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ALLOWABLE STRESS DESIGN OF CONCRETE MASONRY COLUMNS

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Structural   (2001)

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INTRODUCTION

Masonry elements typically support both axial and lateral loads. For structural elements that resist primarily lateral forces, axial load can increase the element's flexural resistance. In this case, axial load is often neglected as a conservative assumption which simplifies the analysis. However, for elements carrying significant axial loads, such as columns, the additional moment due to lateral loads or eccentric axial loads typically reduces the element's axial capacity. In this case, the design must consider the interaction between axial load and moment.

By definition, a column is an isolated vertical member whose horizontal dimension measured at right angles to its thickness does not exceed three times its thickness and whose height is greater than four times its thickness (ref. 1). Columns function primarily as compression members when supporting beams, girders, trusses or similar elements.

COLUMN REQUIREMENTS

Because a column failure has the potential to cause collapse of other structural members, a series of special requirements are imposed on columns in addition to the requirements for reinforced concrete masonry wall design.

Slenderness

The capacity of columns may be reduced due to either buckling or to additional bending moment caused by deflection (*P*-Δ effects). In *Building Code Requirements for Masonry Structures* (ref. 1, referred to hereafter as the *Code*), slenderness effects are included in the calculation of allowable compressive stress for reinforced masonry. For columns, the *Code* also limits the effective height to thickness ratio to 25, and requires a minimum nominal side dimension of 8 in. (203 mm).

The effective height of a column is typically taken as the clear height between supports. If the designer can demonstrate that there is reliable restraint against both translation and rotation at the supports, the effective height may be reduced in accordance with conventional design principles.

Eccentricity also affects the structural capacity of masonry columns. Eccentricity may be introduced by eccentric axial loads, lateral loads, or a column that is out of plumb. As a minimum, the *Code* requires that the design consider an eccentricity of 0.1 times each side dimension, with each axis considered independently. This minimum eccentricity is intended to account for construction tolerances. If the actual eccentricity exceeds this minimum, the actual eccentricity should be used in the design.

Reinforcement

The *Code* (ref. 1) requires a minimum amount of vertical column reinforcement as well as lateral ties to confine the
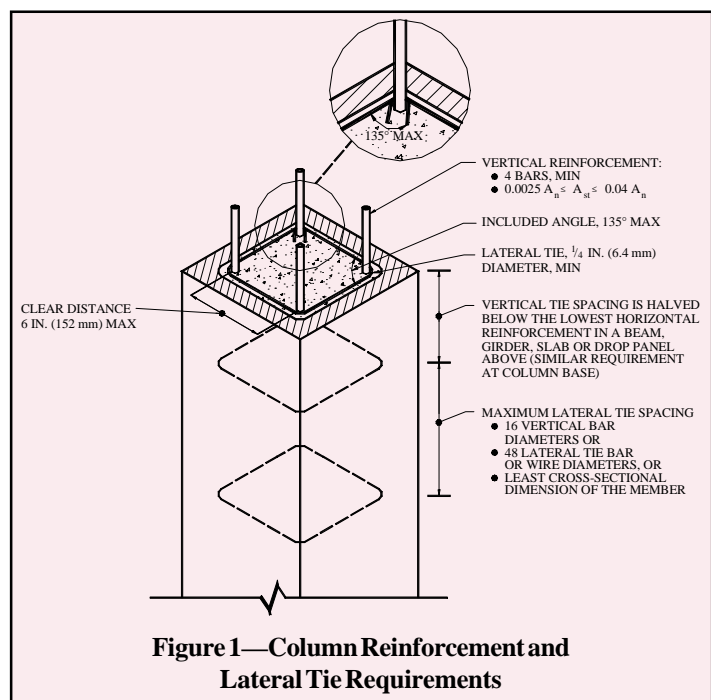

VERTICAL REINFORCEMENT:
- 4 BARS, MIN
- $0.0025 A_n \le A_{st} \le 0.04 A_n$

INCLUDED ANGLE, 135° MAX

LATERAL TIE, $\frac{1}{4}$ IN. (6.4 mm) DIAMETER, MIN

VERTICAL TIE SPACING IS HALVED BELOW THE LOWEST HORIZONTAL REINFORCEMENT IN A BEAM, GIRDER, SLAB OR DROP PANEL ABOVE (SIMILAR REQUIREMENT AT COLUMN BASE)

MAXIMUM LATERAL TIE SPACING
- 16 VERTICAL BAR DIAMETERS OR
- 48 LATERAL TIE BAR OR WIRE DIAMETERS, OR
- LEAST CROSS-SECTIONAL DIMENSION OF THE MEMBER

