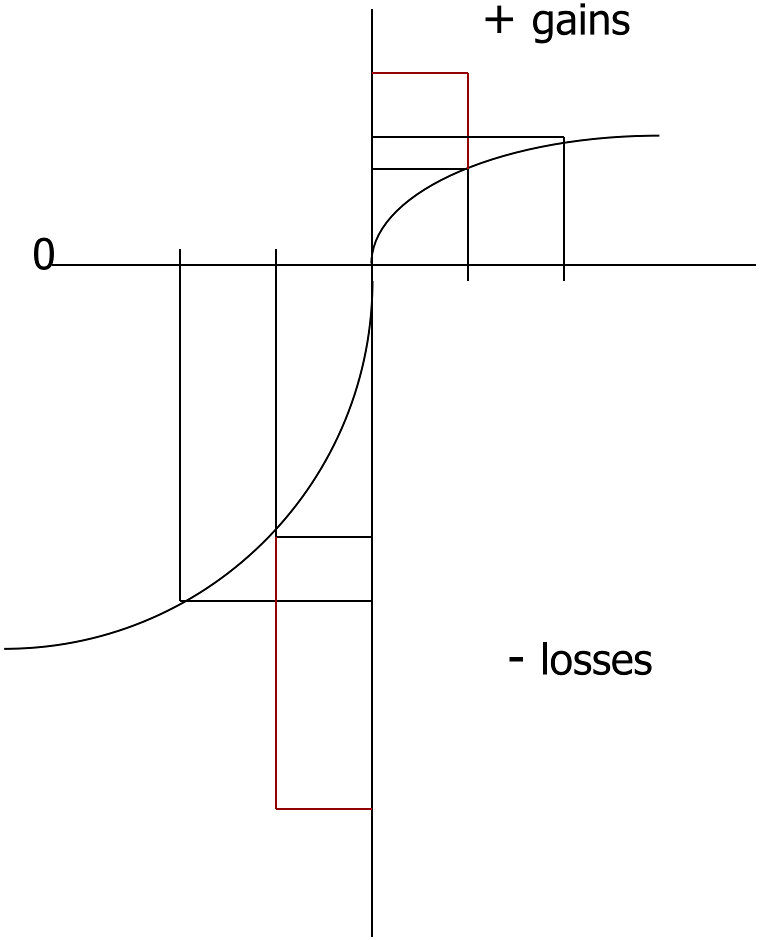
Expected Utility Theory Vs Prospect Theory



Instabilities around 0

Gains and losses are not symmetrical

"Losses loom larger than gains"

Estimates people tend to dislike losses about twice as much as they like equivalent gains

"Aggregate Losses; Segregate Gains"

Contents:

1. The Theory [ASD and SD]

2. The Code [2012 IBC, ASCE 7 –10 and TMS 402-11]

3. The Examples

The Problem – Seismic Design of Walls

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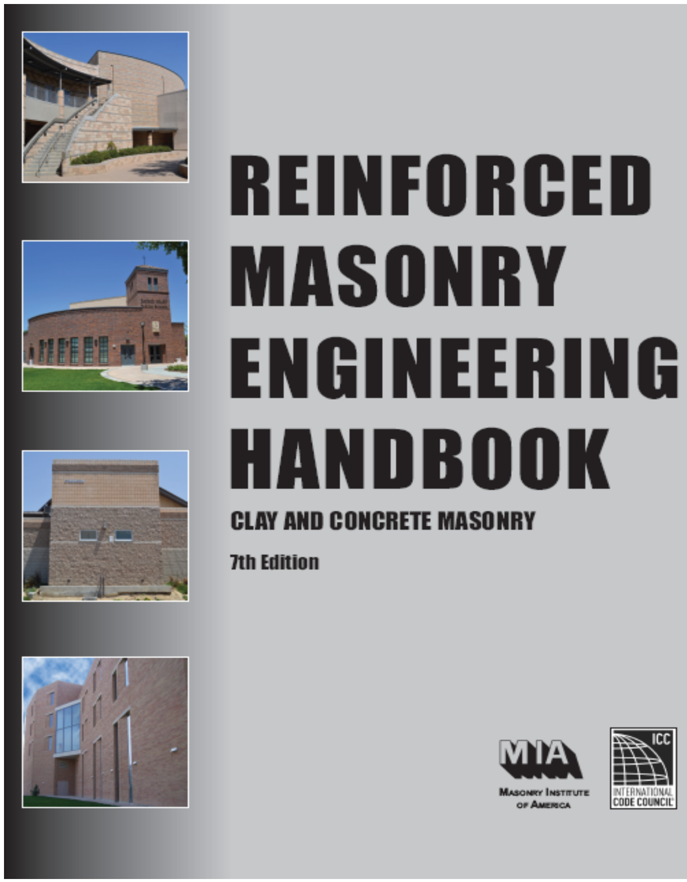
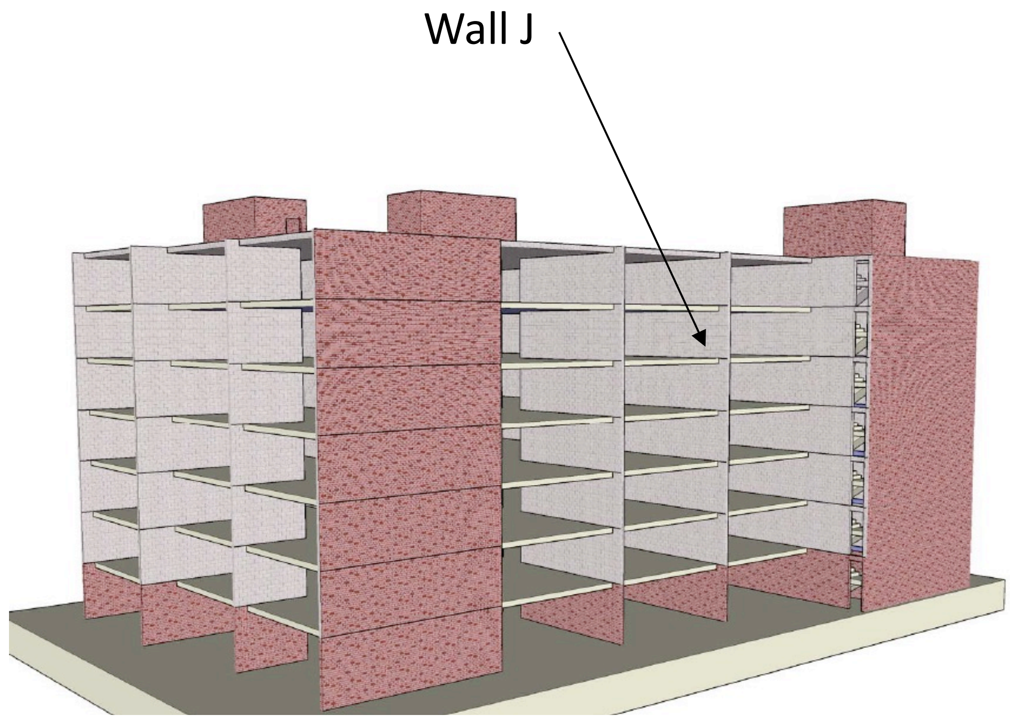
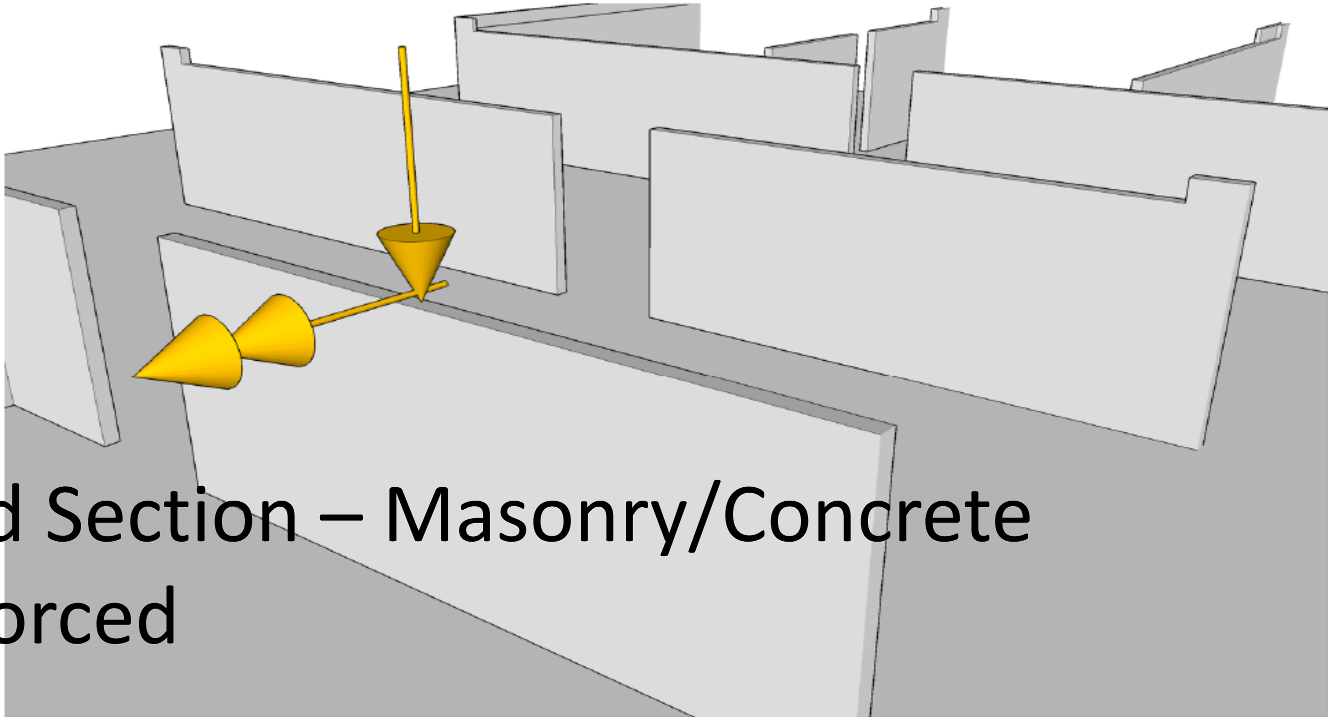
The Problem – Seismic Design of Walls





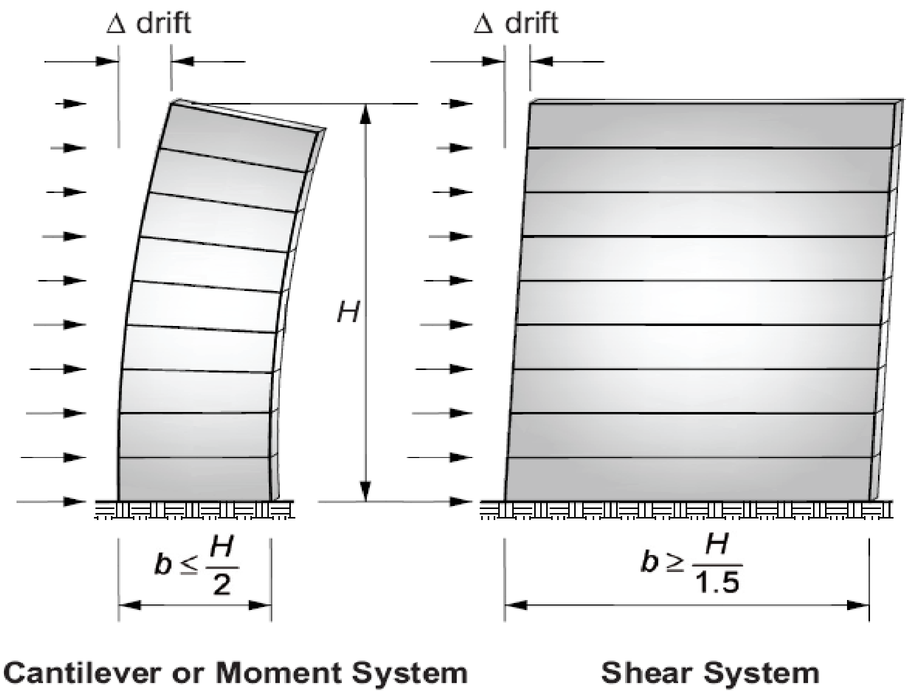
The Problem – Seismic Design of Walls

Bending + Compression



Cracked Section – Masonry/Concrete reinforced

The Problem – Seismic Design of Walls



Story	Seismic Moment - K-ft	Dead Load	F.S. [PL/2]/M
7	136	55.2	5.23
6	558	136.3	3.16
5	1223	217.4	2.30
4	2065	298.4	1.87
3	3069	379.5	1.60
2	4263	460.6	1.40
1	5897	543.7	1.19

Add axial load at L [trim steel].

75k at L results in F.S. of 1.5

Requires 1.2 in^2 of reinforcement.

The Equations:

The Assumptions

The Variables and Solution for the Unknowns

The Limits

The Equations -Assumptions:

Plane Sections Remain Plane [Special Case of an Isotropic Material]

Strains are Compatible

Stress and Strain are Related

The Equations -Assumptions:

Hooks Law

$$\begin{bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ \varepsilon_{yz} \\ \varepsilon_{zx} \\ \varepsilon_{xy} \end{bmatrix} = \begin{bmatrix} S_{11} & S_{12} & S_{13} & S_{14} & S_{15} & S_{16} \\ S_{21} & S_{22} & S_{23} & S_{24} & S_{25} & S_{26} \\ S_{31} & S_{32} & S_{33} & S_{34} & S_{35} & S_{36} \\ S_{41} & S_{42} & S_{43} & S_{44} & S_{45} & S_{46} \\ S_{51} & S_{52} & S_{53} & S_{54} & S_{55} & S_{56} \\ S_{61} & S_{62} & S_{63} & S_{64} & S_{65} & S_{66} \end{bmatrix} \begin{bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{zz} \\ \sigma_{yz} \\ \sigma_{zx} \\ \sigma_{xy} \end{bmatrix},$$

The Equations -Assumptions:

Hooks Law –Isotropic Material

$$\begin{bmatrix}\varepsilon_{xx}\\ \varepsilon_{yy}\\ \varepsilon_{zz}\\ \varepsilon_{yz}\\ \varepsilon_{zx}\\ \varepsilon_{xy}\end{bmatrix} = \frac{1}{E}\begin{bmatrix}1 & -\nu & -\nu & 0 & 0 & 0\\ -\nu & 1 & -\nu & 0 & 0 & 0\\ -\nu & -\nu & 1 & 0 & 0 & 0\\ 0 & 0 & 0 & 1+\nu & 0 & 0\\ 0 & 0 & 0 & 0 & 1+\nu & 0\\ 0 & 0 & 0 & 0 & 0 & 1+\nu\end{bmatrix}\begin{bmatrix}\sigma_{xx}\\ \sigma_{yy}\\ \sigma_{zz}\\ \sigma_{yz}\\ \sigma_{zx}\\ \sigma_{xy}\end{bmatrix}$$

The Equations -Assumptions:

Plane Sections Remain Plane [Special Case of an Isotropic Material]

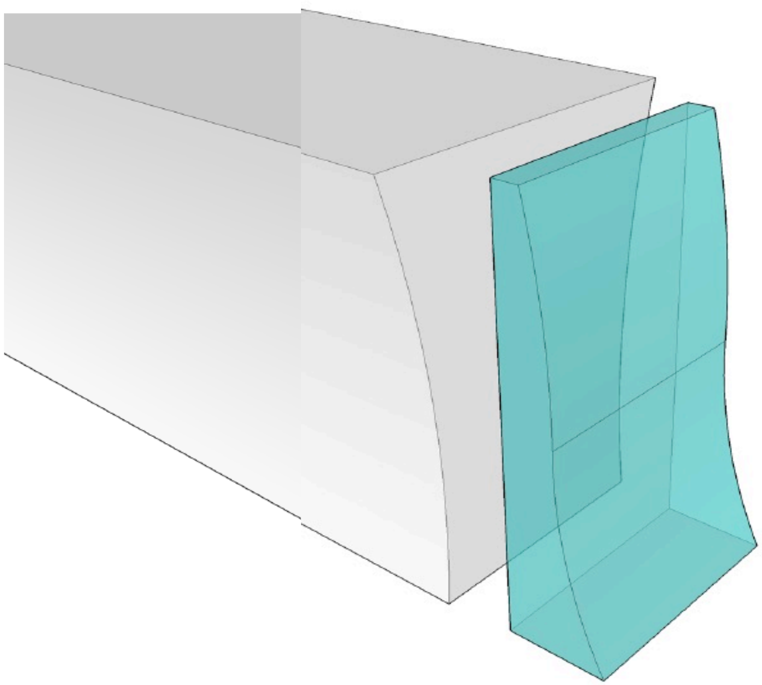
Hooks Law –Plane Sections Remain Plane

$$\begin{bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ \varepsilon_{yz} \\ \varepsilon_{zx} \\ \varepsilon_{xy} \end{bmatrix} = \frac{1}{E} \begin{bmatrix} 1 & \text{[illegible]} & \text{[illegible]} & 0 & 0 & 0 \\ \text{[illegible]} & \text{[illegible]} & \text{[illegible]} & 0 & 0 & 0 \\ \text{[illegible]} & \text{[illegible]} & \text{[illegible]} & 0 & 0 & 0 \\ \text{[illegible]} & \text{[illegible]} & \text{[illegible]} & \text{[illegible]} & \text{[illegible]} & \text{[illegible]} \\ 0 & 0 & 0 & \text{[illegible]} & \text{[illegible]} & \text{[illegible]} \\ 0 & 0 & 0 & \text{[illegible]} & \text{[illegible]} & \text{[illegible]} \end{bmatrix} \begin{bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{zz} \\ \sigma_{yz} \\ \sigma_{zx} \\ \sigma_{xy} \end{bmatrix}$$

$$\in_{xx} = \frac{1}{E} \sigma_{xx}$$

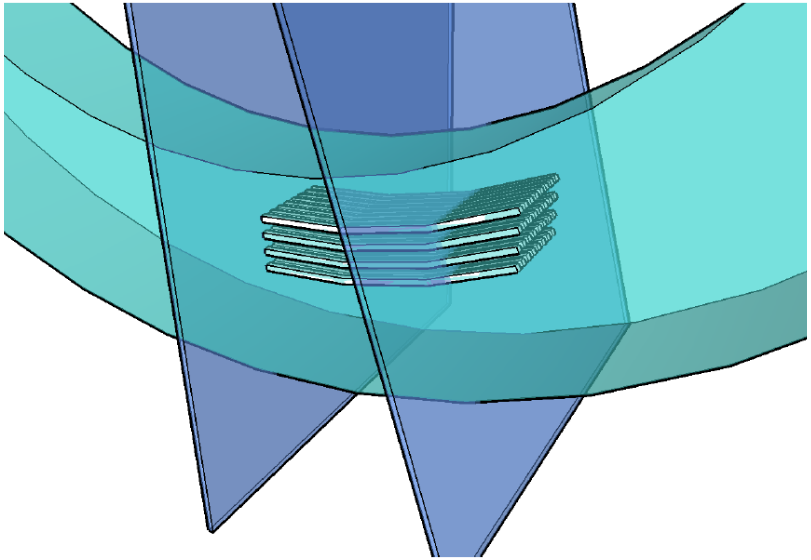
The Equations -Assumptions:

Hooks Law –Plane Sections do not Remain Plane



2.1 The Equations -Assumptions:

Hooks Law –Plane Sections Remain Plane



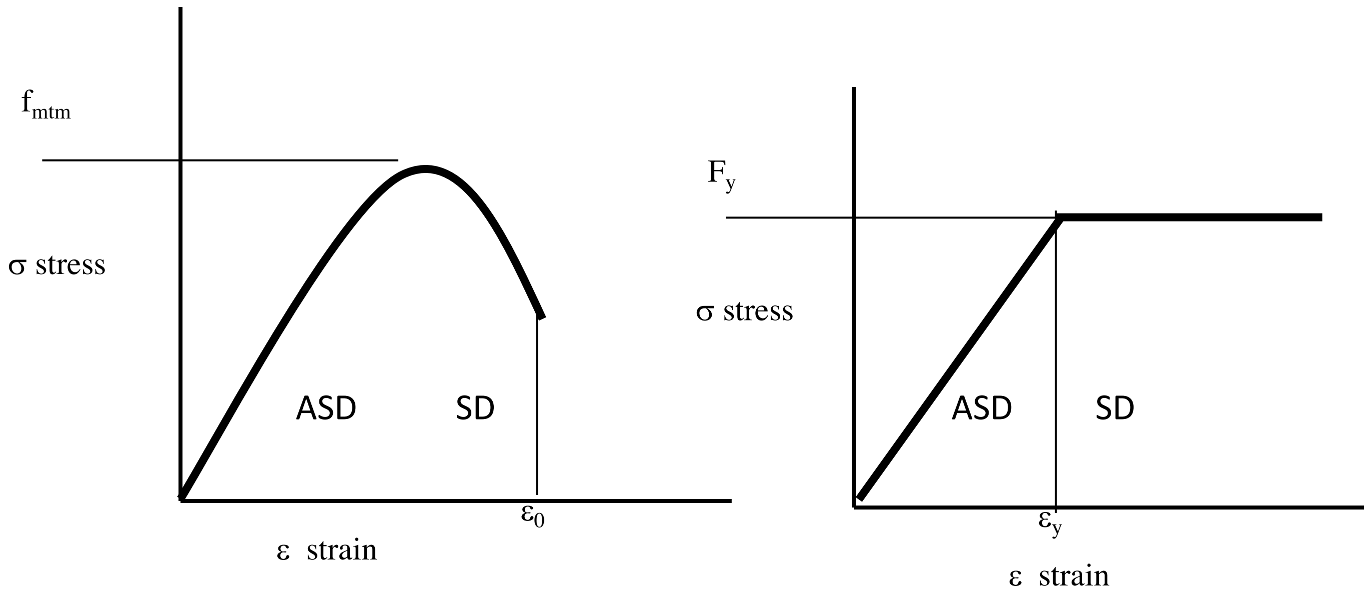
The Equations -Assumptions:

Strains are Compatible

The strain in the masonry/concrete equals the strain in the reinforcement.

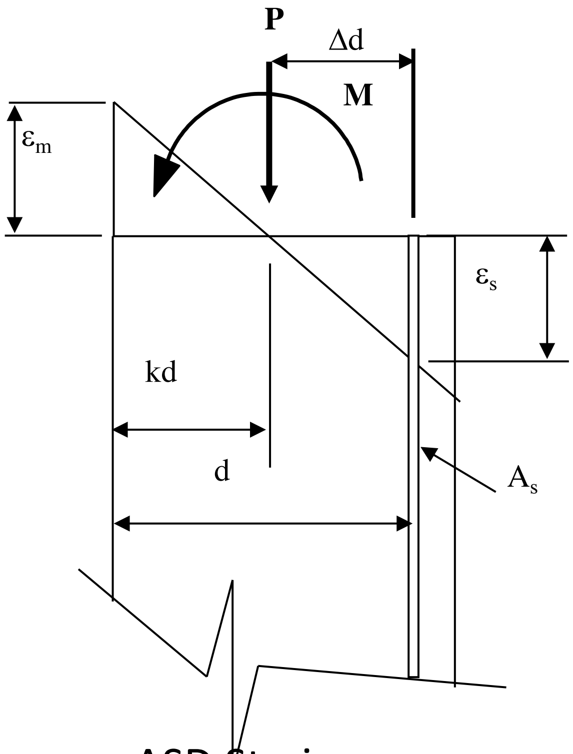
The Equations -Assumptions:

Stress and Strain are Related

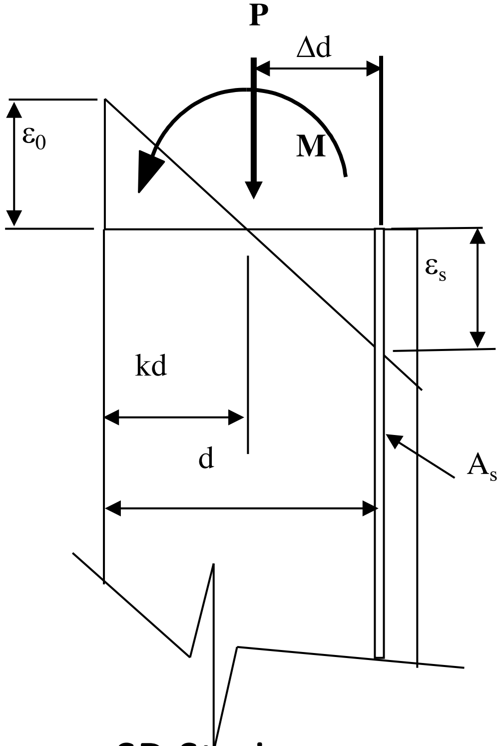


The Equations -Assumptions:

