A Specification for the Design of Steel-Concrete Composite Columns

TASK GROUP 20, STRUCTURAL STABILITY RESEARCH COUNCIL

Subcommittee 20—Composite Columns was designated in 1973 as a standing committee of the Structural Stability Research Council (formerly called the Column Research Council). With an abundant background of experience regarding steel column behavior, the Council recognized that steel-concrete composite compression members should behave almost the same as plain steel columns if, in composite cross sections, the strength and stiffness of the structural steel alone were several times greater than the strength and stiffness of the structural concrete. The Council was also aware that if, in a composite cross section, the strength and stiffness of the concrete alone were significantly greater than the strength and stiffness of structural steel, the composite compression member would behave much the same as an ordinary reinforced concrete column. Design concepts traditionally applied to structural steel involved fundamental differences from those generally applied to reinforced concrete. The consequences of unequal results from the different design concepts required reconciliation within a rational statement of recommended practice for composite column design.

In subsequent years the Council received reports from Subcommittee 20 identifying the major differences between the structural steel (AISC)[1] and reinforced concrete (ACI)[2] approach to regulations each felt should govern the design of composite columns. In May, 1978, a document containing recommendations for a composite column design specification[3] adapted from an earlier paper[4] was presented to the Council. A task group was appointed to review the proposed design rules, and responses from the task group prompted modifications in the recommended design rules.

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This report contains a statement of recommended design rules and a discussion of composite column behavior which serves as a commentary for the recommendations. To facilitate and illustrate applications of the rules, some design examples and design aid charts are added to this report.

A comparison between capacities reported in laboratory tests and allowable loads according to the proposed design rules is appended to this report.

The statement of design requirements for steel-concrete composite columns, as presented here, is in a form intended for incorporation into a general structural steel design document such as the AISC Specification, Part 1. Nomenclature, definitions, the treatment of load cases, and supplementary references to material specifications would be included in the general specification, of which the proposed rules are to be a sub-section. Consequently, only the symbols that are not already defined in the 1978 AISC list of Nomenclature are included in the proposed specification.

PROPOSED DESIGN SPECIFICATION FOR COMPOSITE COLUMNS

Nomenclature

A_{bc} = Area of bearing surface between steel and concrete at connections (square inches)

A_{cc} = Area of concrete effective in composite columns (square inches)

A_{cr} = Area of longitudinal bar reinforcement in a composite column cross section (square inches)

A_g = Gross area included within exterior surfaces of a composite cross section (square inches)

*A_s = Area of steel (shape or tube) in composite design (square inches)

* *Symbols presently defined in Ref. 1 have modified definition.*

$*A_w$ = Web area; for girders or rolled shapes $A_w = d\,t_w$ (square inches)

C_m = Coefficient applied to bending term in interaction formula and dependent upon column curvature caused by applied moments

E = Modulus of elasticity of steel (29,000 kips per square inch)

$*E_c$ = Modulus of elasticity of concrete (kips per square inch)

E_m = Modified modulus of elasticity for composite column (kips per square inch)

F_a = Axial stress permitted in the absence of bending moment (kips per square inch)

F_b = Bending stress permitted in the absence of axial force (kips per square inch)

F_{cr} = Specified yield strength of longitudinal reinforcement in composite column (not greater than 55 ksi) (kips per square inch)

F'_e = Euler stress divided by factor of safety (kips per square inch)

F_{my} = Modified value of yield stress for composite column (kips per square inch)

F_y = Specified minimum yield stress of the type of steel being used (kips per square inch)

*K = Coefficient relating the distance between lateral supports for a column to the effective distance between points of inflection when the column buckles

L = Actual unbraced length (feet)

M = Moment (kip-feet)

M_o = Moment capacity in the absence of axial thrust (kip-feet)

P = Applied load (kips)

P_a = Allowable axial compression force (kips)

P_n = Nominal axial compression capacity (kips)

S_m = Modified section modulus about axis of bending of a composite column (inches^3)

S_{sc} = Elastic section modulus of structural shape, pipe, or tube alone about axis of bending (inches^3)

b = Effective width of concrete slab; actual width of stiffened and unstiffened compression elements (inches)

c_r = Average of distance from compression face to longitudinal reinforcement in that face and distance from tension face to longitudinal reinforcement in that face (inches)

d = Depth of beam or girder (inches)

f_a = Computed axial stress (kips per square inch)

f_b = Computed bending stress (kips per square inch)

f'_c = Specified compression strength of concrete (kips per square inch)

h_1 = Overall thickness of a composite cross section perpendicular to the plane of bending (inches)

h_2 = Overall thickness of a composite cross section in the plane of bending (inches)

l = Actual unbraced length (inches)

r_m = Effective radius of gyration of a composite column (inches)

r_s = Radius of gyration of the structural shape, pipe, or tube in the plane of bending of a composite column (inches)

$*t$ = Girder, beam, or column web thickness; thickness of wall of pipe or tube (inches)

t_w = Web thickness of rolled structural steel shape (inches)

General Requirements

A composite column shall consist of rolled or built up structural steel shapes, pipe or tubing and structural concrete acting together to resist compression or compression plus bending. In order to qualify as a composite column, the cross-section area of the steel shapes, pipe, or tubing must comprise at least 4 percent of the total composite column cross section. (If the ratio $A_s/(A_s + A_{cr} + A_{cc})$ is less than 0.04, the member is defined as reinforced concrete and it is excluded from the design rules that follow.) Concrete shall have a specified compression strength f'_c not less than 3000 psi nor more than 8000 psi, and multiple steel shapes in the same cross section must be connected to one another with lacing, tie plates, or batten plates in conformance with Sect. 1.18.2.

Concrete encasement of structural shapes shall be tied laterally and longitudinally with reinforcement spaced not more than ⅔ the least dimension of the composite cross section and containing both a transverse and a longitudinal cross-section area not less than 0.007 in.^{2} per inch of bar spacing. Concrete encasement of structural steel shapes shall provide at least 1.5 in. of clear cover over lateral and longitudinal reinforcement.

The design yield strength of structural steel in composite columns shall be not greater than 55 ksi. If specified yield strength exceeds this value, 55 ksi shall be used in allowable stress equations. The thickness t of the walls of structural steel pipe or tube filled with concrete shall be limited by $t \geq b\sqrt{F_y/3E}$ for each face of width b in rectangular sections, and $t \geq h\sqrt{F_y/8E}$ for circular sections of outside diameter h.

Allowable Stresses

The allowable compressive axial stress F_a on the structural steel area of a composite cross section shall be determined from Eq. (1.5-1) or (1.5-2), using a modified composite yield stress F_{my} for F_y, a modified composite modulus of elasticity E_m for E, and a radius of gyration r_m for r. The allowable axial compressive force P_a on the composite cross section shall be taken as the product of the area of the structural steel shape A_s and the axial stress F_a.

For concrete filled pipe or tube:

$$F_{my} = F_y + F_{cr}\frac{A_{cr}}{A_s} + 0.85f_c'\frac{A_{cc}}{A_s}$$

with F_y and $F_{cr} < 55$ ksi (A)

$$E_m = 29000 + 0.4E_c\frac{A_{cc}}{A_s} \qquad \text{(B)}$$

$$r_m = r_s$$

For concrete encased structural steel:

$$F_{my} = F_y + 0.7F_{cr}\frac{A_{cr}}{A_s} + 0.6f_c'\frac{A_{cc}}{A_s}$$

with F_y and $F_{cr} < 55$ ksi (C)

$$E_m = 29000 + 0.2E_c\frac{A_{cc}}{A_s} \qquad \text{(D)}$$

$r_m = r_s$, but not less than $0.3h_2$

For composite compression members, the allowable flexural stress shall be:

$$F_b = 0.75F_y \text{ for pipe or tube}$$

$$F_b = 0.6F_y \text{ for steel shapes}$$

Composite compression members subjected to bending in addition to an axial force shall be proportioned to satisfy the expression:

$$\left(\frac{f_a}{F_a}\right)^2 + \frac{C_{mx}}{\left(1 - \frac{f_a}{F'_{ex}}\right)}\frac{f_{bx}}{F_{bx}} + \frac{C_{my}}{\left(1 - \frac{f_a}{F'_{ey}}\right)}\frac{f_{by}}{F_{by}} \le 1 \qquad \text{(E)}$$

where neither $\frac{C_{mx}}{\left(1 - \frac{f_a}{F'_{ex}}\right)}$ nor $\frac{C_{my}}{\left(1 - \frac{f_a}{F'_{ey}}\right)}$ are to be taken less than 1.

