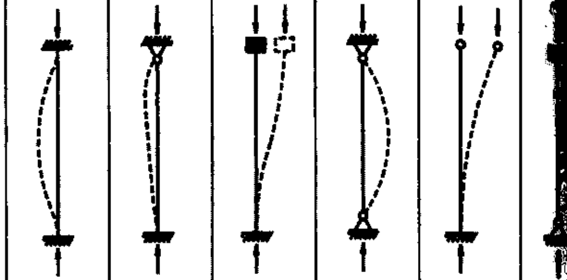
TABLE C-C2.2
Approximate Values of Effective Length Factor, K

Buckled shape of column is shown by dashed line.	(a)	(b)	(c)	(d)	(e)	
						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2
End condition code	Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free					

6. The stiffness parameter $L\sqrt{P/EI}$ of all columns is equal.
7. Joint restraint is distributed to the column above and below the joint in portion to EI/L for the two columns.
8. All columns buckle simultaneously.
9. No significant axial compression force exists in the girders.

The alignment chart for sidesway inhibited frames shown in Figure C-C2. based on the following equation:

$$\frac{G_AG_B}{4}(\pi/K)^2+\left(\frac{G_A+G_B}{2}\right)\left(1-\frac{\pi/K}{\tan(\pi/K)}\right)+\frac{2\tan(\pi/2K)}{(\pi/K)}-1=0$$

The alignment chart for sidesway uninhibited frames shown in Figure C-C2. based on the following equation:

$$\frac{G_AG_B(\pi/K)^2-36}{6(G_A+G_B)}-\frac{(\pi/K)}{\tan(\pi/K)}=0$$

where

$$G=\frac{\Sigma(E_cI_c/L_c)}{\Sigma(E_gI_g/L_g)}=\frac{\Sigma(EI/L)_c}{\Sigma(EI/L)_g}$$

The subscripts A and B refer to the joints at the ends of the column being cons ered. The symbol Σ indicates a summation of all members rigidly connected

that joint and lying in the plane in which buckling of the column is being considered. E_c is the modulus of the column, I_c is the moment of inertia of the column, and L_c is the unsupported length of the column. E_g is the modulus of the girder, I_g is the moment of inertia of the girder, and L_g is the unsupported length of the girder or other restraining member. I_c and I_g are taken about axes perpendicular to the plane of buckling being considered. The alignment chart is valid for different materials if an appropriate effective rigidity, EI, is used in the calculation of G.

For column ends supported by, but not rigidly connected to, a footing or foundation, G is theoretically infinity but unless designed as a true friction-free pin, may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.

Theoretical K values obtained from the alignment charts for various idealized end conditions, rotation fixed or free and translation fixed or free, are shown in Table C-C2.2 along with practical recommendations for use in actual design.

It is important to remember that the alignment charts are based on the assumptions of idealized conditions previously discussed and that these conditions seldom exist

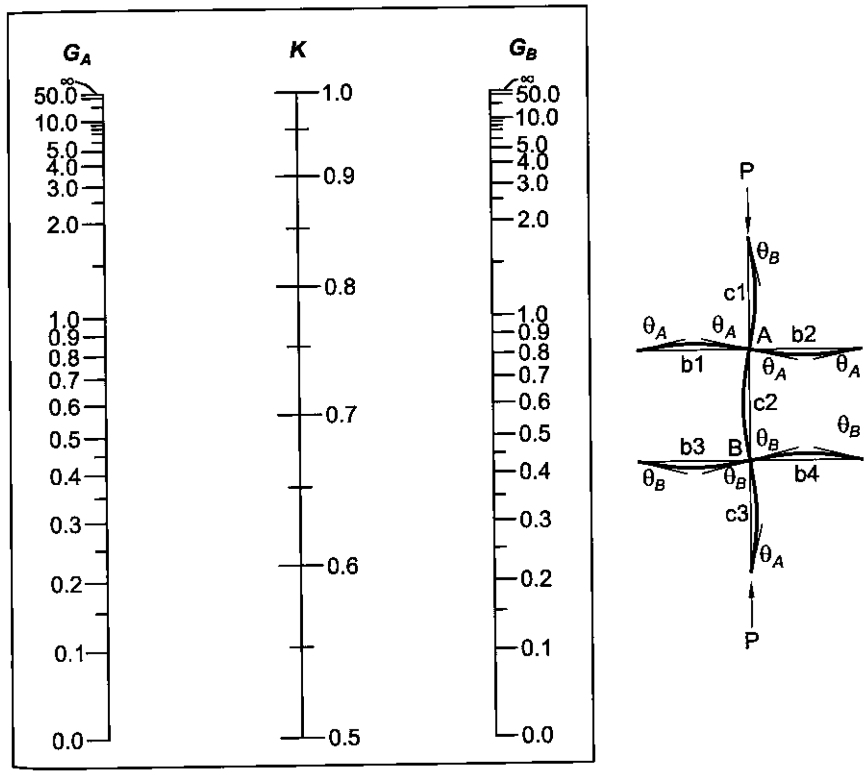


Fig. C-C2.3. Alignment chart—sidesway inhibited (braced frame).

