AISC Column Design Logic Makes Sense for Composite Columns, Too

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STRUCTURAL engineers of North America have regarded the composite column generally as a curious stepchild who does not quite belong to a family, yet appears to behave very well. Fathers of the American Concrete Institute have provided shelter by permitting their own ACI routines to be applied to "concrete compression members reinforced longitudinally with structural steel shape, pipe, or tubing."[1] As a "concrete compression member" the stepchild is restrained from doing its own thing as effectively as its structural steel cousins are allowed to do. The ACI Building Code forbids consideration of axially loaded columns by insisting that all columns also function as beams.

Within the structural steel shape[2] and tubing[3] family, compression members can be assembled for design as axially loaded columns. Composite columns that are incorporated into structures with connections identical to those used for steel shapes or tubes ought to be considered analytically to behave exactly the same as the shapes or tubes.

Composite columns possess better stiffness and local stability than their structural steel cousins, and they are much more reliable in shear and ductility than their reinforced concrete cousins. Even though they cost more to produce than either of the cousins, their potential benefit-cost ratio may make them a far more attractive sibling for any structural family—whether it be concrete, steel, or a new genre as yet unnamed. The following demonstration of composite column analysis and comparison with laboratory behavior is intended to encourage the steel family to consider the adoption of design rules for composite columns.

AXIALLY LOADED COMPOSITE COLUMNS

The logic of the AISC Specification can be applied to axially loaded composite columns if the influence of concrete on the strength, stiffness, and cross-section slenderness of composite columns is incorporated into effective parameters F_y^* for strength, E^* for stiffness and r^* for composite section radius of gyration. Definitions of each equivalence parameter follow:

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$$F_y^* = F_y + 0.85 f'_c \frac{A_c}{A_s} \tag{1}$$

$$E^* = E_s + E_c \frac{A_c}{A_s} \tag{2}$$

$$r^* = \sqrt{\frac{E_s I_s + 0.5 E_c I_c}{E_s A_s + 0.5 E_c A_c}} \tag{3}$$

in which

A_c = area of concrete in composite cross section
A_s = area of steel in composite cross section
E_c = modulus of elasticity for concrete
E_s = modulus of elasticity for steel
f'_c = design compressive strength of concrete standard cylinders
F_y = static yield strength of steel
I_c = moment of inertia of concrete in composite cross section
I_s = moment of inertia of steel in composite cross section

These parameters were applied to 84 concrete-filled tube columns and 30 concrete-encased rolled-shape columns that had been tested in recent years.[4,5,6,7,8] The parameters were then used in AISC column strength formulas in order to compute "allowable" loads on each of the 114 specimens. Significant computed quantities are listed in Tables 1 and 2.

The AISC strength formulas take the following form with the modified parameters:

$$C_c = \pi \sqrt{\frac{2E^*}{F_y^*}} \tag{4}$$

where C_c is the slenderness ratio dividing elastic from inelastic buckling. When the effective length Kl is less than the product r^*C_c, the axial stress F_a should be computed:

$$F_a = \frac{\left[1 - \frac{1}{2}\left(\frac{Kl}{r^*C_c}\right)^2\right] F_y}{\frac{5}{3} + \frac{3}{8}\frac{Kl}{r^*C_c} - \frac{1}{8}\left(\frac{Kl}{r^*C_c}\right)^3} \tag{5}$$