Stress and Strain are Related



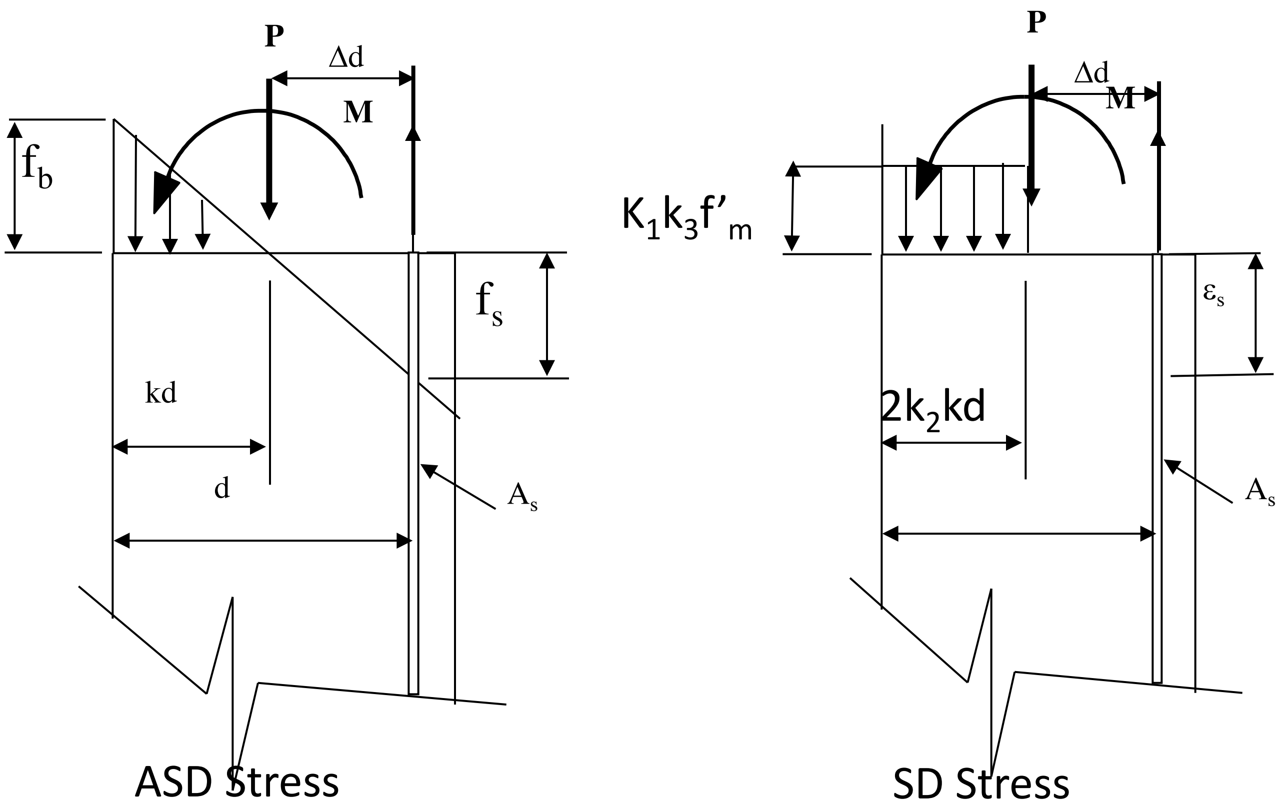
ASD Strain



SD Strain

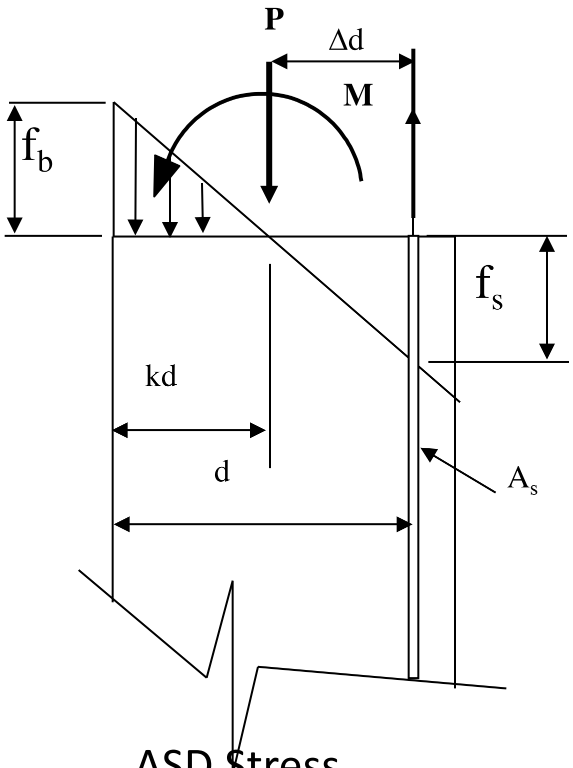
The Equations -Assumptions:

Stress and Strain are Related

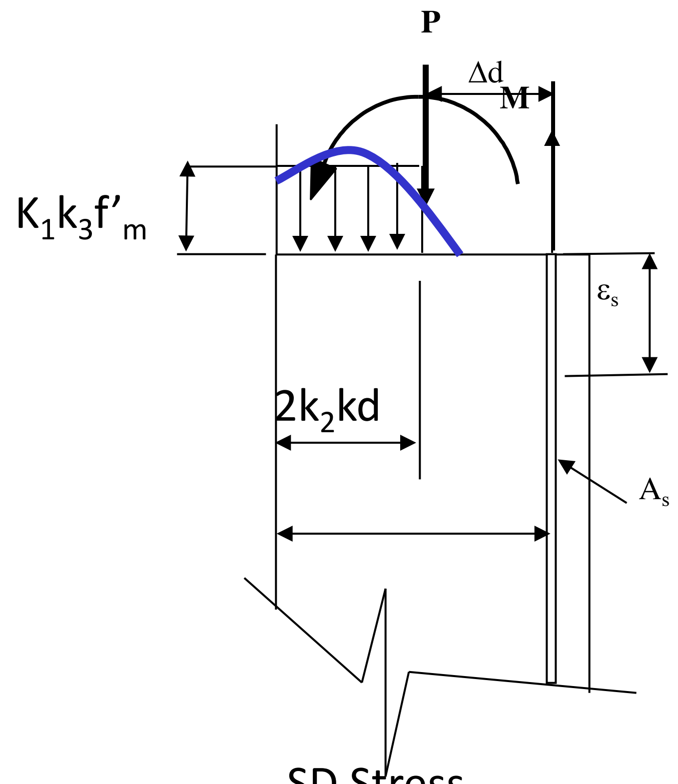


The Equations -Assumptions:

Stress and Strain are Related



ASD Stress



SD Stress

The Variables and Solution for the Unknowns

Knowns:

Guess and Check: A_s,L,b,d

Loads: M,P and V

Unknowns:

Stresses: k, f_b, f_s or k, ϵ_m, ϵ_s

The Variables and Solution for the Unknowns

Equations:

Plane Sections Remain Plane

$\Sigma F = 0$, Internal to external

$\Sigma M = 0$, Internal to external

The Variables and Solution for the Unknowns

Equations:

Plane Sections Remain Plane

$$\frac{\varepsilon_m}{\varepsilon_s} = \frac{k}{1-k}$$

The Variables and Solution for the Unknowns

Equations:

Limits: The Steel Strain is 0

k = 1.0

$$\frac{M}{Pd} = \left(\frac{2}{3} - \Delta\right)$$

k = 1.0

$$\frac{M}{Pd} = \left[1 - k_2 - \Delta\right]$$

The Variables and Solution for the Unknowns

Equations:

Limits: Stress in the steel – Special Case for SD

$$F_s < f_y$$

$$k = \frac{\varepsilon_0}{\left(\varepsilon_0 + \frac{F_y}{E_s}\right)}$$

$$\frac{M}{Pd} = \frac{k_1 k_3 2k_2 kbdf_m^{'}\left(1 - \frac{2k_2 k}{2}\right)}{\left(k_1 k_3 2k_2 kbdf_m^{'} - A_s F_y\right)} - \Delta$$

The Variables and Solution for the Unknowns

Equations:

Limits: Wall is not Cracked

k = L/d

$$\frac{M}{Pd} = \left(1 - \frac{L}{3d} - \Delta\right)$$

k = L/d

$$\frac{M}{Pd} = \left(1 - \frac{k_2 L}{d} - \Delta\right)$$

The Variables and Solution for the Unknowns

Equations:

F = 0, Internal to external

$$E_m \varepsilon_m \frac{bkd}{2} - E_s A_s \varepsilon_s = P$$

$$k_1 k_3 f(\varepsilon_m) 2 k_2 kdb = f(\varepsilon_s) A_s + P$$

The Variables and Solution for the Unknowns

Equations:

$\Sigma F = 0$, Internal to external

$$\varepsilon_{mo} = \frac{P}{E_m bd}$$

$$n = \frac{E_s}{E_m} \qquad \rho = \frac{A_s}{bd}$$

$$k^2 + \left(2n\rho + 2\frac{\varepsilon_{mo}}{\varepsilon_s}\right)k - \left(2n\rho + 2\frac{\varepsilon_{mo}}{\varepsilon_s}\right) = 0$$

Add a limit: Steel yielding

$$\varepsilon_s E_s = F_y$$

$$k = \frac{\left(A_s F_y + P\right)}{2k_2 k_1 k_3 bdf_m^{'}}$$

The Variables and Solution for the Unknowns

Equations:

ΣM = 0, Internal to external

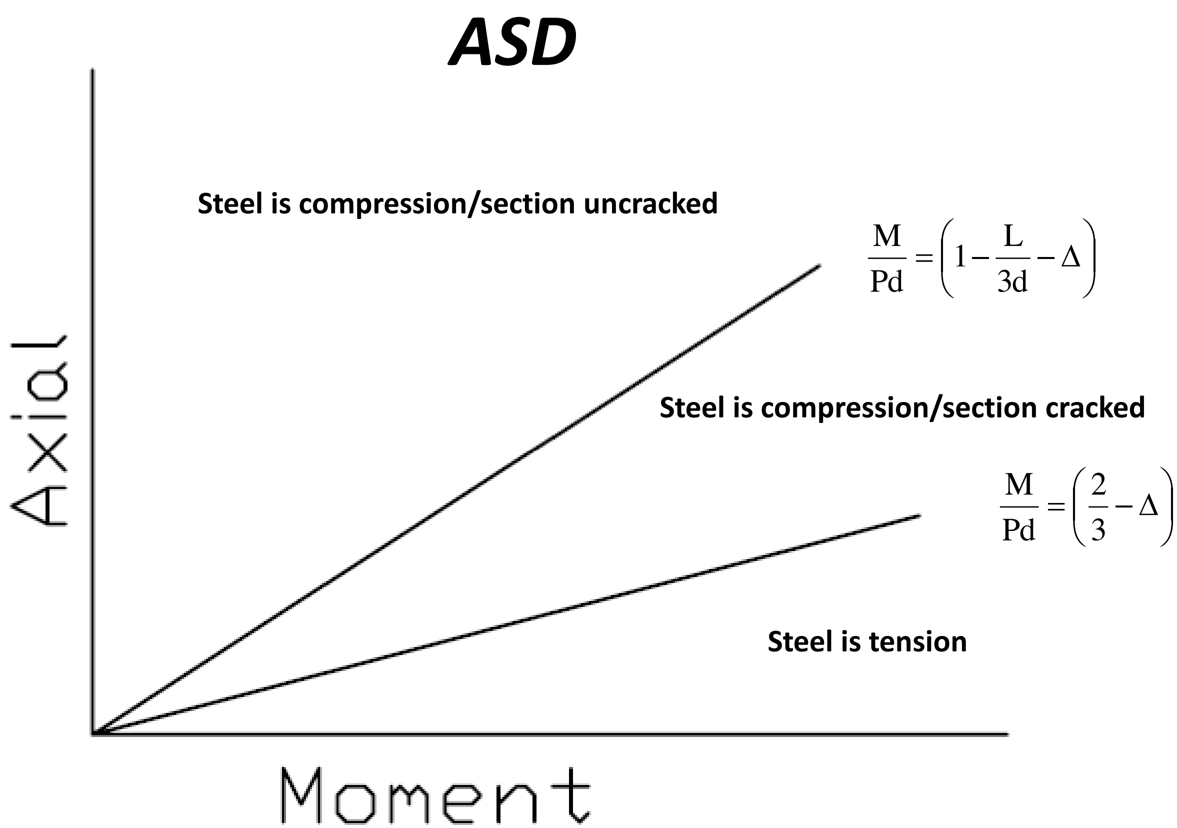
Add a limit: Steel yielding

$$E_s \varepsilon_s A_s d\left(1-\frac{k}{3}\right)=M-P\left(d-\frac{kd}{3}-\Delta d\right)$$

$$\frac{\varepsilon_{m0}}{\varepsilon_s}=\frac{n\rho\left(1-\frac{k}{3}\right)}{\left(\frac{M}{Pd}-\left(1-\frac{k}{3}-\Delta\right)\right)}$$

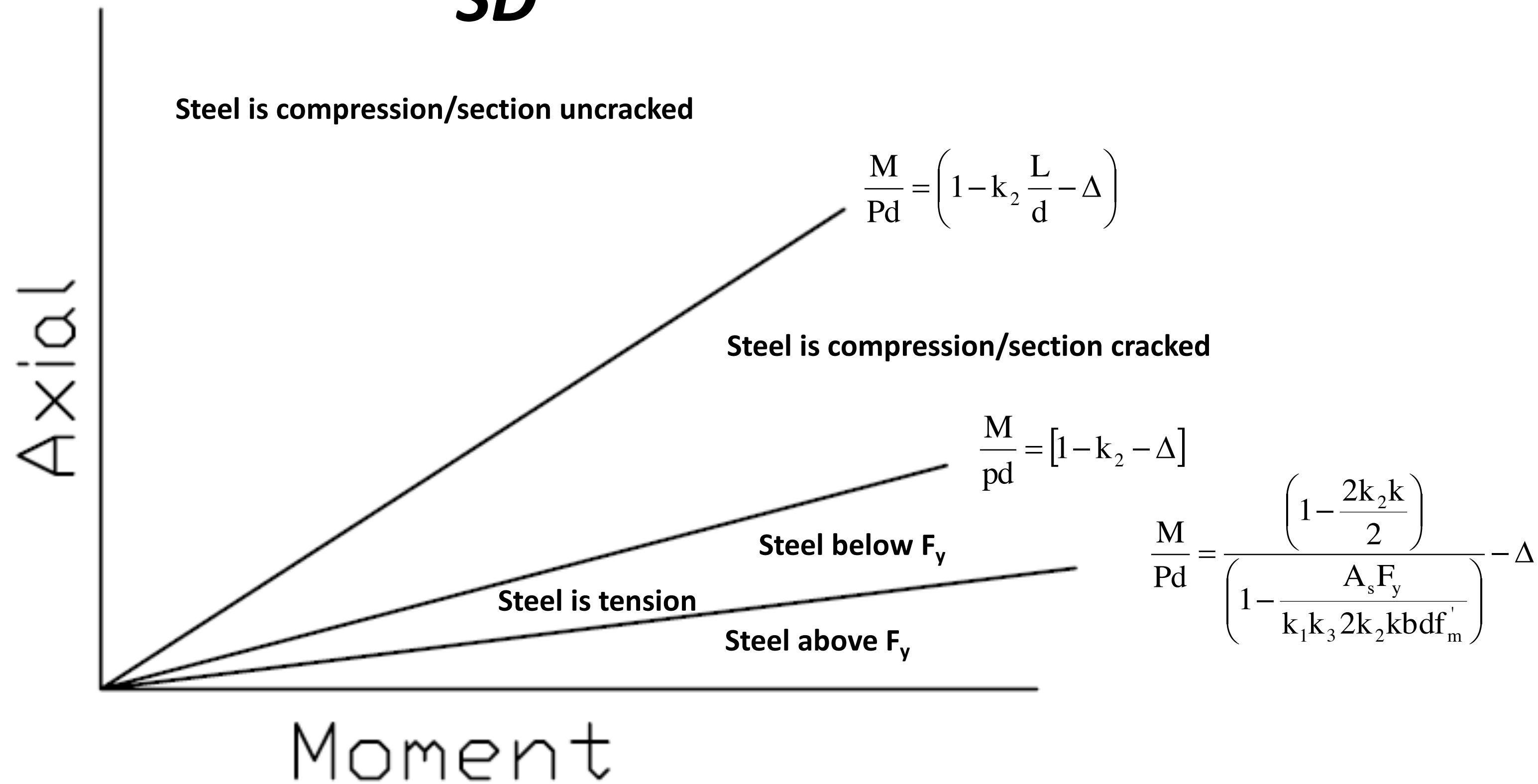
$$M=A_s F_y\left(1-\frac{2k_2k}{2}\right)d+P\left(1-\frac{2k_2k}{2}-\Delta\right)d$$

The Variables and Solution for the Unknowns



The Variables and Solution for the Unknowns

SD

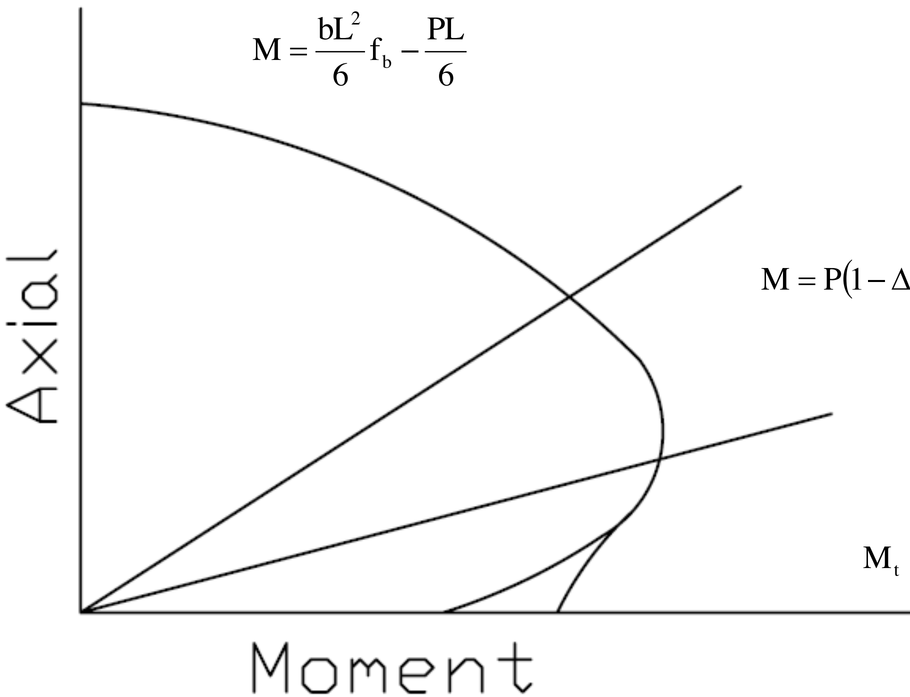


The Variables and Solution for the Unknowns

$$k = \sqrt{\left(n\rho + \frac{\varepsilon_{m0}}{\varepsilon_s}\right)^2 + 2\left(n\rho + \frac{\varepsilon_{m0}}{\varepsilon_s}\right)} - \left(n\rho + \frac{\varepsilon_{m0}}{\varepsilon_s}\right)$$

$$\frac{\varepsilon_{m0}}{\varepsilon_s} = \frac{n\rho\left(1 - \frac{k}{3}\right)}{\left(\frac{M}{Pd} - \left(1 - \frac{k}{3} - \Delta\right)\right)}$$

$$M = \frac{bL^2}{6} f_b - \frac{PL}{6}$$

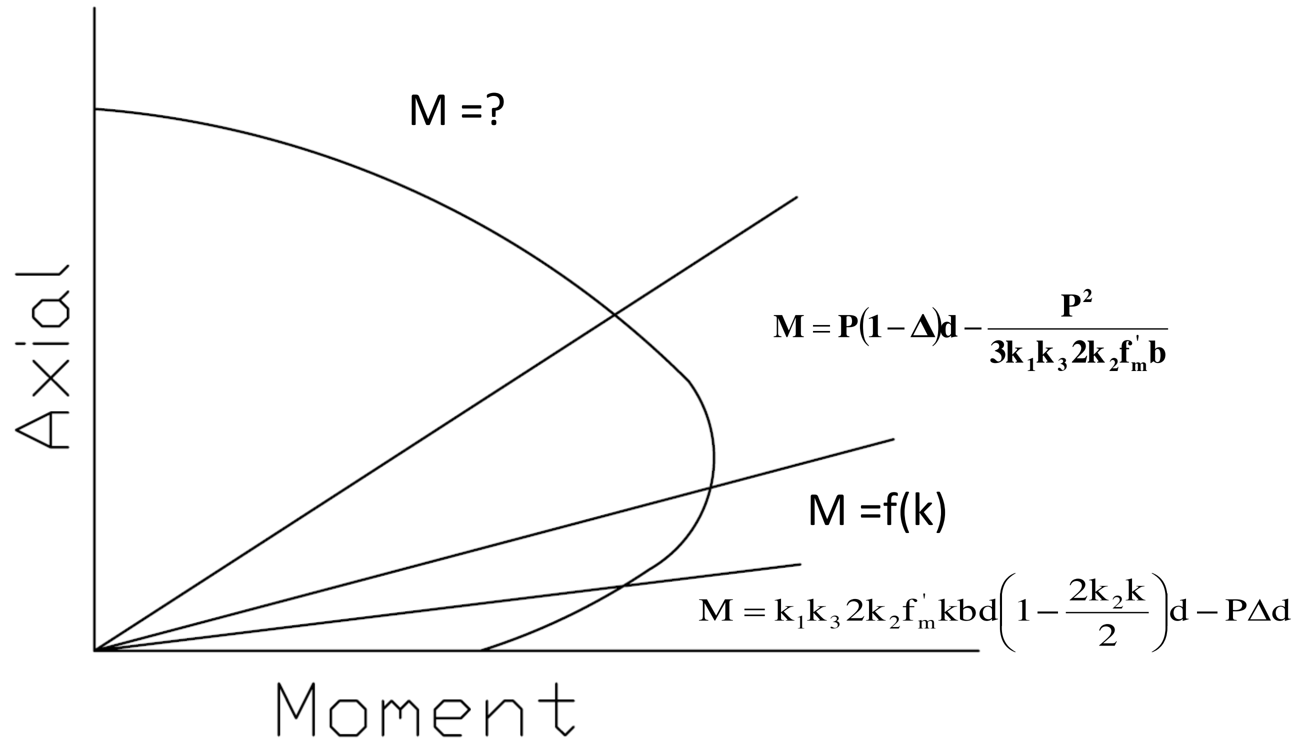


$$M = P(1-\Delta)d - \frac{2}{3}\frac{P^2}{F_b b}$$

$$M_c = \frac{F_b kjbd^2}{2} - P\Delta d$$

$$M_t = A_s F_s jd + P\left(1 - \frac{k}{3} - \Delta\right)d$$

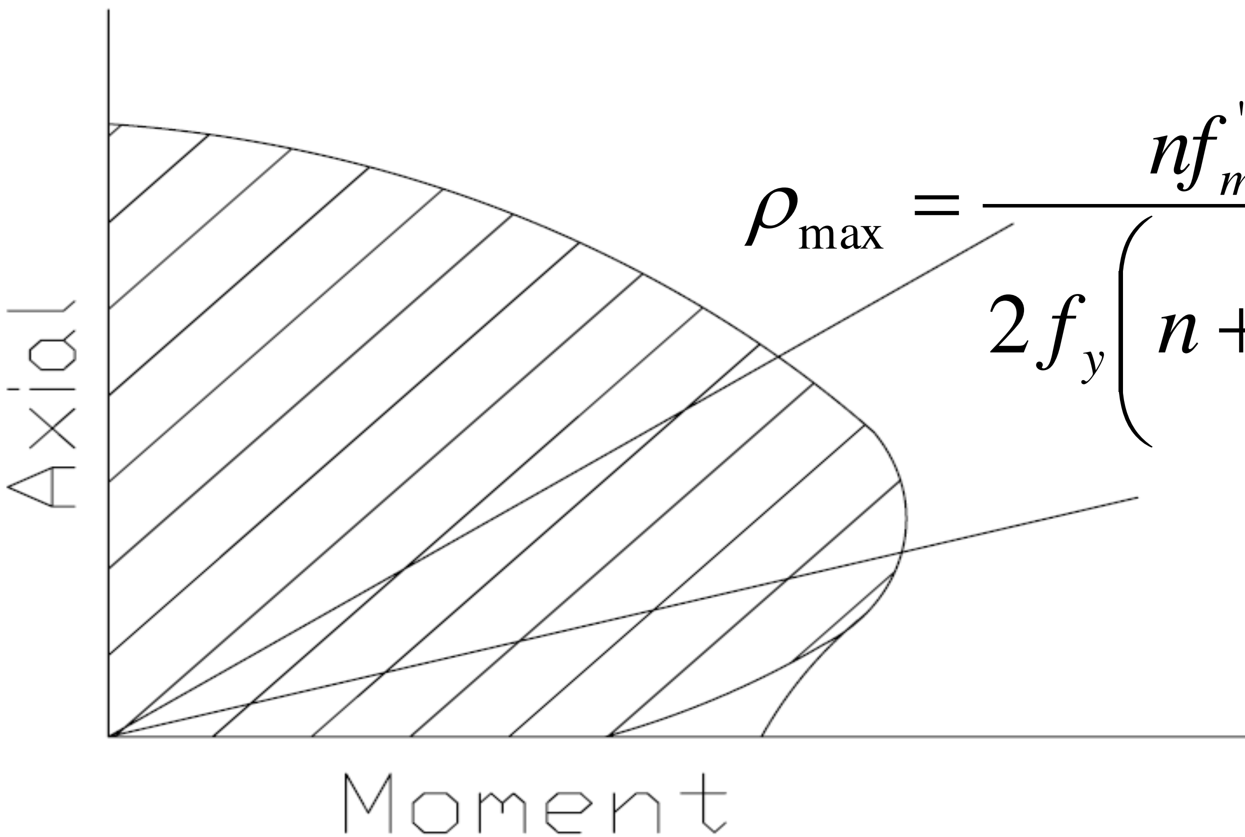
2.2 The Variables and Solution for the Unknowns