For application to Eq. (E), a modified section modulus S_m shall be used for computing bending stresses f_{bx} and f_{by}:

$$S_m = S_{sc} + \frac{1}{3}A_{cr}(h_2 - 2c_r)\frac{F_{cr}}{F_y} + \left(\frac{h_2}{2} - \frac{A_w F_y}{1.7f_c'h_1}\right)A_w \qquad \text{(F)}$$

For steel pipe of tubes, $A_w = 0$.

The index of axial stress $f_a = P/A_s$, and the modified Euler stress becomes:

$$F_e' = \frac{12}{23}\frac{\pi^2 E_m}{(Kl/r_m)^2}$$

Connections

The portion of the column axial force P_a resisted by the concrete at connections must be developed by direct bearing against concrete. Bearing stress against concrete shall be no greater than $0.75f_c'$.

For concrete filled pipe or tube:

$$\frac{0.85f_c'A_{cc}}{A_sF_{my}}\frac{P_a}{A_{bc}} \le 0.75f_c'$$

For concrete encased structural steel:

$$\frac{0.6f_c'A_{cc}}{A_sF_{my}}\frac{P_a}{A_{bc}} \le 0.75f_c'$$

COMMENTARY

Axial Compression Strength of Stocky Columns

The compression strength of composite column cross sections can be estimated accurately as the sum of the compressive capacities from each component part, the concrete, the structural shape or tube, and the longitudinal reinforcement. Superposition of component capacities at ultimate is a reliable procedure if each of the components maintains stiffness to resist increasing strains until the nominal capacity of all components is attained. Longitudinal reinforcing bars and contained steel shapes are restrained from local buckling as long as the concrete remains unspalled or unbroken. Thus, a limit strain taken as 0.0018 at which unconfined concrete remains unspalled and stable serves analytically to define a failure condition for composite cross sections under uniform axial strain. Unless and until laboratory tests might reveal beneficial interactions that promote load sharing among component materials subjected to larger strains for concrete, the upper limit strain of 0.0018 for axially loaded cross sections is recommended. That limit leads in turn to an upper bound on nominal yield strength for structural steel

$$\max F_y = 0.0018E_s \approx 55 \text{ ksi}$$

in composite cross sections. If structural shapes develop yield stresses greater than 55 ksi, it is assumed that the composite concrete is not available to provide local stability and load sharing at the higher levels of steel stress.

The equation that relates an ultimate thrust capacity P_n to the sum of capacities among component parts can be written:

$$P_n = A_sF_y + A_{cr}F_{cr} + 0.85f_c'A_{cc} \qquad (1)$$

The quantity P_n is analogous to the squash load P_o for reinforced concrete sections or P_y for steel sections under purely axial thrust. The strength equation can be transformed into an effective composite stress F_{my} formulation for a composite cross section by dividing both sides by the structural steel area A_s:

$$F_{my} = \frac{P_n}{A_s} = F_y + F_{cr}\frac{A_{cr}}{A_s} + 0.85f_c'\frac{A_{cc}}{A_s} \qquad (2)$$

Equation (2) is the expression that is recommended for filled tube composite members. With the steel encasement always available to provide some lateral confinement to the concrete core, there is no uncertainty that the contained concrete will reach, before spalling, at least as much strength as that reached by concrete in unconfined standard cylinders such as those used in establishing f'_c. In contrast, there is less certainty that the $0.85f'_c$ stress will be attained by unconfined concrete, and if the unconfined concrete fails to reach $0.85f'_c$, the longitudinal reinforcement it stabilizes may not reach its specified strength of F_{cr}. Thus, for applications that rely on unconfined concrete, the ACI Code capacity reduction factor of 70 percent was applied to the concrete and reinforcing bar components of Eq. (1) to obtain the effective stress recommended for concrete encased composite columns. There is a specified upper limit for f'_c, because no test data are available to indicate composite column behavior with f'_c values in excess of 8 ksi. A lower limit $f'_c = 3000$ psi is recommended in order to encourage a degree of quality control commensurate with this readily available and familiar grade of structural concrete.

Column Slenderness

Slenderness can be expressed analytically for columns as a measure of the member flexural stiffness EI/L. The straightforward application of a material stiffness, a cross-section moment of inertia, and an effective length, customary for the design of plain structural steel columns, cannot be used for reinforced concrete members. The contribution of each component is difficult, if not impossible, to define precisely for reinforced concrete columns. The existence and extent of flexural cracking may vary throughout the height of a concrete column. Not only is concrete in a column less homogeneous than steel, but the apparent value of E is altered by sustained loads. Since concrete columns occur in rigid monolithic type frames, the effective length of columns cannot be established easily. Nevertheless, designers of necessity must consider slenderness effects in order to proportion columns that are adequate to support assigned loads. The consideration of slenderness effects in concrete columns requires cautious estimates of concrete stiffness.

The amount of stiffness available from the flexure of concrete contained within a pipe or tube is higher than that which can be anticipated from uncontained concrete. Of more significance, however, the overall stability of a steel tube filled with concrete will be influenced much more by the steel tube than by the contained concrete. Conversely, the overall stability of a concrete encased structural shape composite member will be influenced more significantly by the concrete than by the steel.

The influence of tensile cracking appreciably reduces the effective stiffness of concrete, even when the concrete is confined inside steel tubing. The reliability of attaining a specified quality for concrete is more difficult to control than it is for steel. The expressions for effective stiffness E_m permit the use of 40 percent of the nominal initial stiffness of contained concrete inside steel tubes, while only 20 percent of that stiffness is permitted for unconfined concrete. These coefficients are consistent with values recommended in the ACI Building Code expressions for flexural stiffness EI to be used for estimates of inelastic buckling loads[2] equivalent to $A_sF'_e$. The ACI Code expressions include a parameter for the softening influence of creep in concrete that is subjected to sustained compression loading. Every composite column contains steel in at least 4 percent of the cross section and steel occurs symmetrically on all faces of concrete filled tube columns. The influence of creep as well as the influence of cracking have been accommodated adequately by the 40 percent and the 20 percent coefficients specified in Eqs. (B) and (D) for E_m for filled tubes and encased shapes, respectively.

It should be noted that the expressions for effective stiffness E_m employ ratios, rather than moments of inertia, of areas of each material. Trial calculations for weak axis buckling failure modes of encased rolled shapes involved ratios of moments of inertia that grossly distorted the influence of concrete, whereas the area ratios produced results consistent with those obtained by testing slender composite columns.

The reduction of strength as column slenderness increases has been described analytically by familiar S-shaped curves. The specific shape of "column" curves that most accurately reflect the relationship between thrust capacity and column slenderness for various types of steel cross sections has been the subject of extensive study for decades.[6] It is likely that the variability of concrete stiffness would obscure variations that steel column forms or shapes might produce among strength-slenderness functions. Engineers who are familiar with the form of the AISC column curve for plain steel columns should find the application of the same curve to composite columns convenient and familiar. Use of the curve [Eq. (1.5-1) of the AISC Specification[1]] in design is all but impossible without design tabulations of calculated values obtained from the equation of the curve. Design aids for composite columns can be constructed, and some sample tables and graphs are provided with the example designs in this report.

