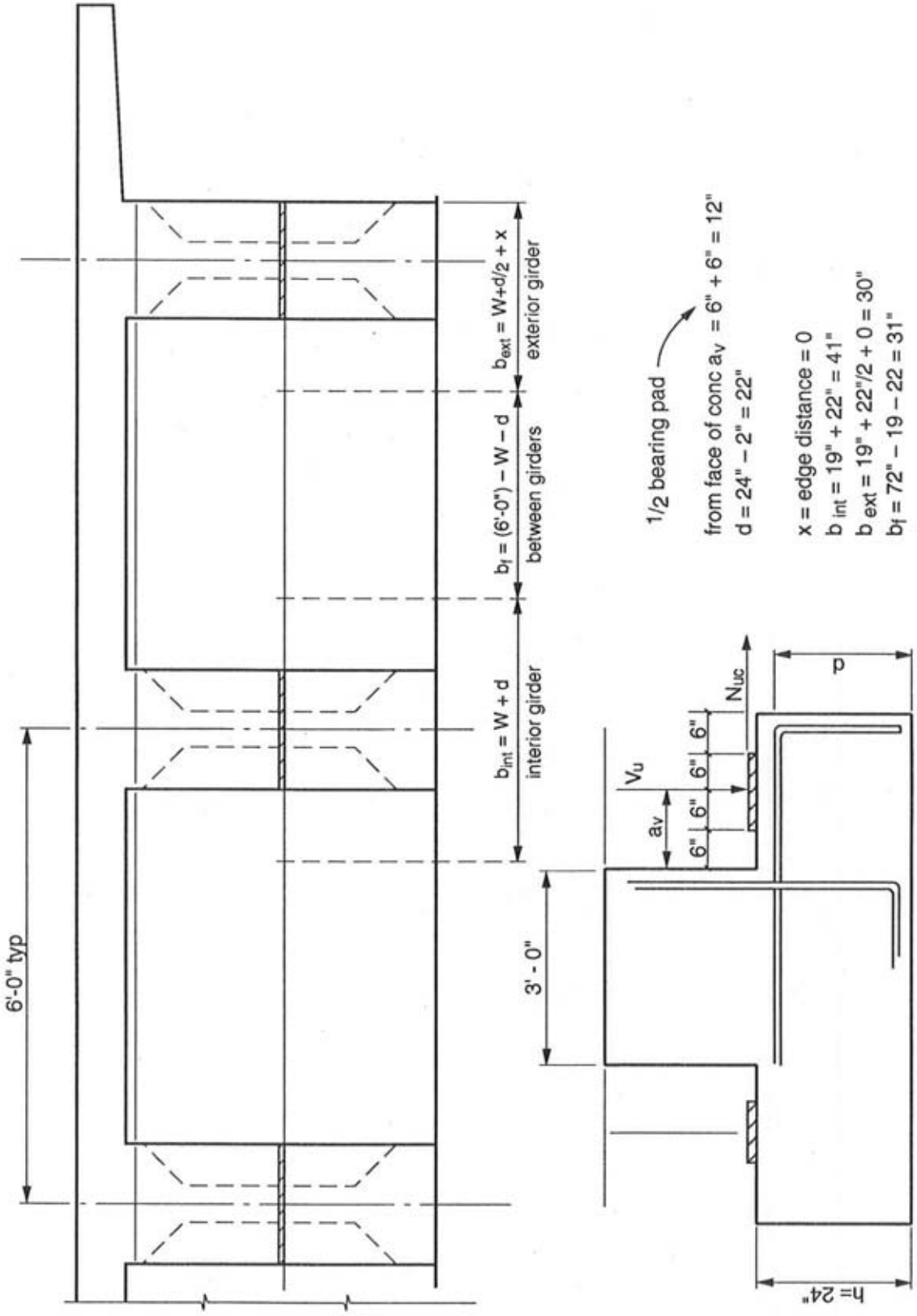
Design procedure for the ledge of the inverted "T" Bent Cap is as follows.

The ledge will be designed at 3 locations along the bent cap:

(a) interior girder

(b) exterior girder

(c) between girders


6'-0" typ
$b_{ext} = W + d/2 + x$
exterior girder
$b_f = (6'-0") - W - d$
between girders
$b_{int} = W + d$
interior girder
3' - 0"
1/2 bearing pad
from face of conc a_V = 6" + 6" = 12"
d = 24" − 2" = 22"
x = edge distance = 0
b_{int} = 19" + 22" = 41"
b_{ext} = 19" + 22"/2 + 0 = 30"
b_f = 72" − 19 − 22 = 31"
V_u
a_V
6"
6"
6"
N_{uc}
p
h= 24"

A. Given

Girder spacing = 6' - 0" o.c.

plain bearing pads $\frac{1}{2} \times 12 \times 19"$

h = 24" say d = 22"

Loads per girder – computed by tributary area method

DL per girder	130 k (includes weight of top deck)
Added DL per girder	30 k
$(LL + I)_{HS}$ per girder	80 k

"P" loads not considered in this example.

B. Design Loads

1. Vertical Shear, V_u

(W_u) Add DL + LL = 1.3 [30 k + $^5/_3$(80 k)]/6 feet

= 35.4 $^k/_1$

Interior Girder

Design for DL + Add DL + LL

$V_u = 1.3\ (130\ k) + (35.4\ ^k/_1)(^{41\ in}/_{12\ in/ft}) = 290\ k$

Exterior Girder

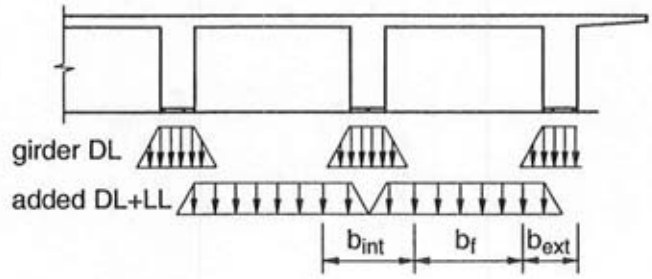
Design for DL + Add DL + LL

$V_u = 1.3\ (130\ k) = (35.4\ ^k/_1)(^{30\ in}/_{12in/ft}) = 258\ k$

Between Girders

Design for Add DL + LL

$V_u = (35.4\ k/1)(^{31\ in}/_{12\ in/ft}) = 92\ k$



2. Horizontal Shear, N_{uc}

$$N_{uc} \geq \begin{cases} \text{horizontal pad shear } \textit{(Memos to Designers 7.1)} \\ 0.2\ V_u \text{ (BDS Art. 8.16.6.8.3)} \end{cases}$$

pad shear $F_s = \frac{G(A)\Delta_s}{t}$ $\quad \Delta s = 0.5 \quad$ $Fs = \frac{(170psi)(12")(19")(0.5")}{0.5"} = 39\text{ k}$

Interior Girder

$$N_{uc} \geq \begin{cases} \text{pad shear} = 39\text{ k} \\ 0.2\ V_u = 0.2\ (290) = 58\text{ k} - \text{controls} \end{cases}$$