Ductility Requirements and the Codes

Axial Load is the same as reinforcement

Masonry ASD



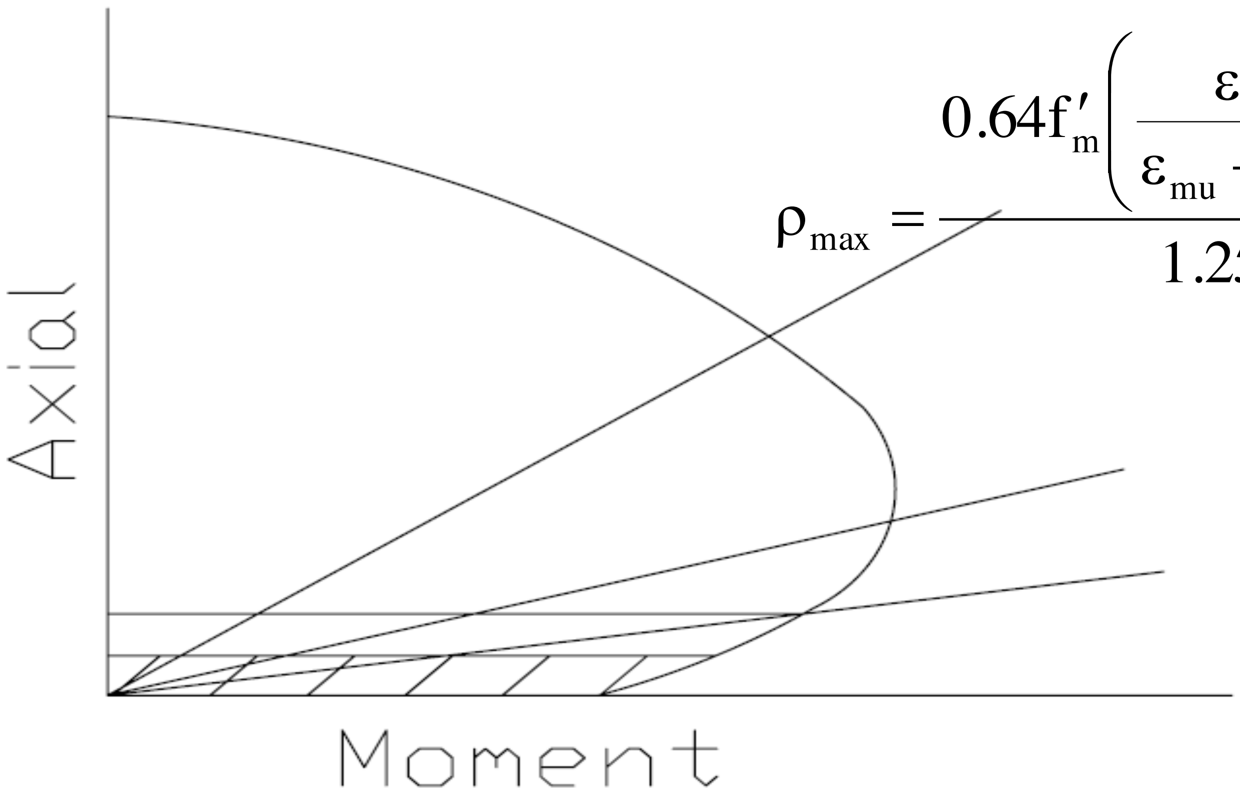
$$\rho_{\max} = \frac{n f_m^{'}}{2 f_y \left(n + \frac{f_y}{f_m^{'}} \right)}$$

Ductility Requirements and the Codes

Axial Load is the same as reinforcement

Masonry SD

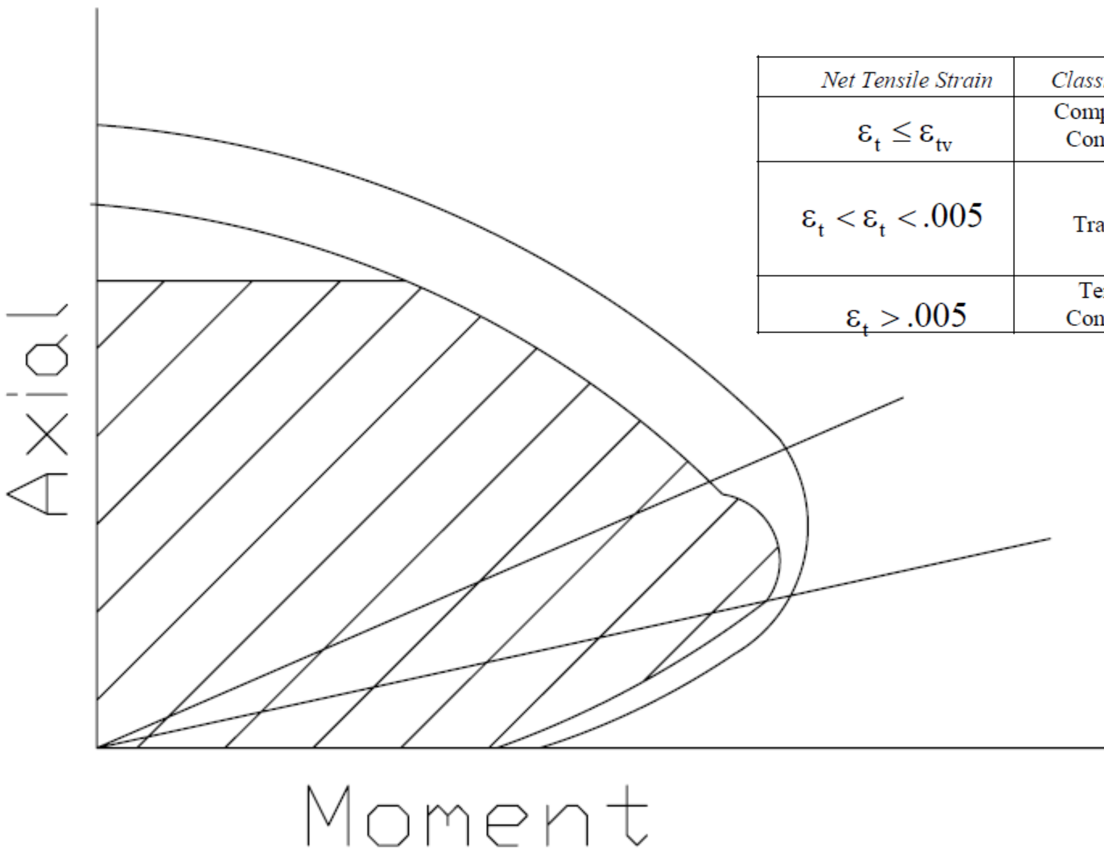
$$\rho_{max} = \frac{0.64 f'_m \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha \varepsilon_y} \right) - \frac{P_{ue}}{bd}}{1.25 f_y}$$



Ductility Requirements and the Codes

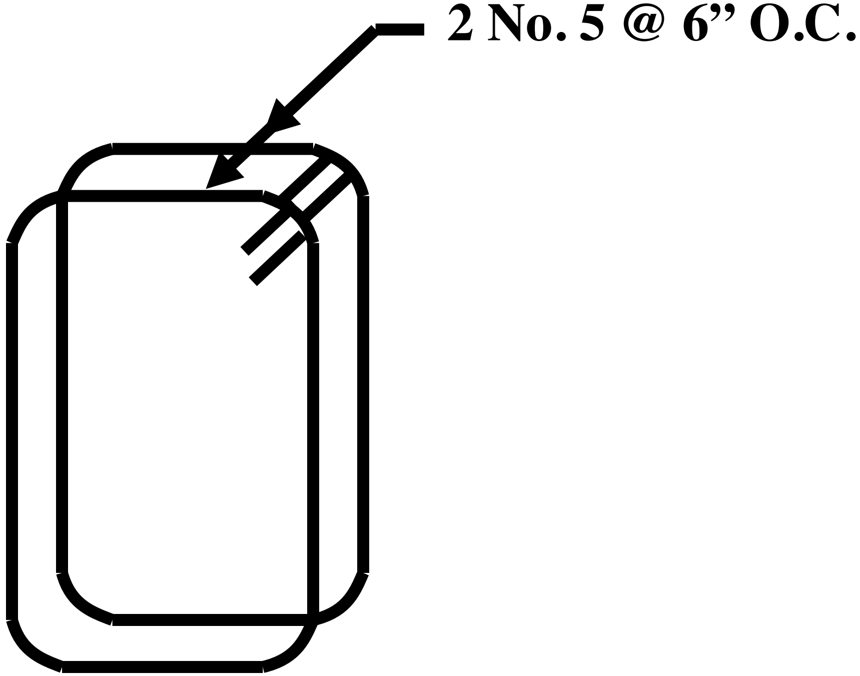
Axial Load is the same as reinforcement

Concrete

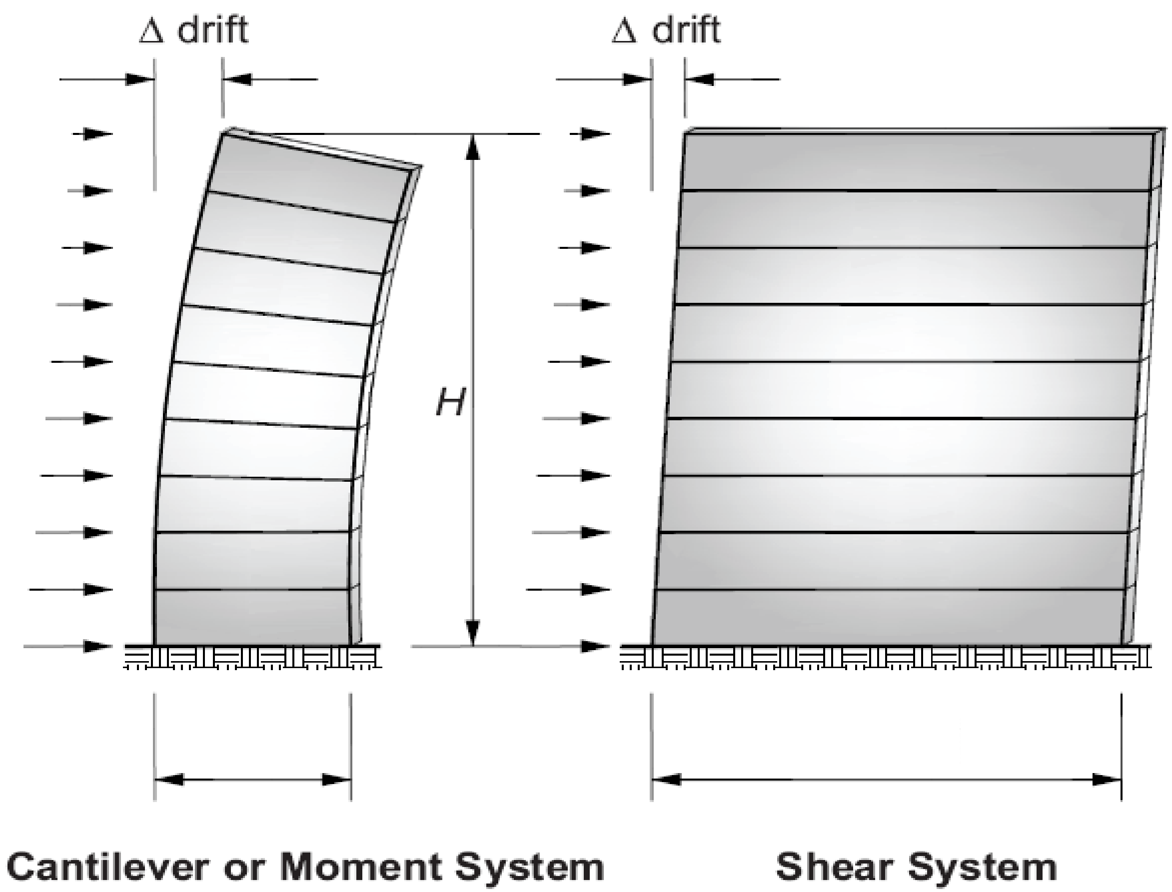


Net Tensile Strain	*Classification*	Φ
$\varepsilon_t \leq \varepsilon_{tv}$	Compression Controlled	.65
$\varepsilon_t < \varepsilon_t < .005$	Transition	$.65 + .25\frac{(\varepsilon_t - \varepsilon_{ty})}{(.005 - \varepsilon_{ty})}$
$\varepsilon_t > .005$	Tension Controlled	.90

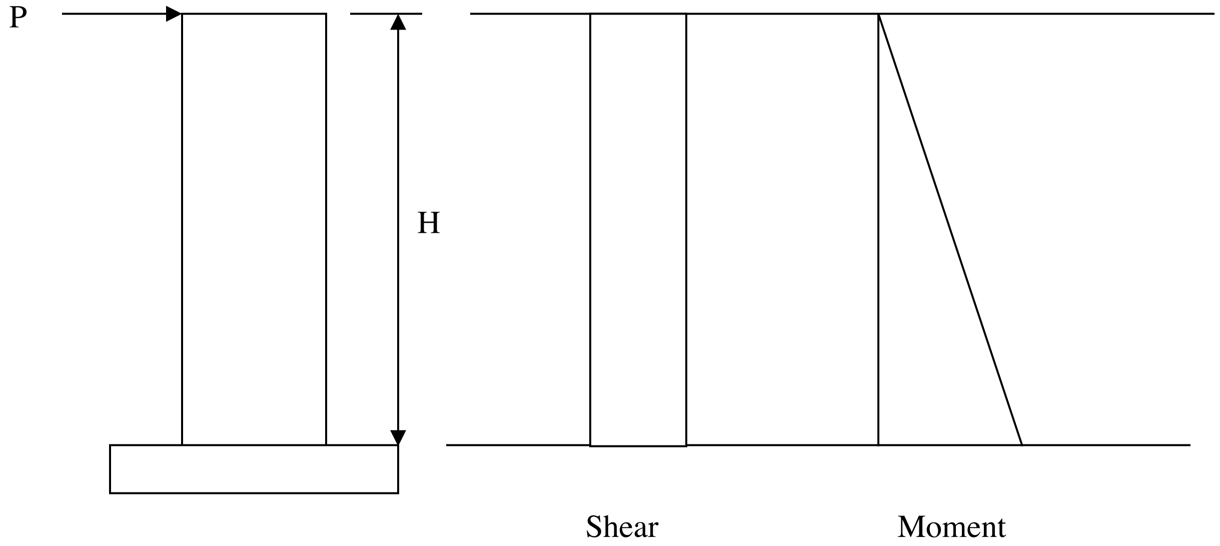
What is wrong with this detail?



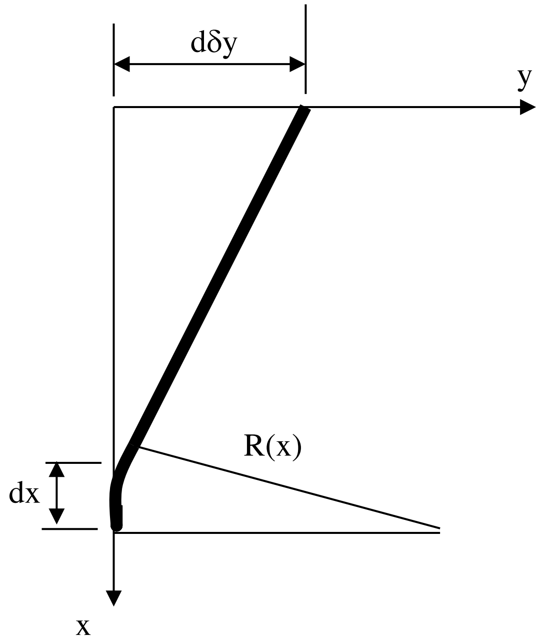
One More Thing – Distribution of Loads



Distribution of Loads



Distribution of Loads - Flexure



$$\frac{dx}{R(x)} = \frac{d\delta_y}{x} \quad \text{or} \quad d\delta_y = \frac{x\,dx}{R(x)}$$

$$\frac{1}{R(x)} = \frac{\varepsilon(x)}{y} \qquad \varepsilon = \varepsilon_0 \frac{x}{H}$$

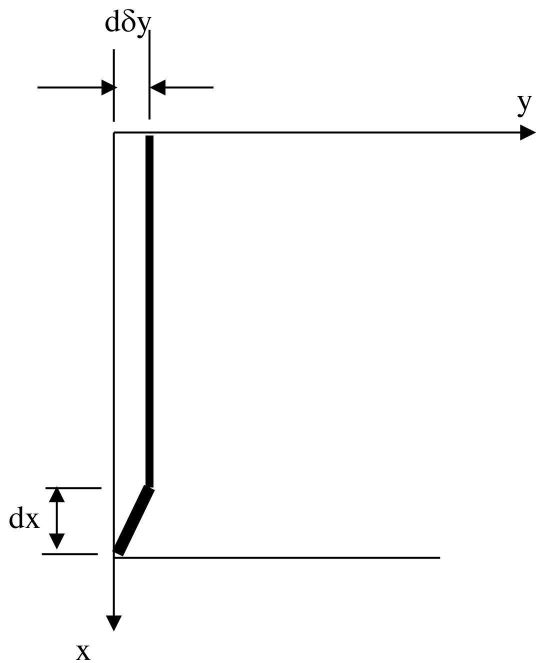
$$d\delta_y = \frac{\varepsilon_0}{yH} x^2 dx$$

$$\delta_y = \frac{\varepsilon_0}{yH}\int_0^H x^2 dx \quad = \frac{\varepsilon_0}{y}\frac{H^2}{3}$$

$$\varepsilon_0 = \frac{Mc}{EI} = \frac{PH\frac{L}{2}}{\frac{E}{12}TL^3} = \frac{6PH}{ETL^2}$$

$$\delta_y = \frac{6PH^2}{ETL^2\frac{L}{2}}\frac{H^2}{3} = \frac{P}{ET}\left[4\left(\frac{H}{L}\right)^3\right]$$

Distribution of Loads - Shear



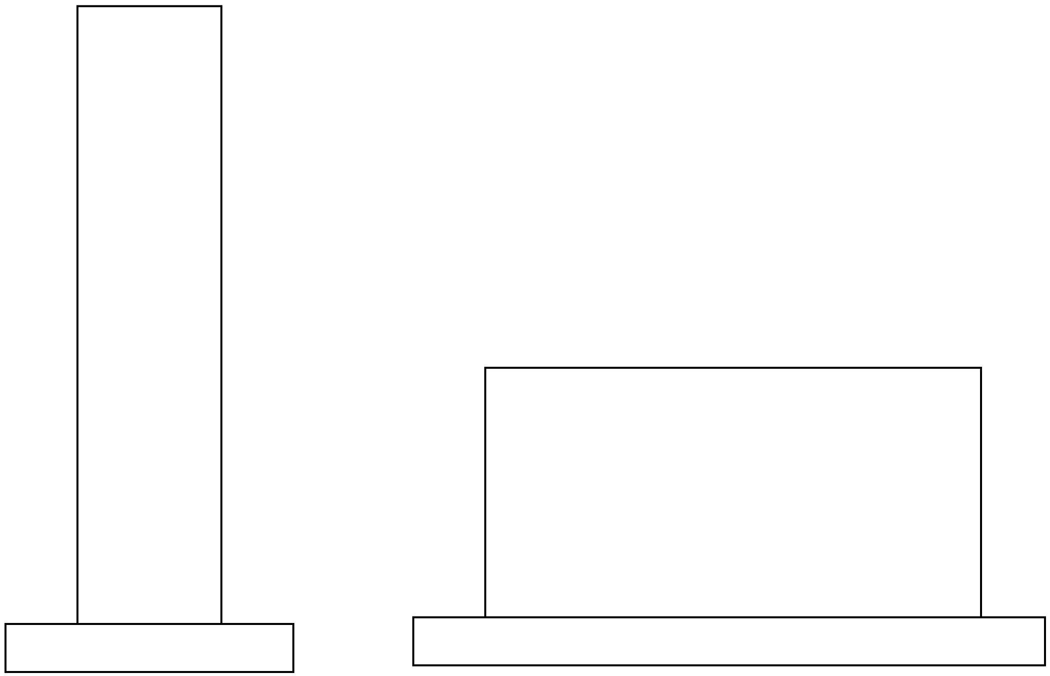
$$\frac{dy}{dx} = \frac{P}{GTL}$$

$$y = \frac{P}{GTL}\int dx = \frac{PH}{GTL}$$

$$G = \frac{E}{2(1+\nu)} \quad \nu = .3$$

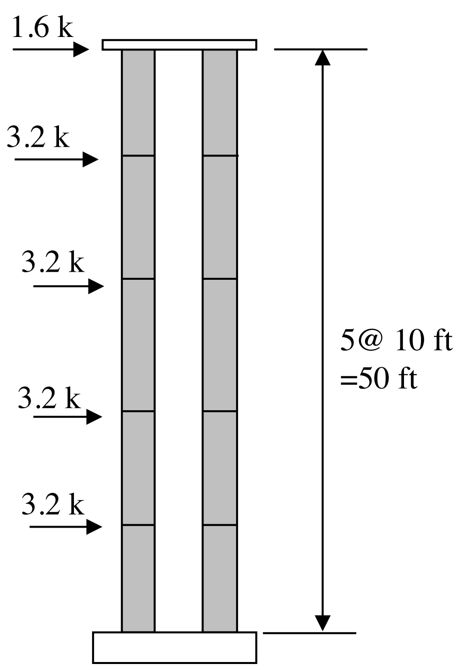
$$y = \frac{P}{GTL}\int dx = \frac{P}{GT}\left[\left(\frac{H}{L}\right)\right] = \frac{P}{ET}\left[2.6\left(\frac{H}{L}\right)\right]$$

Distribution of Loads – Flexure Plus Shear

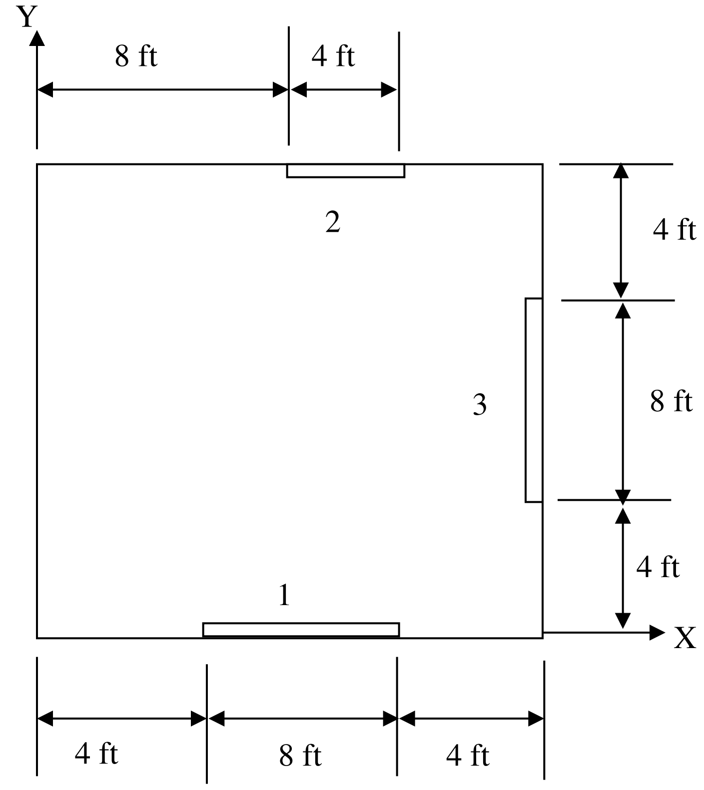


$$\delta_y = \frac{P}{ET}\left[4\left(\frac{H}{L}\right)^3 + 2.6\frac{H}{L}\right]$$

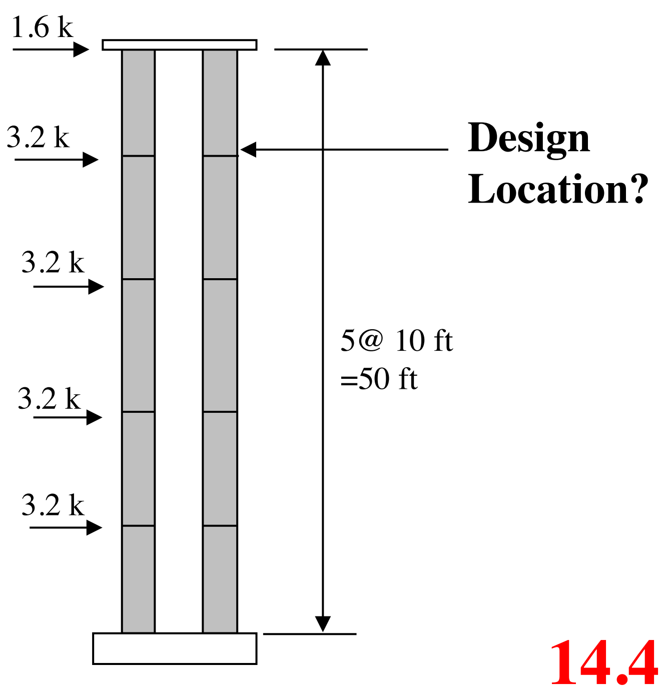
Distribution of Loads – Example

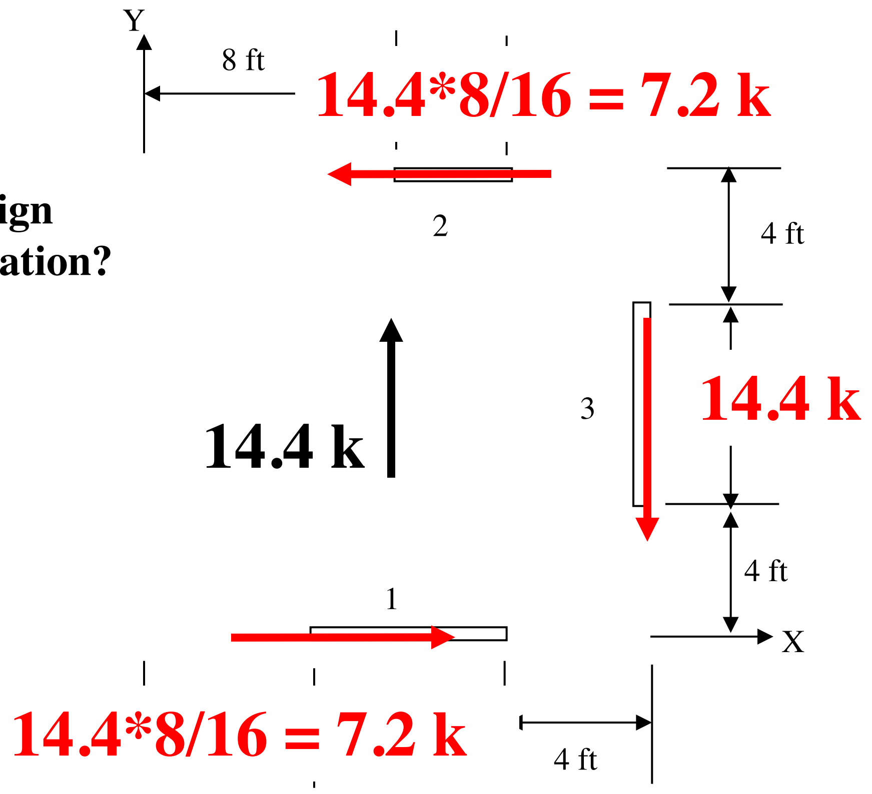


Hose Tower –
Wind Load



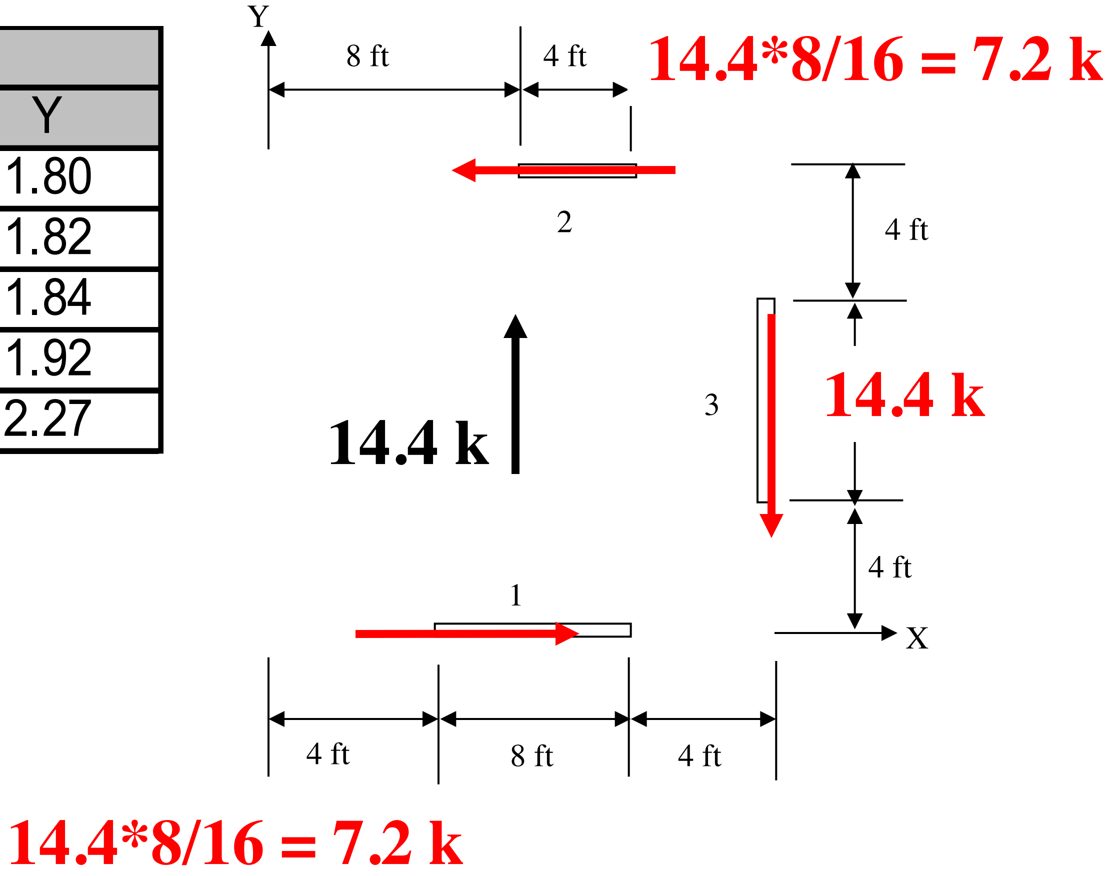
Distribution of Loads – Example





Distribution of Loads – Example

Center of Rigidity		
	X	Y
Roof	15.99	1.80
4th	15.99	1.82
3rd	15.99	1.84
2nd	15.99	1.92
1st	15.99	2.27



Codes – IBC 2012

[IBC 2012]1604.8.2 Structural walls. Walls that provide vertical load-bearing resistance or lateral shear resistance for a portion of the structure shall be anchored to the roof and to all floors and members that provide lateral support for the wall or that are supported by the wall. The connections shall be capable of resisting the horizontal forces specified in Section 1.4.5 of ASCE 7 for walls of structures assigned to Seismic Design Category A and to Section 12.11 of ASCE 7 for walls of structures assigned to all other seismic design categories. Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1609 for wind design requirements and 1613 for earthquake design requirements.