W14

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

Shape		W14×													
Wt/ft		82		74		68		61		53		48		43[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			LRFD		LRFD		LRFD		LRFD		LRFD		LRFD		LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0		1080		980		899		806		702		636		562
	6		1020		922		844		757		633		573		511
	7		995		901		826		740		610		552		491
	8		970		878		804		721		585		529		470
	9		942		853		781		700		557		504		447
	10		912		826		755		677		528		477		423
	11		880		797		728		652		497		449		397
	12		846		766		700		626		465		420		371
	13		810		734		670		599		433		391		345
	14		774		701		639		572		401		361		319
	15		736		667		608		543		369		332		293
	16		698		632		576		515		338		304		267
	17		660		598		544		486		308		276		242
	18		621		563		512		457		278		250		218
	19		583		528		480		428		250		224		196
	20		546		494		448		400		226		202		177
	22		473		428		387		345		186		167		146
	24		403		365		329		293		157		140		123
	26		343		311		281		250		133		120		105
	28		296		268		242		215		115		103		90.2
	30		258		234		211		187		100		89.9		78.6
	32		227		205		185		165		88.1				
	34		201		182		164		146						
	36		179		162		146		130						
	38		161		146		131		117						
	40		145		131		119		105						
Properties															
P_{wo} (kips)			184		155		136		116		116		101		85.5
P_{wi} (kips/in.)			25.5		22.5		20.8		18.8		18.5		17.0		15.3
P_{wb} (kips)			302		208		163		120		115		89.5		64.6
P_{fb} (kips)			206		173		146		117		123		99.6		79.0
L_p (ft)		8.76		8.76		8.69		8.65		6.78		6.75		6.68	
L_r (ft)		33.1		31.0		29.3		27.5		22.2		21.1		20.0	
A_g (in.^{2})		24.0		21.8		20.0		17.9		15.6		14.1		12.6	
I_x (in.^{4})		881		795		722		640		541		484		428	
I_y (in.^{4})		148		134		121		107		57.7		51.4		45.2	
r_y (in.)		2.48		2.48		2.46		2.45		1.92		1.91		1.89	
Ratio r_x/r_y		2.44		2.44		2.44		2.44		3.07		3.06		3.08	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		25200		22800		20700		18300		15500		13900		12300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		4240		3840		3460		3060		1650		1470		1290	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

W12

Shape		W12×											
Wt/ft		336[h]		305[h]		279[h]		252[h]		230[h]		210	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			LRFD		LRFD		LRFD		LRFD		LRFD		LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0		4440		4030		3690		3330		3050		2780
	6		4310		3910		3570		3220		2940		2680
	7		4260		3860		3520		3180		2910		2650
	8		4200		3810		3470		3140		2870		2610
	9		4140		3750		3420		3090		2820		2570
	10		4070		3690		3360		3030		2770		2520
	11		4000		3620		3300		2970		2710		2470
	12		3920		3540		3230		2910		2650		2410
	13		3830		3460		3150		2840		2590		2360
	14		3740		3380		3080		2770		2520		2290
	15		3650		3290		3000		2690		2450		2230
	16		3550		3200		2910		2620		2380		2160
	17		3450		3110		2820		2540		2310		2100
	18		3350		3010		2730		2450		2230		2020
	19		3240		2910		2640		2370		2150		1950
	20		3130		2810		2550		2280		2070		1880
	22		2910		2610		2360		2110		1910		1730
	24		2690		2400		2170		1930		1750		1580
	26		2460		2190		1980		1760		1590		1430
	28		2240		1990		1790		1590		1430		1290
	30		2020		1790		1610		1430		1280		1150
	32		1820		1600		1430		1270		1140		1020
	34		1610		1420		1270		1120		1010		902
	36		1440		1270		1130		1000		898		805
	38		1290		1140		1020		897		806		722
	40		1170		1030		918		810		727		652
Properties													
P_{wo} (kips)			1580		1340		1170		993		857		737
P_{wi} (kips/in.)			88.8		81.3		76.5		69.8		64.3		59.0
P_{wb} (kips)			15000		11500		9590		7270		5680		4400
P_{fb} (kips)			2460		2060		1720		1420		1210		1020
L_p (ft)		12.3		12.1		11.9		11.8		11.7		11.6	
L_r (ft)		150		137		126		114		105		96.0	
A_g (in.^{2})		98.8		89.6		81.9		74.0		67.7		61.8	
I_x (in.^{4})		4060		3550		3110		2720		2420		2140	
I_y (in.^{4})		1190		1050		937		828		742		664	
r_y (in.)		3.47		3.42		3.38		3.34		3.31		3.28	
Ratio r_x/r_y		1.85		1.84		1.82		1.81		1.80		1.80	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		116000		102000		89000		77900		69300		61300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		34100		30100		26800		23700		21200		19000	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