Exterior Girder

$$N_{uc} \geq \begin{cases} \text{pad shear} = 39\text{ k} \\ 0.2\ V_u = 0.2\ (258) = 52\text{ k - controls} \end{cases}$$

Between Girder

$$N_{uc} \geq \begin{cases} \text{pad shear} = 0 \\ 0.2\ V_u = 0.2\ (92) = 18\text{ k} \end{cases}$$

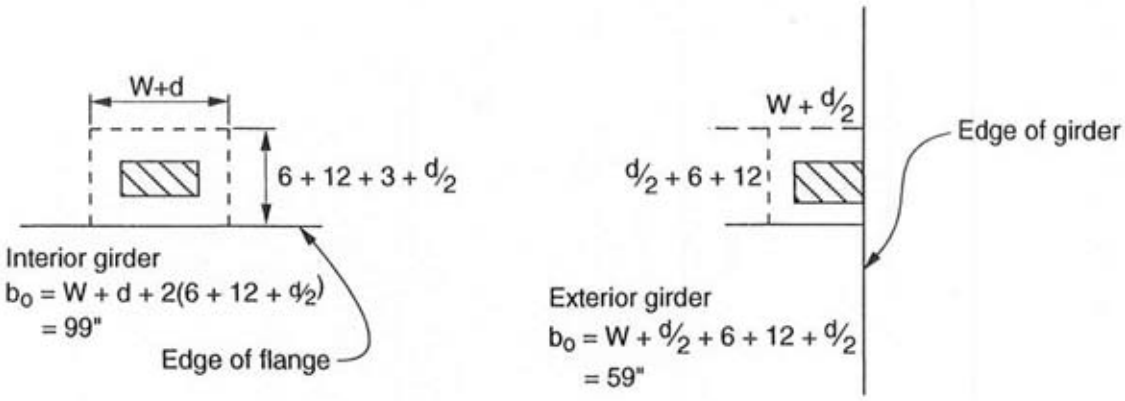
3. Summary

$(V_u)_{int.}$ girder = 290 k $\quad (N_{uc})_{int} = 58$ k

$(V_u)_{ext.}$ girder = 258 k $\quad (N_{uc})_{ext} = 52$ k

$(V_u)_{btwn.}$ girder = 92 k $\quad (N_{uc})_{btwn} = 18$ k

C. Flange Dimension Check



1. Check Punching Shear

$V_u < 0.85\ 4\sqrt{f'_c}\ b_o d$

Exterior Girder

$(V_u)_{ext} < (0.85)\ 4\sqrt{3250}\ (59)\ (22) = 251\ k$

$V_u = 258\ k > 251\ k \rightarrow NG$

Seat inadequate for punching shear. Try increasing depth of flange.

Try h = 30" d = 28"

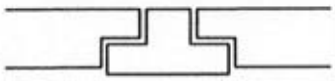
$(V_u)_{ext} \leq 0.85\ (4)\ \sqrt{3250}\ (59)(28) = 220\ k$

$(V_u)_{ext} = 258\ k \rightarrow$ okay

$(V_u)_{int} \leq 0.85\ (4)\ \sqrt{3250}\ (99)(28) = 537\ k$

$(V_u)_{int} = 290k \rightarrow$ okay

∴ Use h = 30 inches

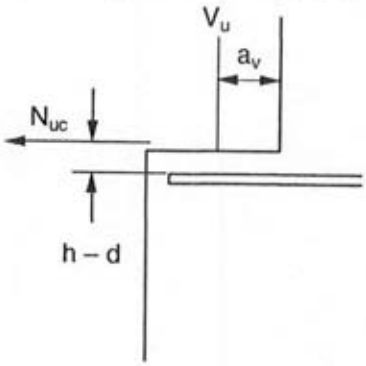


2. $^{av}/_{d} = {}^{12}/_{28} = 0.43 < 1.0$ (BDS Art. 8.16.6.8.1)

∴ Corbel design okay

D. Compute A_s – Primary Tension Reinforcement

(BDS Art. 8.16.6.8.3)



A_s to resist simultaneously $\begin{cases} \text{shear } V_u \\ \text{moment } V_u a_v + N_{uc}\,(h-d) \\ \text{tensile force } N_{uc} \end{cases}$

1. A_{vf} – Shear Friction Reinforcement

Interior Girder

$$V_n = \frac{V_u}{\phi} = \frac{290}{0.85} = 341 \text{ k}$$

$$V_n \le 0.2 f'_c A_{cv} = 0.2\,(3.25)(41 \text{ inches})(28) = 746 \text{ k} \rightarrow \text{okay}$$

$$\le 800\, A_{cv} = 0.800\,(41)\,(28) = 918 \text{ k} \rightarrow \text{okay}$$

$$A_{vf} = \frac{V_n}{f_y}\mu = \frac{341}{60(1.4)} = 4.06 \text{ sq. in.}$$

$\mu = 1.4$ for concrete placed monolithically

Exterior Girder

$$V_n = \frac{258}{0.85} = 304 \text{ k}$$

$$\begin{cases} V_n \le 0.2\, f'_c\, A_{cv} = 0.2\,(3.25)(30)(28) = 546 \text{ k} \rightarrow \text{okay} \\ V_n \le 800\, A_{cv} = 0.8\,(30)(28) = 672 \text{ k} \rightarrow \text{okay} \end{cases}$$

$$A_{vf} = \frac{304}{60(1.4)} = 3.62 \text{ sq. in.}$$

Between Girders

$$V_n = \frac{92}{0.85} = 108 \text{ k}$$

$$A_{vf} = \frac{108}{60(1.4)} = 1.29 \text{ sq. in.}$$

2. A_f – Flexural Reinforcement

Interior Girder

$M_u = [\ V_u\ a_v + N_{uc}\ (h\text{-}d)\ ] = (290)(12) + 58(2\text{ inches}) = 3596\text{ k-in.}$

$M_u = \phi A_f\ f_y\ [d - A_f\ f_y/(1.7\ f'_c\ b)]$

$$3596 = 0.85\ A_f(60)\left[28 - \frac{A_f(60)}{1.7(3.25)(41)}\right]$$

Solving for A_f gives $A_f = 2.59$ sq. in.

Exterior Girder

$M_u = (258)(12) + 52\ (2) = 3200\text{ k-in}$

$$3200 = 0.85\ A_f(60)\left[28 - \frac{A_f(60)}{1.7(3.25)(30)}\right] \rightarrow A_f = 2.31\text{ k-in}$$

Between Girders

$M_u = (92)(12) = 1104\text{ k-in}$

$$1104 = 0.85\ A_f(60)\left[28 - \frac{A_f(60)}{1.7(3.25)(31)}\right] \rightarrow A_f = 0.79\ \text{in}^2$$

3. A_n – Direct Tension Reinforcement

$$N_{uc} \le \phi A_n\ f_y \ \rightarrow\ A_n = \frac{N_{uc}}{0.85\ f_y}$$

Interior Girder $\quad A_n = \dfrac{58}{(0.85)(60)} = 1.14\text{ in}^2$

Exterior Girder $\quad A_n = \dfrac{52}{(0.85)(60)} = 1.02\text{ in}^2$

Between Girders $\quad A_n = 0$

4. Compute A_s

$$A_s \geq \begin{cases} \left(\dfrac{2A_{vf}}{3} + A_n\right) \\ (A_f + A_n) \\ 0.04\left(\dfrac{f'_c}{f_y}\right)bd = 0.0607b \end{cases}$$