The conventional definition of a radius of gyration cannot be applied rigorously to non-homogeneous or composite cross sections. An index of cross section breadth to resist flexure is necessary as a measure of slenderness, nonetheless. The radius of gyration of a solid rectangle is about 30 percent of its depth, and the radius of gyration of a box or W shape can approach 50 percent of the depth of the section. The steel shape and the concrete portions of composite cross sections contribute to resistance against flexural displacement; if the steel predominates, the radius of gyration of the steel is appropriate for the whole section. If flexural deformation is resisted predominantly by concrete, the radius of gyration for concrete is appropriate for slenderness calculations. In either case an effective radius of gyration for the composite section will be somewhat greater than the larger of values for each material taken separately. Until a more rigorous definition is demonstrated, it is recommended that the larger of radius of gyration values for either steel or concrete by used in the slenderness index l/r for composite columns.

Beam-Columns (Axial Load Plus Bending)

The amount of axial force that can be resisted by steel sections or by concrete sections is greatest when there is a concentric axial force without bending applied to the sections. As the bending moment increases, the axial load capacity decreases. The maximum bending resistance of steel sections exists in the absence of any axial force, and small amounts of axial force create very little reduction in bending capacity. Reinforced concrete cross sections achieve their maximum flexural capacity when some axial force is present to help restrain flexural cracking in the concrete. The use of a linear function to represent axial force and moment interaction capacities leads to unacceptably low estimates of failure in composite steel and concrete cross sections. It is acknowledged that even a parabolic function will lead to underestimates of flexural capacity at low levels of axial force where flexural concrete contributes substantially to bending capacity.

The pure flexure capacity of composite cross sections can be estimated accurately only by means of an iterative process that uses stresses compatible with assumed distributions of strain until compressive and/or tensile capacity is reached. The tedious procedures for such an analysis can be aided by computers, but the variety of possible cross sections would necessitate an extensive library of programming. In lieu of an analytically accurate specification for flexural failure in composite columns, an approximating formula is recommended as a part of the definition of an effective section modulus S_m. The equation for S_m is derived from an expression for estimating pure flexural capacity divided by the yield strength of the structural steel:

$$M_o = F_y S_m = S_{sc} F_y + \frac{1}{3} A_{cr} F_{cr} (h_2 - 2C_r) + A_w F_y \left(\frac{h_2}{2} - \frac{A_w F_y}{1.7 f'_c h_1} \right) \quad (3)$$

Each of the three sources of flexural capacity, the steel shape, the longitudinal reinforcement, and the concrete that is compressed along one edge of the cross section, form components of Eq. (3). It is assumed that at least 1/3 of the longitudinal bars in a cross section can be considered concentrated in a position located c_r from the edges of the cross section. In order to obtain the third term of Eq. (3), the web of shapes encased in concrete is considered to be tension reinforcement for a concrete cross section with a flexural depth taken as half the overall thickness in the plane of bending. The mechanism is apparent when bending occurs about the minor axis of the shape, as the web does not contribute to the plastic section modulus used in the first term of Eq. (3). Even though the web contributes a minor portion of the major axis section modulus, at ultimate moment about the major axis of encased shapes the neutral axis is not at mid-depth, but is closer to the compression edge as concrete participates in resisting flexure. The resulting increase in the distance between total internal tension and total compression forces more than offsets the apparent double use of web area as a part of the term S_{sc} and as a part of the third term in Eq. (3). The sidewall regions of round or rectangular filled tubes may permit a similar term for A_w, but no recommendation can be proposed at this time. It is conservative and safe for the present to use $A_w = 0$ for filled tubes.

Secondary moments in beam columns can lead to stress increases or failure conditions that are not revealed by forces obtained from a first order frame analysis. The influence of secondary moments is accommodated in the proposed specification by means of a moment magnifier quantity $C_m/(1 - f_a/F'_e)$, as in the present AISC Specification.

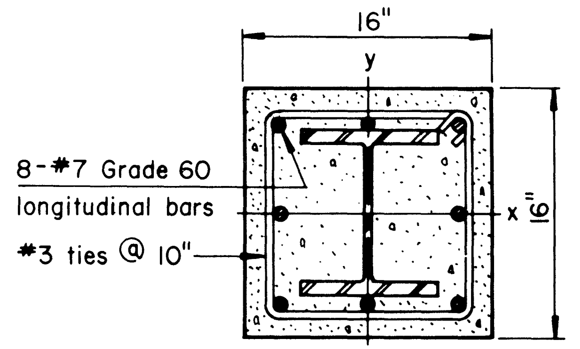
In order to remain consistent with the existing form of beam-column interaction equations in the AISC Specification, the proposed beam-column stress equation is shown in the same general, biaxial bending relationship. The linear addition of apparent stresses that are caused by bending about the y-axis and bending about the x-axis of a cross section leads to an exaggeration of the ratio between strength used and strength available. A less cautious biaxial bending limit can be obtained from the reciprocal axial force relationship suggested for reinforced concrete by Bresler,[5] and the few laboratory tests of biaxially loaded composite members indicate that the reciprocal axial force equation is still safe.

The recommended minimum quantity of transverse reinforcement and longitudinal reinforcement in encasement should be adequate to prevent severe spalling of the surface concrete during fires. Since encased shapes provide considerably more minimum longitudinal steel for reinforced concrete than ACI requires, there is no need for as much as the 1 percent supplementary longitudinal steel as specified by the ACI Building Code.[2] Laboratory data and field experience must be accumulated before improved recommendations can be offered.

The wall thickness minima proposed are derived from relationships identical to those in the present ACI Building Code.[2] The same relationships appear in other design documents.[6,7]