Codes – ASCE 7 2010

12.11 STRUCTURAL WALLS AND THEIR ANCHORAGE

12.11.1 Design for Out-of-Plane Forces. Structural walls and their anchorage shall be designed for a force normal to the surface equal to $F_p = 0.4 S_{DS} I_e$ times the weight of the structural wall with a minimum force of 10% of the weight of the structural wall. Interconnection of structural wall elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

12.11.2 Anchorage of Structural Walls and Transfer of Design Forces into Diaphragms

12.11.2.1 Wall Anchorage Forces. The anchorage of structural walls to supporting construction shall provide a direct connection capable of resisting the following:

$$F_p = 0.4 S_{DS} k_a I_e W_p \qquad (12.11\text{-}1)$$

Codes – ASCE 7 2010

F_p shall not be taken less than $0.2k_aI_eW_p$.

$$k_a = 1.0 + \frac{L_f}{100} \qquad (12.11\text{-}2)$$

k_a need not be taken larger than 2.0.

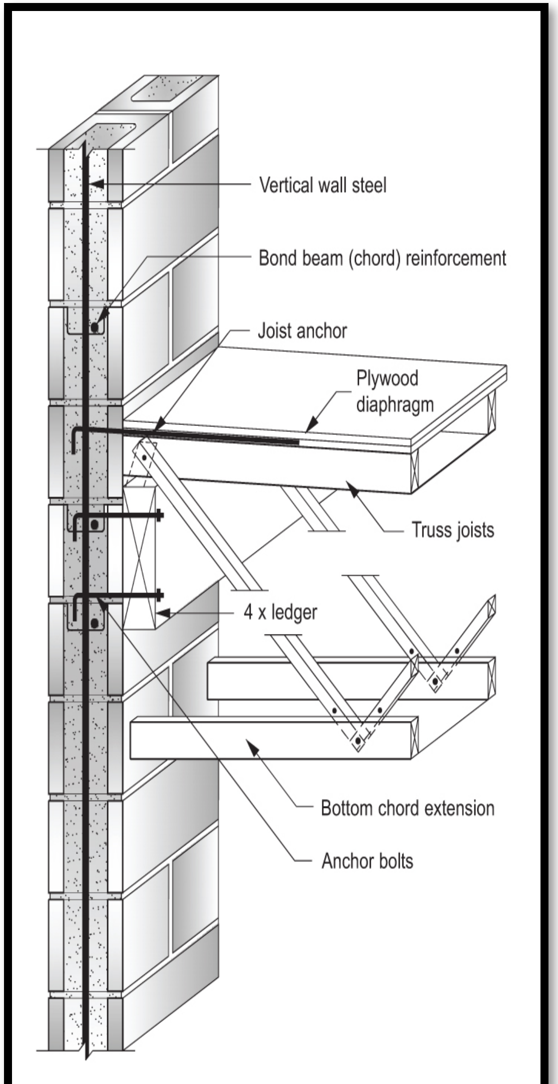
where

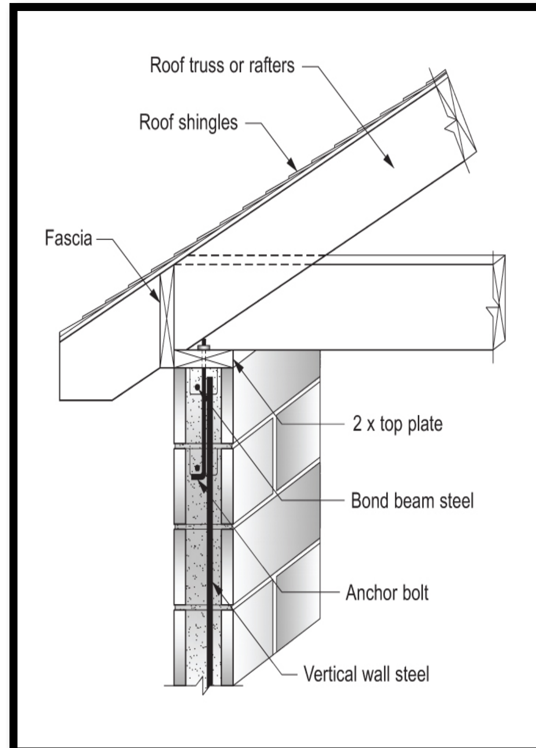
F_p = the design force in the individual anchors
S_{DS} = the design spectral response acceleration parameter at short periods per Section 11.4.4
I_e = the importance factor determined in accordance with Section 11.5.1
k_a = amplification factor for diaphragm flexibility
L_f = the span, in feet, of a flexible diaphragm that provides the lateral support for the wall; the span is measured between vertical elements that provide lateral support to the diaphragm in the direction considered; use zero for rigid diaphragms
W_p = the weight of the wall tributary to the anchor

Where the anchorage is not located at the roof and all diaphragms are not flexible, the value from Eq. 12.11-1 is permitted to be multiplied by the factor $(1 + 2z/h)/3$, where z is the height of the anchor above the base of the structure and h is the height of the roof above the base.

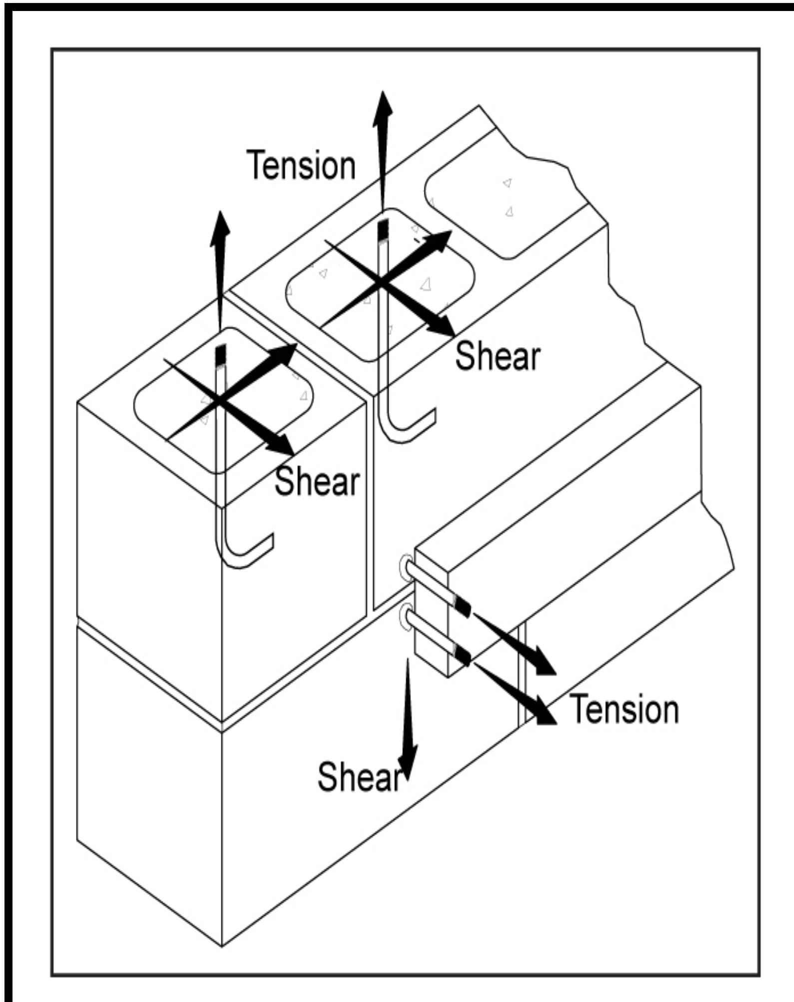
Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1,219 mm).

Anchor Bolt Design



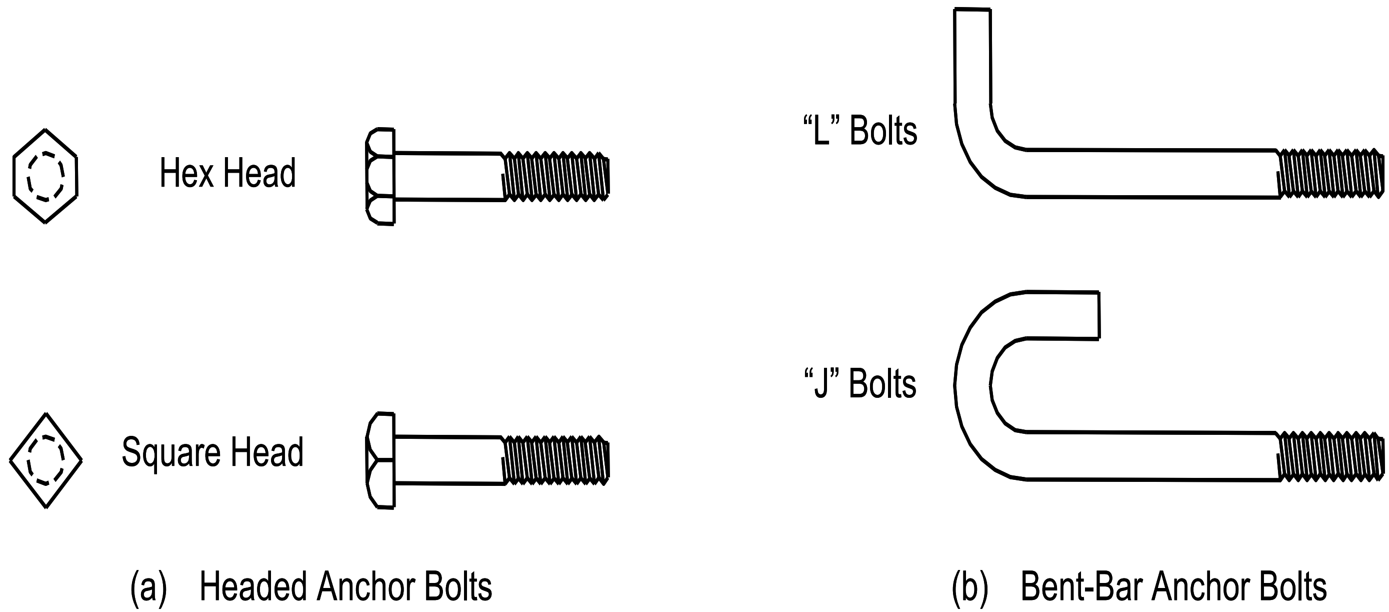


Anchor Bolt Design



Anchorage Design Loads

Anchor Bolt Design



Anchor Bolt Design

- Tension
  - Radial cracking followed by masonry tension breakout
  - Straightening of bolt hook followed by bolt pullout
  - Yielding of bolt followed by bolt fracture
- Shear
  - Radial cracking followed by masonry shear breakout
  - Yielding of bolt followed by bolt fracture

Anchor Bolt Design

- Tension
  - Radial cracking and tension breakout

Anchor Bolt Design

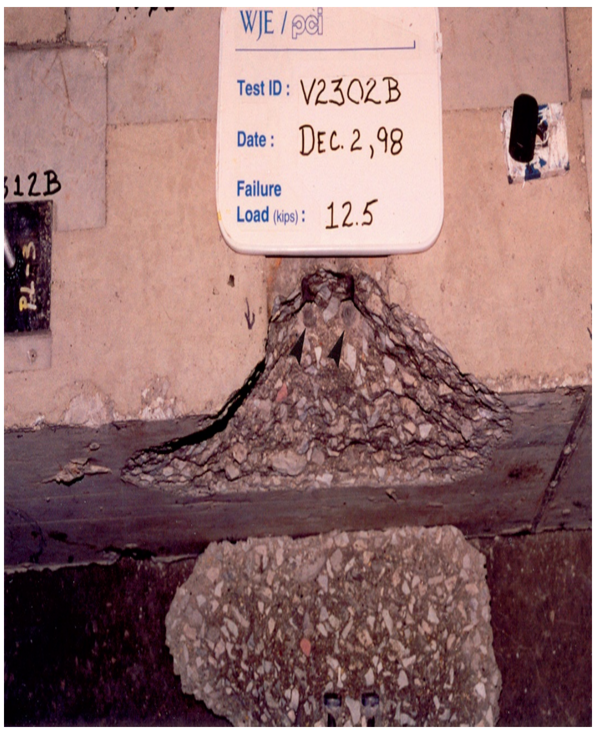
- Tension
  - Hook straightening and bolt pullout

Anchor Bolt Design

- Shear
  - Masonry crushing along with bolt yielding

Anchor Bolt Design

- Shear
  - Shear breakout towards free edge



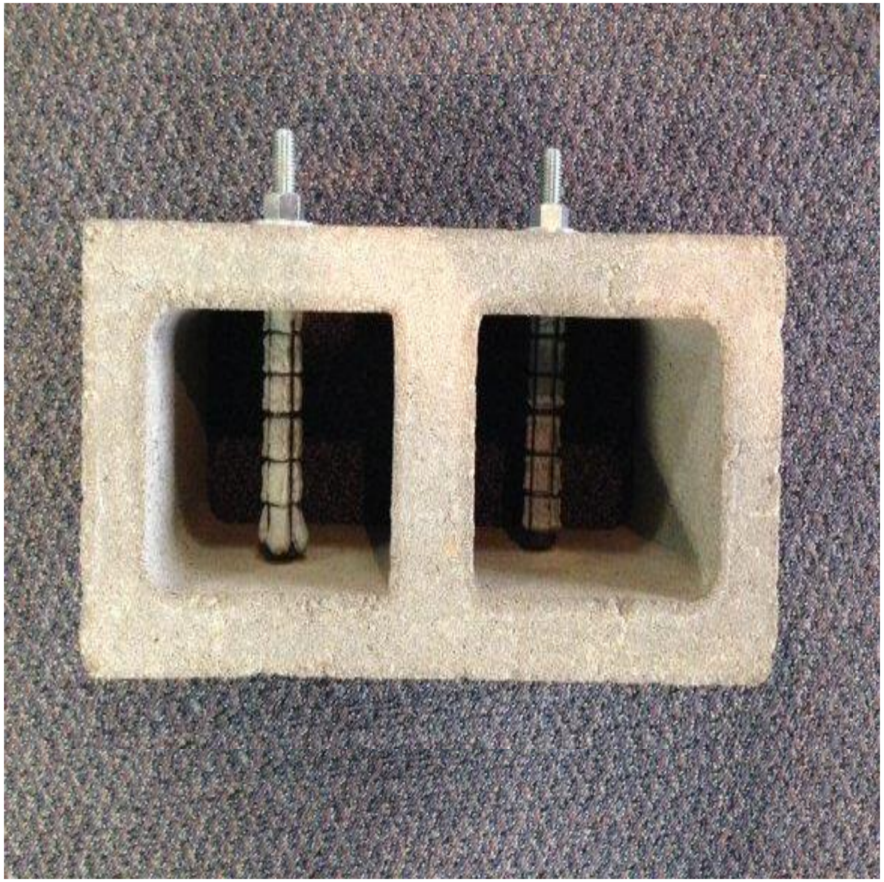
Anchor Bolt Design

(no longer required)

Now OK

Section 6.2.1 - Anchor bolts placed in drilled holes in the face shells of hollow masonry units permitted to contact the masonry unit. [2016 TMS 402]

Anchor Bolt Design

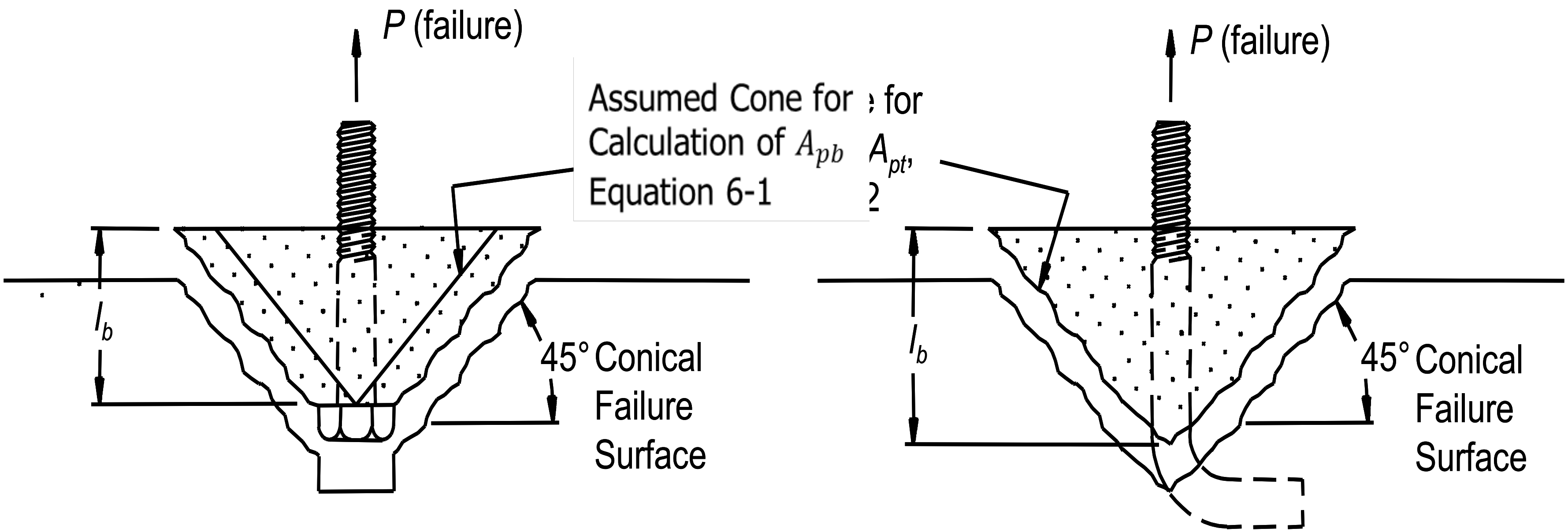


Adhesive anchors in screen tubes

Anchor Bolt Design

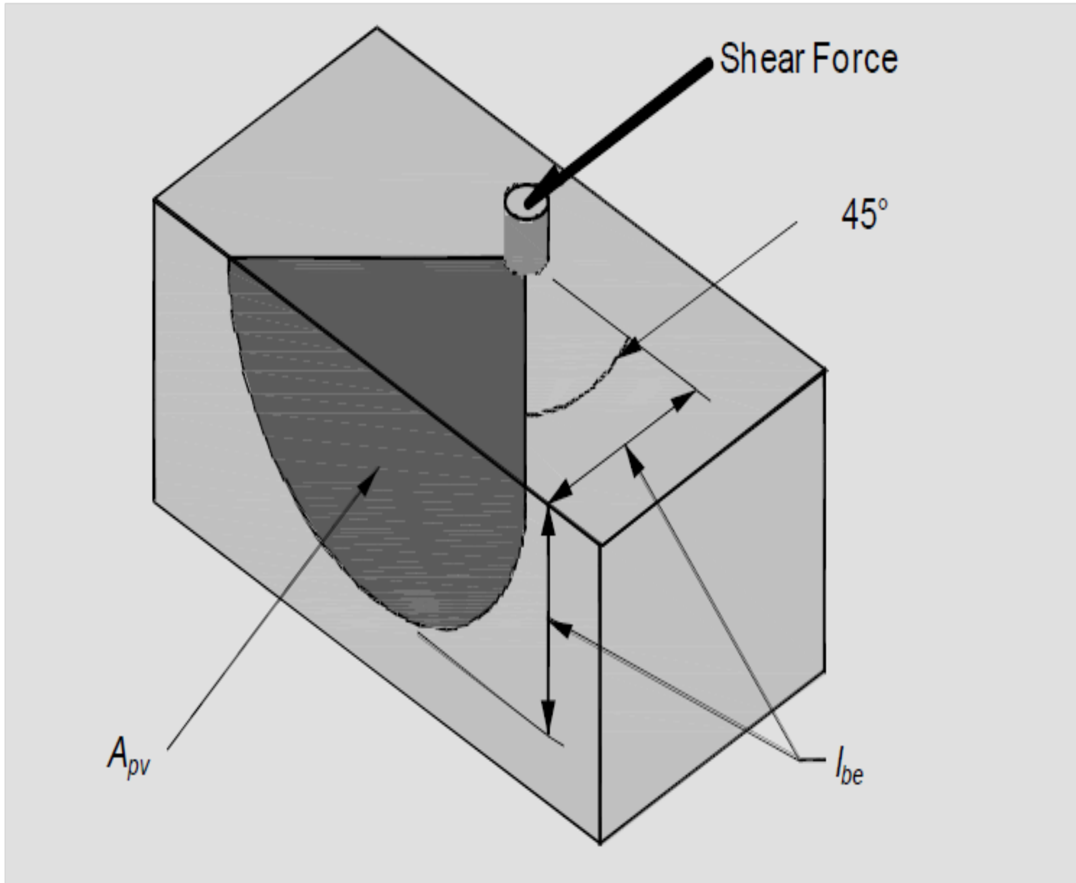
ASD and SD provisions for anchor bolt design are largely the same

Anchor Bolt Design



A_{pt} = projected area in tension on masonry surface of cone = πl_b^2

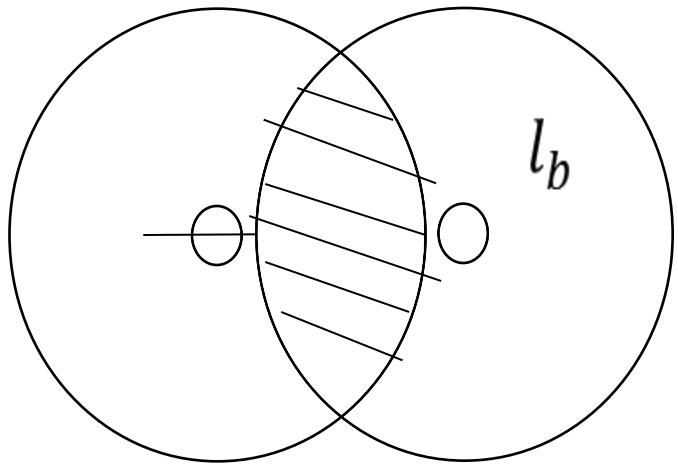
Anchor Bolt Design



$$\frac{\pi l_{be}^2}{2}$$

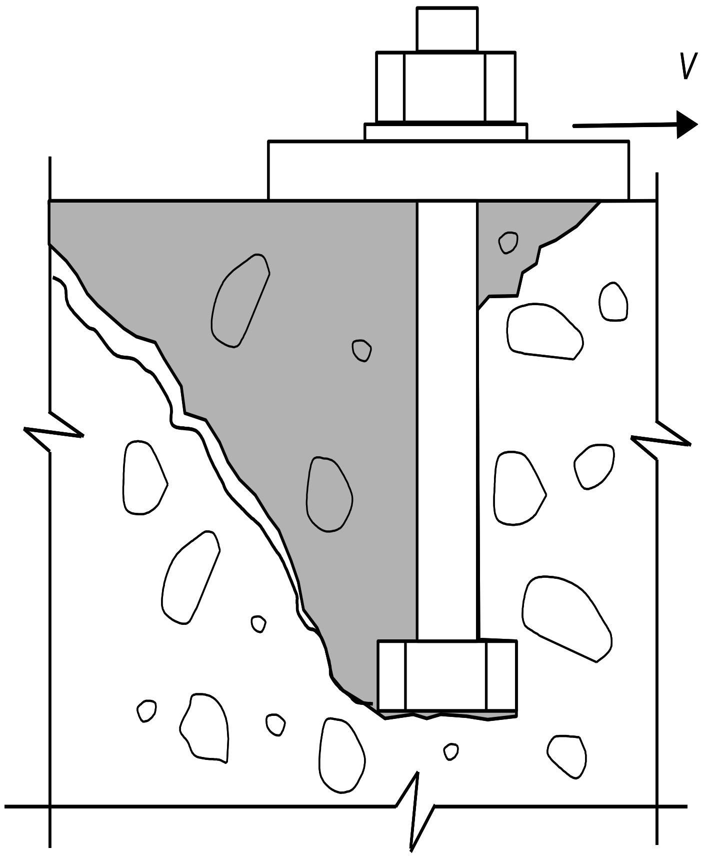
A_{pv} = projected area in shear on the masonry surface of the half cone.