(BDS Art. 8.16.6.8.5)

Interior Girders

$$A_s \geq \begin{cases} \left[\dfrac{2\times(4.06)}{3} + 1.14\right] = 3.85 \text{ in}^2 \\ [2.59 + 1.14] = 3.73 \\ 0.0607(41) = 2.49 \end{cases}$$

$A_s = 3.85$ sq. in. Use #6 tot. 9

Exterior Girders

$$A_s \geq \begin{cases} (2/3)(3.62) + 1.02 = 3.43 \text{ in}^2 \\ 2.31 + 1.02 = 3.33 \text{ in}^2 \\ 0.0607(30) = 1.82 \text{ in}^2 \end{cases}$$

$A_s = 3.43 \text{ in}^2$ Use #6 tot. 8

Between Girders

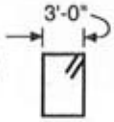
$$A_s \geq \begin{cases} [(2/3)(1.29)] = 0.86 \text{ in}^2 \\ 0.79 \text{ in}^2 \\ 0.0607(31) = 1.88 \text{ in}^2 \end{cases}$$

$A_s = 1.88 \text{ in}^2$ Use #6 tot. 5

E. Compute A_h – Shear Reinforcement (Secondary Tension Reinforcement)

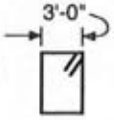
$A_h \geq 0.5\ (A_s - A_n)$ (BDS Art. 8.16.6.8.4)

Interior Girder

$A_h = 0.5(3.85 - 1.14) = 1.36\ in^2$ Use #5 

tot 3

Exterior Girders

$A_h = 0.5\ (3.43 - 1.02) = 1.21\ in^2$ Use #5 

tot 3

Between Girders

$A_h = 0.5\ (1.88) = 0.94\ in^2$ Use #5 tot 4

F. Compute A'_s – Longitudinal Corbel Distribution Reinforcement

Exterior Girder

$(A'_s)_{min} = 0.5\ A_s$

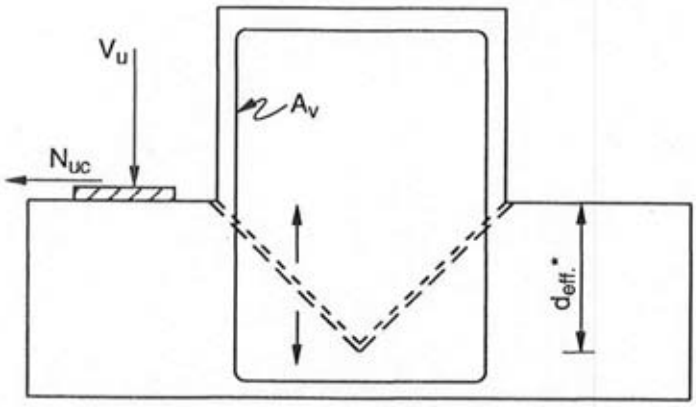
$A'_s = 0.5\ (3.43)$ sq. in. $= 1.72\ in^2$ Use 4 #6 each end.

Other Locations

Provide minimum distribution reinforcement 2 #5 bars.

G. Compute A_v – Diagonal Tension Reinforcement

Diagonal Tension reinforcement is required between columns to cross the diagonal crack. At column supports the shear can be carried through column steel.



$V_u = \phi(V_s + V_c)$

V_c is reduced for concrete in tension

*$d_{eff.}$ is used in calculations of V_c.

1. At Girders – Assume interior girder controls

$$V_c = 2\left[1 + \frac{N_u}{(500A_g)}\right]\sqrt{f'_c}\, b_w d_{eff}$$ (BDS Art. 8.16.6.2.3.)

$N_u = -V_u = -290$ k (tension)

$d_{eff} = 18$ inches

$A_g = (18)(41 \text{ inches}) = 738 \text{ in}^2$

$$V_c = 2\left[1 - \frac{290}{0.5(738)}\right]\sqrt{3250}\,(41)(18) = 18 \text{ k}$$

$(V_s)_{req.} = \frac{V_u}{\phi} - V_c = \frac{290}{0.85} - 18 = 323$ k This force is resisted by reinforcement crossing the tension crack.

Try #6 ⊏⊐ at 18 inches max. bent cap stirrups and 4 #7 ⁄ at each girder

#6 ⊏⊐ $A = (6 \text{ legs})(0.44 \text{ in}^2) = 2.64 \text{ in}^2$ (6 legs effective within b_{int} = 41 inches)

4 #7 ⁄ $A = 4(0.60 \text{ in}^2)(\sin x + \cos x) = 3.38 \text{ in}^2$ for $x = 45°$ (BDS Art. 8.16.6.3.3)

$A_{tot} = 2.64 + 3.38 = 6.02 \text{ in}^2$

$(V_s)_{prov} = A_s \times f_y = (6.02 \text{ in}^2)(60 \text{ ksi}) = 361$ k

$(V_s)_{prov} > (V_s)_{req} \rightarrow$ okay

∴ Use 4 #7 ⁄ @ each girder

#6 ⊏⊐ @ 18 inches max. bent cap stirrups

2. Between Girders

$N_u = -V_u = -92$ k

$$V_c = 2\left(1 - \frac{92}{0.5(738)}\right)\sqrt{3250}\ (41)(18) = 63 \text{ k}$$

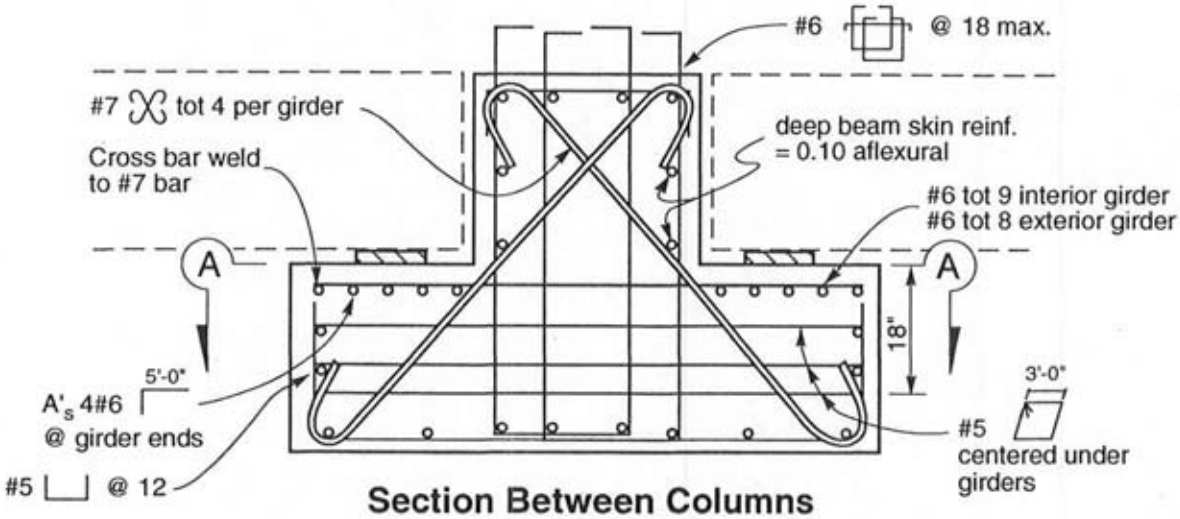
$$(V_s)_{req} = \frac{V_u}{\phi} - V_c = \frac{92}{0.85} - 63 = 45 \text{ k}$$