W12

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

Shape		W12×											
Wt/ft		190		170		152		136		120		106	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	[redacted]	2510	[redacted]	2250	[redacted]	2010	[redacted]	1800	[redacted]	1590	[redacted]	1400
	6	[redacted]	2420	[redacted]	2170	[redacted]	1940	[redacted]	1730	[redacted]	1530	[redacted]	1350
	7	[redacted]	2390	[redacted]	2140	[redacted]	1910	[redacted]	1710	[redacted]	1510	[redacted]	1330
	8	[redacted]	2360	[redacted]	2110	[redacted]	1880	[redacted]	1680	[redacted]	1480	[redacted]	1310
	9	[redacted]	2320	[redacted]	2070	[redacted]	1850	[redacted]	1650	[redacted]	1460	[redacted]	1280
	10	[redacted]	2270	[redacted]	2030	[redacted]	1820	[redacted]	1620	[redacted]	1430	[redacted]	1260
	11	[redacted]	2230	[redacted]	1990	[redacted]	1780	[redacted]	1580	[redacted]	1390	[redacted]	1230
	12	[redacted]	2180	[redacted]	1940	[redacted]	1730	[redacted]	1540	[redacted]	1360	[redacted]	1200
	13	[redacted]	2120	[redacted]	1900	[redacted]	1690	[redacted]	1500	[redacted]	1320	[redacted]	1170
	14	[redacted]	2070	[redacted]	1840	[redacted]	1640	[redacted]	1460	[redacted]	1290	[redacted]	1130
	15	[redacted]	2010	[redacted]	1790	[redacted]	1600	[redacted]	1420	[redacted]	1250	[redacted]	1100
	16	[redacted]	1950	[redacted]	1730	[redacted]	1540	[redacted]	1370	[redacted]	1210	[redacted]	1060
	17	[redacted]	1880	[redacted]	1680	[redacted]	1490	[redacted]	1320	[redacted]	1160	[redacted]	1020
	18	[redacted]	1820	[redacted]	1620	[redacted]	1440	[redacted]	1280	[redacted]	1120	[redacted]	985
	19	[redacted]	1750	[redacted]	1560	[redacted]	1390	[redacted]	1230	[redacted]	1080	[redacted]	946
	20	[redacted]	1690	[redacted]	1500	[redacted]	1330	[redacted]	1180	[redacted]	1030	[redacted]	907
	22	[redacted]	1550	[redacted]	1380	[redacted]	1220	[redacted]	1080	[redacted]	944	[redacted]	828
	24	[redacted]	1420	[redacted]	1250	[redacted]	1110	[redacted]	979	[redacted]	855	[redacted]	749
	26	[redacted]	1280	[redacted]	1130	[redacted]	1000	[redacted]	881	[redacted]	768	[redacted]	672
	28	[redacted]	1150	[redacted]	1010	[redacted]	895	[redacted]	786	[redacted]	684	[redacted]	597
	30	[redacted]	1020	[redacted]	902	[redacted]	793	[redacted]	695	[redacted]	602	[redacted]	525
	32	[redacted]	904	[redacted]	794	[redacted]	698	[redacted]	611	[redacted]	530	[redacted]	462
	34	[redacted]	800	[redacted]	704	[redacted]	618	[redacted]	541	[redacted]	469	[redacted]	409
	36	[redacted]	714	[redacted]	628	[redacted]	551	[redacted]	483	[redacted]	418	[redacted]	365
	38	[redacted]	641	[redacted]	563	[redacted]	495	[redacted]	433	[redacted]	376	[redacted]	327
	40	[redacted]	578	[redacted]	508	[redacted]	446	[redacted]	391	[redacted]	339	[redacted]	295
Properties													
P_{wo} (kips)		[redacted]	618	[redacted]	518	[redacted]	435	[redacted]	365	[redacted]	302	[redacted]	242
P_{wi} (kips/in.)		[redacted]	53.0	[redacted]	48.0	[redacted]	43.5	[redacted]	39.5	[redacted]	35.5	[redacted]	30.5
P_{wb} (kips)		[redacted]	3190	[redacted]	2370	[redacted]	1760	[redacted]	1320	[redacted]	958	[redacted]	608
P_{fb} (kips)		[redacted]	847	[redacted]	684	[redacted]	551	[redacted]	439	[redacted]	343	[redacted]	276
L_p (ft)		11.5		11.4		11.3		11.2		11.1		11.0	
L_r (ft)		87.3		78.5		70.6		63.3		56.5		50.7	
A_g (in.^{2})		55.8		50.0		44.7		39.9		35.3		31.2	
I_x (in.^{4})		1890		1650		1430		1240		1070		933	
I_y (in.^{4})		589		517		454		398		345		301	
r_y (in.)		3.25		3.22		3.19		3.16		3.13		3.11	
Ratio r_x/r_y		1.79		1.78		1.77		1.77		1.76		1.76	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		54100		47200		40900		35500		30600		26700	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		16900		14800		13000		11400		9870		8620	