Anchor Bolt Design



Overlapping anchor bolt breakout cones—subtract hatched area from A_{pt}

Anchor Bolt Design



Anchor bolt shear pryout

Anchor Bolt Design

$$B_{ab} = 1.25\, A_{pt} \sqrt{f'_m}$$

$$B_{as} = 0.6\ A_b f_y$$

$$B_{ap} = 0.6 f'_m\, e_b\, d_b + 120\,\pi\,(l_b + e_b + d_b)d_b$$

Allowable tensile capacity B_a = smallest of (B_{ab}, B_{as}, B_{ap})

$$B_{vb} = 1.25\, A_{pv} \sqrt{f'_m}$$

$$B_{vc} = 350 \sqrt[4]{f'_m A_b}$$

$$B_{vpry} = 2.5\, A_{pt} \sqrt{f'_m}$$

$$B_{vs} = 0.36\ A_b\ f_y$$

Allowable shear capacity B_v = smallest of ($B_{vb}, B_{vc}, B_{vpry}, B_{vs}$)

Anchor Bolt Design

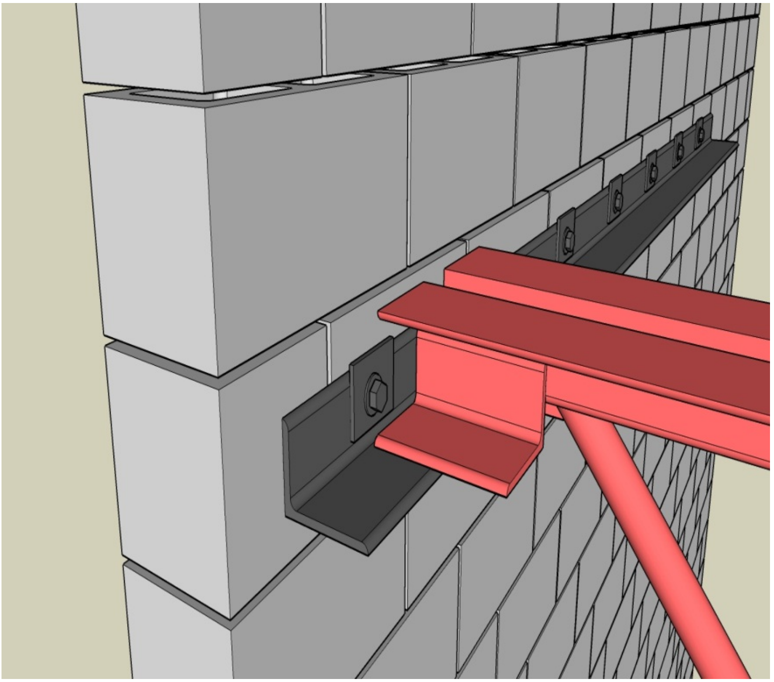
- Anchor bolts subjected to combined shear and tension must satisfy a linear interaction equation

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \leq 1$$

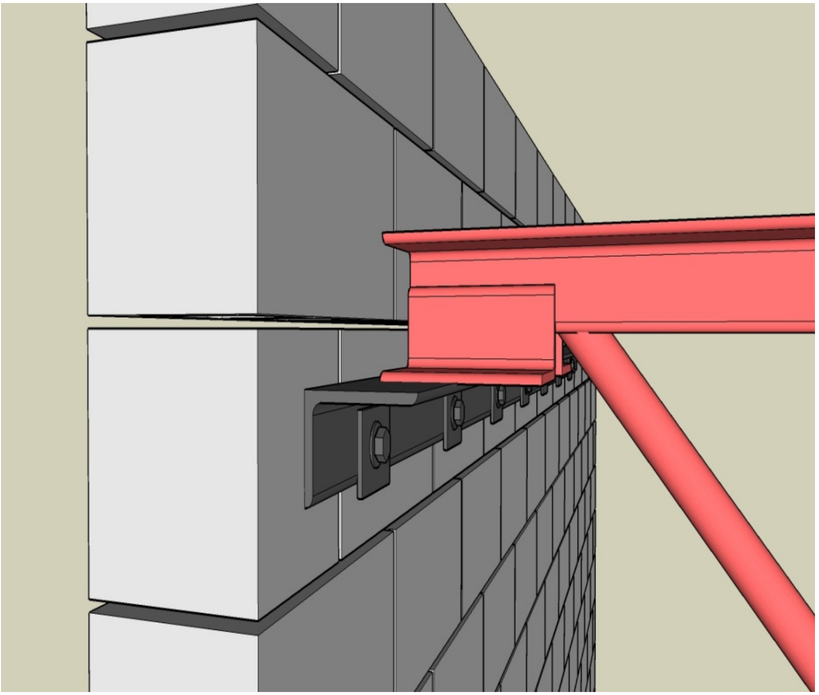
Anchor Bolt Design

Or

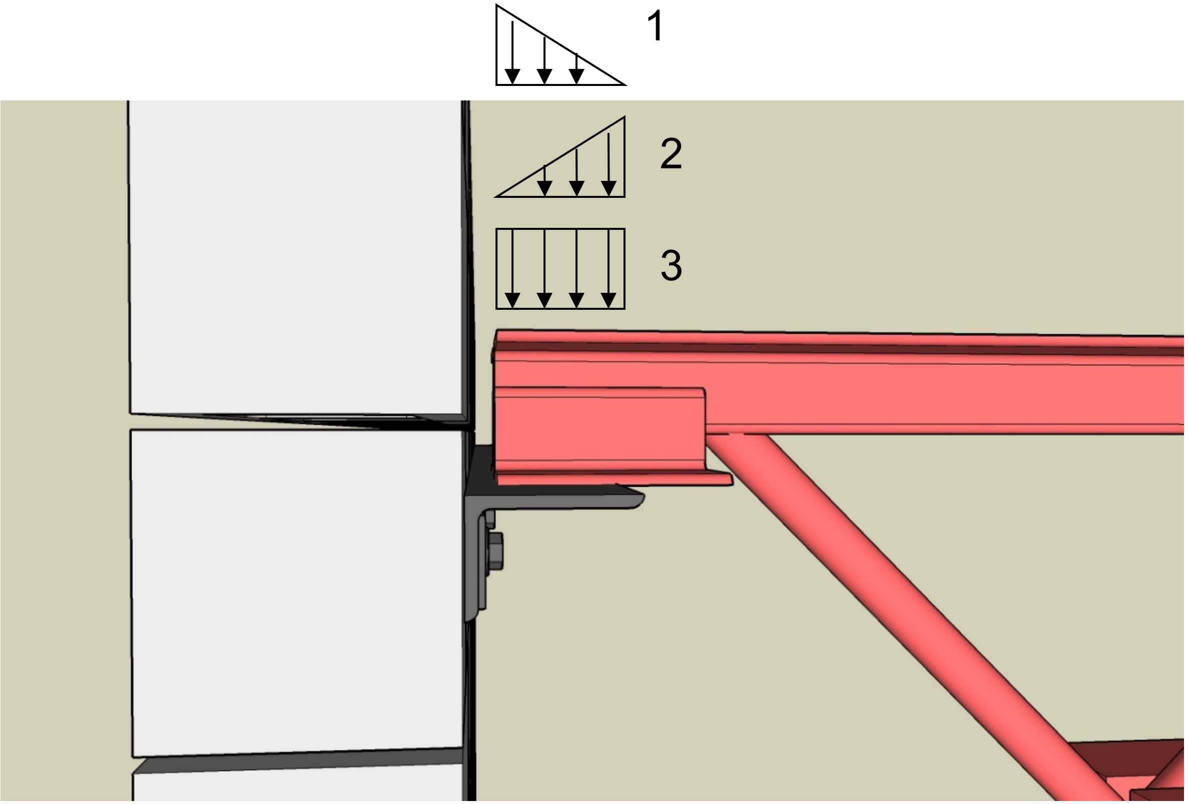
Angle Pointed Up



Angle Pointed Down



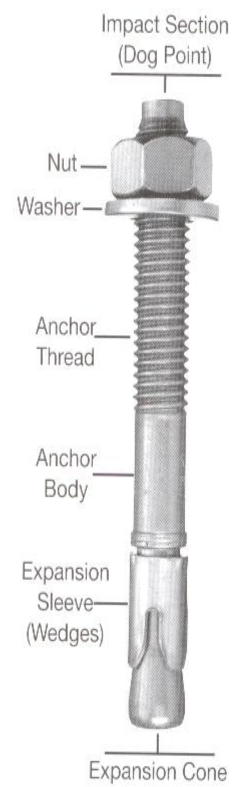
Anchor Bolt Design



Question: 1, 2 or 3 ?

Anchor Bolt Design

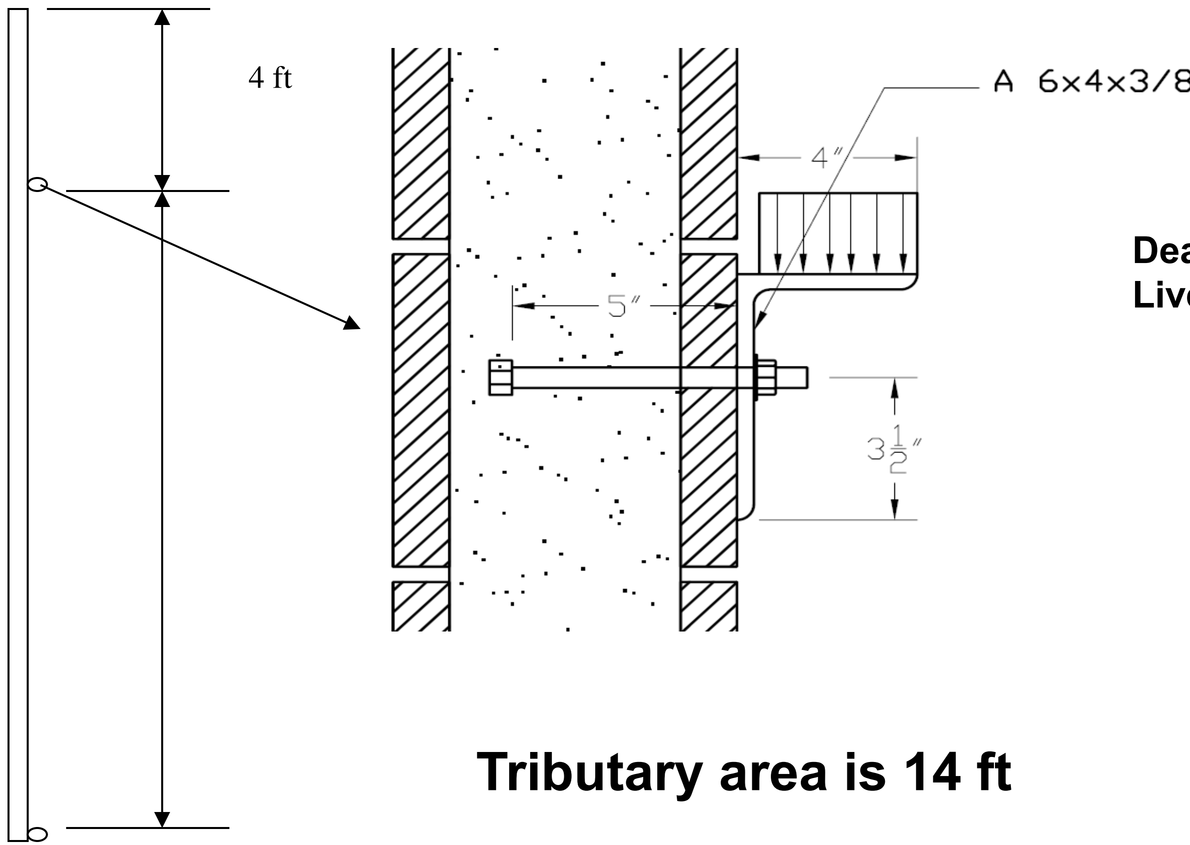
*ICC Reports Available



Adhesive Anchor

Mechanical Anchors

Anchor Bolt Example



Dead load = 240 lb/ft
Live load = 144 lb/ft

Tributary area is 14 ft

14*80 = 1,120 Lbs/ft

Anchor Bolt Example

ASD

$D + F$ **(Equation 16-8)**					
$D + H + F + L$ **(Equation 16-9)**					
$D + H + F + (Lr \text{ or } S \text{ or } R)$ **(Equation 16-10)**					
$D + H + F + 0.75(L) + 0.75(Lr \text{ or } S \text{ or } R)$ **(Equation 16-11)**					
$D + H + F + (0.6W \text{ or } 0.7E)$ **(Equation 16-12)**					
$D + H + F + 0.75(0.6W) + 0.75L + 0.75(Lr \text{ or } S \text{ or } R)$ **(Equation 16-13)**					
$D + H + F + 0.75(0.7E) + 0.75L + 0.75S$ **(Equation 16-14)**					
$0.6D + 0.6W + H$ **(Equation 16-15)**					
$0.6(D + F) + 0.7E + H$ **(Equation 16-16)**					

ASCE 7-10 Section 12.11 [Ref. By IBC]

$$F_p = .4S_{DS}k_aI_eW_p$$

$$k_a = 1 + \frac{L_f}{100} \leq 2.0$$

Use k_a = 2.0
S_{DS} = 1.5
I_e = 1.0

$$F_p = .4*1.5*2.0*1*1{,}120 = 1{,}344 \text{ Lb/ft}$$

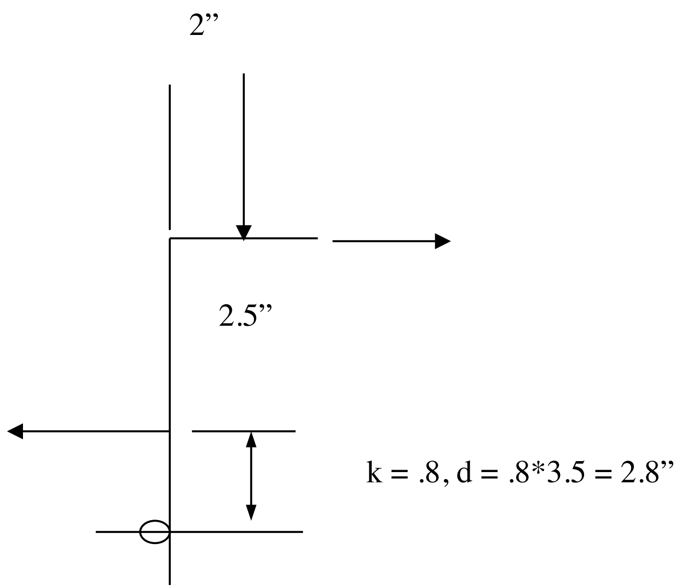
Strength Level

D + .75(.7E)+.75L

Space bolts at 16 in O.C.

D = 240*16/12 = 320 Lb
L = .75*144*16/12 = 144 Lb
E = .75*.7*1344*16/12 = 940 Lb

Anchor Bolt Example



Tension on the bolt

$M = (320+144)*2 + 940*2.5 = 3{,}278$ Lb-in

$T = [3{,}278/2.8 = 1{,}170] + 940 = 2{,}110$ Lb

)+144 = 464 lb

Anchor Bolt Example

Tension allowable:

$$B_{ab} = 1.25 A_{pt} \sqrt{f'_m}$$

$$A_{pt} = \pi l_b^2 = 3.14 * 5^2 = 78.5$$

$$B_{ab} = 1.25 * 78.5 \sqrt{1500} = 3{,}800 \text{ lb}$$

Use 5/8 inch bolt. Area is conservatively .8*.31 = .25 in^2

$$B_{ab} = .6 A_b F_y$$

$$B_{ab} = .6 * .25 * 27{,}000 = 4{,}050 \text{ lb}$$

Anchor Bolt Example

Shear allowable

$$B_{vb} = 1.25 A_{pv} \sqrt{f'_m}$$

$$A_{pv} = \pi l_{be}^2 / 2$$

L_{be} is very large. Distance from the center of the bolt to the edge of the masonry.

$$B_{vc} = 350 \sqrt[4]{f'_m A_b}$$

$$B_{vc} = 350 \sqrt[4]{1500 * .25} = 1{,}540 \text{ lb}$$

$$B_{vpry} = 2.0 B_{ab} = 8{,}100 \text{ lb}$$

$$B_{vs} = .35 A_b f_y = .35 * .25 * 27{,}000 = 2{,}362 \text{ lb}$$

Anchor Bolt Example

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \leq 1.0$$

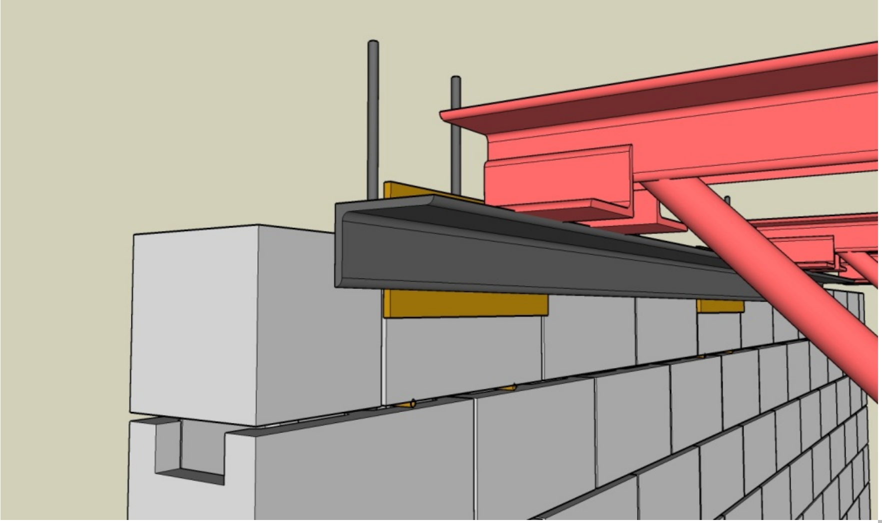
$$\frac{2{,}110}{3{,}800} + \frac{464}{1{,}540} = .55 + .30 = .85$$

5/8" Dia. Bolts at 16 inch O.C.

Codes – ASCE 7 2010

12.11.2.2.5 Embedded Straps. Diaphragm to structural wall anchorage using embedded straps shall be attached to, or hooked around, the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

Anchor Bolt Design



Design embed using break out and pull out concepts for bolts.

Install embeds with the masonry and grout

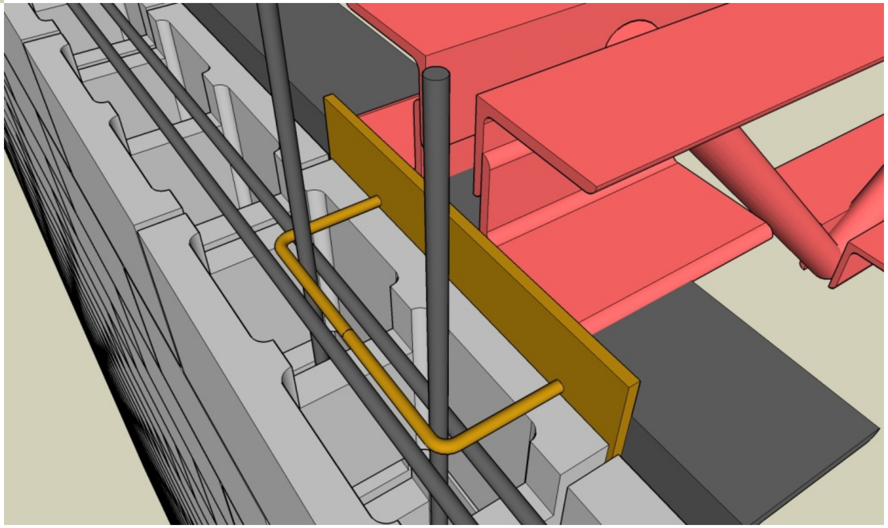
Hooks around reinforcement

Weld ledger angle to the embed

If ledgers not spliced at control joints, ledge can be the diaphragm cord tie.

Cost maybe less with cooperative contactor.

Some masons do not do misc. iron

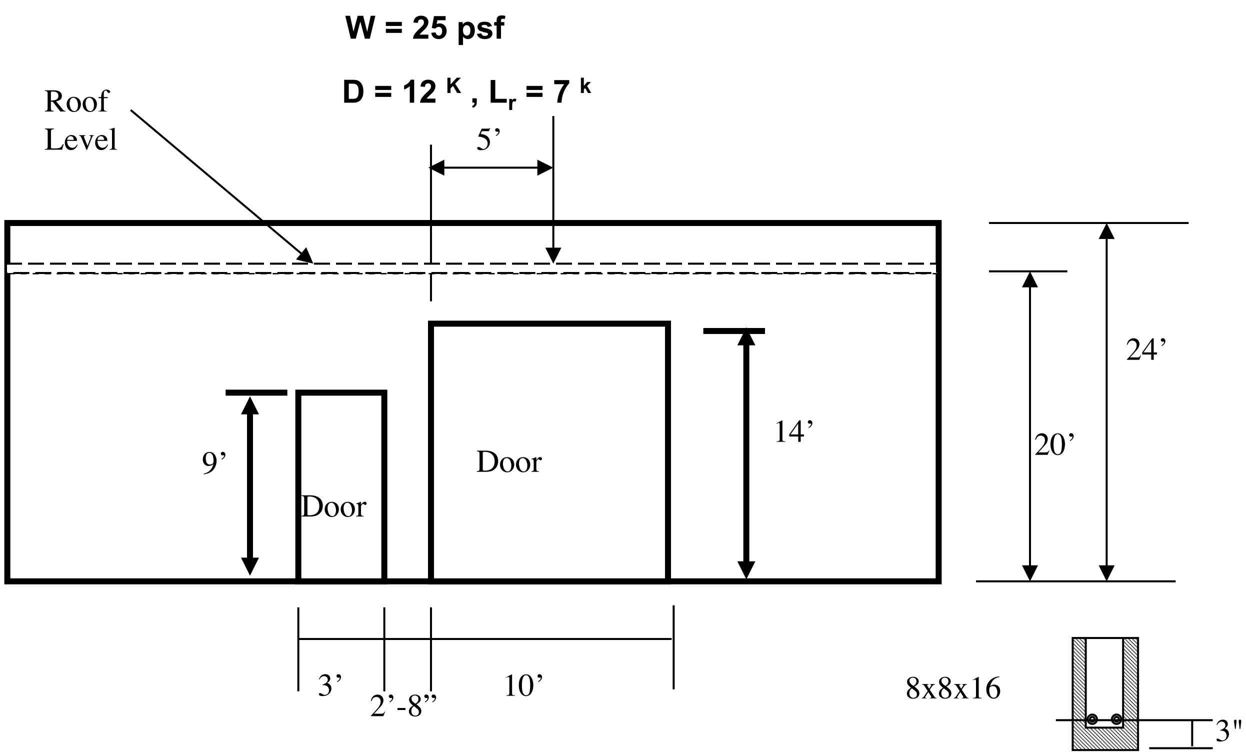


Example

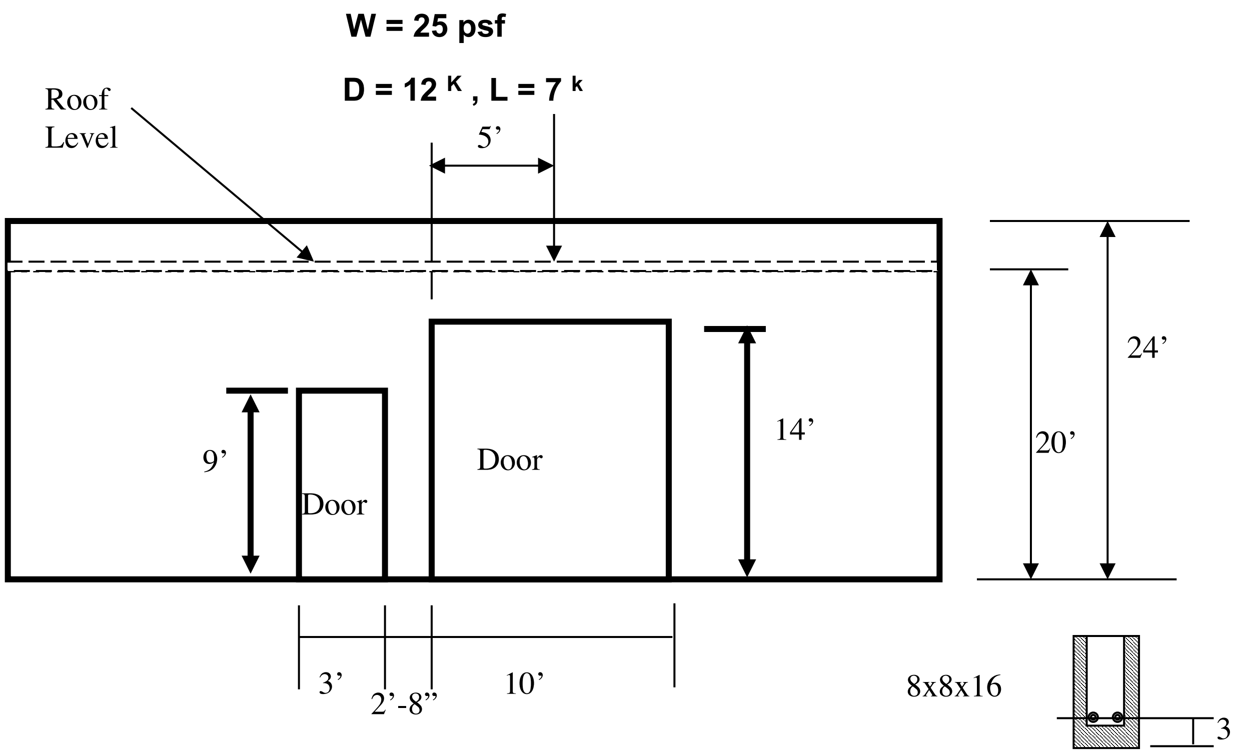
Future 2,000 psi

f'_m=1500 psi (CMU F_b= .45 f'_m =675 psi), F_y = 60,000psi (F_s = 32,000 psi)

