Architecture 504
Masonry Structures

Columns and Pilasters

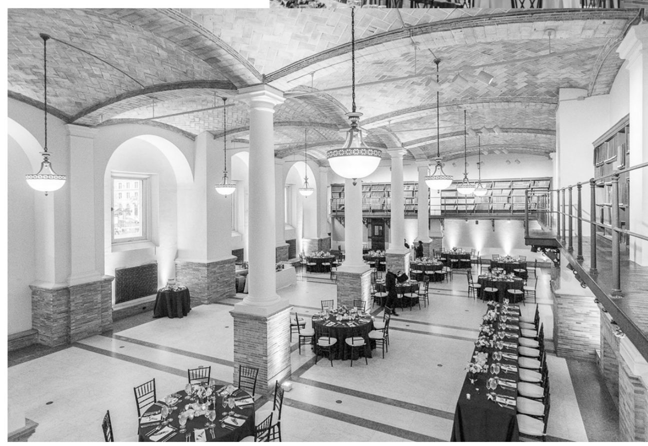
- Concentric axial
- Interaction
- Bearing walls

Guastavino Vaulting

Boston Public Library

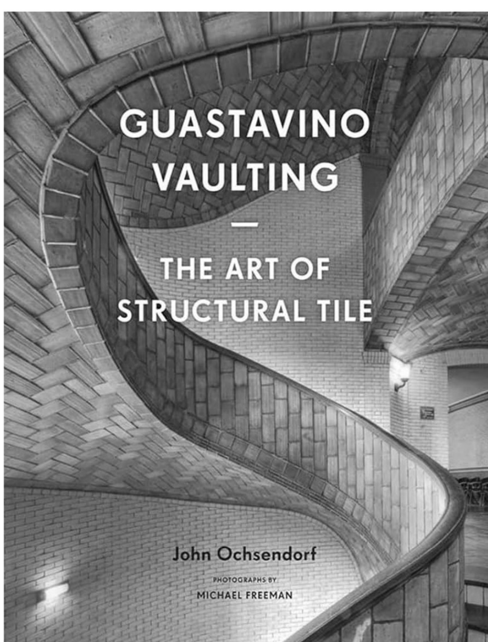
Guastavino Vaulting

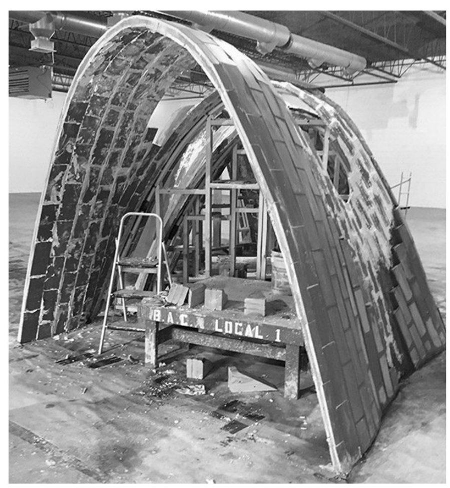
Boston Public Library

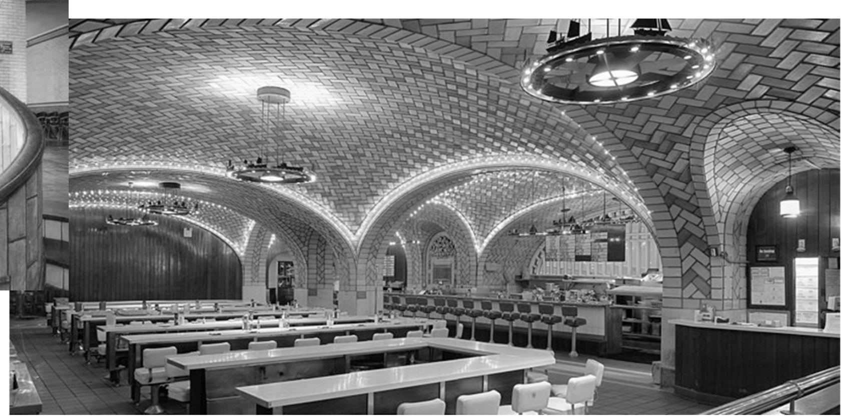


Guastavino Vaulting

John Ochsendorf



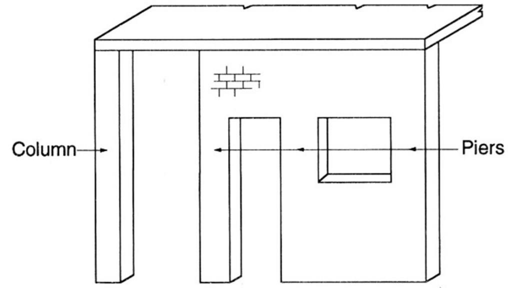




Types of Vertical Supports

Columns
- width/thickness <= 3
- separate member

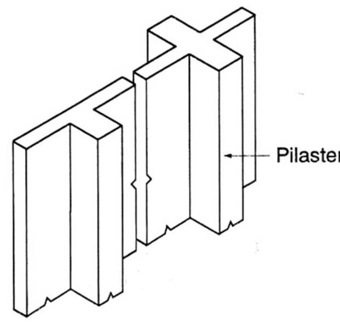
Piers
- like a column but within wall



(a) Piers and Column

Pilasters
- integral with wall
- resists out of plane bending
- either or both sides project from wall
- stiffer than wall so carry more moment
- expansion joints prevent cracking

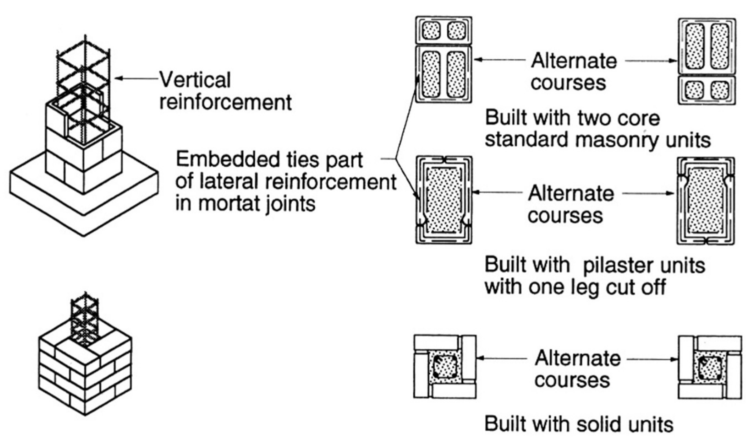
Buttresses
- tapered top to bottom
- enlarged at base to resist overtuning
- if thrust line is within kern, all compression



(b) Pilasters as Parts of a Wall

Columns

carry vertical load but also moment
- eccentric loading
- can be unreinforced (short)
- reinforcement required by TMS 402
- ρ min = 0.0025
- ρ max = 0.04
- ρ = As/bd
- ties – TMS 402 5.3.1.4.
- better in contact w/ bars
- running bond



(a) Columns

Concentric Axial Compression

A_{st} = area of **laterally tied steel**

TMS 402 CODE

5.3.1.4 *Lateral ties* — Lateral ties shall conform to the following:

(a) Vertical reinforcement shall be enclosed by lateral ties at least 1/4 in. (6.4 mm) in diameter.

(b) Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 lateral tie bar or wire diameters, or least cross-sectional dimension of the member.

(c) Lateral ties shall be arranged so that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a lateral tie with an included angle of not more than 135 degrees. No bar shall be farther than 6 in. (152 mm) clear on each side along the lateral tie from such a laterally supported bar. Lateral ties shall be placed in grout. Where longitudinal bars are located around the perimeter of a circle, a complete circular lateral tie is permitted. Lap length for circular ties shall be 48 tie diameters.

(d) Lateral ties shall be located vertically not more than one-half lateral tie spacing above the top of footing or slab in any story, and shall be spaced not more than one-half a lateral tie spacing below the lowest horizontal reinforcement in beam, girder, slab, or drop panel above.