Table 1. Axially Loaded Steel Tubes Filled with Concrete

Ref.	O.D. (in.)	A_s (in.²)	A_c (in.²)	f_y (ksi)	f'_c (ksi)	Kl (in.)	$\frac{Kl}{r^*C_c}$	P_{ax} (kips)	P_{test} (kips)	$\frac{P_{test}}{P_{ax}}$
Ref. 7. Tests by Klöppel and Goder, Knowles and Park, Sims and Calani, Furlong, Jacobsen, and Gardner	3.74	5.07	5.92	39.9	2.94	33.9	0.353	113.0	229.0	2.02
						55.9	0.582	96.9	209.0	2.16
						78.0	0.812	76.4	203.0	2.66
	3.74	1.63	9.36	50.7	3.62	33.9	0.383	57.3	150.0	2.62
						55.9	0.632	47.6	131.0	2.75
						78.0	0.882	35.6	119.0	3.34
	8.50	4.22	52.5	42.3	3.32	87.4	0.421	164.0	371.0	2.26
				41.7	4.32	87.4	0.443	182.0	509.0	2.79
	8.50	6.13	50.6	56.8	3.32	87.4	0.453	241.0	549.0	2.27
				50.8	4.32	87.4	0.449	245.0	645.0	2.63
	3.74	1.63	9.36	49.0	3.49	80.0	0.890	34.0	104.0	3.06
	4.76	2.14	15.6	45.2	3.06	41.3	0.348	72.0	162.0	2.25
					3.51	41.3	0.354	75.7	192.0	2.54
					3.06	91.0	0.768	51.0	143.0	2.80
					3.51	91.0	0.781	53.0	163.0	3.08
	4.76	3.11	14.7	49.8	3.06	41.3	0.359	100.6	227.0	2.26
					3.51	41.3	0.354	99.7	245.0	2.46
					3.06	91.0	0.791	69.8	180.0	2.58
					3.51	91.0	0.781	69.9	195.0	2.79
	1.00	0.11	0.68	76.0	4.04	42.0	1.44	1.31	3.52	2.69
	1.50	0.48	1.29	76.0	4.04	42.0	1.00	10.7	24.7	2.31
	2.00	0.40	2.74	76.0	4.04	42.0	0.736	15.3	27.1	1.77
	3.00	0.59	6.47	76.0	3.95	42.0	0.474	32.3	72.0	2.24
	14.00	18.7	135.0	51.5	5.52	22.0	0.169	910.0	2576.0	2.83
					4.76	22.0	0.165	361.0	2408.0	2.80
	14.00	8.07	146.0	40.1	3.04	21.1	0.153	402.0	791.0	1.97
	14.00	13.50	140.0	51.5	3.40	21.5	0.187	622.0	1671.0	2.69
	5.01	0.99	18.7	53.8	9.6	19.7	0.222	115.0	289.0	2.49
				47.7	9.6	19.7	0.218	112.0	289.0	2.58
	5.00	1.78	17.9	53.8	9.6	20.0	0.206	136.0	293.0	2.15
				47.7	9.6	20.0	0.201	130.0	293.0	2.25
	4.00	1.49	11.1	87.8	4.95	60.0	0.566	80.2	184.0	2.29
					4.52	60.0	0.563	78.6	180.0	2.29
	4.76	2.33	15.5	65.5	4.99	41.3	0.245	121.0	260.0	2.16
					4.29	41.3	0.290	113.0	246.0	2.18
					3.76	41.3	0.286	109.0	214.0	1.96
	6.00	2.29	26.0	60.2	3.03	66.0	0.348	107.0	211.0	1.85
									198.0	2.39
	3.01	0.63	6.5	52.7	3.62	60.0	0.616	23.0	55.0	2.57
					5.93	24.0	0.266	36.0	92.5	2.51
					3.76	24.0	0.248	29.6	74.2	1.89
	4.50	1.72	14.2	60.0	4.20	33.0	0.238	84.5	160.0	2.01
									170.0	1.79
	5.00	1.46	18.2	42.0	5.10	59.0	0.346	81.7	141.0	1.73
									140.0	1.71
									148.0	1.81
	6.00	1.14	27.1	48.0	3.05	59.0	0.303	72.9	153.0	2.10
					3.75	59.0	0.312	82.2	162.0	1.97
									165.0	2.01
	5.51	6.14	17.7	38.5	4.66	16.0	0.079	181.0	663.0	3.67
				39.0	4.66	16.0	0.079	183.0	663.0	3.63
	5.53	3.25	20.8	41.9	4.74	16.0	0.079	130.0	410.0	3.15
				43.2	4.74	16.0	0.079	132.0	410.0	3.10
	6.62	3.62	30.8	43.2	4.56	32.0	0.139	160.0	451.0	2.81
					6.26	32.0	0.152	184.0	502.0	2.73
									475.0	2.58
					3.34	32.0	0.137	141.0	392.0	2.79

Table 1 (cont'd)