[redacted]	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 50 ksi

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

W12

Shape		W12×									
Wt/ft		96		87		79		72		65	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	[redacted]	1270	[redacted]	1150	[redacted]	1040	[redacted]	951	[redacted]	859
	6	[redacted]	1220	[redacted]	1110	[redacted]	1000	[redacted]	913	[redacted]	824
	7	[redacted]	1200	[redacted]	1090	[redacted]	987	[redacted]	899	[redacted]	811
	8	[redacted]	1180	[redacted]	1070	[redacted]	971	[redacted]	884	[redacted]	798
	9	[redacted]	1160	[redacted]	1050	[redacted]	952	[redacted]	867	[redacted]	782
	10	[redacted]	1140	[redacted]	1030	[redacted]	932	[redacted]	849	[redacted]	765
	11	[redacted]	1110	[redacted]	1010	[redacted]	910	[redacted]	828	[redacted]	747
	12	[redacted]	1080	[redacted]	980	[redacted]	887	[redacted]	807	[redacted]	727
	13	[redacted]	1050	[redacted]	953	[redacted]	862	[redacted]	784	[redacted]	706
	14	[redacted]	1020	[redacted]	924	[redacted]	836	[redacted]	761	[redacted]	685
	15	[redacted]	990	[redacted]	895	[redacted]	809	[redacted]	736	[redacted]	662
	16	[redacted]	957	[redacted]	864	[redacted]	781	[redacted]	710	[redacted]	639
	17	[redacted]	923	[redacted]	833	[redacted]	752	[redacted]	684	[redacted]	615
	18	[redacted]	888	[redacted]	801	[redacted]	723	[redacted]	657	[redacted]	591
	19	[redacted]	852	[redacted]	769	[redacted]	694	[redacted]	630	[redacted]	566
	20	[redacted]	816	[redacted]	736	[redacted]	664	[redacted]	603	[redacted]	541
	22	[redacted]	744	[redacted]	670	[redacted]	603	[redacted]	548	[redacted]	491
	24	[redacted]	672	[redacted]	605	[redacted]	544	[redacted]	493	[redacted]	442
	26	[redacted]	602	[redacted]	541	[redacted]	486	[redacted]	440	[redacted]	393
	28	[redacted]	534	[redacted]	479	[redacted]	430	[redacted]	389	[redacted]	347
	30	[redacted]	469	[redacted]	420	[redacted]	376	[redacted]	340	[redacted]	303
	32	[redacted]	412	[redacted]	369	[redacted]	331	[redacted]	299	[redacted]	267
	34	[redacted]	365	[redacted]	327	[redacted]	293	[redacted]	265	[redacted]	236
	36	[redacted]	326	[redacted]	292	[redacted]	261	[redacted]	236	[redacted]	211
	38	[redacted]	292	[redacted]	262	[redacted]	234	[redacted]	212	[redacted]	189
	40	[redacted]	264	[redacted]	236	[redacted]	212	[redacted]	191	[redacted]	171
Properties											
P_{wo} (kips)		[redacted]	206	[redacted]	181	[redacted]	157	[redacted]	136	[redacted]	117
P_{wi} (kips/in.)		[redacted]	27.5	[redacted]	25.8	[redacted]	23.5	[redacted]	21.5	[redacted]	19.5
P_{wb} (kips)		[redacted]	445	[redacted]	366	[redacted]	278	[redacted]	213	[redacted]	159
P_{fb} (kips)		[redacted]	228	[redacted]	185	[redacted]	152	[redacted]	126	[redacted]	103
L_p (ft)		10.9		10.8		10.8		10.7		11.9	
L_r (ft)		46.6		43.0		39.9		37.4		35.1	
A_g (in.^{2})		28.2		25.6		23.2		21.1		19.1	
I_x (in.^{4})		833		740		662		597		533	
I_y (in.^{4})		270		241		216		195		174	
r_y (in.)		3.09		3.07		3.05		3.04		3.02	
Ratio r_x/r_y		1.76		1.75		1.75		1.75		1.75	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		23800		21200		18900		17100		15300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		7730		6900		6180		5580		4980	

[redacted]	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

5

COMPOSITE PIPE 3½–PIPE 3

Table 4–20 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Pipe

F_y = 35 ksi
f'_c = 5 ksi

Shape		Pipe 3½				Pipe 3					
		XS		Std		XXS		XS		Std	
Wall Thickness, in.		0.296		0.211		0.559		0.280		0.201	
Steel Wt/ft		12.5		9.12		18.6		10.3		7.58	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			LRFD		LRFD		LRFD		LRFD		LRFD
Effective length (KL) (ft)	0		123		102		151		98.5		81.5
	6		102		84.4		117		77.5		63.9
	7		95.6		78.8		107		71.0		58.5
	8		88.5		72.9		96.1		64.3		52.9
	9		81.2		66.8		85.1		57.4		47.3
	10		73.6		60.5		74.4		50.5		41.5
	11		66.1		54.2		64.1		43.9		36.0
	12		58.8		48.1		54.3		37.6		30.8
	13		51.7		42.3		46.3		32.1		26.2
	14		45.0		36.7		39.9		27.6		22.6
	15		39.2		31.9		34.8		24.1		19.7
	16		34.4		28.1		30.5		21.2		17.3
	17		30.5		24.9		27.1		18.7		15.3
	18		27.2		22.2				16.7		13.7
	19		24.4		19.9				15.0		12.3
	20		22.0		18.0						
	21		20.0		16.3						
	22				14.8						
Properties											
$\phi_b M_n$	kip-ft		10.7		7.96		12.8		7.64		5.75
$P_e(KL)^2/10^4$	kip-in.²	193		157		171		119		97.1	
	LRFD	Note: Heavy line indicates Kl/r equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$										