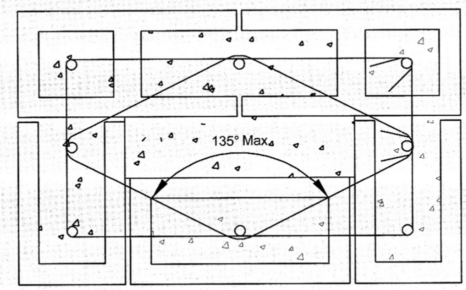


Figure CC-5.3-2 — Example of a lateral tie included angle for a CMU column

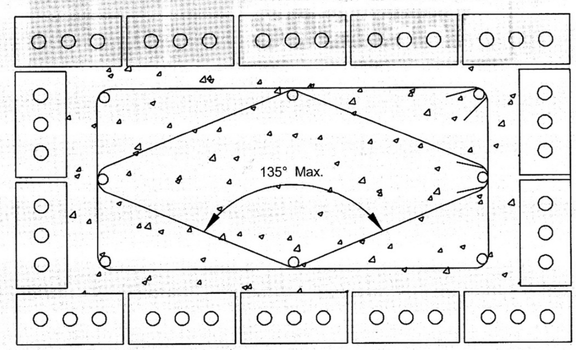
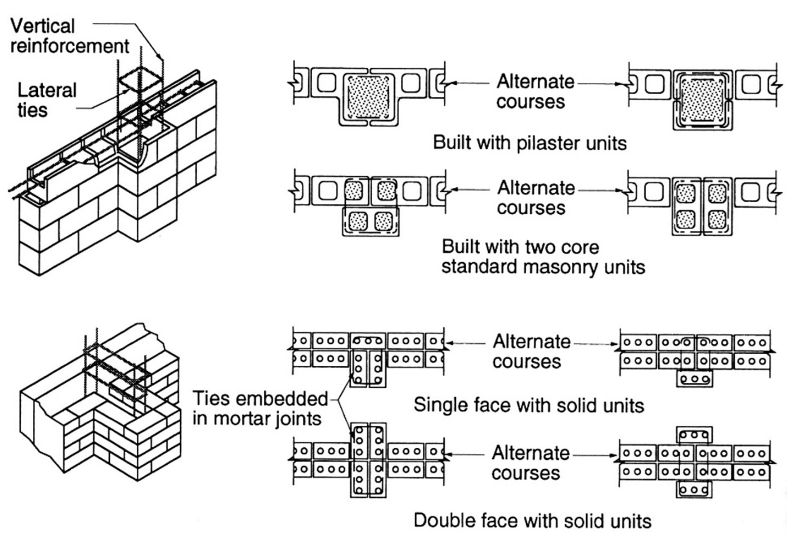


Figure CC-5.3-3 — Example of a lateral tie included angle for a clay masonry column

Pilasters

carry vertical load but also moment

- integral with wall
- resists out of plane bending
- either or both sides project from wall
- stiffer than wall so carry more moment
- expansion joints prevent cracking
- running bond w/ wall
- anchored to wall at 48 in. o.c. max.

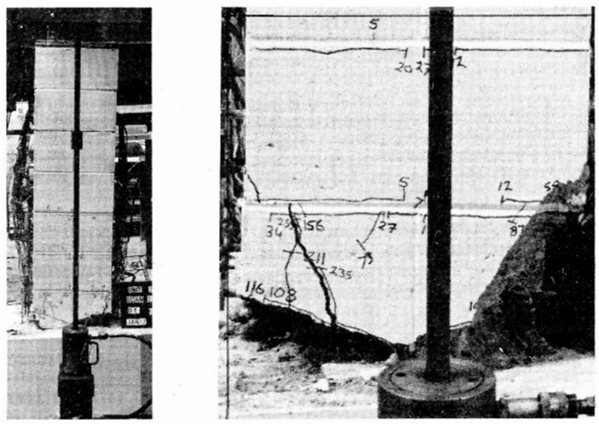


(b) Pilasters

Column Behavior

modes of failure:

- vertical splitting of masonry shell and grout core – typical for unreinforced columns
- simultaneous splitting (as above) and buckling of vertical reinforcement between ties
- like above + failure of ties (at hooks) with reinforcement buckling over two or more courses



a) Test under cyclic lateral load

b) Full capacity retained at large lateral displacement

Effect of Slenderness

- effects buckling
- more susceptible to additional moment: P-Δ effect
- TMS 402 uses the ψ factor

9.3.5.4.3 The strength level moment, M_u, shall be determined either by a second-order analysis, or by a first-order analysis and Equations 9-27 through 9-29.

$$M_u = \psi M_{u,0} \qquad \text{(Equation 9-27)}$$

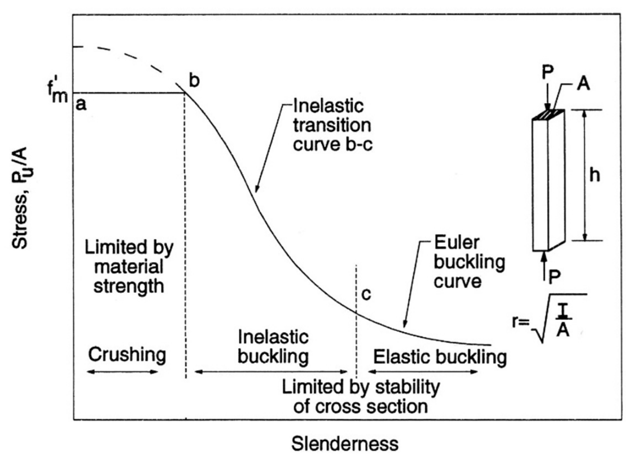
Where $M_{u,0}$ is the strength level moment from first-order analysis.

$$\psi = \frac{1}{1-\frac{P_u}{P_e}} \qquad \text{(Equation 9-28)}$$

Where:

$$P_e = \frac{\pi^2 E_m I_{eff}}{h^2} \qquad \text{(Equation 9-29)}$$

For $M_u<M_{cr}$, I_{eff} shall be taken as $0.75I_n$. For $M_u \geq M_{cr}$, I_{eff} shall be taken as I_{cr}. P_u/P_e cannot exceed 1.0.