Ref.	O.D. (in.)	A_s (in.^{2})	A_c (in.^{2})	f_y (ksi)	f'_c (ksi)	Kl (in.)	$\frac{Kl}{r^*C_c}$	P_{ax} (kips)	P_{test} (kips)	$\frac{P_{test}}{P_{ax}}$
Ref. 7. (cont'd)	3.50	2.36	7.26	58.0	5.81	68.0	0.758	64.9	138.0	2.13
					5.75	56.0	0.625	74.1	160.0	2.16
					5.65	44.0	0.491	81.6	161.0	1.97
					6.06	32.0	0.358	90.8	206.0	2.27
					5.92	20.0	0.223	96.6	223.0	2.31
	3.50	0.55	7.74	70.0	6.00	68.0	0.805	27.7	50.5	1.82
					5.36	56.0	0.638	31.4	66.2	2.11
					5.92	44.0	0.512	36.6	80.0	2.19
						32.0	0.372	40.1	90.0	2.24
						20.0	0.232	43.1	110.0	2.55
						10.0	0.116	45.1	119.2	2.64
Ref. 6. Spiral steel tubes tested by Gardner	6.64	2.13	32.5	43.2	2.60	12.0	0.051	97.2	298.0	3.06
						78.0	0.333	86.6	185.0	2.14
					4.95	12.0	0.058	136.0	274.0	2.01
						78.0	0.375	118.0	206.0	1.74
	6.64	2.13	32.5	46.0	5.30	12.0	0.059	145.0	294.0	2.03
						78.0	0.384	125.0	170.0	1.36
					4.87	12.0	0.058	138.0	299.0	2.17
						78.0	0.379	120.0	155.0	1.29
	6.62	2.89	31.5	32.1	3.86	12.0	0.046	117.0	350.0	3.00
						90.0	0.345	103.0	213.0	2.24
					4.75	12.0	0.048	131.0	322.0	2.46
						90.0	0.359	115.0	236.0	2.05
	6.64	3.98	30.6	37.8	4.77	12.0	0.045	163.0	442.0	2.71
						90.0	0.337	145.0	254.0	1.75
					3.98	12.0	0.044	151.0	446.0	2.95
						90.0	0.328	135.0	262.0	1.94

When the effective length Kl is greater than r^*C_c, the axial stress F_a should be taken as:

$$F_a = \frac{12}{23}\frac{\pi^2 E^*}{(Kl/r^*)^2} \tag{6}$$

Finally, the allowable axial load P_{ax} is determined as the product of allowable stress and *steel* area:

$$P_{ax} = F_a A_s \tag{7}$$

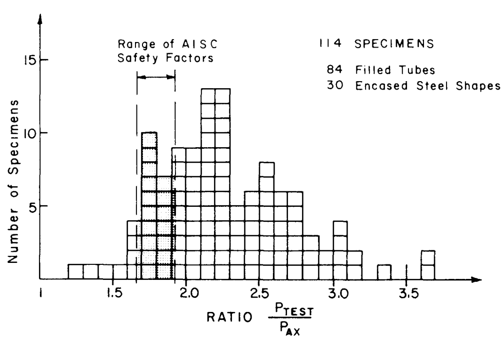

Fig. 1. Axially loaded composite columns

The right-hand columns of Table 1 and Table 2 contain ratios of the reported test load P_{ax} to the computed values P_{test}. The ratios vary from a low value of 1.29 to a high value of 3.67. The mean value for 114 composite specimens was 2.28, with an 18.4 percent coefficient of variation.

AISC safety factors are intended to be in the range 1.67 to 1.92. The frequency distribution for all 114 tests appears in Fig. 1, and the AISC safety domain is shown as a shaded region of that diagram. The two ratios that were less than 1.40 involved steel tubes fabricated from welded spiral plate stock, and perhaps such tubes require special attention.

AISC FORMAT FOR BEAM-COLUMNS

Frequently it is necessary to proportion compression members to resist flexural forces in addition to thrust. All columns that are used in frames constructed with moment resistant connections must be considered as beam-columns.

The AISC requirements for designing beam-columns are based on limits to the total stress generated by both thrust and flexure. However, composite columns present