Table 4–21
Stiffness Reduction Factor τ_a

	LRFD	F_y, ksi									
$\dfrac{P_s}{A_g}$	$\dfrac{P_u}{A_g}$	35		38		42		46		50	
			LRFD		LRFD		LRFD		LRFD		LRFD
	45		–		–		–		–		–
	44		–		–		–		–		0.0599
	43		–		–		–		–		0.118
	42		–		–		–		–		0.175
	41		–		–		–		0.0262		0.231
	40		–		–		–		0.0905		0.285
	39		–		–		–		0.153		0.338
	38		–		–		–		0.214		0.389
	37		–		–		0.0570		0.274		0.438
	36		–		–		0.127		0.331		0.486
	35		–		–		0.194		0.387		0.532
	34		–		–		0.260		0.441		0.577
	33		–		–		0.323		0.492		0.620
	32		–		0.0334		0.384		0.542		0.660
	31		0.0429		0.115		0.443		0.590		0.699
	30		0.127		0.194		0.500		0.636		0.736
	29		0.207		0.270		0.554		0.679		0.771
	28		0.285		0.344		0.606		0.720		0.804
	27		0.360		0.414		0.655		0.759		0.835
	26		0.431		0.481		0.701		0.796		0.863
	25		0.500		0.545		0.745		0.830		0.890
	24		0.564		0.606		0.786		0.861		0.913
	23		0.626		0.663		0.823		0.890		0.934
	22		0.683		0.716		0.858		0.915		0.953
	21		0.736		0.766		0.890		0.938		0.969
	20		0.786		0.811		0.917		0.957		0.982
	19		0.831		0.853		0.942		0.974		0.992
	18		0.871		0.890		0.962		0.986		0.998
	17		0.907		0.922		0.979		0.996		1.00
	16		0.937		0.949		0.991		1.00		
	15		0.962		0.971		0.999				
	14		0.982		0.988		1.00				
	13		0.995		0.998						
	12		1.00		1.00						
	11										
	10										
	9										
	8										
	7										
	6										
	5										

– Indicates stiffness reduction factor is not applicable because the required strength exceeds the available strength for Kl/r = 0.

in real structures. Therefore, adjustments are required when these assumptions [are] violated and the alignment charts are still to be used. Adjustments for com[mon] design conditions that apply to both sidesway conditions are:

1. To account for inelasticity in columns, replace (E_cI_c) with $\tau_a(E_cI_c)$ fo[r] columns in the expression for G_A and G_B. The stiffness reduction factor, is discussed later in this section.
2. For girders containing significant axial load, multiply the $(EI/L)_g$ by factor $(1-Q/Q_{cr})$ where Q is the axial load in the girder and Q_{cr} is in-plane buckling load of the girder based on $K = 1.0$.

For sidesway inhibited frames, these adjustments for different beam end conditi[ons] may be made:

1. If the far end of a girder is fixed, multiply the $(EI/L)_g$ of the member by 2
2. If the far end of the girder is pinned, multiply the $(EI/L)_g$ of the mem[ber] by 1.5.

For sidesway uninhibited frames and girders with different boundary conditi[ons] the modified girder length, L'_g, should be used in place of the actual girder len[gth]

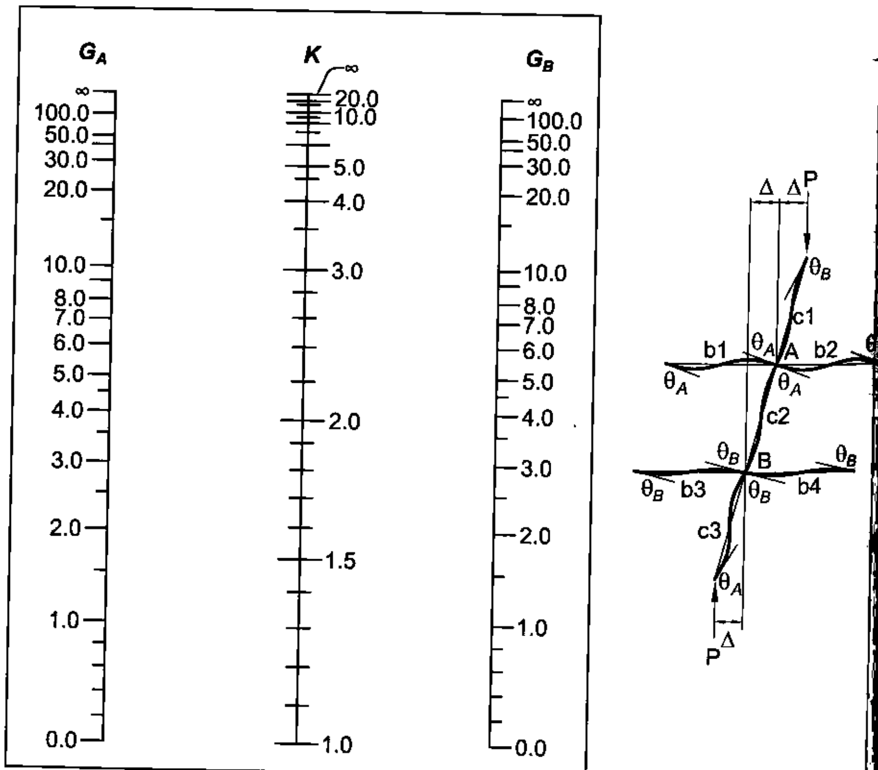


Fig. C-C2.4. Alignment chart—sidesway uninhibited (moment frame).

where

$$L'_g = L_g\,(2 - M_F/M_N)$$

M_F is the far end girder moment and M_N is the near end girder moment from a first-order lateral analysis of the frame. The ratio of the two moments is positive if the girder is in reverse curvature. If M_F/M_N is more than 2.0, then L'_g becomes negative, in which case G is negative and the alignment chart equation must be used.