135° MAX

CLEAR DISTANCE 6 IN. (152 mm) MAX

Figure 1—Column Reinforcement and Lateral Tie Requirements

Column size, in. (mm)	No. 4 (M 13)	No.5 (M 16)	No. 6 (M 19)	No. 7 (M 22)	No. 8 (M 25)	No. 9 (M 29)	No. 10 (M 32)	No. 11 (M 36)
8 x 8 (203 x 203)	4 - 10	4	4	N/A	N/A	N/A	N/A	N/A
8 x 16 (203 x 406)	4 - 12	4 - 12	4 - 10	4 - 8	4 - 6	4	N/A	N/A
8 x 24 (203 x 610)	4 - 12	4 - 12	4 - 12	4 - 12	4 - 8	4 - 6	4	4
10 x 16 (254 x 406)	4 - 12	4 - 12	4 - 12	4 - 10	4 - 6	4 - 6	4	N/A
10 x 24 (254 x 610)	4 - 12	4 - 12	4 - 12	4 - 12	4 - 10	4 - 8	4 - 6	4
12 x 12 (305 x 305)	4 - 12	4 - 12	4 - 12	4 - 8	4 - 6	4	4	N/A
12 x 16 (305 x 406)	4 - 12	4 - 12	4 - 12	4 - 12	4 - 8	4 - 6	4	4
12 x 24 (305 x 610)	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 10	4 - 8	4 - 6
12 x 32 (305 x 813)	6 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 10	4 - 8
16 x 16 (406 x 406)	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 8	4 - 6	4 - 8
16 x 24 (406 x 610)	6 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 10	4 - 8
16 x 32 (406 x 813)	8 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12
24 x 24 (610 x 610)	8 - 12	6 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12
24 x 32 (610 x 813)	10 - 12	8 - 12	6 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12
24 x 40 (610 x 1016)	12	8 - 12	6 - 12	4 - 12	4 - 12	4 - 12	4 - 12	4 - 12

Table 1—Allowable Column Reinforcement

Number of reinforcing bars permitted, based on $0.0025A_n \leq A_{st} \leq 0.04A_n$, for bar sizes:

vertical steel. The basic requirements are illustrated in Figure 1. In addition, Table 1 lists allowable reinforcement for various column sizes, based on the *Code* required minimum and maximum vertical steel area. The requirement for at least four vertical bars allows the lateral ties to provide a confined core of masonry.

Lateral ties enclose and support the vertical reinforcement. The size and spacing requirements ensure the ties prevent buckling of reinforcement acting in compression as well as provide shear resistance to columns subjected to lateral loads. Vertical lateral tie spacing is halved above the top of the footing or slab in any story, as well as below the lowest horizontal reinforcement in a beam, girder, slab, or drop panel above. Where beams or brackets frame into a column from four directions, the lateral ties must be placed within 3 in. (76 mm) below the lowest reinforcement in the shallowest beam or bracket.

The *Code* allows lateral ties to be placed in either mortar or grout, although placement in grout more effectively prevents buckling and results in more ductile behavior. For this reason, the *Code* requires ties to be embedded in grout in Seismic Performance Categories D and E.

When more than four vertical bars are used, additional requirements apply. In this case, in addition to the requirement for corner bars to be laterally supported by the corner of a lateral tie, alternate bars must also be supported. In addition, bars not supported by a lateral tie corner must be spaced 6 in. (152 mm) or closer on each side along the lateral tie from the laterally supported bar. Where the longitudinal bars are placed in a circle, circular ties are permitted, provided they have a minimum lap length of 48 tie diameters.

Additional Requirements in Seismic Performance Categories (SPC) C, D and E

Columns in buildings that require higher levels of seismic protection are subject to additional design require-ments to help prevent structural failure during an earth-quake. To ensure proper anchorage between columns and horizontal structural elements, the *Code* requires connec-tors to transfer forces in SPC C, D and E. Where anchor bolts are used for this purpose, they must be enclosed by the vertical reinforcement and lateral ties. In addition, at least two No. 4 (M 13) lateral ties must be provided within the top 5 in. (127 mm) of the column.

Adequate lateral restraint is important for column rein-forcement subjected to seismic forces. For this reason, in SPC D and E 3/8 in. (9.5 mm) minimum diameter lateral ties are required to be embedded in grout and spaced vertically no more than 8 in. (203 mm) on center.

These requirements are illustrated in Figure 2.