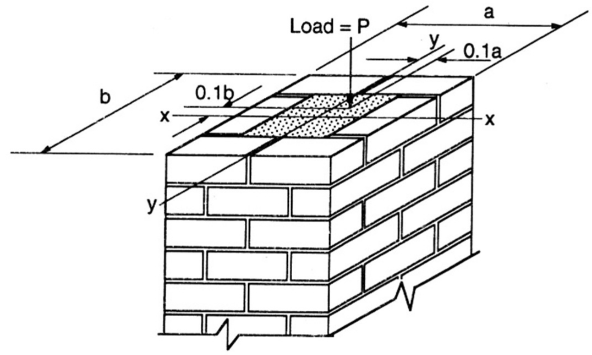


TMS 402 CODE	COMMENTARY
4.3.3 *Radius of gyration* Radius of gyration shall be calculated using the average net cross-sectional area of the member considered. $r = \sqrt{I/A}$	**4.3.3** *Radius of gyration* The radius of gyration is the square root of the ratio of bending moment of inertia to cross-sectional area. Because stiffness is based on the average net cross-sectional area of the member considered, this same area should be used in the calculation of radius of gyration.

Design Considerations

- minimum eccentricity: 0.1 x side dimension of column
- minimum side distance = 8"

- effective height – based on boundary conditions top and bottom
- when in doubt use clear height between floors



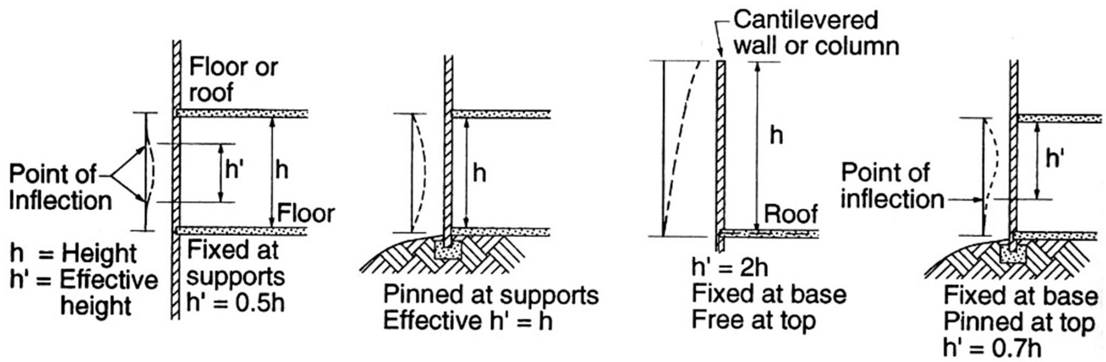
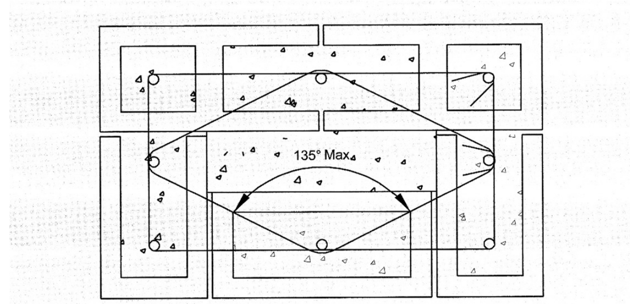


Figure 9.6 Effective height of masonry columns (from Ref. 9.1).

Design Considerations

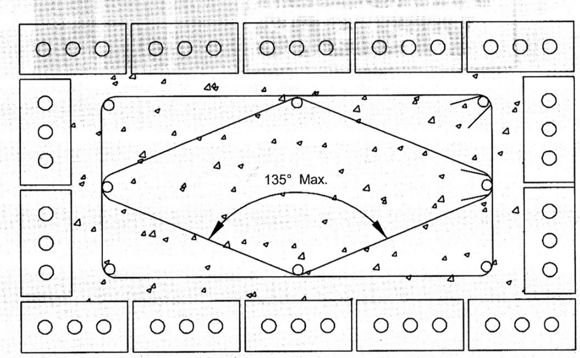
reinforcement

- reinforcement required by TMS 402
- min. of 4 bars
- ρ min = 0.0025
- ρ max = 0.04
- ρ = As/bd
- ties – TMS 402 5.3.1.4.
- better in contact w/ bars



Clear space between bars greater than 6 in.