E_m = 900 f'_m [ASD no longer 1/3 stress increase]



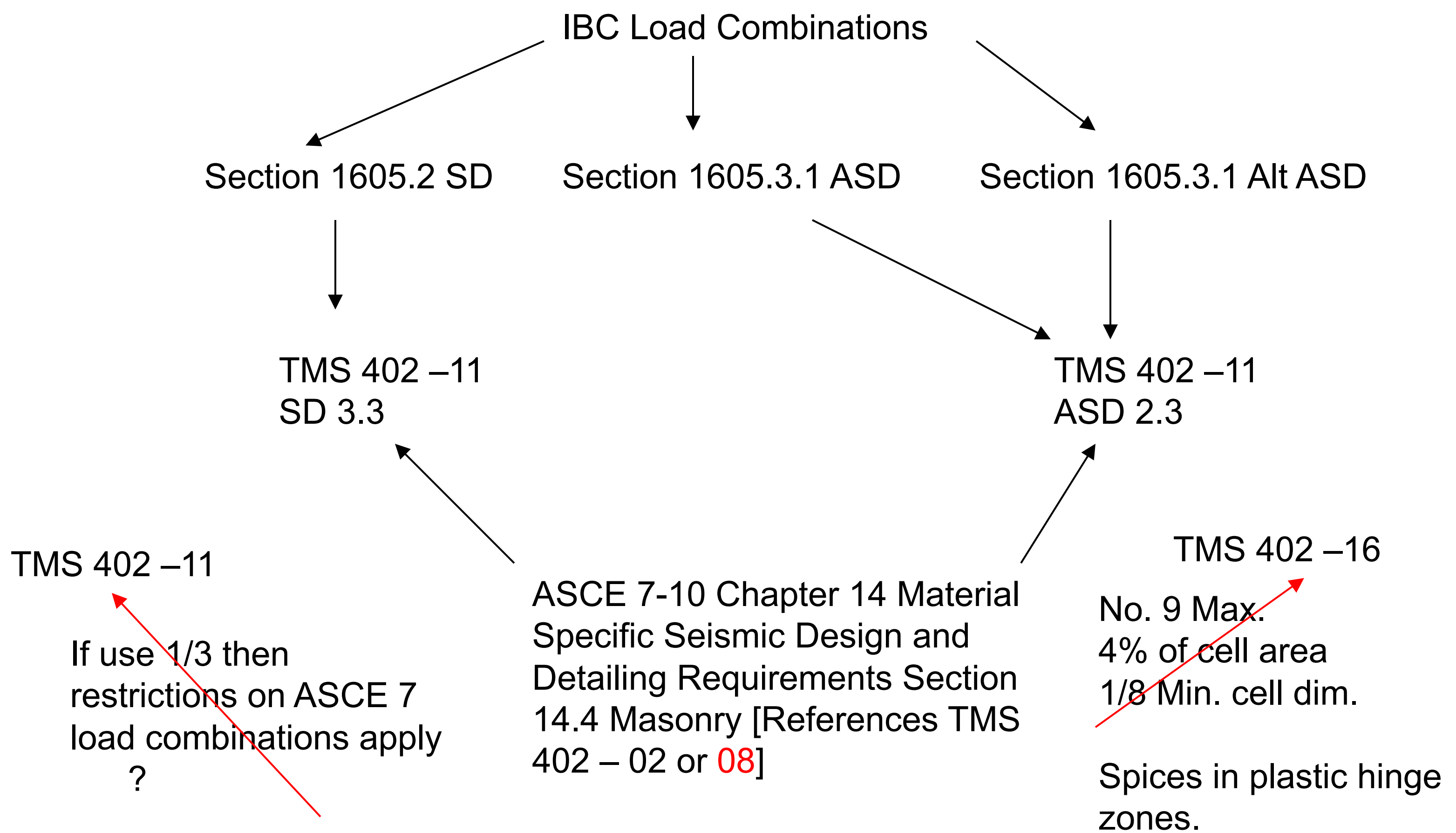
Example



The objective is to show the appropriate reinforcement in the wall. There are many correct answers. There are many different methods to analyze the wall.

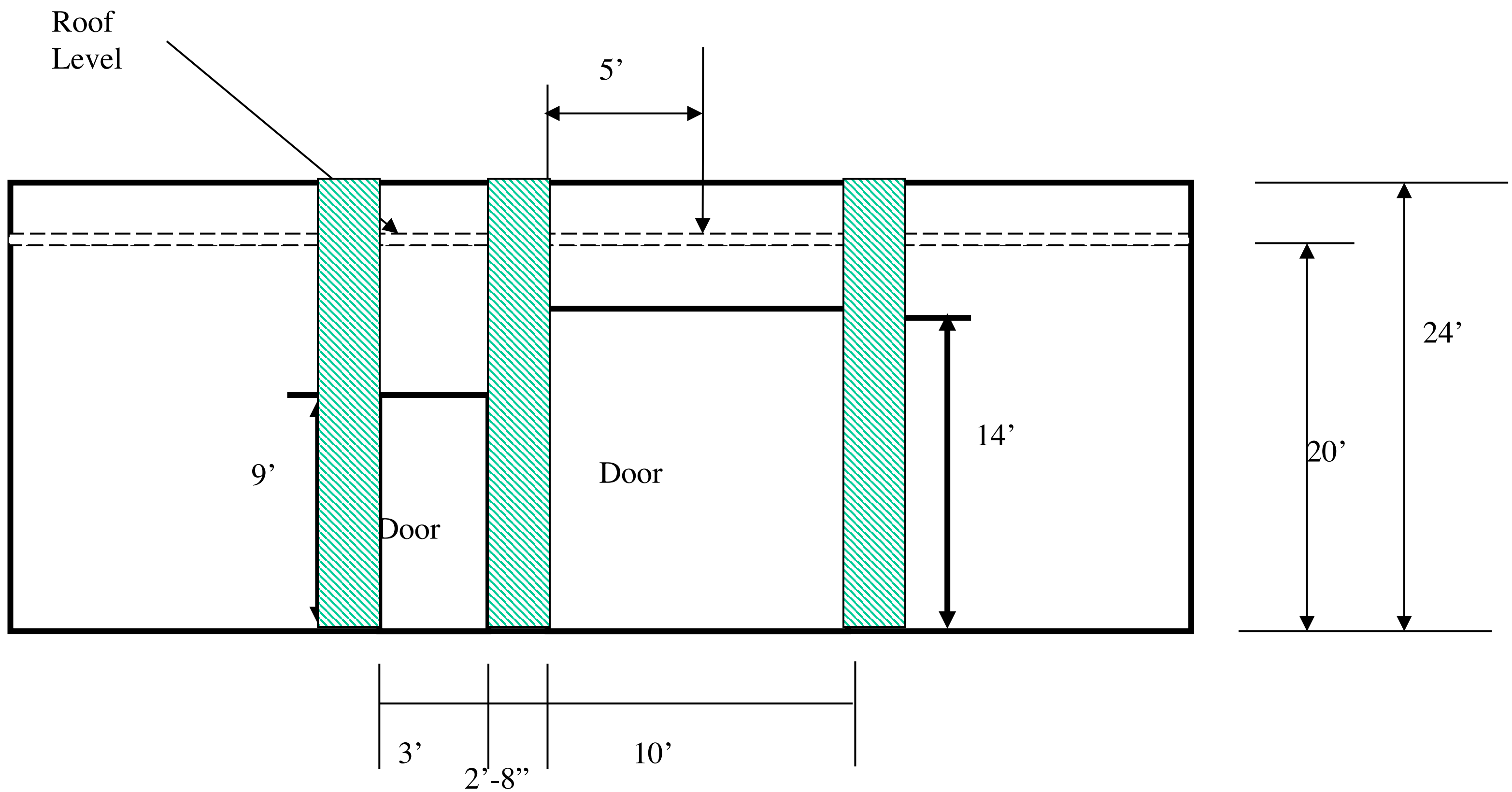
Example

Code 2012 IBC which includes TMS 402-11 [ACI530-11] and ASCE 7 -10



Example

Next we need to decide what element to design.



1.9.6 *Effective compressive width per bar*

1.9.6.1 For masonry not laid in running bond and having bond beams spaced not more than 48 in. (1219 mm) center-to-center, and for masonry laid in running bond, the width of the compression area used to calculate element capacity shall not exceed the least of:

(a) Center-to-center bar spacing.

(b) Six multiplied by the nominal wall thickness.

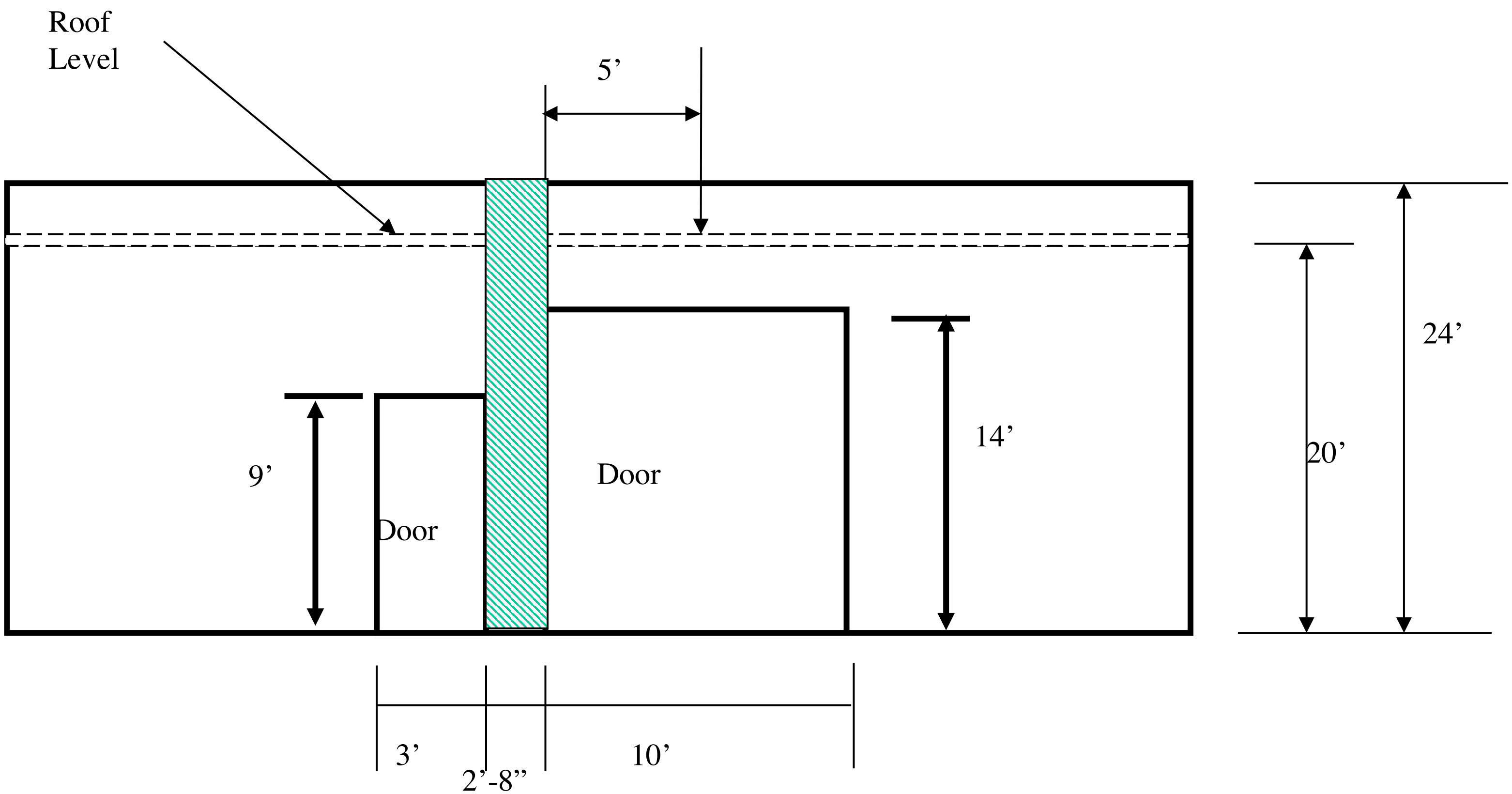
6x8 in = 48 in

(c) 72 in. (1829 mm).

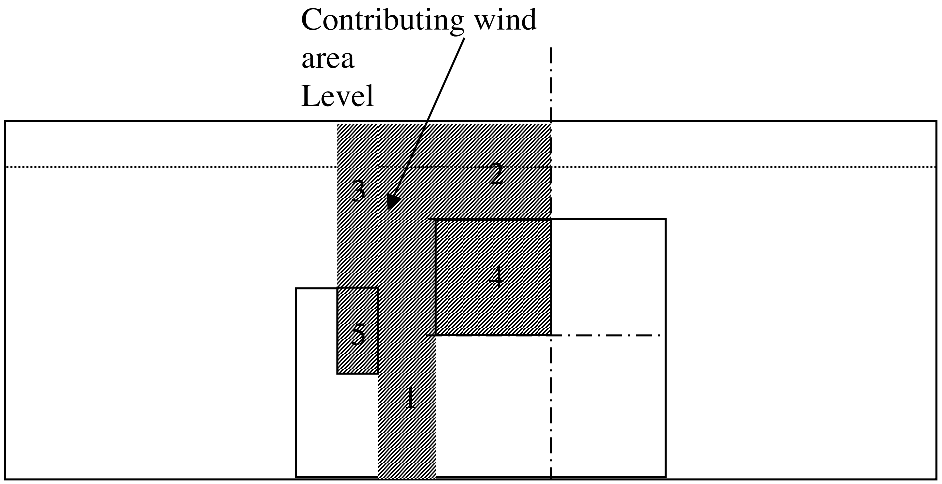
1.9.6.2 For masonry not laid in running bond and having bond beams spaced more than 48 in. (1219 mm) center-to-center, the width of the compression area used to calculate element capacity shall not exceed the length of the masonry unit.

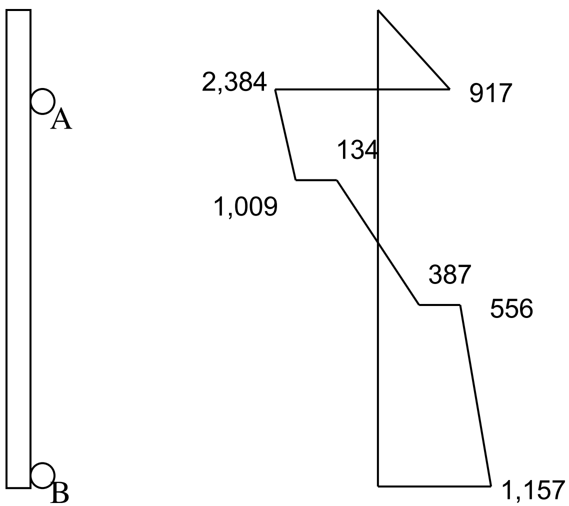
Example

If this element works the others work.



Example





Shear Diagram

$\Sigma M_B = 0$

$25*2.67*24^2/2 = 19,223$ lbs-ft
$25*5*10*(14+ 5) = 23,750$ lbs-ft
$25*1.5*15*(9+7.5)= 9,281$ lbs-ft
$25*7*5*14 = 12,250$ lbs-ft
$25*1.5*4.5*9 = 1,519$ lbs-ft
$\Sigma = 66,024$ Lb-ft and $R_A = 66024/20 = 3,300$ lbs

$\Sigma M_A = 0$
$25*2.67*20^2/2 – 53.4*4^2/2] = 12,815$ lb-ft
$25*5*6^2/2 – 100*4^2/2 = 1,250$ lb-ft
$25*1.5*11^2/2 – 30*4^2/2 = 1,968$ lb-ft
$25*7*5*6 = 5,250$ lb-ft
$25*1.5*4.5*11 =1,856$ lb-ft
$\Sigma = 23,140$ lb-ft and $R_A = 23,240/20 = 1,160$ lbs

$\Sigma F = 25*2.67*24+25*5*10 + 25*1.5*15 +25*7*5 + 25*1.5*4.5 – 3,300 – 1,160 = 0$ OK

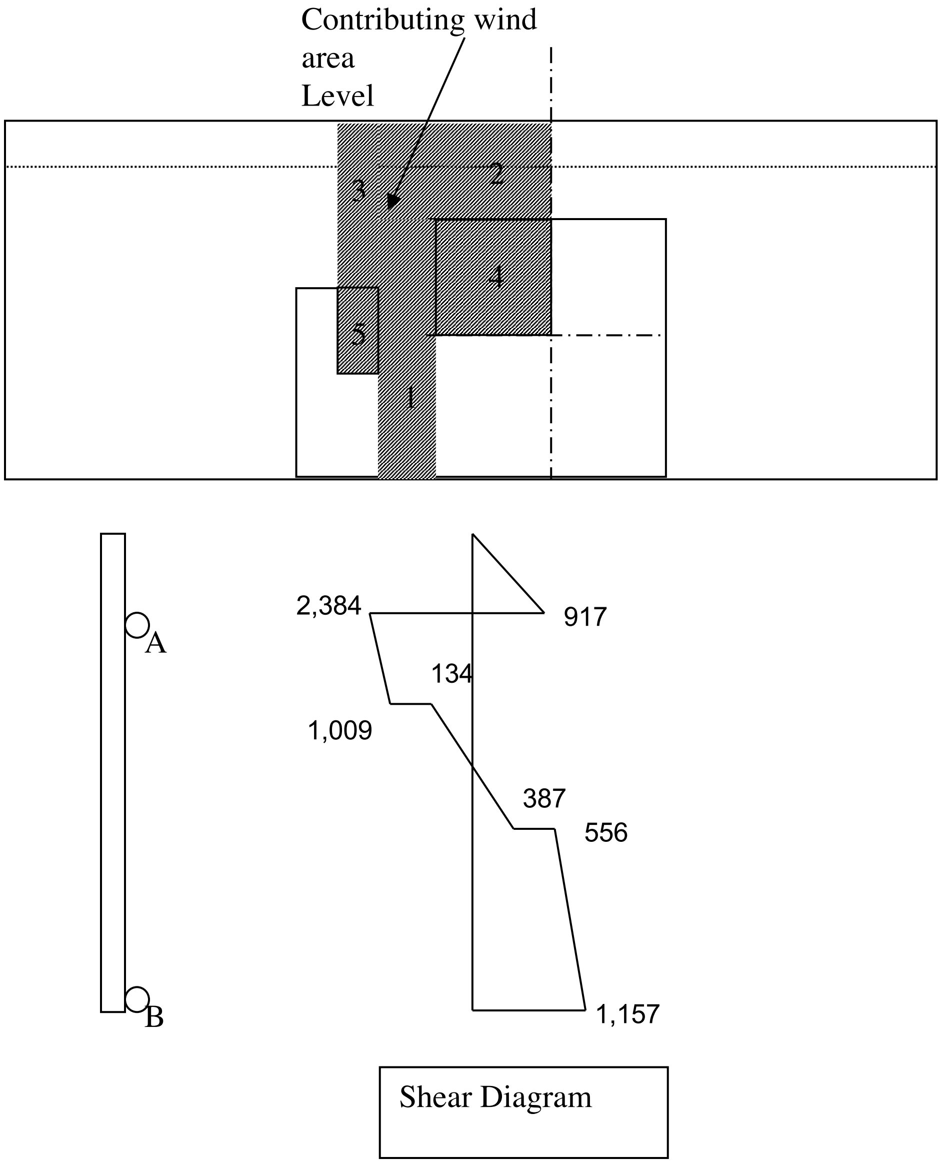
Max Moment:

X = 12.72 ft (see shear diagram)

$M_{max} = 387*3.72/2+556*9+(1,157-556)*9/2$

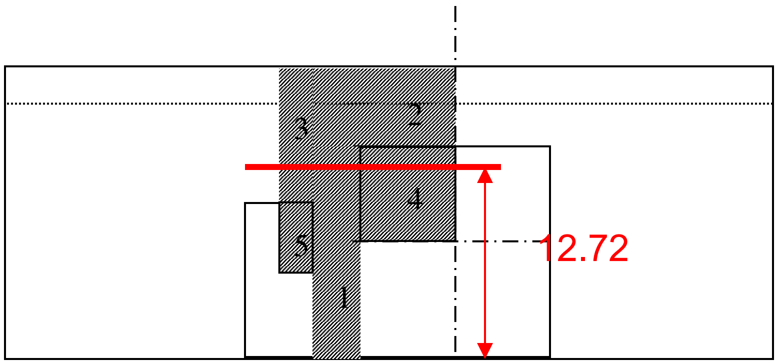
= 8,430 lb-ft

Example



The maximum moment occurs above the 3 foot door, so the width for design is greater than the 2’-8”. We need to check the minimum section at 2’-8” and calculate the moment at the top of the door ie check the two cases, or check the 2’-8” width for the maximum moment and if it works it is OK. It is conservative

Example



Axial load:						
	Dead Load					
		Roof Beam	12/2 = 6 k			
		Wall Weight				
		(1.5+2.67)*(24-12.72)*80 = 3.76 k				
			5*10*80 = 4.0 k			
			Total		13.8 k	
	Live Load					
			7/2		3.5 k	
	Lateral Load:					
		Wind Flexure			8,430 Lb ft	

Example

First we need the load combinations. Lets start with the ASD load combinations form the 2012 IBC [ASCE 7-10 does not apply]. The following provision applies:

$D + F$ **(Equation 16-8)**
$D + H + F + L$ **(Equation 16-9)**
$D + H + F + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-10)**
$D + H + F + 0.75(L) + 0.75(L_r \text{ or } S \text{ or } R)$ **(Equation 16-11)**
$D + H + F + (0.6W \text{ or } 0.7E)$ **(Equation 16-12)**
$D + H + F + 0.75(0.6W) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$ **(Equation 16-13)**
$D + H + F + 0.75(0.7E) + 0.75L + 0.75S$ **(Equation 16-14)**
$0.6D + 0.6W + H$ **(Equation 16-15)**
$0.6(D + F) + 0.7E + H$ **(Equation 16-16)**

Maximum and Minimum Axial Load Control

16-13 and 13-15

Example

Try minimum first: 16-15

P = .6*D = .6*13.8 = 8.3 k

M = .6*8.430 = 5.1 k-ft

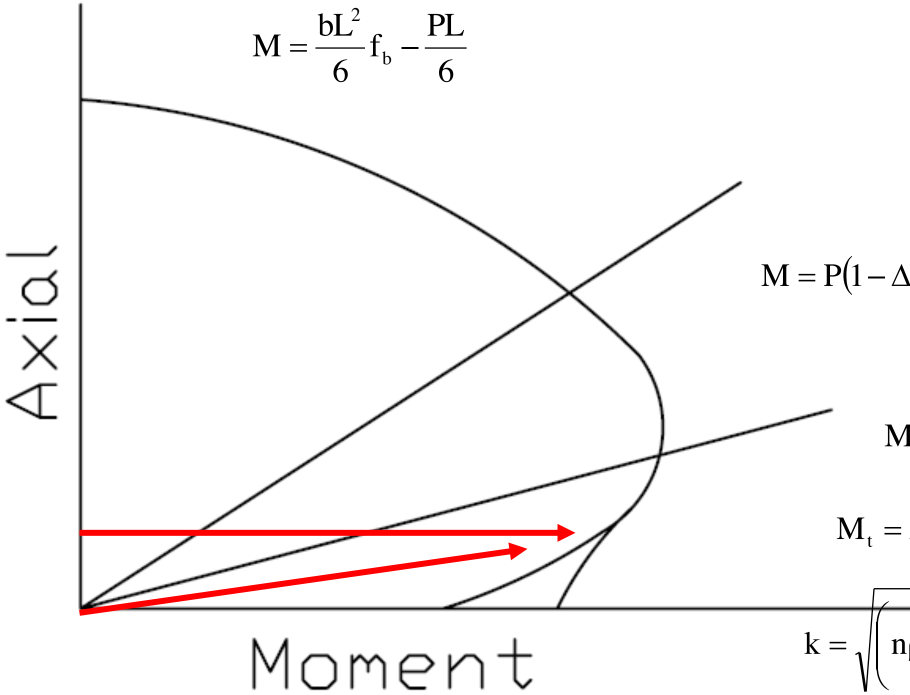
d = 3.8125 in

M/Pd = 5.1*12/[8.3*3.8125] = 1.9 > 2/3-Δ

b = 32 in

Assume the axial load P is independent from the moment. Hold P constant and find the moment at the interaction diagram. Compare the moment to the applied moment.