using #6 ⊏⊐ @ 18 along cap

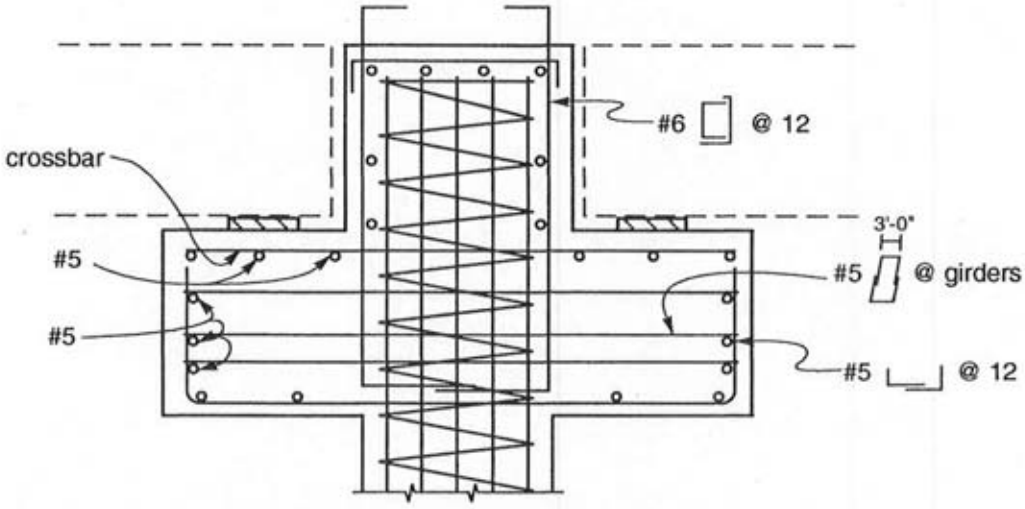
for b = 30 inches, 4 legs effective

$V_s = 4(0.44 \text{ in}^2)(60 \text{ ksi}) = 105 \text{ k} > (V_s)_{req} \rightarrow$ okay