Table 2. Axially Loaded Encased Steel Shapes

Reference	Concr. Size (in.)	Steel Size (in.)	A_s (in.^{2})	A_c (in.^{2})	f'_c (ksi)	f_y (ksi)	$\frac{Kl}{r^*C_c}$	P_{ax} (kips)	P_{test} (kips)	$\frac{P_{test}}{P_{ax}}$
Ref. 4. Tests by Stevens	5 × 3.5	3 × 1.5	1.18	16.32	2.60	36.0	0.398	40.0	81.4	2.04
							0.550	36.0	71.5	1.99
							0.707	31.2	63.0	2.02
							0.864	25.8	43.6	1.69
							1.015	20.0	50.6	2.53
							1.172	15.0	36.1	2.41
							1.329	11.6	33.9	2.92
	7 × 6.5	5 × 4.5	5.88	39.6	1.60	36.0	0.054	177.0	352.0	1.99
							0.272	163.0	308.0	1.89
							0.490	143.0	317.0	2.22
							0.707	119.0	288.0	2.42
							0.917	90.6	231.0	2.55
	10 × 8	8 × 6	10.3	69.7	2.60	36.0	0.472	255.0	568.0	2.22
	12 × 10	8 × 6	10.3	110.0	2.60	36.0	0.479	300.0	704.0	2.34
	14 × 12	8 × 6	10.3	158.0	2.60	36.0	0.441	354.0	836.0	2.42
	16 × 12	12 × 8	19.1	173.0	2.60	36.0	0.091	627.0	1051.0	1.68
							0.183	608.0	990.0	1.63
							0.274	583.0	926.0	1.59
							0.365	556.0	937.0	1.69
							0.457	525.0	933.0	1.78
Ref. 5. Tests by Janss	9.5 × 9.5	5.5 × 5.5	6.66	82.6	4.66	41.5	0.664	255.0	482.0	1.89
					4.28	42.7	0.545	268.0	526.0	1.96
					4.77	40.2	0.381	310.0	590.0	1.90
					4.29	40.0	0.157	271.0	572.0	2.11
					4.24	55.0	0.561	294.0	528.0	1.80
					4.24	72.6	0.779	298.0	528.0	1.77
					4.27	70.8	0.635	322.0	554.0	1.72
					4.77	72.5	0.442	384.0	545.0	1.42
					4.39	41.5	0.576	239.0	513.0	2.14
					4.30	70.7	0.694	303.0	517.0	1.71

unique problems associated with estimates of stress, either in concrete or in steel. Again a pseudo-allowable bending stress, F_b^*, can be defined as the product of allowable moment and the steel section modulus, and all other components of AISC design equations can be applied.

The moment capacity of composite cross sections can be determined analytically only by means of rather tedious calculations involving the equilibrium of post-elastic stress conditions associated with compatible strains near ultimate flexural loads. Estimates of a reliable moment capacity M_o can be taken far more simply by using the product of yield stress and the plastic section modulus of the steel in the cross section of filled tubes and encased shapes that are bent about the major axis. Minor axis bending strength estimates should include not only the weak axis plastic moment for steel, but also the capacity of a concrete section reinforced by the web of the steel shape. For encased shapes bent about the minor axis,

$$M_o = Z_y F_y + A_w F_y \left(\frac{b}{2} - \frac{A_w F_y}{1.7 f_c h}\right) \qquad (7)$$

where

Z_y = weak axis plastic section modulus of rolled shape
b = width of concrete section parallel to steel flanges
h = depth of concrete section in the web direction
A_w = area of web of steel shape

The allowable flexural stress for composite sections should be taken as

$$F_b^* = \frac{3}{5}\frac{M_o}{S} \qquad (8)$$

where S = section modulus of steel in plane of bending.

The AISC equation for allowable combinations of thrust and flexural stress then can be used:

$$\frac{f_a}{F_a} + \frac{C_m}{\left(1 - \frac{f_a}{F_e^*}\right)} \times \frac{f_b}{F_b^*} \leq 1 \qquad (9)$$