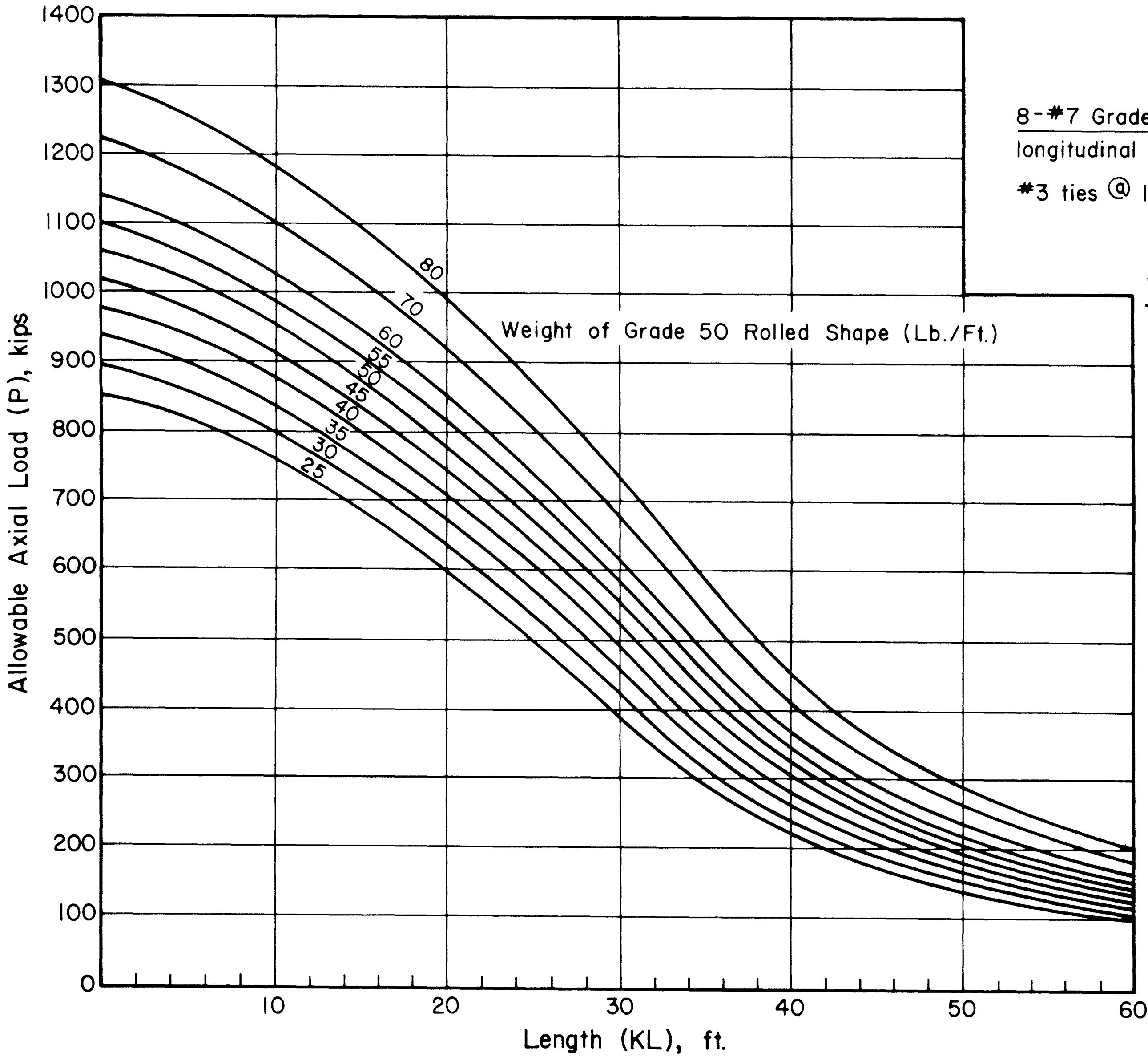
DESIGN AIDS

At the design stage, the material qualities F_y and f'_c, as well as general configurations of composite cross sections, are known or assumed. Equations for allowable axial load can be solved for specific types of cross sections, and values of section modulus S_m from Eq. (3) can be determined. Table 1 contains values of allowable axial force P_a for W8 and W10 shapes of A36 steel encased in 16-in. square cross sections with f'_c = 3000 psi concrete and almost 1 percent longitudinal Grade 60 reinforcement.



W8 and W10 Encased Shapes

F_y = 50 ksi, f'_c = 6 ksi



Modified Section Modulus		
Shape	S_{mx} in.^{3}	S_{my} in.^{3}
W8x24	53.7	38.5
28	59.1	41.4
31	62.3	44.1
35	67.2	46.6
40	73.8	50.5
48	83.4	55.1
58	96.8	63.1
67	108.0	68.7
W10x26	65.5	42.5
30	72.4	45.8
33	73.3	47.5
39	81.8	51.8
45	90.7	54.9
49	95.7	59.8
54	103.0	63.3
60	112.0	68.3
68	124.0	74.2
77	137.0	80.7

Fig. 1. Column design aid—allowable axial load vs. effective length

The strong axis and the weak axis values of modified radius of gyration for W8 and W10 shapes encased in 16 in. of concrete are governed by the lower bound $0.3h = 4.8$ in. Therefore, the allowable axial force is a function of area of the steel shape, not any additional geometrical property of the shape. Area is easily converted to weight per foot in order to develop the graph of Fig. 1. For Fig. 1 a strong concrete encasement with $f'_c = 6000$ psi and longitudinal bars comprising 1.9 percent of the cross-section area were used, and $F_y = 50$ ksi was used for the shapes. This type composite column should permit more axial force than that which can be supported in the same size column reinforced only with longitudinal bars.

The overall cost of concrete filled steel tube composite columns will be increased a negligible amount if the quality of concrete $f'_c = 5000$ psi instead of 3000 psi. Table 1 contains values of allowable axial force and values of S_m for round tubing of $F_y = 35$ ksi filled with $f'_c = 5000$ psi concrete.

The design aids of Tables 1 and 2, and Fig. 1 are presented as examples of data useful in the design process. Alternate presentations of similar data can be generated with relative ease from specified allowable stress equations. The design examples that follow will illustrate applications of the recommended rules and the design aids. Some comparisons among AISC and ACI design results accompany examples.

Some examples of column design will illustrate applications of the proposed composite column specification. Subsequent comparisons with all steel or all reinforced concrete cross sections adequate for the same load conditions illustrate the relative effectiveness of composite columns.

Example 1—Large Axial Load, No Moment

Select a 16-in. square cross section for an axial load comprised of 445 kips dead load and 160 kips live load for an unsupported length of 12 ft. Use 3000 psi concrete and an A36 steel core.

From Table 1, the required service load thrust of 445 + 160 = 605 kips can be supported with a W10x60 core and a column length $KL = 12$ ft.

If only steel were used, Table 1, pg. 3-15 of the AISC Manual,[1] indicates that a W14x111 would be the necessary size of an A36 steel shape.

For the ultimate axial force, $P_u = (1.4 \times 445) + (1.7 \times 160) = 895$ kips, regulations of the ACI Building Code[2] would require a W10x8 core in the 16-in. square cross section reinforced longitudinally with the same 8 #5 Grade 60 bars. Since moments are negligible, the ACI creep coefficient β_d could be taken as zero, but the minimum eccentricity requirements of Sect. 10.11.5.4 and moment magnification for a column in single curvature must be applied when the unsupported length exceeds 22 times the least radius of gyration, taken here to be $0.3 \times 16 = 4.8$ in. A core size of W10x77 would have been acceptable by ACI rules if the length Kl were $4.8 \times 22 = 106$ in. or less.

Example 2—Large Moment, Less Axial Load

Select a 16-in. square cross section for an axial load comprised of 84 kips dead load and 60 kips live load together with a dead load moment of 54 kip-ft and a live load moment of 110 kip-ft. The unsupported length is 12 ft, $C_m = -0.5$, and 3000 psi concrete is to be used as well as an A36 core shape.

Since the member is in double curvature ($C_m = -0.5$), slenderness is not likely to be of concern. The specified axial force of 84 + 60 = 144 kips appears to be well within the allowable loads of Table 1. The bending moment of 54 + 110 = 164 kip-ft, however, may require some of the larger values of S_m listed in Table 1. Assume that f_a/F_a might be near 1/3, such that $1 - (P/P_a)^2 = 8/9$ as the ratio that might be available for f_b/F_b.

$$\text{Estimated required } P_a = \frac{P}{1/3} = \frac{144}{0.33} = 432 \text{ kips}$$

$$\text{Estimated required } S_m = \frac{M}{(8/9)F_b} = \frac{16 \times 12}{0.89 \times 22} = 101 \text{ in.}^3$$

The W10x54 shows a value $P_a = 591$ kips for $KL = 12$ ft. in Table 1, and a value $S_m = 95.7$ is shown in Table 1 for the same shape. Check Eq. (E) with $C_m/[1 - (f_a/F'_c)]$ taken as unity.

$$\left(\frac{f_a}{F_a}\right)^2 + \frac{f_b}{F_b} = \left(\frac{P}{P_a}\right)^2 + \frac{M}{S_m(0.6F_y)} = \left(\frac{144}{591}\right)^2 + \frac{162 \times 12}{95.7(0.6)36} = 1.011$$

The W10x60 must be used as the core shape, or perhaps longitudinal reinforcement could be made larger.

A more detailed analysis of the cross section capacity in accordance with the ACI Building Code would permit an ultimate moment of 262 kip-ft when the ultimate thrust is $(1.4 \times 84) + (60 \times 1.7) = 220$ kips. The required ultimate moment of $(54 \times 1.4) + (1.7 \times 110)$ is 263 kip-ft. Therefore, the proposed regulations would require the same cross section as that which the ACI Code would require for this example.