Figure CC-5.3-2 — Example of a lateral tie included angle for a CMU column



Clear space between bars greater than 6 in.

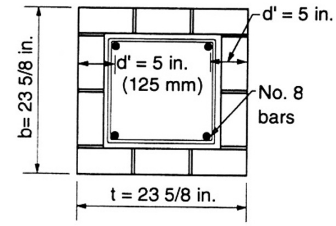
Figure CC-5.3-3 — Example of a lateral tie included angle for a clay masonry column

Design for Axial Load and Bending

methods based on degree of eccentricity

if load is within the kern (middle 1/3) then section is in compression. This eccentricity = e_k

e_k = Section modulus / Area

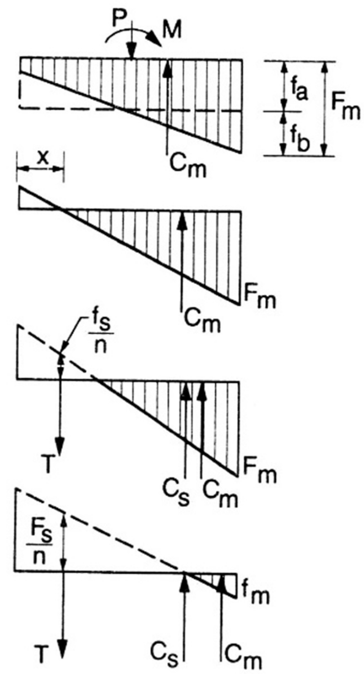


Case 1:
compression controls
$e < e_k$ full section in compression

Case 2: Category I
compression controls
$e > e_k$ initial cracking – steel in compression

Case 2: Category II
compression controls
$e > e_k$ more cracking – steel in tension

Case 2: Category III
tension controls
$e > e_k$ more cracking – steel in tension



Combined Bending and Axial Load flowchart

zero when A_s at center

Calculate

$$a = d - \sqrt{d^2 - \frac{2[P_u(d - t_{sp}/2) + M_u]}{\phi(0.8 f'_m b)}} \qquad c = \frac{a}{0.8}$$

Is $c \geq c_{bal}$?

For CMU, Grade 60 steel $c_{bal} = \frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_y} d$

$c_{bal} = 0.547d$

YES

$$A_{s,reqd} = \frac{0.8 f'_m ba - P_u/\phi}{\varepsilon_{mu} E_s \left(\frac{d-c}{c}\right)}$$

Compression controls

NO

$$A_{s,reqd} = \frac{0.8 f'_m ba - P_u/\phi}{f_y}$$

Tension controls

Arches

TEK 14-14

Minor Arch
- 6 ft span limit
- rise to span ratio less than 0.15 about 11" height for 6' span
- can carry 1500 lbs / ft. of span

Major Arch
- deeper
- longer span

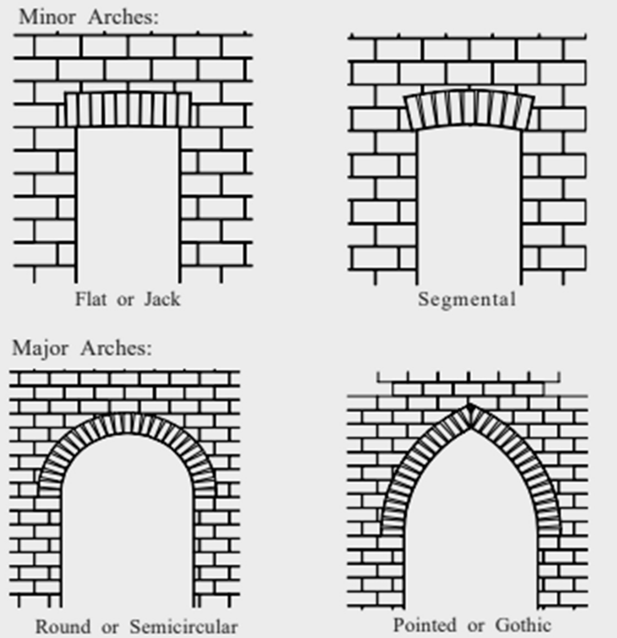


Figure 1—Masonry Arch Forms