DESIGN

Allowable stress design of concrete masonry columns must comply with Section 2.3 of the *Code*, which governs reinforced masonry design. Allowable forces and stresses are as follows:

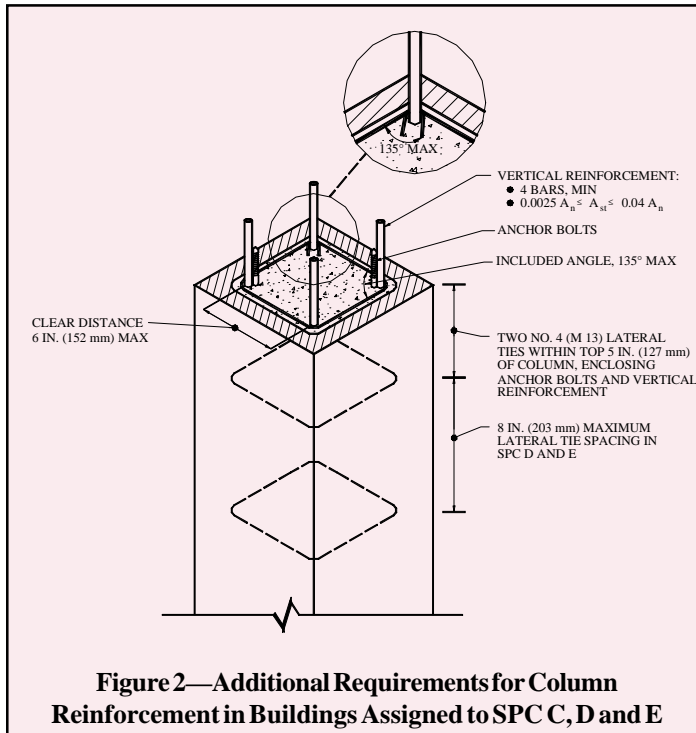$F_s = 24{,}000 \text{ psi (165.5 MPa) for Grade 60 steel}$

$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)[1 - (h/140r)^2], \text{ for } h/r \leq 99$

$\quad = (0.25f'_m A_n + 0.65A_{st}F_s)(70r/h)^2, \text{ for } h/r > 99$

$F_{b/a} = \tfrac{1}{3}f'_m$

The allowable compressive force, P_a, includes the contri-bution of vertical reinforcement in the term $0.65A_{st}F_s$. This assumes proper confinement of the vertical steel using lateral ties, as described above.

Masonry columns may be connected to horizontal elements of the structure and may rely on these connec-tions for lateral support. Forces at the connection may be transferred by masonry/mortar bond, mechanical anchor-age, friction, bearing, or a combination of these. The col-umns must be designed to resist all loads, moments and

Figure 2—Additional Requirements for Column Reinforcement in Buildings Assigned to SPC C, D and E

Figure labels:
- 135° MAX
- VERTICAL REINFORCEMENT:
  - 4 BARS, MIN
  - $0.0025 A_n \leq A_{st} \leq 0.04 A_n$
- ANCHOR BOLTS
- INCLUDED ANGLE, 135° MAX
- CLEAR DISTANCE 6 IN. (152 mm) MAX
- TWO NO. 4 (M 13) LATERAL TIES WITHIN TOP 5 IN. (127 mm) OF COLUMN, ENCLOSING ANCHOR BOLTS AND VERTICAL REINFORCEMENT
- 8 IN. (203 mm) MAXIMUM LATERAL TIE SPACING IN SPC D AND E

Table 2—Allowable Column Compressive Force for Concentrically Loaded Concrete Masonry Columns up to 20 ft (6.1 m) High[1]

Column size, in. (mm)	Allowable column compressive force, kip (kN)
8 x 8 (203 x 203)	18[2] (80)
8 x 16 (203 x 406)	37[2,3] (165)
8 x 24 (203 x 610)	56[2,4] (249)
10 x 16 (254 x 406)	46[5] (205)
10 x 24 (254 x 610)	71[6] (316)
12 x 12 (305 x 305)	42 (186)
12 x 16 (305 x 406)	56 (249)
12 x 24 (305 x 610)	85 (378)
12 x 32 (305 x 813)	114 (507)
16 x 16 (406 x 406)	76 (338)
16 x 24 (406 x 610)	115 (511)
16 x 32 (406 x 813)	154 (685)
24 x 24 (610 x 610)	174 (773)
24 x 32 (610 x 813)	233 (1030)
24 x 40 (610 x 1016)	292 (1300)

Notes:

[1] The table assumes the element is in pure compression, i.e., that the axial load falls within the center one-third of the section, under a minimum design eccentricity of $0.1t$ for each axis as required by the *Code*. The designer must ensure the section is in compression prior to using the table. $f'_m = 1500$ psi (10.3 MPa). $F_s = 24,000$ psi (165 MPa) (Grade 60 steel). One kip = 1,000 lb (4.4 kN).