Example 3—Very High Axial Load; Composite Column in Lieu of Reinforced Concrete

In the lowest elevation of a reinforced concrete frame with 16-in. square columns throughout, a service load thrust of 814 kips plus a service load moment of 131 kip-ft (with ACI load factors $P_u = 1206$ kips and $M_u = 200$ kip-ft) requires more longitudinal reinforcement than can be placed in the available space, even with $f'_c = 6$ ksi. Design a composite section with a 50 ksi steel core shape in $f_c = 6$ ksi encasement. The unsupported length $KL = 11$ ft and $C_m = 0.8$.

Table 1. Composite Column Allowable Loads (kips)

Column Data: 16 in. × 16 in.; enclosed A36 steel shapes; f'_c = 3 ksi; longitudinal bars 8 #5 grade 60

	W8										W10								
Wt/ft	28	31	35	40	48	58	67	22	26	30	33	39	45	49	54	60	68	77	88
6	478	495	519	547	595	654	706	443	465	490	507	543	579	601	629	664	712	764	830
8	468	486	509	536	584	643	694	433	455	480	497	532	568	590	617	653	700	751	816
10	458	475	498	525	572	630	681	423	445	469	486	521	556	578	605	640	686	737	801
12	446	463	486	513	559	616	666	412	434	457	474	509	543	565	591	626	672	721	784
14	434	451	473	499	545	602	650	400	421	445	462	496	530	551	577	611	656	705	767
16	420	437	459	485	530	586	634	387	408	432	448	482	515	536	562	595	639	687	748
18	406	423	445	470	514	569	616	373	395	417	434	467	500	520	545	578	622	669	729
20	392	408	429	455	498	552	598	359	380	403	418	451	484	503	528	561	603	649	708
22	376	392	413	438	481	533	579	344	365	387	403	435	466	486	510	542	584	629	686
24	360	376	396	421	462	514	558	328	348	370	386	417	449	468	492	522	563	607	663
26	343	358	379	403	443	494	537	312	332	353	368	399	430	448	472	502	542	585	640
28	325	340	360	384	424	473	515	294	314	335	350	380	410	428	451	481	520	562	615
30	307	321	341	364	403	451	492	276	296	317	331	361	390	408	430	459	497	538	589
32	287	302	321	344	381	428	468	257	277	297	311	340	369	386	408	436	473	512	562
34	267	282	300	322	359	405	443	238	257	277	291	319	347	364	385	412	448	486	535
36	246	260	279	300	336	380	418	217	236	256	269	297	324	341	361	387	422	459	506
38	225	238	256	277	312	355	391	196	214	234	247	274	301	316	336	362	395	431	476
40	203	216	233	253	287	328	364	177	193	211	224	250	276	291	311	335	368	402	446
42	184	196	211	230	261	301	335	161	175	192	203	227	251	264	284	308	339	372	414
44	168	178	192	209	238	274	306	146	160	175	185	207	229	242	259	281	309	341	381
46	153	163	176	191	218	251	280	134	146	160	169	189	209	221	237	257	283	312	348
48	141	150	162	176	200	231	257	123	134	147	156	174	192	203	217	236	260	286	320
50	130	138	149	162	185	213	237	113	124	135	143	160	177	187	200	217	240	264	295
S_{mx} (in.^{3})	52.9	56.1	60.9	67.3	76.7	89.5	99.8	53.3	59.1	65.7	66.8	75.2	83.8	88.9	95.7	105	116	128	143
S_{my} (in.^{3})	35.3	37.9	40.3	44.0	48.4	55.8	60.8	34.1	36.1	39.1	41.0	44.4	48.0	53.0	56.3	60.9	66.3	72.1	79.2

Effective Length KL (ft)

Table 2. Allowable Axial Loads on Concrete-Filled Steel Tube Columns (kips)

Steel Tube: E_y = 35 ksi   Concrete: f'_c = 5 ksi

O.D. (in.)	6				8				10				12			
Wall t (in.)	1/8	1/4	3/8	1/2	1/8	1/4	3/8	1/2	1/8	1/4	3/8	1/2	1/4	3/8	1/2	5/8
3	109	148	186	221	179	233	285	335	264	333	399	464	449	530	609	687
6	101	139	174	207	170	222	272	320	255	321	386	449	437	516	594	669
9	92	126	159	189	159	209	257	302	243	308	370	431	422	500	575	649
12	80	112	141	168	147	194	239	282	229	292	352	410	406	481	554	625
15	67	95	120	144	133	177	219	259	214	274	332	387	387	460	531	599
18	52	76	98	116	117	158	197	233	197	255	309	362	367	437	505	570
21	44	62	79	94	99	138	172	204	179	234	285	338	344	412	477	539
24	38	54	69	82	80	115	146	173	159	210	259	304	321	385	447	506
27	34	48	61	73	71	98	122	145	137	186	230	271	295	356	415	470
30		43	55	66	64	88	110	131	116	159	199	236	268	326	380	432
33					58	80	100	119	105	140	172	203	239	293	344	392
36					54	73	92	109	96	128	158	186	208	259	305	349
39						68	85	100	89	118	146	172	188	229	267	304
42									83	110	135	159	175	212	248	282
45									77	103	126	149	163	198	232	263
48										96	119	140	153	186	217	247
51											112	131	144	175	204	232
54													136	165	193	220
57													129	156	183	208
														149	174	198
S_m	3.32	6.23	8.78	11.0	5.99	11.4	16.4	20.8	9.46	18.2	26.3	33.8	26.6	38.6	49.9	60.4

Effective Length KL (ft)

Values below heavy rule:

$$\frac{kL}{rC_c} > 1.0$$

No values are listed for:

$$\frac{kL}{rC_c} > 1.6$$

Use Fig. 1 with 8 #7 bars in $f'_c = 6$ ksi encasement. Assume that f_b/F_b is about 50% and f_a/F_a will be about 70%.

$$\text{Estimated } P_a = \frac{P}{0.7} = \frac{814}{0.7} = 1162 \text{ kips}$$

With $F_b = 30$ ksi:

$$\text{Estimated } S_m = \frac{M}{0.5F_b} = \frac{131 \times 12}{0.5(30)} = 105 \text{ in.}^3$$

From Fig. 1, note that $S_m = 112$ in.^{3} for W10x60, but P_a for $KL = 11$ ft and a 60 lb shape is only 1012 kips.

Try the W10x68:

$P_a = 1077$ kips when $KL = 11$ ft

$S_m = 124$ in.^{3}

The value of $A_sF'_e$ can be obtained from Fig. 1 by using P_a at a value KL greater than C_c, and then multiplying that value by the square of the ratio between KL values:

$A_sF'_e = 335$ when $KL = 44$

$A_sF'_e = 335(44/11)^2 = 5360$ kips when $KL = 11$ ft

Eq. (E) becomes

$$\left(\frac{814}{1077}\right)^2 + \left[\frac{131 \times 12}{124} \times \frac{1}{30}\left(\frac{0.8}{1 - \dfrac{814}{5360}}\right)\right]$$

$$= 0.571 + [0.423 \times (\text{use } 1)] = 0.994 \textbf{ o.k.}$$

Note that the moment magnification term cannot be taken less than 1.

Use W10x68 in the 16 x 16-in. $f'_c = 6$ ksi encasement.

The design procedure of the ACI Building Code requires a more precise (and considerably more tedious) evaluation of cross section capacity. The "old" W10x72 Grade 50 core shape with 8 #7 Grade 60 longitudinal bars in a 16-in. square encasement of $f'_c = 6000$ psi concrete can resist a moment of 240 kip-ft where the axial force P_u is 1206 kips. Thus the ACI procedure and the proposed technique would result in a selection of the same cross section for this example.