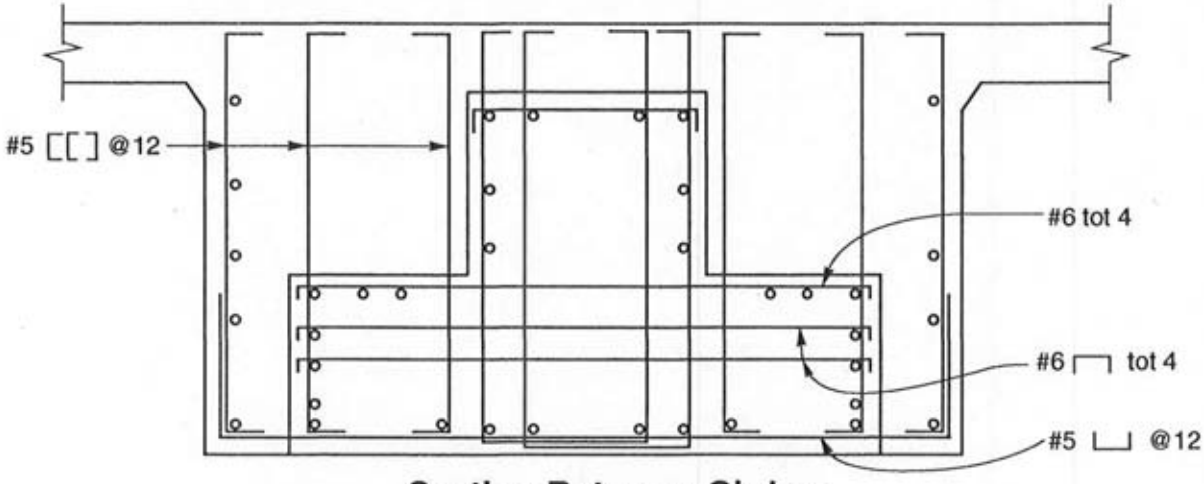
∴ No diagonal bars required



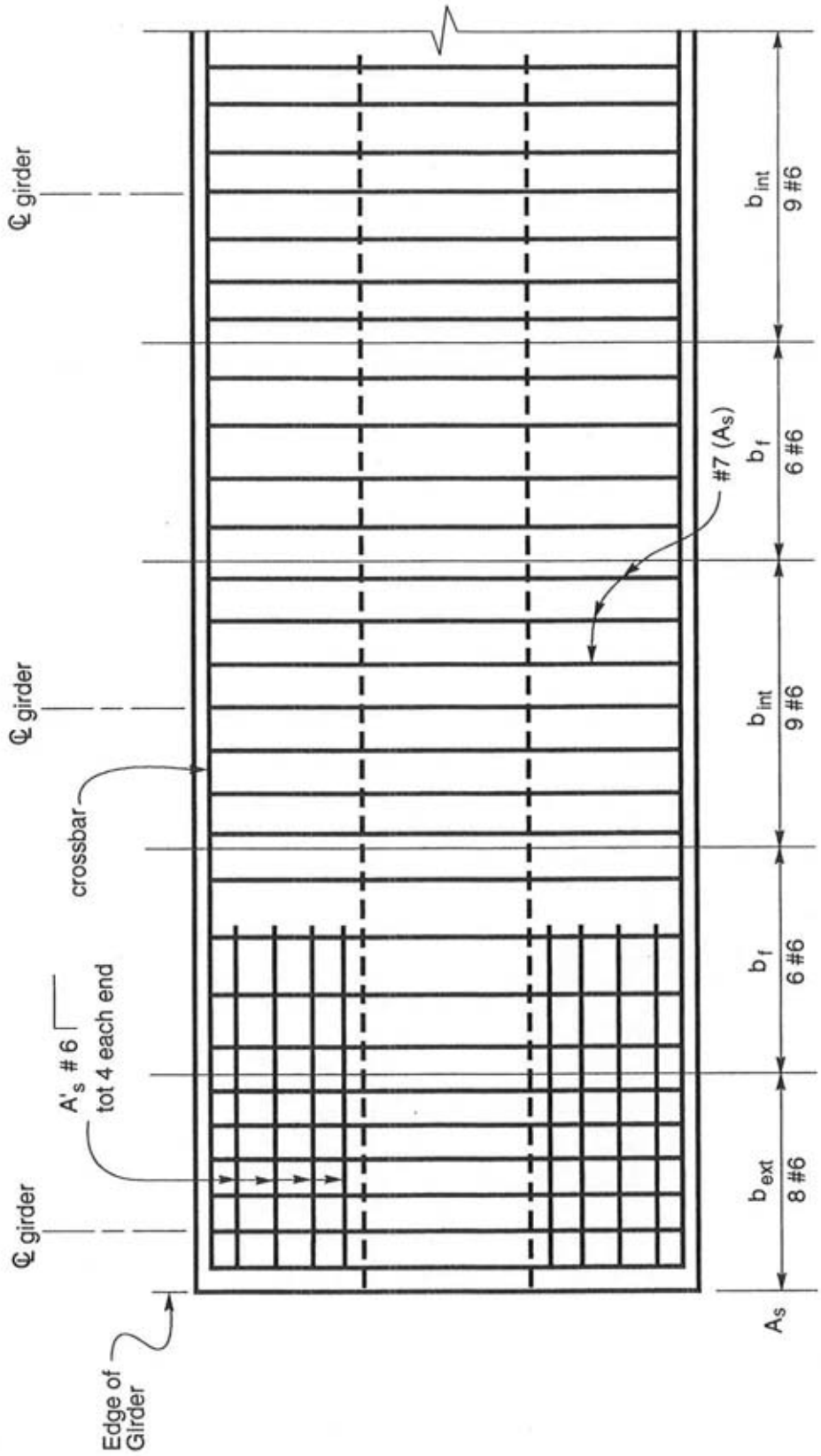
Section Between Columns



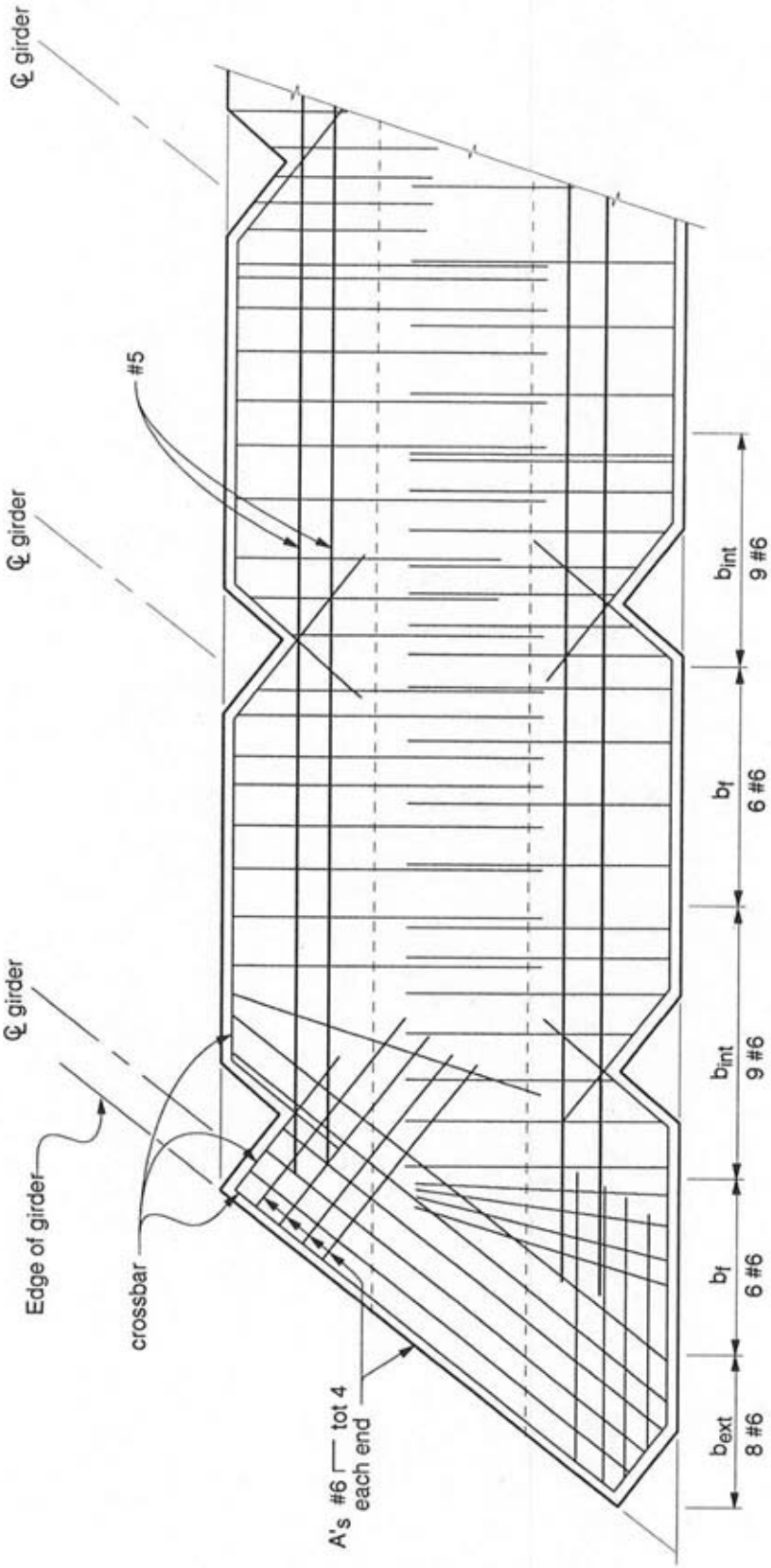
Section @ Column



Section Between Girders

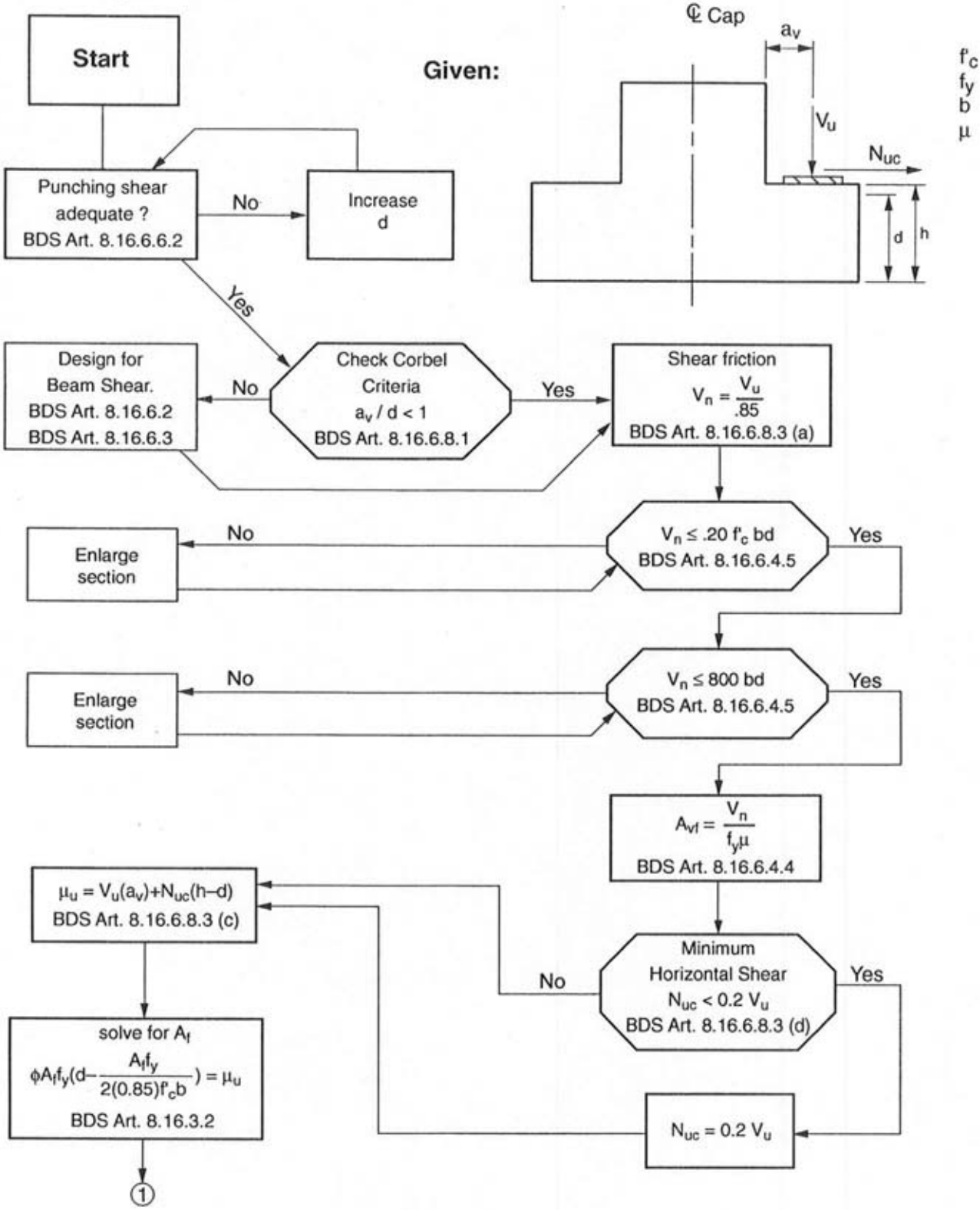


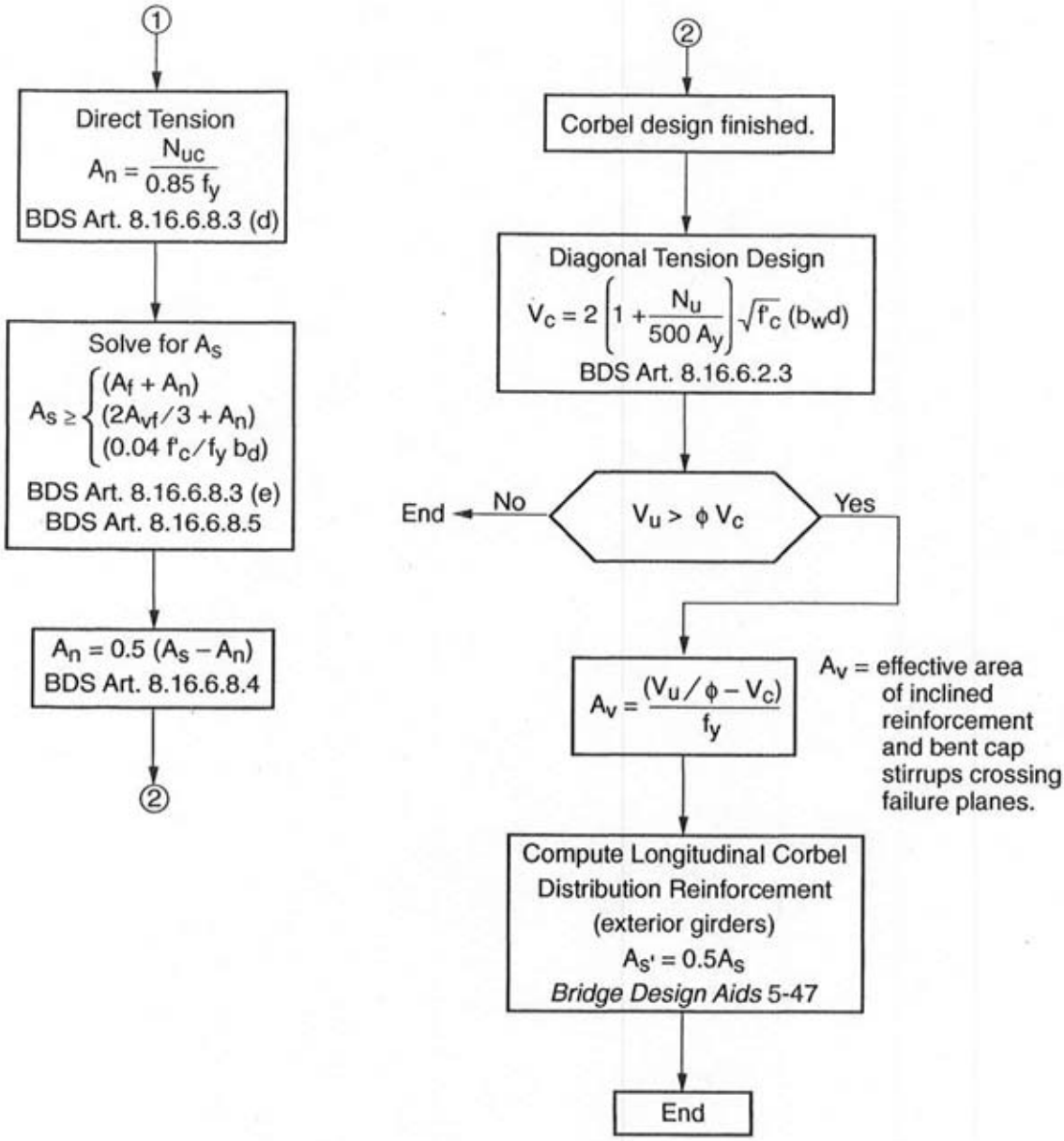
Section A-A (Orthogonal)



Section A-A (Skewed)

III. Design Flow Chart




1
Direct Tension
$A_n = \frac{N_{uc}}{0.85 f_y}$
BDS Art. 8.16.6.8.3 (d)
Solve for A_s
$A_s \geq \begin{cases} (A_f + A_n) \\ (2A_{vf}/3 + A_n) \\ (0.04 f'_c/f_y b_d) \end{cases}$
BDS Art. 8.16.6.8.3 (e)
BDS Art. 8.16.6.8.5
$A_n = 0.5 (A_s - A_n)$
BDS Art. 8.16.6.8.4
2
2
Corbel design finished.
Diagonal Tension Design
$V_c = 2\left(1 + \frac{N_u}{500 A_y}\right)\sqrt{f'_c}\,(b_w d)$
BDS Art. 8.16.6.2.3
$V_u > \phi V_c$
No
End
Yes
$A_v = \frac{(V_u/\phi - V_c)}{f_y}$
A_v = effective area of inclined reinforcement and bent cap stirrups crossing failure planes.
Compute Longitudinal Corbel Distribution Reinforcement (exterior girders)
$A_{s'} = 0.5A_s$
Bridge Design Aids 5-47
End