1. If the far end of a girder is fixed, multiply the $(EI/L)_g$ of the member by $2/3$.
2. If the far end of the girder is pinned, multiply the $(EI/L)_g$ of the member by 0.5.

One important assumption in the development of the alignment charts is that all beam-column connections are fully restrained (FR connections). As seen above, when the far end of a beam does not have an FR connection that behaves as assumed, an adjustment must be made. When a beam connection at the column under consideration is a shear only connection—that is, there is no moment—then that beam can not participate in the restraint of the column and it cannot be considered in the $\Sigma(EI/L)_g$ term of the equation for G. Only FR connections can be used directly in the determination of G. PR connections with a documented moment-rotation response can be utilized, but the $(EI/L)_g$ of each beam must be adjusted to account for the connection flexibility. The ASCE Task Committee on Effective Length (ASCE, 1997) provides a detailed discussion of frame stability with PR connections.

Amplified First-Order Elastic Analysis (Section C2.1b). In this application of the effective length factor, K is used in the determination of the elastic critical buckling load, P_{e1}, for a member, or ΣP_{e2}, for a building story. These elastic critical buckling loads are then used for calculation of the corresponding amplification factors B_1 and B_2.

B_1 is used to estimate the P-δ effects on the nonsway moments, M_{nt}, in axially loaded members. K_1 is calculated in the plane of bending on the basis of no translation of the ends of the member and is normally set to 1.0, unless a smaller value is justified on the basis of analysis. There are also P-δ effects on the sway moments, M_{lt}, as explained previously in the discussion of Equation C2-6b.

B_2 is used to determine the P-Δ effect on the various components of moment, braced and/or combined framing systems. K_2 is calculated in the plane of bending through a sidesway buckling analysis. K_2 may be determined from the sidesway uninhibited alignment chart, Figure C-C2.4, without any correction for story buckling discussed later. ΣP_{e2} from the lateral load resisting columns with K_2 calculated in this way is an accurate estimate of the story elastic sidesway buckling strength. The contribution to the story sidesway buckling strength from leaning columns is zero, and therefore, these columns are not included in the summation in Equation C2-6a. However, the total story vertical load, including all columns in the story, is used for $\alpha\Sigma P_{nt}$ in Equation C2-3.

W14

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

Shape		W14×											
Wt/ft		257		233		211		193		176		159	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	[redacted]	3400	[redacted]	3080	[redacted]	2790	[redacted]	2550	[redacted]	2330	[redacted]	2100
	6	[redacted]	3330	[redacted]	3010	[redacted]	2720	[redacted]	2500	[redacted]	2280	[redacted]	2050
	7	[redacted]	3300	[redacted]	2990	[redacted]	2700	[redacted]	2480	[redacted]	2260	[redacted]	2040
	8	[redacted]	3270	[redacted]	2960	[redacted]	2680	[redacted]	2450	[redacted]	2230	[redacted]	2020
	9	[redacted]	3230	[redacted]	2930	[redacted]	2650	[redacted]	2430	[redacted]	2210	[redacted]	1990
	10	[redacted]	3200	[redacted]	2890	[redacted]	2620	[redacted]	2400	[redacted]	2180	[redacted]	1970
	11	[redacted]	3160	[redacted]	2860	[redacted]	2580	[redacted]	2360	[redacted]	2150	[redacted]	1940
	12	[redacted]	3110	[redacted]	2810	[redacted]	2540	[redacted]	2330	[redacted]	2120	[redacted]	1910
	13	[redacted]	3060	[redacted]	2770	[redacted]	2500	[redacted]	2290	[redacted]	2090	[redacted]	1880
	14	[redacted]	3010	[redacted]	2720	[redacted]	2460	[redacted]	2250	[redacted]	2050	[redacted]	1850
	15	[redacted]	2960	[redacted]	2680	[redacted]	2420	[redacted]	2210	[redacted]	2010	[redacted]	1810
	16	[redacted]	2900	[redacted]	2620	[redacted]	2370	[redacted]	2170	[redacted]	1970	[redacted]	1780
	17	[redacted]	2850	[redacted]	2570	[redacted]	2320	[redacted]	2120	[redacted]	1930	[redacted]	1740
	18	[redacted]	2780	[redacted]	2510	[redacted]	2270	[redacted]	2070	[redacted]	1890	[redacted]	1700
	19	[redacted]	2720	[redacted]	2460	[redacted]	2220	[redacted]	2030	[redacted]	1840	[redacted]	1660
	20	[redacted]	2660	[redacted]	2400	[redacted]	2160	[redacted]	1980	[redacted]	1800	[redacted]	1620
	22	[redacted]	2520	[redacted]	2270	[redacted]	2050	[redacted]	1870	[redacted]	1700	[redacted]	1530
	24	[redacted]	2380	[redacted]	2150	[redacted]	1930	[redacted]	1760	[redacted]	1600	[redacted]	1440
	26	[redacted]	2240	[redacted]	2020	[redacted]	1810	[redacted]	1660	[redacted]	1500	[redacted]	1350
	28	[redacted]	2100	[redacted]	1890	[redacted]	1690	[redacted]	1540	[redacted]	1400	[redacted]	1260
	30	[redacted]	1950	[redacted]	1750	[redacted]	1570	[redacted]	1430	[redacted]	1300	[redacted]	1160
	32	[redacted]	1810	[redacted]	1620	[redacted]	1450	[redacted]	1320	[redacted]	1200	[redacted]	1070
	34	[redacted]	1670	[redacted]	1490	[redacted]	1340	[redacted]	1220	[redacted]	1100	[redacted]	983
	36	[redacted]	1530	[redacted]	1370	[redacted]	1220	[redacted]	1110	[redacted]	1000	[redacted]	896
	38	[redacted]	1390	[redacted]	1250	[redacted]	1110	[redacted]	1010	[redacted]	909	[redacted]	812
	40	[redacted]	1260	[redacted]	1130	[redacted]	1010	[redacted]	913	[redacted]	820	[redacted]	733