If steel alone were to be used, a W14x132 Grade 50 shape is necessary for the specified loading condition.

Example 4—Concrete-filled Steel Tube Column

Select a concrete-filled steel tube to support an axial load of 395 kips if the unsupported length is 14 ft.

Table 2 indicates that a 12-in. steel tube with ³⁄₈-in. wall or a 10-in. tube with ½-in. wall would be adequate if either is filled with $f'_c = 5$ ksi concrete. None of the A36 steel pipe columns listed in the AISC Manual is adequate, without concrete fill, to carry the 395-kip load with a 14-ft unsupported height. A square tube 10 x 10 x ⁵⁄₈ would be adequate without being filled with concrete.

The specified slenderness of a 12-in. tube 14 ft long would require a minimum eccentricity of 0.96 in. that must be magnified about 30 percent according to the ACI regulations, such that a ⁵⁄₈-in. thick 12-in. tube would be needed to satisfy the ACI Building Code. The empty 12-in. tube with a ⁵⁄₈-in. wall thickness would be adequate for the 395-kip load when the AISC Specification is used.

Example 5—Analysis Without Design Aids (See Fig. 2)

a. Determine allowable axial load if $KL = 18$ ft-4 in.

Section properties:

Gross Area $A_g = (8 \times 28) + (10 \times 12) = 344$ in.^{2}
$A_s = 14.7$ in.^{2}
$A_{cr} = 10 \times 0.44 = 4.40$ in.^{2}
$A_{cc} = 344 - 14.7 - 4.4 = 324.9$ in.^{2}

Is A_s large enough for section to qualify as composite? $A_s = 14.7$ in.^{2}, which is more than 4 percent of $A_g = 0.04 \times 344 = 13.76$ in.^{2}, and section does qualify as composite.

From Eq. (C):

$$F_{my} = F_y + 0.7\, F_{cr}\frac{A_{cr}}{A_s} + 0.6 f'_c \frac{A_{cc}}{A_s}$$

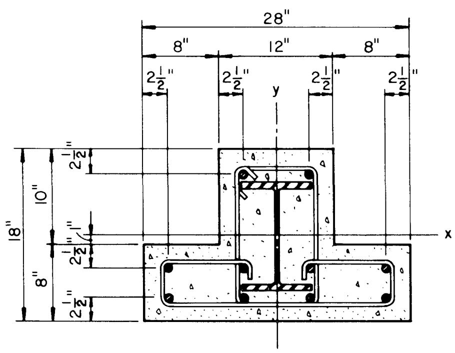
$$= 44 + 0.7(55)\frac{4.40}{14.7} + 0.6(5.0)\frac{324.9}{14.7}$$

$$= 121.8 \text{ ksi}$$

From Eq. (D):

$$E_m = 29{,}000 + 0.2\, E_c \frac{A_{cc}}{A_s}$$

$$= 29{,}000 + 0.2(4050)\frac{324.9}{14.7} = 46{,}900 \text{ ksi}$$



Concrete: $f'_c = 5$ ksi ($E_c = 4050$ ksi)
Reinforcing bars: 10-#6 Grade 60 (Use $F_{cr} = 55$ ksi)
Core shape: W12x50, $F_y = 44$ ksi

Fig. 2. Design Example 5

Radius of gyration:

Strong Axis: $r_{sx} = 5.18$ in.
Concrete $I_x = 9030$ in.^{4}; $r_{cx} = 5.12$ in.

Weak Axis: $r_{sy} = 2.17$ in.
Concrete $I_y = 16{,}070$; $r_{cy} = 6.84$ in.

Potential buckling is more likely to occur about the x-axis (strong axis of core shape), and since $r_{sx} > r_{cx}$, use $r_m = r_{sx}$.

$$\frac{Kl}{r} = \frac{(18 \times 12) + 4}{5.18} = 42.5$$

$$C_c = \pi \sqrt{\frac{2E_m}{F_{my}}} = \pi \sqrt{\frac{2 \times 46{,}900}{121.8}} = 87.2$$

$$F_a = \frac{\left[1 - \frac{1}{2}\left(\frac{Kl}{r_m C_c}\right)^2\right] F_{my}}{\frac{5}{3} + \frac{3}{8}\left(\frac{Kl}{r_m C_c}\right) - \frac{1}{8}\left(\frac{Kl}{r_m C_c}\right)^3}$$

$$= \frac{\left[1 - \frac{1}{2}\left(\frac{42.5}{87.2}\right)^2\right] 121.8}{\frac{5}{3} + \frac{3}{8}\left(\frac{42.5}{87.2}\right) - \frac{1}{8}\left(\frac{42.5}{87.2}\right)^2} = 58.5 \text{ ksi}$$

$$P_a = A_s F_a = 14.7 \times 58.5 = 860 \text{ kips}$$

b. Determine allowable longitudinal force if an end moment of 114 kip-ft is applied at one end, placing the top of the cross section in compression. Use $KL = 18$ ft-4 in. and $C_m = 0.6$.

$$S_m = S_{sc} + \frac{1}{3}A_{cr}(h - 2C_r) + \left(\frac{h_2}{2} - \frac{A_w F_y}{1.7 f'_c h_1}\right) A_w \qquad \text{(F)}$$

The unsymmetric cross section does not fit the rectangular model from which Eq. (F) was developed. In lieu of an analysis with compatible failure strains, as per ACI Code practice, replace the second term of Eq. (F) with a strength equivalent for the 8 reinforcing bars that will be in tension under pure bending at failure. Estimate the distance from centroid of tension bars to centroid of compression on the cross section as $18 - 4 - 4 = 10$ in. Since the top is in compression, $h_1 = 12$ in. and $h_2 = 18$ in. for the third term of Eq. (F). $A_w = 0.37\ (12.19) = 4.51$ in.^{2}

$$S_m = 64.7 + \frac{1}{3}(8)(0.44)(10) + \left(\frac{18}{2} - \frac{4.51 \times 44}{1.7 \times 5 \times 12}\right) 4.51 = 108 \text{ in.}^3$$

$$f_b = \frac{M}{S_m} = \frac{114 \times 12}{108} = 12.7 \text{ ksi}$$

$$F'_e = \frac{12\pi^2 E_m}{23\,(Kl/r)^2} = \frac{12\pi^2(46{,}900)}{23(42.5)^2} = 134 \text{ ksi}$$

$$F_{by} = 0.6 \times 44 = 26.4 \text{ ksi}$$

From Eq. (E), solve for f_a in

$$\left(\frac{f_a}{F_a}\right)^2 + \left[\frac{C_m}{\left(1 - \frac{f_a}{F'_e}\right)} \frac{f_b}{F_b}\right] = 1$$

$$\left(\frac{f_a}{58.5}\right)^2 + \left[\frac{0.6}{\left(1 - \frac{f_a}{134}\right)} \times \frac{12.7}{26.4}\right] = 1$$

If $f_a < 80$ ksi, $\dfrac{C_m}{\left(1 - \frac{f_a}{F'_e}\right)} < 1$; $\therefore$ use 1.

$$\left(\frac{f_a}{58.5}\right)^2 + (1)\frac{12.7}{26.4} = 1$$

$$f_a = 42.1 \text{ ksi}$$

$$P_a = A_s f_a = 14.7\ (42.1) = 619 \text{ kips}$$

The examples illustrate design applications of the proposed regulations for composite columns. Examples 1 and 4 show that the proposed regulations permit, on axially loaded members, loads significantly larger than those permitted by ACI regulations. Examples 2 and 3 showed that the proposed regulation and the ACI Building Code produce almost the same allowable forces for the eccentrically loaded condition of a beam-column that must resist a significant amount of moment in addition to axial load.