SLENDERNESS LIMITATIONS:

[2] The maximum allowable height for 8 in. columns is 15.9 ft (4.8 m).

	Height ft (m)	Number of bars	Bar size	Maximum load kips (kN)
[3]	15.1-15.9 (4.6-4.8)	4	No. 4 (M13)	34 (151)
[4]	14.0-15.9 (4.3-4.8)	4	No. 4 (M13)	48 (213)
	15.1-15.9 (4.6-4.8)	6	"	52 (231)
	15.3-15.9 (4.6-4.8)	4	No. 5 (M16)	53 (236)
[5]	18.6-20 (5.6-6.1)	4	No. 4 (M13)	42 (186)
[6]	16.9-18 (5.1-5.5)	4	No. 4 (M13)	67 (298)
	18.0-20 (5.5-6.1)	4	"	60 (266)
	18.2-20 (5.5-6.1)	6	"	64 (284)
	19.3-20 (5.9-6.1)	8	"	68 (302)
	18.3-20 (5.6-6.1)	4	No. 5 (M16)	64 (284)
	19.7-20 (6.0-6.1)	6	"	70 (311)
	19.7-20 (6.0-6.1)	4	No. 6 (M19)	69 (307)

shears applied at intersections with horizontal members, using a force of at least 1,000 lb (4.4 kN).

The design approach depends on the magnitude of the axial load relative to the bending moment. The section will either be in pure compression, with the allowable axial load governed by P_a; be subject to combined axial load and flexure with the allowable moment and allowable axial force governed by the allowable flexural compressive stress in masonry, F_b; or be subject to combined axial load and flexure, but governed by the allowable tensile stress in the reinforcement, F_s.

Section in Compression

An eccentricity located within the kern (center one-third) of the column places the entire section in compression. In this case, capacity is determined by the equations for P_a listed above, and Table 2 can be used for design for columns up to 20 ft (6.1 m) high. The table assumes the element is in pure compression under a minimum design eccentricity of $0.1t$ for each axis, as required by the *Code*. The designer is responsible for confirming this.

The values in Table 2 are independent of vertical steel area because in all cases except those noted in the table footnotes, the allowable compressive stress in masonry governs the column design.

Design Example—Compression Only

Design a 20-ft (6.1 m) high column to carry a concentric axial force of 45,000 lb (200 kN), based on $f'_m = 1,500$ psi (10 MPa) and Grade 60 steel.

First, check the minimum eccentricity:

$0.1t = 0.1(8 \text{ in.}) = 0.8 \text{ in.} (20 \text{ mm})$

At a minimum (for an 8 x 8 in. (203 x 203 mm) column), the kern is bounded by $t/6 = 8$ in./6 = 1.3 in. (33 mm).

Because the design eccentricity falls within the kern, pure compression results and Table 2 can be used.

From Table 2, an 8 x 24 in. (203 x 610 mm) column has adequate capacity but is limited to 15.9 ft. Slenderness effects of a 10 x 16 in. column (254 x 406 mm) with four No. 4 (M 13) reduce the capacity to 42 kips (186 kN) - not adequate. With four No. 5 (M 16), it can support 46 kips (205 kN) > 45 kips (200 kN). Checking Table 1, four No. 5 (M 16) bars will meet the reinforcing area requirements. Use four No. 5 (M 16).

Combined Axial Compression and Flexure

For larger eccentricities, the section is subjected to flexure, resulting in both net compression and tension. Therefore, the interaction of the vertical load and the bending moment must be accounted for, typically using interaction diagrams or iterative computer solutions. Further description of the design methodology, as well as interaction diagrams for columns are contained the *Masonry Designer's Guide* (ref. 2).

NOTATIONS:

A_n = net cross-sectional area of masonry, in.^{2} (mm^2)

A_{st} = total area of laterally tied longitudinal reinforcing steel in a reinforced masonry column, in.^{2} (mm^2)

e = eccentricity of axial load, in. (mm)

F_b = allowable compressive stress due to flexure only, psi (MPa)

$F_{b/a}$ = allowable compressive stress in masonry due to combined flexure and axial load

F_s = allowable tensile stress in reinforcement, psi (MPa)

P = compressive force due to axial load, lb (N)

P_a = allowable compressive force in reinforced masonry due to axial load, lb (N)

r = radius of gyration, in. (mm)

t = thickness of section, in. (mm)

D = deflection

REFERENCES

1. *Building Code Requirements for Masonry Structures*, ACI 530-99/ASCE 5-99/TMS 402-99. Reported by the Masonry Standards Joint Committee, 1999.
2. *Masonry Designer's Guide*. Matthys, John H., ed. The Masonry Society and the American Concrete Institute, 1999.