Properties

Property	257		233		211		193		176		159	
P_{wo} (kips)	[redacted]	731	[redacted]	620	[redacted]	529	[redacted]	453	[redacted]	396	[redacted]	333
P_{wi} (kips/in.)	[redacted]	58.8	[redacted]	53.5	[redacted]	49.0	[redacted]	44.5	[redacted]	41.5	[redacted]	37.3
P_{wb} (kips)	[redacted]	3700	[redacted]	2790	[redacted]	2150	[redacted]	1610	[redacted]	1300	[redacted]	943
P_{fb} (kips)	[redacted]	1000	[redacted]	832	[redacted]	684	[redacted]	583	[redacted]	483	[redacted]	398
L_p (ft)	14.6		14.5		14.4		14.3		14.2		14.1	
L_r (ft)	104		94.9		86.4		79.7		73.2		66.7	
A_g (in.^{2})	75.6		68.5		62.0		56.8		51.8		46.7	
I_x (in.^{4})	3400		3010		2660		2400		2140		1900	
I_y (in.^{4})	1290		1150		1030		931		838		748	
r_y (in.)	4.13		4.10		4.07		4.05		4.02		4.00	
Ratio r_x/r_y	1.62		1.62		1.61		1.60		1.60		1.60	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	97300		86200		76100		68700		61300		54400	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	36900		32900		29500		26600		24000		21400	

[redacted]	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

W14

Shape		W14×											
Wt/ft		145		132		120		109		99		90	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD	[redacted]	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	[redacted]	1920	[redacted]	1740	[redacted]	1590	[redacted]	1440	[redacted]	1310	[redacted]	1190
	6	[redacted]	1870	[redacted]	1700	[redacted]	1550	[redacted]	1400	[redacted]	1280	[redacted]	1160
	7	[redacted]	1860	[redacted]	1680	[redacted]	1530	[redacted]	1390	[redacted]	1260	[redacted]	1150
	8	[redacted]	1840	[redacted]	1660	[redacted]	1510	[redacted]	1370	[redacted]	1250	[redacted]	1130
	9	[redacted]	1820	[redacted]	1640	[redacted]	1500	[redacted]	1360	[redacted]	1230	[redacted]	1120
	10	[redacted]	1800	[redacted]	1620	[redacted]	1470	[redacted]	1340	[redacted]	1210	[redacted]	1100
	11	[redacted]	1770	[redacted]	1590	[redacted]	1450	[redacted]	1320	[redacted]	1200	[redacted]	1090
	12	[redacted]	1740	[redacted]	1570	[redacted]	1430	[redacted]	1290	[redacted]	1170	[redacted]	1070
	13	[redacted]	1720	[redacted]	1540	[redacted]	1400	[redacted]	1270	[redacted]	1150	[redacted]	1050
	14	[redacted]	1690	[redacted]	1510	[redacted]	1370	[redacted]	1240	[redacted]	1130	[redacted]	1020
	15	[redacted]	1650	[redacted]	1480	[redacted]	1340	[redacted]	1220	[redacted]	1100	[redacted]	1000
	16	[redacted]	1620	[redacted]	1440	[redacted]	1310	[redacted]	1190	[redacted]	1080	[redacted]	978
	17	[redacted]	1580	[redacted]	1410	[redacted]	1280	[redacted]	1160	[redacted]	1050	[redacted]	954
	18	[redacted]	1550	[redacted]	1370	[redacted]	1250	[redacted]	1130	[redacted]	1020	[redacted]	928
	19	[redacted]	1510	[redacted]	1330	[redacted]	1210	[redacted]	1100	[redacted]	995	[redacted]	902
	20	[redacted]	1470	[redacted]	1300	[redacted]	1180	[redacted]	1070	[redacted]	966	[redacted]	876
	22	[redacted]	1390	[redacted]	1220	[redacted]	1100	[redacted]	1000	[redacted]	906	[redacted]	821
	24	[redacted]	1310	[redacted]	1140	[redacted]	1030	[redacted]	932	[redacted]	844	[redacted]	765
	26	[redacted]	1230	[redacted]	1050	[redacted]	956	[redacted]	864	[redacted]	782	[redacted]	708
	28	[redacted]	1140	[redacted]	973	[redacted]	881	[redacted]	797	[redacted]	720	[redacted]	652
	30	[redacted]	1060	[redacted]	892	[redacted]	807	[redacted]	730	[redacted]	659	[redacted]	596
	32	[redacted]	972	[redacted]	814	[redacted]	735	[redacted]	664	[redacted]	599	[redacted]	542
	34	[redacted]	890	[redacted]	738	[redacted]	666	[redacted]	601	[redacted]	542	[redacted]	490
	36	[redacted]	811	[redacted]	663	[redacted]	598	[redacted]	540	[redacted]	486	[redacted]	439
	38	[redacted]	734	[redacted]	595	[redacted]	537	[redacted]	484	[redacted]	436	[redacted]	394
	40	[redacted]	663	[redacted]	537	[redacted]	484	[redacted]	437	[redacted]	393	[redacted]	355