Table 3. Eccentric Loads on Filled Tubes

Reference	Tube Size (in.)	A_s (in.^{2})	A_c (in.^{2})	f_y (ksi)	f'_c (ksi)	$\frac{Kl}{r^*C_c}$	P_u (kips)	M_o (kip-in.)	P_{test} (kips)	M_{test} (kip-in.)	$\frac{P_{test}}{P_u}$	$\frac{M_{test}}{M_o}$
Ref. 9. Tests by Furlong	4.50 Dia.	1.72	14.2	60.0	4.20	0.238	150	143	100	100	0.67	0.70
									90	106	0.60	0.74
									75	131	0.50	0.92
									50	141	0.33	0.99
									25	144	0.17	1.01
	6.00 Dia.	1.14	27.1	48.0	3.75	0.318	134	103	128	88	0.95	0.85
									95	158	0.71	1.52
									64	153	0.48	1.48
					3.05	0.308	119	103	30	143	0.26	1.39
									30	133	0.26	1.29
	5.00 Dia.	1.40	18.2	42.0	5.10	0.288	132	97	128	78	0.97	0.82
									120	112	0.91	1.16
									90	141	0.68	1.46
									79	140	0.60	1.45
									79	126	0.60	1.31
									78	141	0.59	1.45
									69	151	0.52	1.56
									60	156	0.46	1.61
									59	156	0.44	1.61
									39	146	0.30	1.51
									20	141	0.15	1.46
									10	130	0.07	1.35
	5.00 Sq.	1.85	23.2	70.3	6.50	0.308	246	461	250	310	1.02	0.67
									150	365	0.61	0.79
									150	430	0.61	0.93
									100	450	0.41	0.98
	4.00 Sq.	1.31	14.7	48.0	3.40	0.389	102	93	84	44	0.82	0.44
									84	45	0.82	0.45
									54	92	0.53	0.92
									20	105	0.20	1.05
									20	114	0.20	1.14
	4.00 Sq.	1.94	14.1	48.0	4.18	0.383	133	135	98	119	0.74	0.88
									69	162	0.52	1.20
									68	162	0.51	1.20
									59	190	0.44	1.41
									29	209	0.22	1.55
									29	193	0.22	1.43

Again, f_a and f_b are to be computed as the total force divided by steel area and total moment divided by steel section modulus, and

$$F_e^* = \frac{12}{23}\frac{E^*\pi^2}{(K/r^*)^2} \tag{10}$$

Results from tests of 37 eccentrically loaded steel tubes filled with concrete were compared with strength estimates adopted from Eq. (9). Specimen characteristics and computed strength values are given in Table 3. The reported flexural capacities already include slenderness effects, and no moment magnification would be appropriate for these filled tube specimens. The axial capacity P_u was estimated as

$$P_u = F_a A_s \times (\text{F.S.}) \tag{11}$$

The ratios P_{test}/P_u and M_{test}/M_o are plotted as data points identified by dots in boxes or circles in Fig. 2.