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APPENDIX A

Allowable Loads Compared to Test Loads

Tests of composite columns have been reported from several laboratories during the past two decades. On the basis of cross sections and material properties that were described in the test reports, it was possible to compare test loads with the allowable loads determined in accordance with the proposed design specification. Tabulations of specimen properties, test loads, and allowable loads are presented for four categories of composite column tests. The right hand column of each tabulation contains the ratio between test load and allowable load for each test cited. In some cases the reported data represents the average of material strengths and test loads on three identical specimens. In computing allowable loads a maximum value, F_y = 55 ksi was used when reported values exceeded 55 ksi.

Table 3 contains data from tests on axially loaded concrete filled steel tubes of round or square cross section.[8–13] Among the 73 elements of test data there are two unacceptably low ratios between test load P_{test} and allowable load P_a. The ratios 1.28 and 1.34 were obtained from a set of specimens involving spiral welded tubing.[11] Although the proposed specification does not prohibit spiral welded tubing applications for slender composite columns, it should be assumed that all tubing employed for structural columns would satisfy the applicable ASTM regulations related to mechanical properties of the material. The average value of the ratio between test load and allowable load was an acceptable 2.26, with a standard deviation equal to 20 percent of the average.

Data shown in Table 4 involves tests of axially loaded encased shapes with slenderness ratios Kl/r_m between 5 in. and 147 in. and structural shapes which occupied 5 to 13 percent of the gross area of the concrete encased cross sections.[14–17] The lowest of the 29 ratios between test load and allowable load was 1.70, and the average value was 2.04 with a standard deviation equal to 16.8 percent of the average value.

Eccentrically loaded concrete-filled steel tubes were employed for the data[12] given in Table 5. The allowable eccentric load was evaluated from trial and error solutions of Eq. (E) after values of P_a , S_m , and F'_e had been determined for each specimen. With a low value 1.90 and an average value 2.50 for the ratio between test load and allowable load, the expected underestimation of beam column capacity from the parabolic equation for load-moment interaction at ultimate becomes apparent. The standard deviation of 15 percent of the average value indicates that the underestimation of capacity produced a range of results the same as that observed for the axially loaded specimens. Specimens included slenderness ratios from 19 to 24 in. and tubing that occupied between 16 and 33 percent of the gross cross-section area.

Test reports[14–18] for eccentrically loaded encased shape composite columns are listed in Table 6. Among the 60 sets of data that are given, slenderness ratios varied from 15 in. to 70 in. and the percentage of steel in cross sections varied from 2.6 to 13. As for Table 5, an iterative procedure was used to solve Eq. (E) for the value of allowable loads for specimens that had axial loads applied at a constant eccentricity. The data displays results for eight specimens on which the total bending force was held constant while loads were increased until failure occurred. Generally before axial loads were increased, the magnitude of moment that was applied produced ratios f_b/F_b so high that very little apparent capacity remained for load P according to Eq. (E). Ratios between test loads and allowable loads thus established were too high to be meaningful, and they were not included in the average value calculation shown at the bottom of Table 6. These eight test specimens do, however, indicate that the expression proposed for an effective section modulus produces allowable moment values that are quite safe. The average ratio between test loads and allowable loads for the eccentrically loaded encased shapes was again acceptable at 2.02, with a standard deviation 15.3 percent of the average value. None of the ratios was less than 1.48.

Table 3. Axially Loaded Concrete-Filled Tubes

O.D. (in.)	A_s (in.^{2})	A_c (in.^{2})	F_y (ksi)	f'_c (ksi)	I_s (in.^{4})	Kl (in.)	P_{test} (kips)	P_a (kips)	$\frac{P_{test}}{P_a}$
3.74	5.07	5.92	39.9	2.94	6.79	33.9	229	120	1.91
						55.9	209	110	1.90
						78.0	203	98	2.07
	1.63	9.36	50.7	3.62	2.63	53.9	150	61.2	2.45
						55.9	131	55.8	2.85
						78.0	119	49.2	2.42
8.50	4.22	52.5	42.3	3.32	36.7	87.4	371	178	2.09
				4.32			509	201	2.53
	6.13	50.6	56.8	3.32	52.3		549	264	2.08
			50.8	4.32			645	268	2.41
3.74	1.63	9.36	49.0	3.49	2.63	80	104	47.3	2.20
4.76	2.14	15.6	45.2	3.06	5.71	41.3	162	76.2	2.13
				3.51			192	79.4	2.42
				3.06		91	143	69.7	2.21
				3.51			163	67.2	2.43
	3.11	14.7	49.8	3.06	8.04	41.3	227	107	2.13
				3.51			245	110	2.23
				3.06		91	180	89.4	2.01
				3.51			195	91.7	2.13
1.00	0.11	0.68	76.0	4.04	0.0124	42	3.52	1.96	1.80
1.50	0.48	1.29			0.116		24.7	11.1	2.22
2.00	0.40	2.74			0.185		27.1	15.9	1.70
3.00	0.59	6.47		3.95	0.646		72.0	32.7	2.20
14.0	18.7	135	51.5	5.52	769	22	2576	950	2.71
				4.76			2408	898	2.69
	13.5	140		3.40	431	21.1	1671	654	2.55
	8.07	146	40.1	3.04	316	21.5	791	417	1.90
5.01	0.99	18.7	53.8	9.60	3.04	19.7	289	119	2.43
			47.7				289	115	2.51
5.00	1.78	17.9	53.8		5.31	20.0	293	140	2.10
			47.7				293	134	2.19
4.00	1.49	11.1	87.8	4.95	2.78	60	184	88.5	2.23
							180		2.18
4.76	2.33	15.5	65.5	4.99	6.16	413	260	119	2.19
				4.29			246	114	2.16
				3.76			214	110	1.96
6.00	2.29	26.0	60.2	3.03	9.88	89.4	211	91.1	2.32
							198		2.17
3.01	0.63	6.5	52.7	3.62	0.88	55	55	83.4	2.25
				5.93		24	95.2	36.2	2.63
				3.76			74.2	29.9	2.48
4.50	1.72	14.2	60.0	4.20	4.11	33	165	85.5	1.93
5.00	1.46	18.2	42.0	5.10	4.40	59	143	73.2	1.95
6.00	1.14	27.1	48.0	3.05	5.02		153	67.5	2.27
				3.75			163	75.8	2.15
5.51	6.14	17.7	38.5	4.66	20.0	16	663	180	3.68
			39.0				663	182	3.64
5.53	3.25	20.8	41.9	4.74	11.6		410	129	3.17
			43.2				410	132	3.12
6.62	3.62	30.6		4.56	18.7	32	451	159	2.84
				6.26			489	184	2.66
				3.34			392	141	2.79
3.50	2.36	7.26	58.0	5.81	3.17	68	138	74.6	1.85
				5.75		56	160	81.4	1.97
				5.65		44	161	87.3	1.84
				6.06		32	206	94.0	2.19
				5.92		20	223	98.3	2.27
3.25	0.55	7.74	70.0	6.00	0.705	68	50.5	30.1	1.68
				5.36		56	66.2	32.6	2.03
				5.92		44	80.0	37.5	2.13
						32	90.0	40.7	2.21

(Cont'd next page)

O.D. (in.)	A_s (in.^{2})	A_c (in.^{2})	F_y (ksi)	f'_c (ksi)	I_s (in.^{4})	Kl (in.)	P_{test} (kips)	P_a (kips)	$\frac{P_{test}}{P_a}$
						20	110.0	43.3	2.54
						10	119.2	45.1	2.64
6.64	2.13	32.5	43.2	2.60	11.4	12	298	97.1	3.07
						78	185	87.1	2.12
				4.95		12	274	135	2.02
						78	206	119	1.73
			46.0	5.30		12	294	145	2.03
						78	170	127	1.34
				4.87		12	299	136	2.17
						78	155	121	1.28
	3.98	30.6	32.1	4.72	20.9	90	236	131	1.80
	2.89	31.5	37.8	4.75	15.3	90	254	121	2.10
								Avg.	2.26
								Std. Dev.	0.45
									(20%)