AREAS AND PERIMETERS FOR VARIOUS BAR SIZES AND NUMBER OF BARS

TOP NUMBERS ARE AREAS BOTTOM NUMBERS ARE PERIMETERS

Size / No.	#3	#4	#5	#6	#7	#8	#9	#10	#11	#14	#18	Size / No.
1	0.11 1.18	0.20 1.57	0.31 1.96	0.44 2.36	0.60 2.75	0.79 3.14	1.00 3.54	1.27 3.99	1.56 4.43	2.25 5.32	4.00 7.09	1
2	0.22 2.36	0.40 3.14	0.62 3.93	0.88 4.71	1.20 5.50	1.58 6.28	2.00 7.09	2.54 7.98	3.12 8.86	4.50 10.63	8.00 14.18	2
3	0.33 3.53	0.60 4.71	0.93 5.89	1.32 7.07	1.80 8.25	2.37 9.43	3.00 10.63	3.81 11.97	4.68 13.29	6.75 15.95	12.00 21.26	3
4	0.44 4.71	0.80 6.28	1.24 7.85	1.76 9.42	2.40 11.00	3.16 12.57	4.00 14.18	5.08 15.96	6.24 17.72	9.00 21.26	16.00 28.35	4
5	0.55 5.89	1.00 7.86	1.55 9.82	2.20 11.78	3.00 13.75	3.95 15.71	5.00 17.72	6.35 19.95	7.80 22.15	11.25 26.58	20.00 35.44	5
6	0.66 7.07	1.20 9.43	1.86 11.78	2.64 14.14	3.60 16.49	4.74 18.85	6.00 21.26	7.62 23.94	9.36 26.58	13.50 31.90	24.00 42.53	6
7	0.77 8.25	1.40 11.00	2.17 13.74	3.08 16.49	4.20 19.24	5.53 21.99	7.00 24.81	8.89 27.93	10.92 31.01	15.75 37.21	28.00 49.62	7
8	0.88 9.42	1.60 12.57	2.48 15.70	3.52 18.85	4.80 21.99	6.32 25.14	8.00 28.35	10.16 31.92	12.48 35.44	18.00 42.53	32.00 56.70	8
9	0.99 10.60	1.80 14.14	2.79 17.67	3.96 21.20	5.40 24.74	7.11 28.28	9.00 31.90	11.43 35.91	14.04 39.87	20.25 47.84	36.00 63.79	9
10	1.10 11.78	2.00 15.71	3.10 19.63	4.40 23.56	6.00 27.49	7.90 31.42	10.00 35.44	12.70 39.90	15.60 44.30	22.50 53.16	40.00 70.88	10
11	1.21 12.96	2.20 17.28	3.41 21.59	4.84 25.92	6.60 30.24	8.69 34.56	11.00 38.98	13.97 43.89	17.16 48.73	24.75 58.48	44.00 77.97	11
12	1.32 14.14	2.40 18.85	3.72 23.56	5.28 28.27	7.20 32.99	9.48 37.70	12.00 42.53	15.24 47.88	18.72 53.16	27.00 63.79	48.00 85.06	12
13	1.43 15.31	2.60 20.42	4.03 25.52	5.72 30.63	7.80 35.74	10.27 40.85	13.00 46.07	16.51 51.87	20.28 57.59	29.25 69.11	52.00 92.14	13
14	1.54 16.49	2.80 21.99	4.34 27.48	6.16 32.98	8.40 38.49	11.06 43.99	14.00 49.62	17.78 55.86	21.84 62.02	31.50 74.42	56.00 99.23	14
15	1.65 17.67	3.00 23.57	4.65 29.45	6.60 35.34	9.00 41.24	11.85 47.13	15.00 53.16	19.05 59.85	23.40 66.45	33.75 79.74	60.00 106.32	15
16	1.76 18.85	3.20 25.14	4.96 31.41	7.04 37.70	9.60 43.98	12.64 50.27	16.00 56.70	20.32 63.84	24.96 70.88	36.00 85.06	64.00 113.41	16
17	1.87 20.03	3.40 26.71	5.27 33.37	7.48 40.05	10.20 46.73	13.43 53.41	17.00 60.25	21.59 67.83	26.52 75.31	38.25 90.37	68.00 120.50	17
18	1.98 21.20	3.60 28.28	5.58 35.33	7.92 42.41	10.80 49.48	14.22 56.56	18.00 63.79	22.86 71.82	28.08 79.74	40.50 95.69	72.00 127.58	18
19	2.09 22.38	3.80 29.85	5.89 37.30	8.36 44.76	11.40 52.23	15.01 59.70	19.00 67.34	24.13 75.81	29.64 84.17	42.75 101.00	76.00 134.67	19
20	2.20 23.56	4.00 31.42	6.20 39.26	8.80 47.12	12.00 54.98	15.80 62.84	20.00 70.88	25.40 79.80	31.20 88.60	45.00 106.32	80.00 141.76	20
21	2.31 24.74	4.20 32.99	6.51 41.22	9.24 49.48	12.60 57.73	16.59 65.98	21.00 74.42	26.67 83.79	32.76 93.03	47.25 111.64	84.00 148.85	21
22	2.42 25.92	4.40 34.56	6.82 43.19	9.68 51.83	13.20 60.48	17.38 69.12	22.00 77.97	27.94 87.78	34.32 97.46	49.50 116.95	88.00 155.94	22
23	2.53 27.09	4.60 36.13	7.13 45.15	10.12 54.19	13.80 63.23	18.17 72.27	23.00 81.51	29.21 91.77	35.88 101.89	51.75 122.27	92.00 163.02	23
24	2.64 28.27	4.80 37.70	7.44 47.11	10.56 56.54	14.40 65.98	18.96 75.41	24.00 85.06	30.48 95.76	37.44 106.32	54.00 127.58	96.00 170.11	24
25	2.75 29.45	5.00 39.28	7.75 49.08	11.00 58.90	15.00 68.73	19.75 78.55	25.00 88.60	31.75 99.75	39.00 110.75	56.25 132.90	100.00 177.20	25
26	2.86 30.63	5.20 40.85	8.06 51.04	11.44 61.26	15.60 71.47	20.54 81.69	26.00 92.14	33.02 103.74	40.56 115.18	58.50 138.22	104.00 184.29	26
27	2.97 31.81	5.40 42.42	8.37 53.00	11.88 63.61	16.20 74.22	21.33 84.83	27.00 95.69	34.29 107.73	42.12 119.61	60.75 143.53	108.00 191.38	27
28	3.08 32.98	5.60 43.99	8.68 54.96	12.32 65.97	16.80 76.97	22.12 87.98	28.00 99.23	35.56 111.72	43.68 124.04	63.00 148.85	112.00 198.46	28
29	3.19 34.16	5.80 45.56	8.99 56.93	12.76 68.32	17.40 79.72	22.91 91.12	29.00 102.78	36.83 115.71	45.24 128.47	65.25 154.16	116.00 205.55	29
30	3.30 35.34	6.00 47.13	9.30 58.89	13.20 70.68	18.00 82.47	23.70 94.26	30.00 106.32	38.10 119.70	46.80 132.90	67.50 159.48	120.00 212.64	30
No. / Size	#3	#4	#5	#6	#7	#8	#9	#10	#11	#14	#18	No / Size