Example



$$M = \frac{bL^2}{6} f_b - \frac{PL}{6}$$

$$M = P(1-\Delta)d - \frac{2}{3}\frac{P^2}{F_b b}$$

$$M_c = \frac{F_b k j b d^2}{2} - P\Delta d$$

$$M_t = A_s F_s jd + P\left(1 - \frac{k}{3} - \Delta\right)d$$

$$k = \sqrt{\left(n\rho + \frac{\varepsilon_{m0}}{\varepsilon_s}\right)^2 + 2\left(n\rho + \frac{\varepsilon_{m0}}{\varepsilon_s}\right)} - \left(n\rho + \frac{\varepsilon_{m0}}{\varepsilon_s}\right)$$

$$\frac{\varepsilon_{m0}}{\varepsilon_s} = \frac{n\rho\left(1 - \frac{k}{3}\right)}{\left(\frac{M}{Pd} - \left(1 - \frac{k}{3} - \Delta\right)\right)}$$

Example

Guess 2 No.5 bars at the jamb.
Find K

$$k = \sqrt{\left(n\rho + \frac{\varepsilon_{m0}}{\varepsilon_s}\right)^2 + 2\left(n\rho + \frac{\varepsilon_{m0}}{\varepsilon_s}\right)} - \left(n\rho + \frac{\varepsilon_{m0}}{\varepsilon_s}\right) \qquad \frac{\varepsilon_{m0}}{\varepsilon_s} = \frac{n\rho\left(1 - \frac{k}{3}\right)}{\left(\frac{M}{Pd} - \left(1 - \frac{k}{3} - \Delta\right)\right)}$$

2 No. 5 bars A_s = .62 in^2
ρ = .62/(32*3.8125) = .0051
n = 29,000,000/(900*1500) = 21.5
nρ = .11
Assume k = .3

$$\frac{\varepsilon_{mo}}{\varepsilon_s} = \frac{.11(1 - .3/3)}{(1.92 - (1 - .3/3))} = .097 \qquad k = \sqrt{(.11 + .097)^2 + 2(.11 + .097)} - (.11 + .097) = .46$$

$$\frac{\varepsilon_{mo}}{\varepsilon_s} = \frac{.11(1 - .46/3)}{(1.92 - (1 - .46/3))} = ..086 \qquad k = \sqrt{(.11 + .086)^2 + 2(.11 + .086)} - (.11 + .086) = .46$$

$$M_t = .62 * 32{,}000 * \left(1 - \frac{.46}{3}\right) 3.8125/12 + 8{,}300 * \left(1 - \frac{.46}{3}\right) * 3.8125/12 = 7{,}600\ \text{lb} - \text{ft}$$

$$M_c = \left(\frac{675 * .46 * (1 - .46/3) * 32 * 3.8125^2}{2} - 0\right) / 12 = 5{,}100\ \text{lb} - \text{ft}$$

OK

At the critical section b is larger.

b = 32 – 2*4 +2*3*8 = 72 in

Example

Try maximum: 16-13

$P = D + .75*L_r = 13.8 + .75*3.5 = 16.4$ k

$M = .75*.6*8.430 = 3.8$ k-ft

$d = 3.8125$ in

$M/Pd = 3.8*12/[16.4*3.8125] = .73 > 2/3-\Delta$

Section slightly cracked at the reinforcement

At top of door

$P = D + .75*L_r = 15.0 + .75*3.5 = 17.6$ k

$M = .75*.6*7.703 = 3.5$ k-ft

$M/Pd = 3.5*12/[17.6*3.8125] = .62 < 2/3-\Delta$

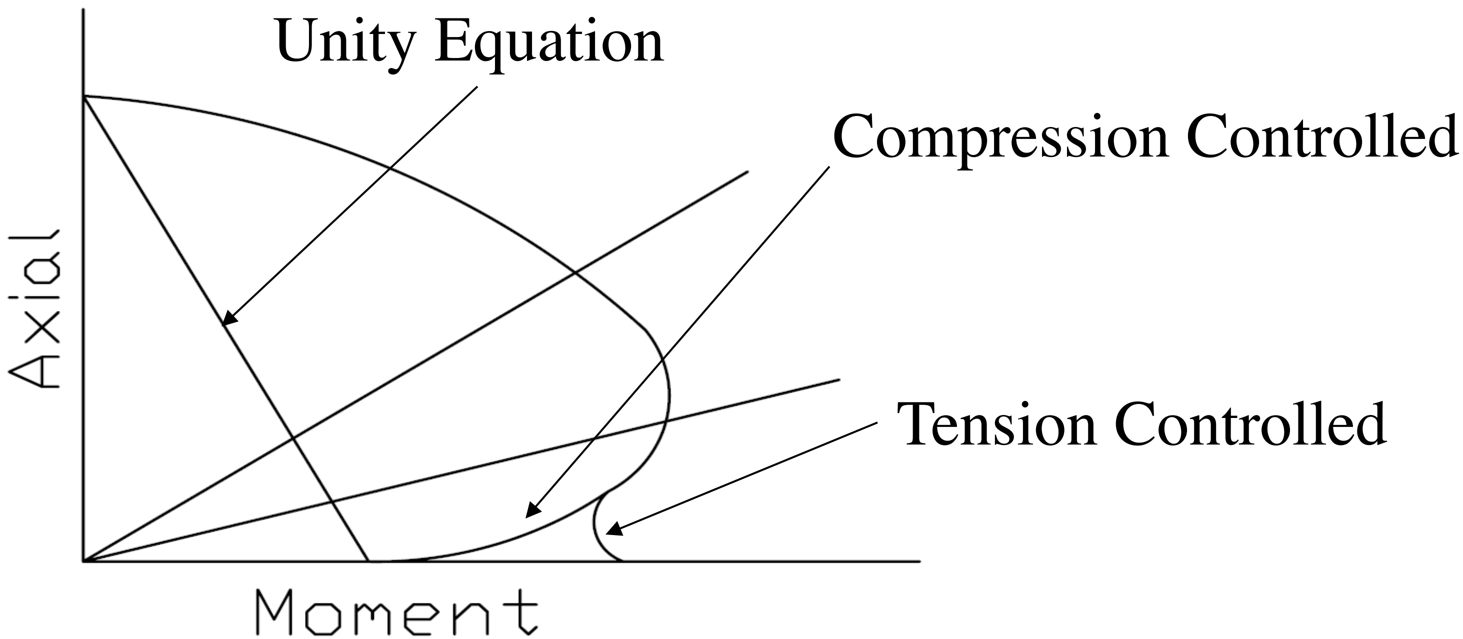
Section not cracked at the reinforcement

Example

M/Pd is close to 2/3 and therefore K is close to 1.0. The system does not converge rapidly. Try the unity equation.

Unity equation is no longer in the code, but it is acceptable to use it.

$$\frac{P}{P_a}\Big|_{M=0} + \frac{M}{M_a}\Big|_{P=0} \leq 1.0$$



Example

$$P_a = \left(.25f'_m A_n + .65A_{st}F_s\right)\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \frac{h}{r} \le 99$$

$$P_a = \left(.25f'_m A_n + .65A_{st}F_s\right)\left[\frac{70r}{h}\right]^2 \quad \frac{h}{r} \succ 99$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{1}{12}32*7.625^3}{32*7.625}} = 2.2 \quad \frac{h}{r} = \frac{240}{2.2} = 109$$

$$P_a = \left(.25*1500*7.625*32+0\right)\left[\frac{70*2.2}{240}\right]^2 = 37{,}600\ \text{lb}$$

$$P_a = \left(.25*1500*7.625*32+0\right)\left[1-\left(\frac{240}{140*2.2}\right)^2\right] = 35{,}900\ \text{lb}$$

Example

$$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{900 * 1500} = 21.5 \quad \rho = \frac{A_s}{bd} = \frac{.62}{32 * 3.8125} = .0051$$

$$n\rho = 21.5 * .0051 = .11$$

$$k = \sqrt{(n\rho)^2 + 2(n\rho)} - n\rho = \sqrt{(.11)^2 + 2(.11)} - .11 = .37$$

$$j = \left(1 - -\frac{k}{3}\right) = \left(1 - -\frac{.37}{3}\right) = .88$$

$$M_t = A_s jF_s d = .62 * .88 * 32{,}000 * 3.8125 / 12{,}000 = 5.54\ k - ft$$

$$M_c = \frac{bd^2}{2} kjF_b = \frac{32 * 3.8125^2}{2} .37 * .88 * .45 * 1500 / 12{,}000 = 4.26\ k - ft$$

Example

$$\frac{P}{P_a}\Big|_{M=0} + \frac{M}{M_a}\Big|_{P=0} \leq 1.0 = \frac{16.4}{37.6} + \frac{3.79}{4.26} = .44 + .88 = 1.32$$

Refine analysis:

1. **Refine axial load**
2. **Add (2) No. 5**
3. **Refine conservative moment. At top of door = 7,710 Lb-ft**

$$\frac{P}{P_a}\Big|_{M=0} + \frac{M}{M_a}\Big|_{P=0} \leq 1.0 = \frac{17.6}{37.6} + \frac{3.46}{5.25} = .47 + .66 = 1.13$$

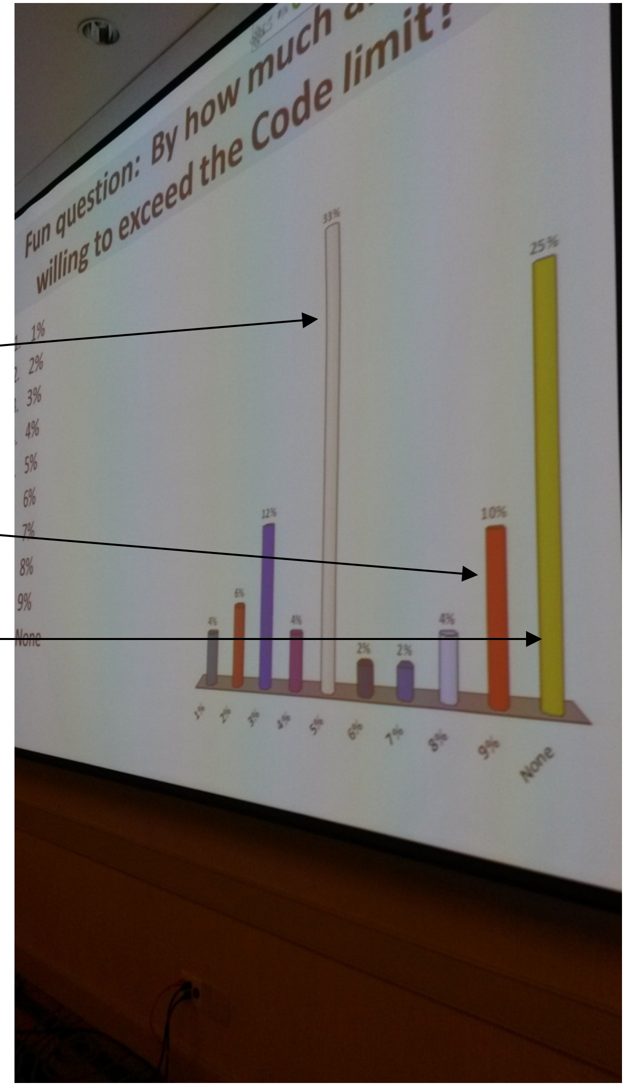
Example

By how much is the engineer will to exceed the Code limit?

5% = 33%

9% = 10%

None = 25%

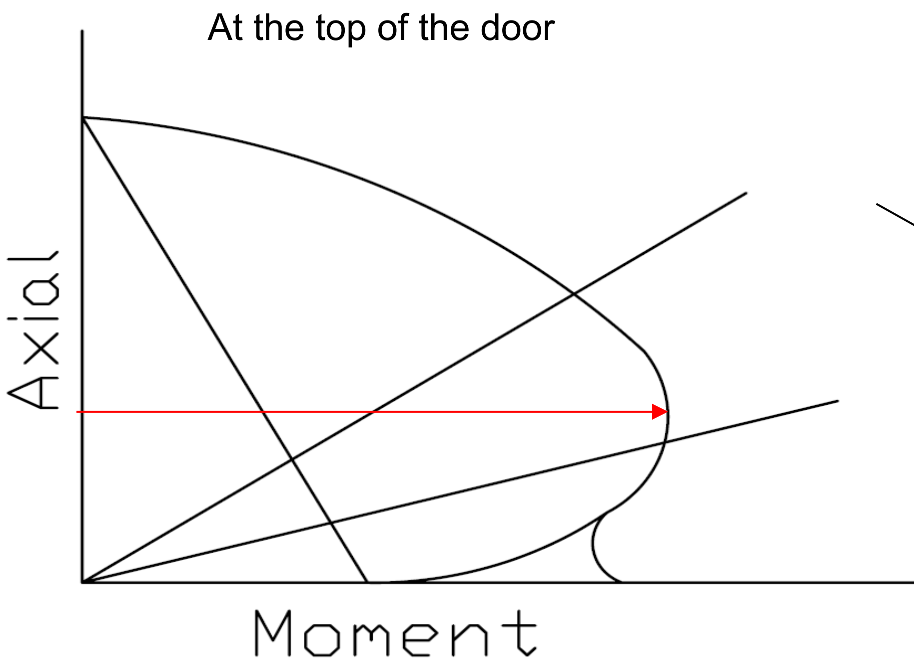


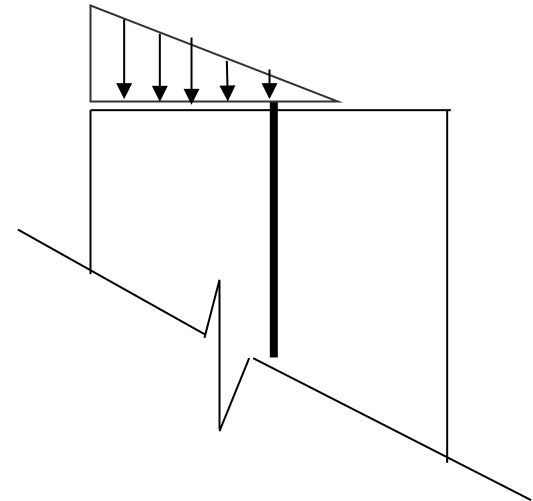
At maximum moment, iterative equations with (2) No. 5 result in k = .886

M_c = 8.17 k-ft

M_t= 8.12 k-ft

Example





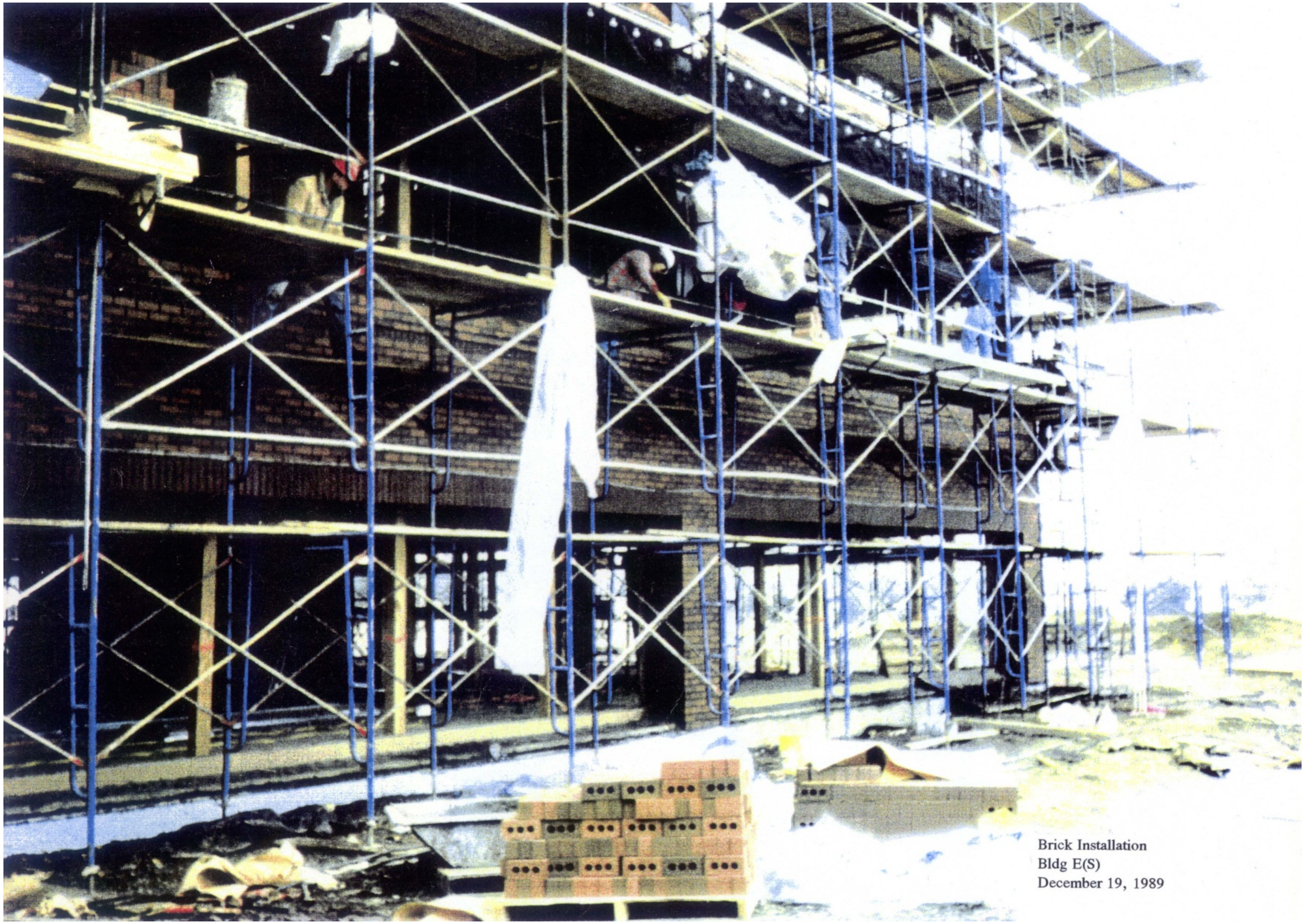
$$P = \frac{F_b kbd}{2}$$

$$k = \frac{2P}{F_b bd}$$

$$M = P(1-\Delta)d - \frac{2}{3}\frac{P^2}{F_b b}$$

$$M = 17{,}600(1-0)*3.8125 - \frac{2}{3}\frac{17{,}600^2}{*675*32} = 4.79\ \text{k}-\text{ft}$$

OK



Brick Installation
Bldg E(S)
December 19, 1989

Example

Try maximum: 16-12

P = D = 13.8 k

M = .6*8.430 = 5.05 k-ft

d = 3.8125 in

M/Pd = 5.05*12/[13.8*3.8125] = 1.15 > 2/3-Δ

M_t = 8.7 k-ft

M_c = 6.0 k-ft

OK

Example

The SD load combinations form the 2012 IBC [ASCE 7-10 does not apply]. The following provision applies:

$1.4(D+F)$ **(Equation 16-1)**
$1.2(D+F)+1.6(L+H)+0.5(L_r \text{ or } S \text{ or } R)$ **(Equation 16-2)**
$1.2(D+F)+1.6(L_r \text{ or } S \text{ or } R)+1.6H+(f_1L \text{ or } 0.5W)$ **(Equation 16-3)**
$1.2(D+F)+1.0W+f_1L+1.6H+0.5(L_r \text{ or } S \text{ or } R)$ **(Equation 16-4)**
$1.2(D+F)+1.0E+f_1L+1.6H+f_2S$ **(Equation 16-5)**
$0.9D+1.0W+1.6H$ **(Equation 16-6)**
$0.9(D+F)+1.0E+1.6H$ **(Equation 16-7)**

Maximum and Minimum Axial Load Control

16-4 and 13-6

Example

Try minimum first: 16-4

P = .9*D = .9*13.8 = 12.4 k

M = 1.0*8.430 = 8.43 k-ft

d = 3.8125 in

M/Pd = 8.43*12/[12.4*3.8125] = 2.1 > 1-k/2-Δ

b = 32 in

Example

2 No. 5 bars A_s = .62 in^2

Limit on reinforcement to be above yield

$$k = \frac{\varepsilon_0}{\left(\varepsilon_0 + \frac{F_y}{E_s}\right)}$$

ε_0 = .0025 CMU

ε_0 = .0035 Brick

$2k_2$=.8

K_1k_3 = .8

$$k = \frac{.0025}{\left(.0025 + \frac{60{,}000}{29{,}000{,}000}\right)} = .547$$

Example

2 No. 5 bars A_s = .62 in^2

Limit on reinforcement to be above yield

$$\frac{M}{Pd} = \frac{\left(1 - \frac{2k_2k}{2}\right)}{\left(1 - \frac{A_sF_y}{k_1k_3 2k_2kbdf'_m}\right)} - \Delta$$

$$\frac{M}{Pd} = \frac{\left(1 - \frac{2*.4*.547}{2}\right)}{\left(1 - \frac{.62*60{,}000}{.8*2*.4*.547*32*3.8125*1500}\right)} - 0 = 1.86$$

M/Pd > 1.86 so steel is at yield

Example

Calculate Moment Capacity

$$k = \frac{(A_s F_y + P)}{2k_2 k_1 k_3 b d f'_m} = \frac{(.62 * 60{,}000 + 12{,}400)}{.8 * .8 * 32 * 3.8125 * 1500} = .423$$

$$M = A_s F_y \left(1 - \frac{2k_2 k}{2}\right) d + P \left(1 - \frac{2k_2 k}{2} - \Delta\right) d$$

$$M = .62 * 60{,}000 \left(1 - \frac{.8 * .423}{2}\right) 3.8125 + 12{,}400 \left(1 - \frac{.8 * .423}{2} - 0\right) 3.8125$$

$$M = 117{,}800 + 39300 = 157{,}000 \text{ Lb} - \text{in} = 13.1 \text{ k} - \text{ft}$$

M_u= 11.8 k-ft

OK – Bigger margin than ASD

Example

Try maximum: 16-4

$P = 1.2*D + .75*L_r = 1.2*13.8 + .5*3.5 = 18.3$ k

M = 1.0*8.430 = 8,430 k-ft

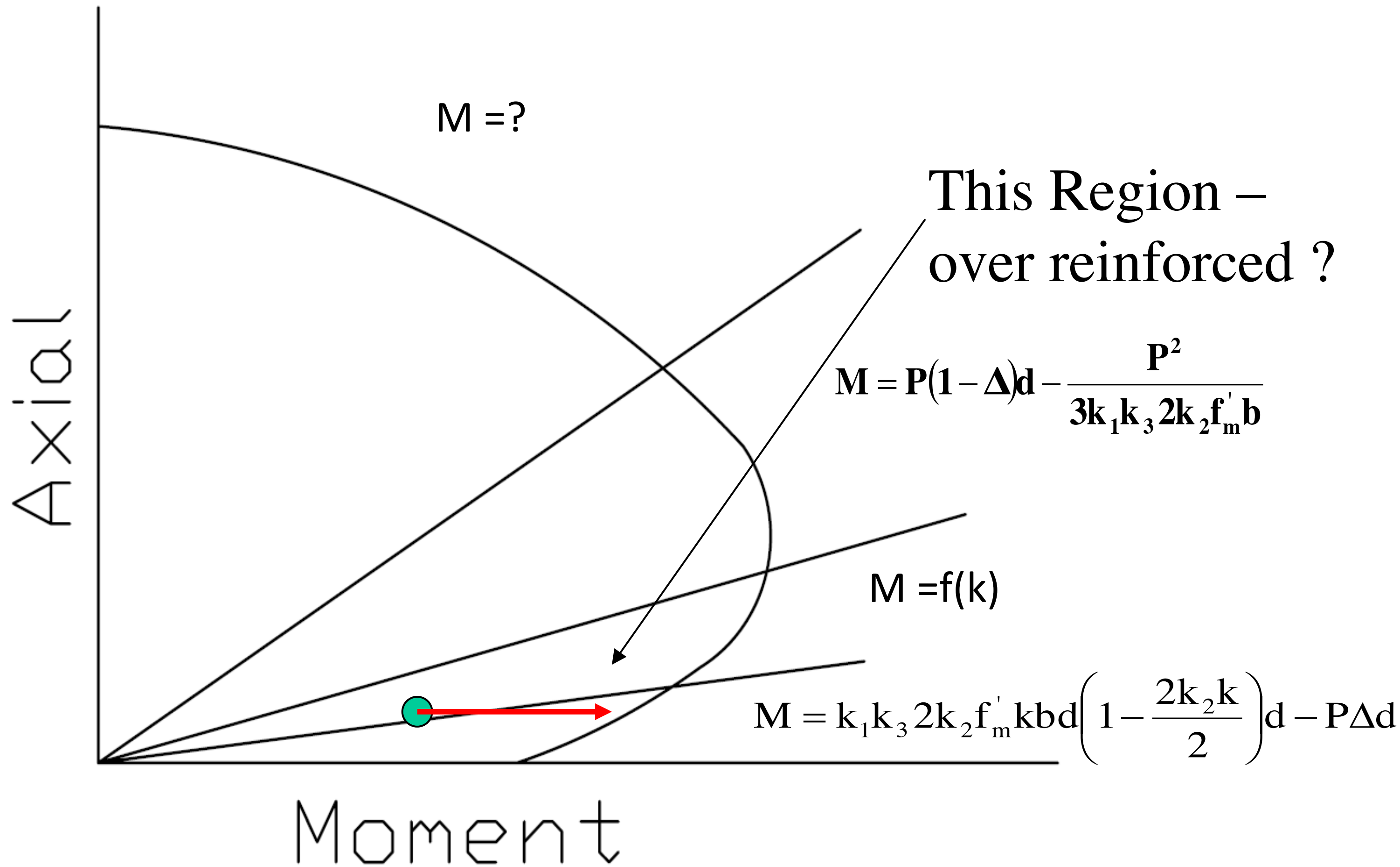
d = 3.8125 in

M/Pd = 8,430*12/[18.3*3.8125] = 1.44

Less than 1.86 so steel is in tension but is stresses lower than yield.