Properties

Property	145		132		120		109		99		90	
P_{wo} (kips)	[redacted]	287	[redacted]	263	[redacted]	227	[redacted]	191	[redacted]	167	[redacted]	144
P_{wi} (kips/in.)	[redacted]	34.0	[redacted]	32.3	[redacted]	29.5	[redacted]	26.3	[redacted]	24.3	[redacted]	22.0
P_{wb} (kips)	[redacted]	717	[redacted]	612	[redacted]	468	[redacted]	330	[redacted]	260	[redacted]	194
P_{fb} (kips)	[redacted]	334	[redacted]	298	[redacted]	249	[redacted]	208	[redacted]	171	[redacted]	142
L_p (ft)	14.1		13.3		13.2		13.2		13.5		15.2	
L_r (ft)	61.7		56.0		52.0		48.4		45.3		42.6	
A_g (in.^{2})	42.7		38.8		35.3		32.0		29.1		26.5	
I_x (in.^{4})	1710		1530		1380		1240		1110		999	
I_y (in.^{4})	677		548		495		447		402		362	
r_y (in.)	3.98		3.76		3.74		3.73		3.71		3.70	
Ratio r_x/r_y	1.59		1.67		1.67		1.67		1.66		1.66	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	48900		43800		39500		35500		31800		28600	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	19400		15700		14200		12800		11500		10400	

[redacted]	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

E4. COMPRESSIVE STRENGTH FOR TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up *columns* with *compact* and *noncompact sections*, as defined in Section B4 for uniformly compressed elements. These provisions are not required for single angles, which are covered in Section E5.

The *nominal compressive strength*, P_n, shall be determined based on the *limit states* of *flexural-torsional* and *torsional buckling*, as follows:

$$P_n = F_{cr} A_g \tag{E4-1}$$

(a) For double-angle and tee-shaped compression members:

$$F_{cr} = \left(\frac{F_{cry} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}}\right] \tag{E4-2}$$

where F_{cry} is taken as F_{cr} from Equation E3-2 or E3-3, for *flexural buckling* about the y-axis of symmetry and $\frac{KL}{r} = \frac{KL}{r_y}$, and

$$F_{crz} = \frac{GJ}{A_g \bar{r}_o^2} \tag{E4-3}$$

(b) For all other cases, F_{cr} shall be determined according to Equation E3-2 or E3-3, using the torsional or flexural-torsional elastic buckling *stress*, F_e, determined as follows:

(i) For doubly symmetric members:

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right]\frac{1}{I_x + I_y} \tag{E4-4}$$

(ii) For singly symmetric members where y is the axis of symmetry:

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right] \tag{E4-5}$$

(iii) For unsymmetric members, F_e is the lowest root of the cubic equation:

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0$$

where

A_g = gross area of member, in.^{2} (mm^2)

C_w = warping constant, in.^{6} (mm^6)

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \tag{E4-7}$$

$$H = 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \tag{E4-8}$$

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L}{r_x}\right)^2} \tag{E4-9}$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y}\right)^2} \tag{E4-10}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \tag{E4-11}$$

G = shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)

I_x, I_y = moment of inertia about the principal axes, in.^{4} (mm^4)

J = torsional constant, in.^{4} (mm^4)

K_z = *effective length factor* for torsional buckling

x_o, y_o = coordinates of shear center with respect to the centroid, in. (mm)

$\bar{r}_o$ = polar radius of gyration about the shear center, in. (mm)

r_y = radius of gyration about y-axis, in. (mm)

User Note: For doubly symmetric I-shaped sections, C_w may be taken as $I_y h_o^2/4$, where h_o is the distance between flange centroids, in lieu of a more precise analysis. For tees and double angles, omit term with C_w when computing F_{ez} and take x_o as 0.

SINGLE ANGLE COMPRESSION MEMBERS

The *nominal compressive strength*, P_n, of single angle members shall be determined in accordance with Section E3 or Section E7, as appropriate, for axially loaded members, as well as those subject to the slenderness modification of Section E5(a) or E5(b), provided the members meet the criteria imposed.

The effects of eccentricity on single angle members are permitted to be neglected when the members are evaluated as axially loaded compression members using one of the effective slenderness ratios specified below, provided that: (1) members are loaded at the ends in compression through the same one leg; (2) members are attached by welding or by minimum two-bolt *connections*; and (3) there are no