Table 4. Axially Loaded Concrete-Encased Steel Shapes

Steel Shape	h_1 (in.)	h_2 (in.)	A_s (in.^{2})	A_c (in.^{2})	f'_c (ksi)	F_y (ksi)	Kl (in.)	P_{test} (kips)	P_a (kips)	$\frac{P_{test}}{P_a}$
3 × 1½	5	3.5	1.18	16.32	2.60	36.0	46	81.4	36.5	2.23
							64	71.5	33.3	2.15
							82	63.0	29.3	2.15
							100	43.6	24.5	1.78
							118	50.6	18.9	2.68
							136	36.1	14.2	2.54
							154	33.9	11.1	3.06
5 × 4½	7	6.5	5.88	39.6	2.60	36.0	9	352	163	2.15
							46	308	158	1.95
							82	307	150	2.05
							118	288	138	2.09
							153	231	123	1.88
8 × 6	10	8	10.3	69.7	2.60	36.0	84	572	269	2.13
	12	10		110			84	726	310	2.35
	14	12		158			84	856	356	2.41
	16	12	19.1	173			36	1051	568	1.85
							72	990	558	1.77
							108	926	544	1.70
							144	937	526	1.78
							180	933	504	1.85
5½ × 5½	9.5	9.5	6.66	82.6	4.66	41.5	169	482	240	2.01
					4.28	42.7	137	526	253	2.08
					4.77	40.2	98	590	276	2.13
					4.29	40.0	50	572	279	2.05
					4.24	55.0	137	528	287	1.84
					4.24	72.6	168	528	298	1.77
					4.27	70.8	137	554	329	1.68
					4.77	72.5	98	545	382	1.43
					4.39	41.5	136	513	253	2.03
					4.30	70.7	136	517	331	1.56
									Avg.	2.04
									Std. Dev.	0.344
										(16.8%)

Table 5. Eccentrically Loaded Concrete-Filled Steel Tubes

O.D. (in.)	A_s (in.^{2})	A_c (in.^{2})	f_y (ksi)	f'_c (ksi)	r_s (in.)	S_s (in.^{3})	Kl (in.)	P_{test} (kips)	θ (in.)	P_a (kips)	M_o (kip-in.)	P_{all} (kips)	$\frac{P_{test}}{P_{all}}$
4.50	1.72	14.2	55.0	4.20	1.55	1.83	30	100	1.00	75.6	75.5	46.7	2.14
								90	1.18			43.2	2.09
								75	1.75			34.3	2.19
								30	2.82			24.1	2.08
								25	5.76			12.7	1.96
6.00	1.14	27.1	48.0	3.75	2.10	1.67	40	128	0.69	77.4	60.1	50.3	2.54
								95	1.66			30.6	3.11
								64	2.39			22.9	2.79
				3.05				30	4.77	68.6	60.1	12.2	2.46
								30	4.43			13.1	2.29
5.00	1.40	18.2	42.0	5.10	1.77	1.76	42	128	0.61	71.6	55.4	48.8	2.63
								120	0.93			40.5	2.96
								90	1.57			29.4	3.06
								79	1.77			26.9	2.94
								79	1.59			29.1	2.71
								78	1.81			26.5	2.95
								69	2.19			22.8	3.03
								60	2.60			19.7	3.04
								39	3.74			14.2	2.74
								20	7.05			7.8	2.57
								10	13.0			4.25	2.35
5.00	1.85	23.2	55.0	6.50	1.67	5.60	42	250	1.24	116.0	231	85.4	2.93
								150	2.43			65.1	2.30
								150	2.87			59.4	2.53
								100	4.50			44.0	2.29
4.00	1.31	14.7	48.0	3.40	1.60	1.68	42	84	0.52	52.5	60.5	48.0	2.00
								54	1.70			26.5	2.04
								20	5.48			10.6	1.89
4.00	1.94	14.1	48.0	4.18	1.58	2.42	42	98	1.21	60.5	87.1	43.3	2.27
								68	2.38			29.7	2.29
								59	3.22			23.8	2.48
								29	6.93			12.2	2.38
												Avg. Std. Dev.	2.50 0.375 (15.0%)

Table 6. Eccentrically Loaded Concrete-encased Steel Shapes

h_1 (in.)	h_2 (in.)	A_s (in.^{2})	A_c (in.^{2})	A_w (in.^{2})	S_s (in.^{3})	Kl (in.)	f'_c (ksi)	f_y (ksi)	P_{test} (kips)	e (in.)	P_a (kips)	M_o (kip-in.)	P_{all} (kips)	$\frac{P_{test}}{P_{all}}$
9.45	9.45	6.66	82.6	1.52	4.7	135.9	4.80	41.5	251	1.57	248	265	117	2.15
							4.63		265		245	264	116	2.29
							4.03		240		232	259	112	2.14
							4.50	55	265		277	334	138	1.92
							4.36		251		274	332	137	1.83
							4.03		223		267	327	135	1.66
12.60	8.27	5.18	99.0	2.01	2.2	96.5	4.64	39.5	269		244	211	103	2.62
							4.36		234		236	209	101	2.32
							4.28		229		234	208	100	2.28
16.0	12.0	19.1	172.9	5.16	16.3	120	2.52	32.3	672	1.00	478	673	331	2.03
							2.36		486	2.00	470	656	235	2.06
							3.92		515	2.00	553	760	273	1.88
							2.68		361	3.00	487	687	188	1.92
							2.68		296	4.00	487	687	151	1.96
							2.80		262	5.00	493	697	127	2.06
							2.72		231	6.00	489	691	107	2.15
							3.08		199	7.00	514	726	98.3	2.02
							3.00		168	8.00	510	721	86.2	1.95
7.0	6.5	5.88	39.6	1.45	2.93	82	2.80	33.6	161	0.75	138	111	85.0	1.89
									168	0.80			82.6	2.03
						28.6			202	0.75	153		92.9	2.18
									228	0.80			90.1	2.53
						28.6			166	1.00			80.1	2.07
						45.5			224	0.50	14.9		106	2.11
									164	1.00			78.6	2.09
						82			141	1.00	138		73.9	1.91
						118			161	0.50	126		89.5	1.80
									119	1.00			67.8	1.75
						153			99	1.00	111		60.8	1.63
									78	1.50			49.1	1.59
									74	2.00			41.0	1.81
8.0	7.0	2.94	53.1	0.96	0.88	84	3.71	40.7	195	0.40	121	85.4	88.2	2.21
							3.28	45.6	108	0.80	121	90.2	68.9	1.57
							4.20	39.3	88	1.50	127	85.0	46.1	1.91
						120	4.58	39.5	201	0.20	117	86.6	97.2	2.07
						120	4.31	39.5	135	0.40	114	85.8	80.8	1.67
						120	3.25	42.7	88	0.80	106	85.8	59.4	1.48
						120	4.28	39.5	68	1.50	113	85.7	42.8	1.59
							4.28	42.4	211	0.40	132	90.8	95.1	2.22
							3.91		130	0.80	126	89.2	69.9	1.86
		1.47	54.5	0.68	0.37	84	2.89	43.0	116	0.40	79.8	57.9	58.4	1.99
							3.81		108	0.80	93.6	61.0	49.3	2.19
7.0	8.0			0.96	2.15		3.81	39.5	214	0.40	91.4	113.9	75.7	2.83
							3.46		175	0.80	86.1	112.3	61.0	2.87
													Avg.	2.02
													Std. Dev.	0.31 (15.3%)