AREAS AND PERIMETERS FOR VARIOUS BAR SIZES AND SPACING

TOP NUMBERS ARE AREAS BOTTOM NUMBERS ARE PERIMETERS

Spacing	#3	#4	#5	#6	#7	#8	#9	#10	#11	#14	#18	Spacing
3"	0.44 4.7	0.80 6.3	1.24 7.8	1.76 9.4	2.40 11.0	3.16 12.6	4.00 14.2					3"
3¼"	0.41 4.4	0.74 5.8	1.14 7.2	1.62 8.7	2.22 10.2	2.92 11.6	3.69 13.1					3¼"
3½"	0.38 4.0	0.69 5.4	1.06 6.7	1.51 8.1	2.06 9.4	2.71 10.8	3.43 12.1	4.36 13.7				3½"
3¾"	0.35 3.8	0.64 5.0	0.99 6.3	1.41 7.5	1.92 8.8	2.53 10.0	3.20 11.3	4.06 12.8	4.99 14.2			3¾"
4"	0.33 3.5	0.60 4.7	0.93 5.9	1.32 7.1	1.80 8.3	2.37 9.4	3.00 10.6	3.81 12.0	4.68 13.3	6.75 16.0		4"
4¼"	0.31 3.3	0.56 4.4	0.88 5.5	1.24 6.7	1.69 7.8	2.23 8.9	2.82 10.0	3.59 11.3	4.40 12.5	6.35 15.0	11.30 20.0	4¼"
4½"	0.29 3.1	0.53 4.2	0.83 5.2	1.17 6.3	1.60 7.3	2.11 8.4	2.67 9.5	3.39 10.6	4.16 11.8	6.00 14.2	10.68 18.9	4½"
4¾"	0.28 3.0	0.51 4.0	0.78 5.0	1.11 6.0	1.52 6.9	2.00 7.9	2.53 9.0	3.21 10.1	3.94 11.2	5.68 13.4	10.10 17.9	4¾"
5"	0.26 2.8	0.48 3.8	0.74 4.7	1.06 5.7	1.44 6.6	1.90 7.5	2.40 8.5	3.05 9.6	3.74 10.6	5.40 12.8	9.60 17.0	5"
5¼"	0.25 2.7	0.46 3.6	0.71 4.5	1.01 5.4	1.37 6.3	1.81 7.2	2.29 8.1	2.90 9.1	3.57 10.1	5.14 12.2	9.14 16.2	5¼"
5½"	0.24 2.6	0.44 3.4	0.68 4.3	0.96 5.1	1.31 6.0	1.72 6.9	2.18 7.7	2.77 8.7	3.40 9.7	4.91 11.6	8.73 15.5	5½"
5¾"	0.23 2.5	0.42 3.3	0.65 4.1	0.92 4.9	1.25 5.7	1.65 6.6	2.09 7.4	2.65 8.3	3.26 9.2	4.69 11.1	8.34 14.8	5¾"
6"	0.22 2.4	0.40 3.1	0.62 3.9	0.88 4.7	1.20 5.5	1.58 6.3	2.00 7.1	2.54 8.0	3.12 8.9	4.50 10.6	8.00 14.2	6"
6½"	0.20 2.2	0.37 2.9	0.57 3.6	0.81 4.4	1.11 5.1	1.46 5.8	1.85 6.5	2.35 7.4	2.88 8.2	4.15 9.8	7.39 13.1	6½"
7"	0.19 2.0	0.34 2.7	0.53 3.4	0.75 4.0	1.03 4.7	1.35 5.4	1.71 6.1	2.18 6.8	2.67 7.6	3.86 9.1	6.86 12.2	7"
7½"	0.18 1.9	0.32 2.5	0.50 3.1	0.70 3.8	0.96 4.4	1.26 5.0	1.60 5.7	2.03 6.4	2.50 7.1	3.60 8.5	6.40 11.3	7½"
8"	0.17 1.8	0.30 2.4	0.47 2.9	0.66 3.5	0.90 4.1	1.19 4.7	1.50 5.3	1.91 6.0	2.34 6.7	3.38 8.0	6.00 10.6	8"
8½"	0.16 1.7	0.28 2.2	0.44 2.8	0.62 3.3	0.85 3.9	1.12 4.4	1.41 5.0	1.79 5.6	2.20 6.3	3.18 7.5	5.65 10.0	8½"
9"	0.15 1.6	0.27 2.1	0.41 2.6	0.59 3.1	0.80 3.7	1.05 4.2	1.33 4.7	1.69 5.3	2.08 5.9	3.00 7.1	5.34 9.5	9"
9½"	0.14 1.5	0.25 2.0	0.39 2.5	0.56 3.0	0.76 3.5	1.00 4.0	1.26 4.5	1.60 5.0	1.97 5.6	2.84 6.7	5.06 9.0	9½"
10"	0.13 1.4	0.24 1.9	0.37 2.4	0.53 2.8	0.72 3.3	0.95 3.8	1.20 4.3	1.52 4.8	1.87 5.3	2.70 6.4	4.80 8.5	10"
10½"	0.13 1.3	0.23 1.8	0.35 2.2	0.50 2.7	0.69 3.1	0.90 3.6	1.14 4.0	1.45 4.6	1.78 5.1	2.57 6.1	4.57 8.1	10½"
11"	0.12 1.3	0.22 1.7	0.34 2.2	0.48 2.6	0.65 3.0	0.86 3.4	1.09 3.9	1.39 4.4	1.70 4.8	2.45 5.8	4.36 7.7	11"
11½"		0.21 1.6	0.32 2.0	0.46 2.5	0.63 2.9	0.82 3.3	1.04 3.7	1.33 4.2	1.63 4.6	2.35 5.6	4.17 7.4	11½"
12"		0.20 1.6	0.31 2.0	0.44 2.4	0.60 2.8	0.79 3.1	1.00 3.5	1.27 4.0	1.56 4.4	2.25 5.3	4.00 7.1	12"
13"		0.18 1.4	0.29 1.8	0.41 2.2	0.55 2.5	0.73 2.9	0.92 3.3	1.17 3.7	1.44 4.1	2.08 4.9	3.69 6.5	13"
14"		0.17 1.3	0.27 1.7	0.38 2.0	0.51 2.4	0.68 2.7	0.86 3.0	1.09 3.4	1.24 3.8	1.93 4.6	3.43 6.1	14"
15"		0.16 1.3	0.25 1.6	0.35 1.9	0.48 2.2	0.63 2.5	0.80 2.8	1.02 3.2	1.25 3.5	1.80 4.3	3.20 5.7	15"
16"		0.15 1.2	0.23 1.5	0.33 1.8	0.45 2.1	0.59 2.4	0.75 2.7	0.95 3.0	1.17 3.3	1.69 4.0	3.00 5.3	16"
17"		0.14 1.1	0.22 1.4	0.31 1.7	0.42 1.9	0.56 2.2	0.71 2.5	0.90 2.8	1.10 3.1	1.59 3.8	2.82 5.0	17"
18"		0.13 1.1	0.21 1.3	0.29 1.6	0.40 1.8	0.53 2.1	0.67 2.4	0.85 2.7	1.04 2.9	1.50 3.6	2.67 4.7	18"