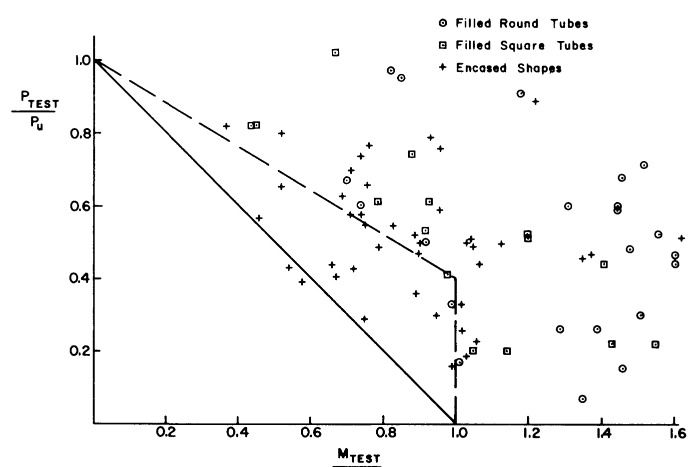

Fig. 2. Eccentrically loaded composite columns

Table 4. Eccentric Loads on Encased Shapes

Ref.	Col. Size (in.)	Steel Shape	A_s (in.^{2})	A_c (in.^{2})	f_y (ksi)	f'_c (ksi)	$\frac{Kl}{r^*C_c}$	P_u (kips)	M_o (kip-in.)	P_{test} (kips)	M_{test} (kip-in.)	$\frac{P_{test}}{P_u}$	δ	$\frac{\delta M_{test}}{M_o}$
Ref. 5. Janss	9.45 ×	HEB	6.66	82.6	41.5	4.80	0.577	511	512	251	394	0.49	1.37	1.05
	9.45	140				4.64	0.477	533	508	265	416	0.50	1.25	1.03
						4.03	0.569	468	503	240	377	0.51	1.38	1.04
					70.7	4.50	0.657	617	819	265	416	0.43	1.41	0.72
						4.36	0.660	608	816	251	394	0.41	1.39	0.67
						4.03	0.659	590	806	223	350	0.39	1.33	0.58
	8.27 ×	IPE	5.18	99.0	39.5	4.64	0.509	281	340	269	422	0.52	1.31	1.62
	12.60	220				4.36	0.505	271	338	234	367	0.47	1.26	1.37
						4.28	0.503	268	337	229	360	0.46	1.26	1.35
Ref. 10. Stevens	12.0 ×	12 × 8	19.1	171.0	32.3	2.52	0.362	961	1415	672	672	0.70	1.40	0.71
	16.0	65 lb.				2.36	0.367	935	1395	486	972	0.52	1.27	0.89
						2.92	0.353	1026	1455	515	1032	0.50	1.26	0.90
						2.68	0.358	988	1432	361	1083	0.36	1.18	0.89
						2.68	0.358	988	1432	296	1184	0.30	1.14	0.95
						2.80	0.354	1007	1444	262	1310	0.26	1.12	1.02
						2.72	0.358	993	1436	231	1386	0.23	1.11	1.06
						3.08	0.349	1052	1466	199	1393	0.19	1.09	1.03
						3.00	0.350	1039	1461	168	1344	0.16	1.07	0.99
Ref. 10. Stevens	6.50 ×	5 × 4½	5.88	39.6	33.6	2.80	0.492	256	242	161	121	0.63	1.36	0.69
	7.00	20 lb.					0.492	256	242	168	134	0.66	1.38	0.76
							0.492	256	242	202	151	0.79	1.50	0.93
							0.492	256	242	228	183	0.89	1.61	1.22
							0.171	287	242	166	166	0.58	1.03	0.71
							0.273	281	242	224	112	0.80	1.13	0.52
							0.273	281	242	164	164	0.58	1.09	0.74
							0.492	256	242	141	141	0.55	1.30	0.75
							0.710	218	242	161	81	0.74	2.24	0.74
							0.710	218	242	119	119	0.55	1.69	0.83
							0.921	168	242	99	99	0.59	2.33	0.96
							0.921	168	242	78	118	0.47	1.83	0.90
							0.921	168	242	74	148	0.44	1.75	1.07
Ref. 11. Loke	7.00 ×	4 × 3	2.94	53.1	40.7	3.71	0.491	252	152	195	78	0.77	1.49	0.76
	8.00	10 lb.			45.6	3.28	0.502	246	164	108	86	0.44	1.24	0.66
					39.3	4.20	0.494	268	150	88	132	0.33	1.16	1.02
					39.5	4.58	0.701	244	152	201	40	0.82	1.44	0.37
					39.5	4.31	0.697	235	151	135	54	0.57	1.27	0.46
					42.7	3.25	0.696	206	156	88	70	0.43	1.19	0.54
					39.5	4.28	0.697	234	151	68	101	0.29	1.12	0.75
					42.4	4.27	0.500	278	167	211	84	0.76	1.49	0.76
					42.4	3.91	0.497	264	167	130	104	0.49	1.27	0.79
	7.00 ×	4 × 1¾	1.47	54.5	43.0	2.89	0.443	178	89	116	46	0.65	1.30	0.52
	8.00	5 lb.			43.0	3.81	0.455	215	93	108	86	0.50	1.23	1.13

Results from 42 eccentrically loaded encased shapes are tabulated in Table 4, together with strength estimates based on Eqs. (9) and (11), except that an ultimate strength moment magnifier δ was computed with an effective F_e^* equal to $^{23}/_{12}$ of the value from Eq. (10). The ratios of P_{test}/P_u and $\delta M_{test}/M_o$ are plotted as symbols + in Fig. 2.

Data points illustrated in Fig. 2 show that the allowable stress interaction Eq. (9) is definitely valid. When values of M_o are taken as low as the plastic moment strength of steel tubes filled with concrete, the coefficient C_m could be taken safely as low as 0.60, with the further restriction that the ratio $f_b/F_b^* \leq 1$.

SUMMARY

The logic of the AISC Specification can be applied to composite columns that are loaded in either direct or eccentric compression. The proposed relationships for an effective yield strength, F_y^*, material stiffness, E^*, and cross-section stiffness, r^*, will provide for margins of safety generally larger than those resulting from the AISC column design rules applied to steel alone. The AISC beam-column equations likewise provide a reliable index of capacity. The proposed relationships for estimating flexural strength, M_o, tend to underestimate actual capacity, particularly for concrete-filled steel tubes.

As the amount or strength of concrete in a composite member is reduced analytically to zero, each of the proposed design equations provides safe estimates for the strength of steel by itself. There remain no transition inconsistencies between the strength of steel alone and the augmented strength from composite action.

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