This is an indication it is over reinforced

Example


M =?
This Region – over reinforced ?
$\mathbf{M = P(1 - \Delta)d - \frac{P^2}{3k_1 k_3 2k_2 f'_m b}}$
M =f(k)
$M = k_1 k_3 2k_2 f'_m kbd\left(1 - \frac{2k_2 k}{2}\right)d - P\Delta d$
Axial
Moment

Example

$$k = \frac{(A_s F_y + P)}{2k_2 k_1 k_3 b d f'_m} = \frac{(.62 * 60{,}000 + 18{,}300)}{.8 * .8 * 32 * 3.8125 * 1500} = .474$$

$$M = A_s F_y \left(1 - \frac{2k_2 k}{2}\right) d + P \left(1 - \frac{2k_2 k}{2} - \Delta\right) d$$

$$M = .62 * 60{,}000 \left(1 - \frac{.8 * .474}{2}\right) 3.8125 + 18{,}300 \left(1 - \frac{.8 * .474}{2} - 0\right) 3.8125$$

$$M = 114{,}900 + 56{,}500 = 171{,}400 \text{ Lb} - \text{in} = 14.2 \text{ k} - \text{ft}$$

M_u= 12.8 k-ft

OK – Bigger margin than ASD

Example

$$\rho_{max} = \frac{A_S}{bd} = \frac{.64f'_m\left(\frac{\varepsilon_0}{\varepsilon_0 + \alpha\varepsilon_y}\right) - \frac{P}{bd}}{f_y}$$

TMS 402 Code 3.3.3.5

$$\rho_{max} = \frac{A_S}{bd} = \frac{.64 * 1500\left(\frac{.0025}{.0025 + 1.5 * .00207}\right) - \frac{18,300}{32 * 3.8125}}{60,000} = .00464$$

$$\rho = \frac{.62}{32 * 3.8125} = .00508 > .00464$$

Does not meet requirements, but the code does not use the actual axial load.

$$P = D + ,75L + .525Q_E = 13.8k$$

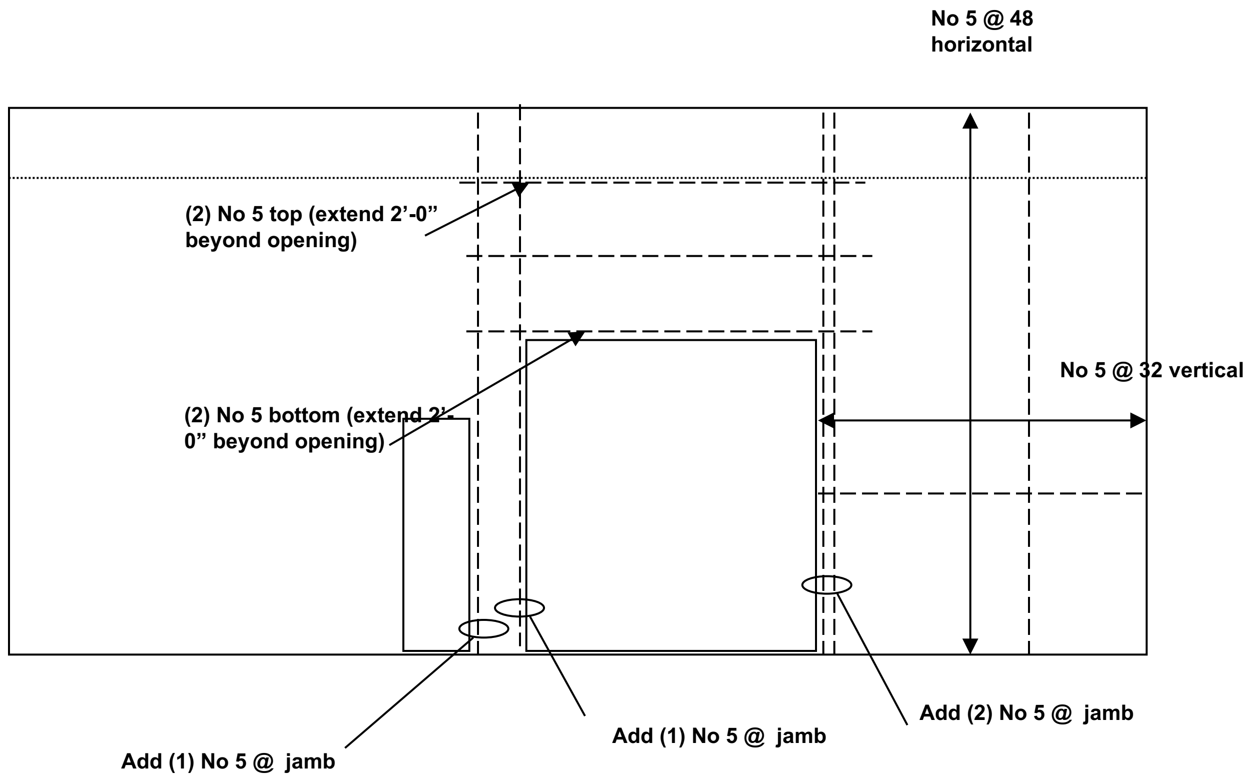
TMS 402 Code 3.3.3.5

$$\rho_{max} = \frac{A_S}{bd} = \frac{.64 * 1500\left(\frac{.0025}{.0025 + 1.5 * .00207}\right) - \frac{13,800}{32 * 3.8125}}{60,000} = .00525$$

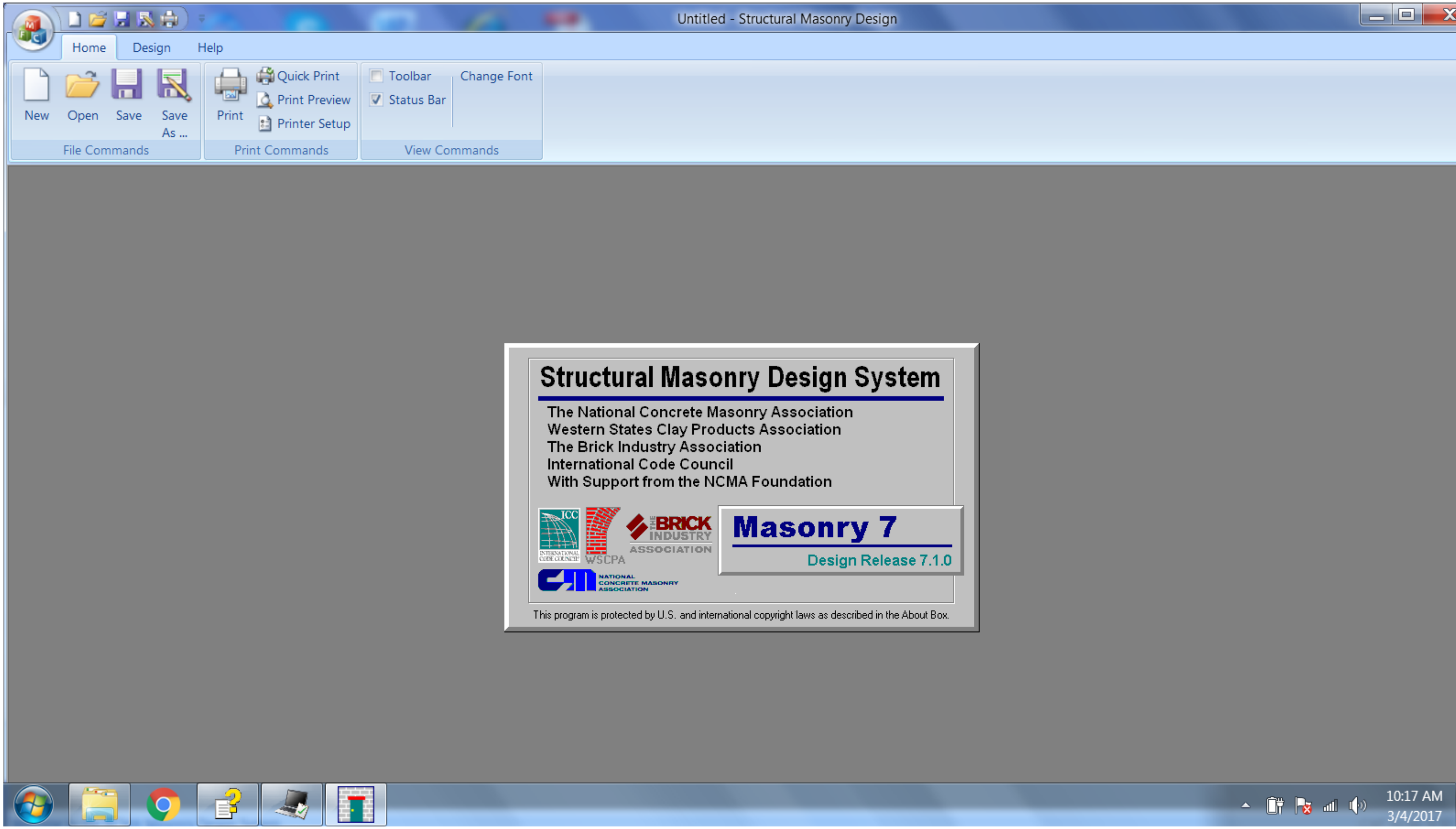
OK

Example

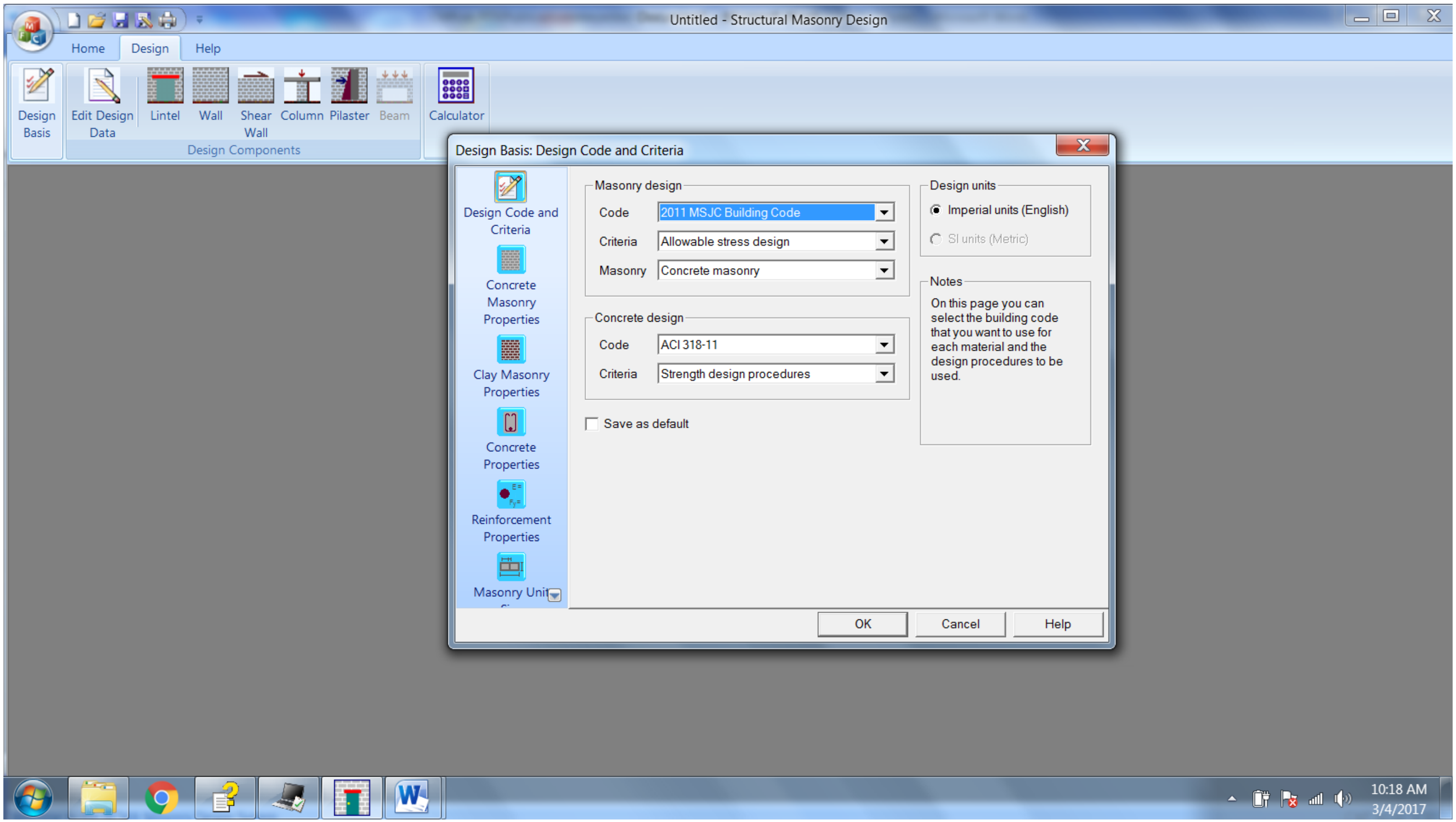
TMS 402 ASD Section 2.3.4.4 Maximum reinforcement does not apply to out-of-plane loads.



Example

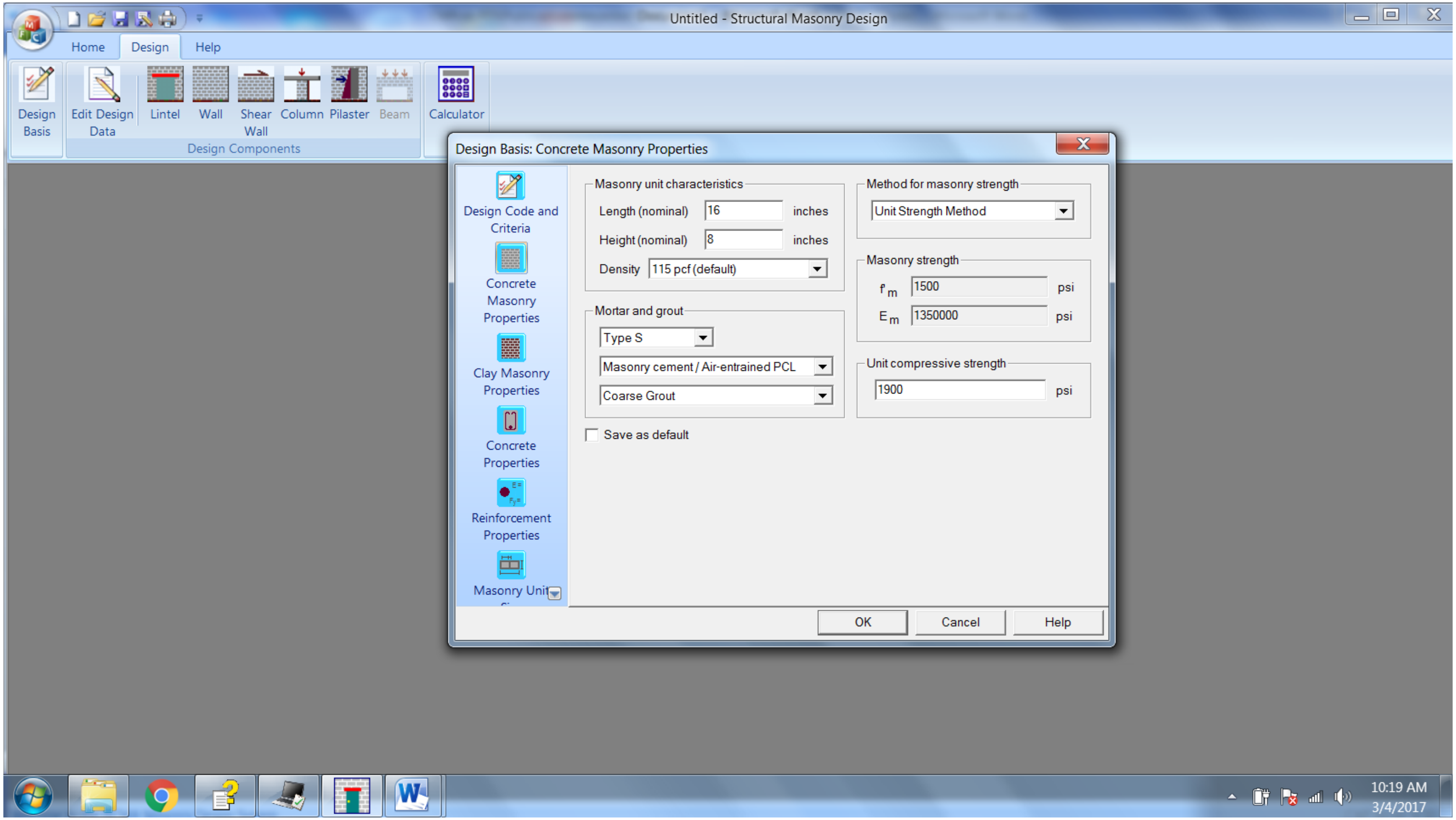

Untitled - Structural Masonry Design
Home
Design
Help
New
Open
Save
Save As ...
File Commands
Print
Quick Print
Print Preview
Printer Setup
Print Commands
Toolbar
Status Bar
Change Font
View Commands
Structural Masonry Design System
The National Concrete Masonry Association
Western States Clay Products Association
The Brick Industry Association
International Code Council
With Support from the NCMA Foundation
Masonry 7
Design Release 7.1.0
This program is protected by U.S. and international copyright laws as described in the About Box.
10:17 AM
3/4/2017

Example



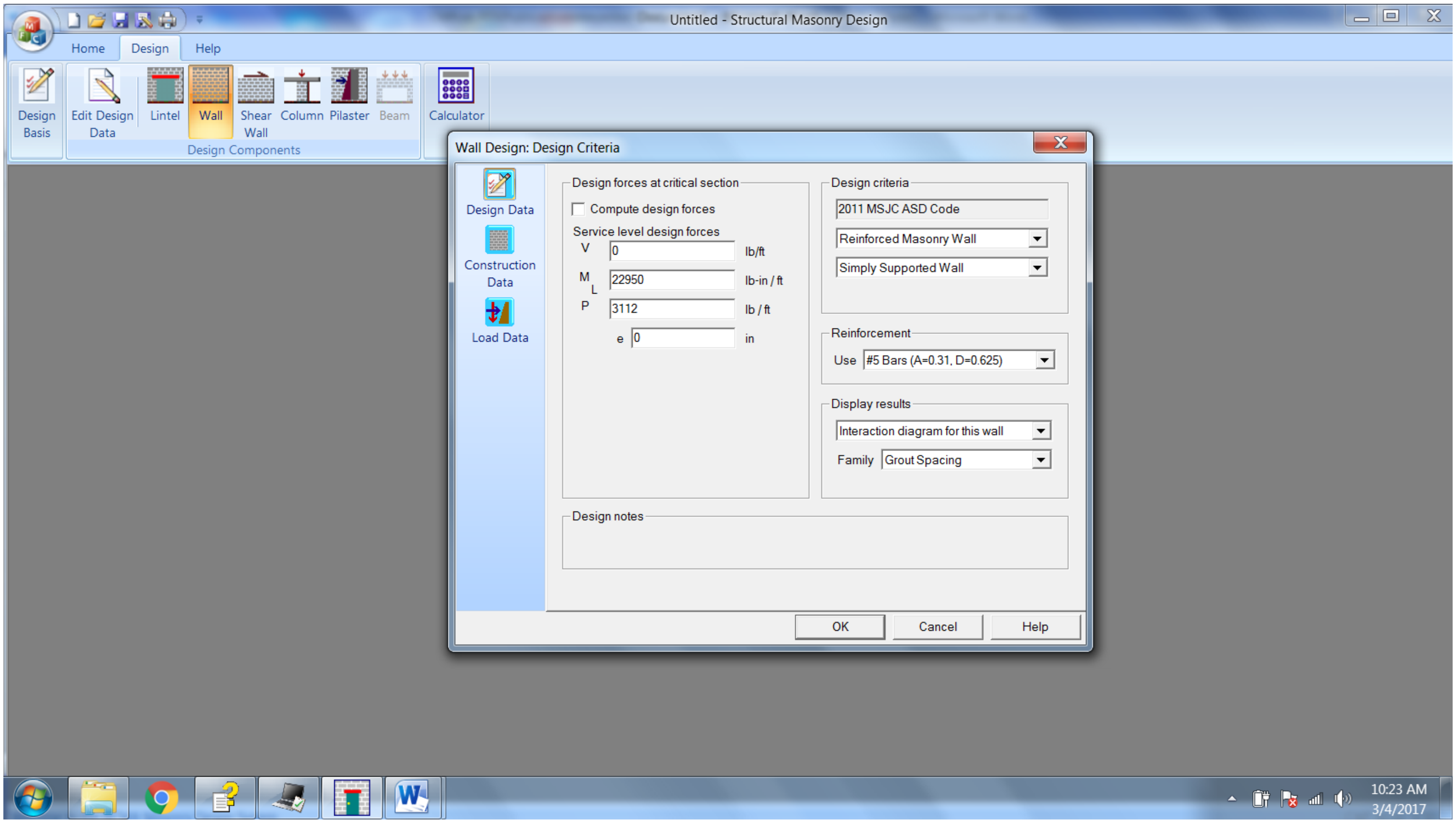
Untitled - Structural Masonry Design
Home
Design
Help
Design Basis
Edit Design Data
Lintel
Wall
Shear Wall
Column
Pilaster
Beam
Calculator
Design Components
Design Basis: Design Code and Criteria
Design Code and Criteria
Concrete Masonry Properties
Clay Masonry Properties
Concrete Properties
Reinforcement Properties
Masonry Unit
Masonry design
Code
2011 MSJC Building Code
Criteria
Allowable stress design
Masonry
Concrete masonry
Concrete design
Code
ACI 318-11
Criteria
Strength design procedures
Save as default
Design units
Imperial units (English)
SI units (Metric)
Notes
On this page you can select the building code that you want to use for each material and the design procedures to be used.
OK
Cancel
Help
10:18 AM
3/4/2017

Example

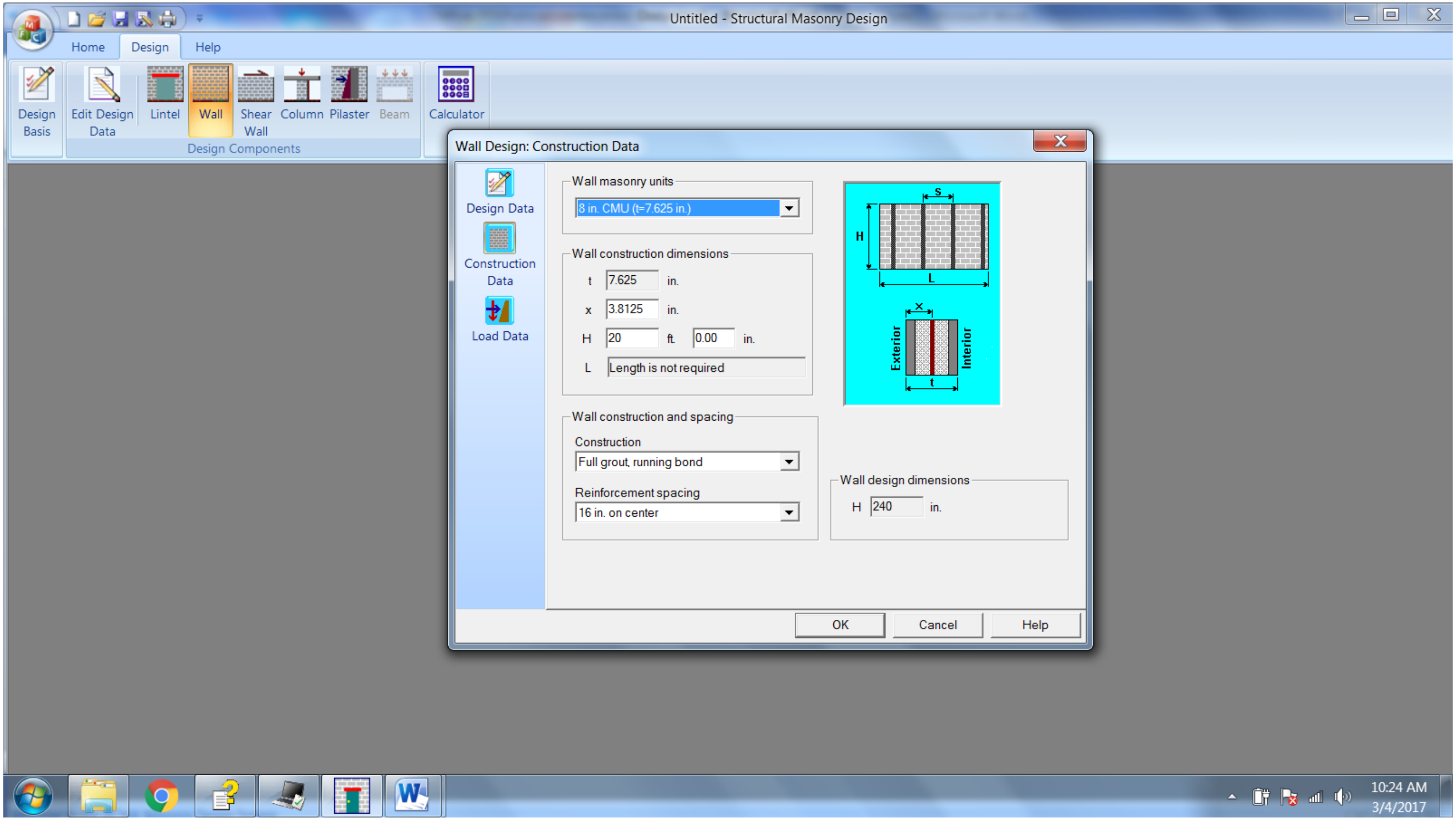


Untitled - Structural Masonry Design
Home
Design
Help
Design Basis
Edit Design Data
Lintel
Wall
Shear Wall
Column
Pilaster
Beam
Calculator
Design Components
Design Basis: Concrete Masonry Properties
Design Code and Criteria
Concrete Masonry Properties
Clay Masonry Properties
Concrete Properties
Reinforcement Properties
Masonry Unit
Masonry unit characteristics
Length (nominal) 16 inches
Height (nominal) 8 inches
Density 115 pcf (default)
Mortar and grout
Type S
Masonry cement / Air-entrained PCL
Coarse Grout
Save as default
Method for masonry strength
Unit Strength Method
Masonry strength
f'm 1500 psi
Em 1350000 psi
Unit compressive strength
1900 psi
OK
Cancel
Help
10:19 AM
3/4/2017

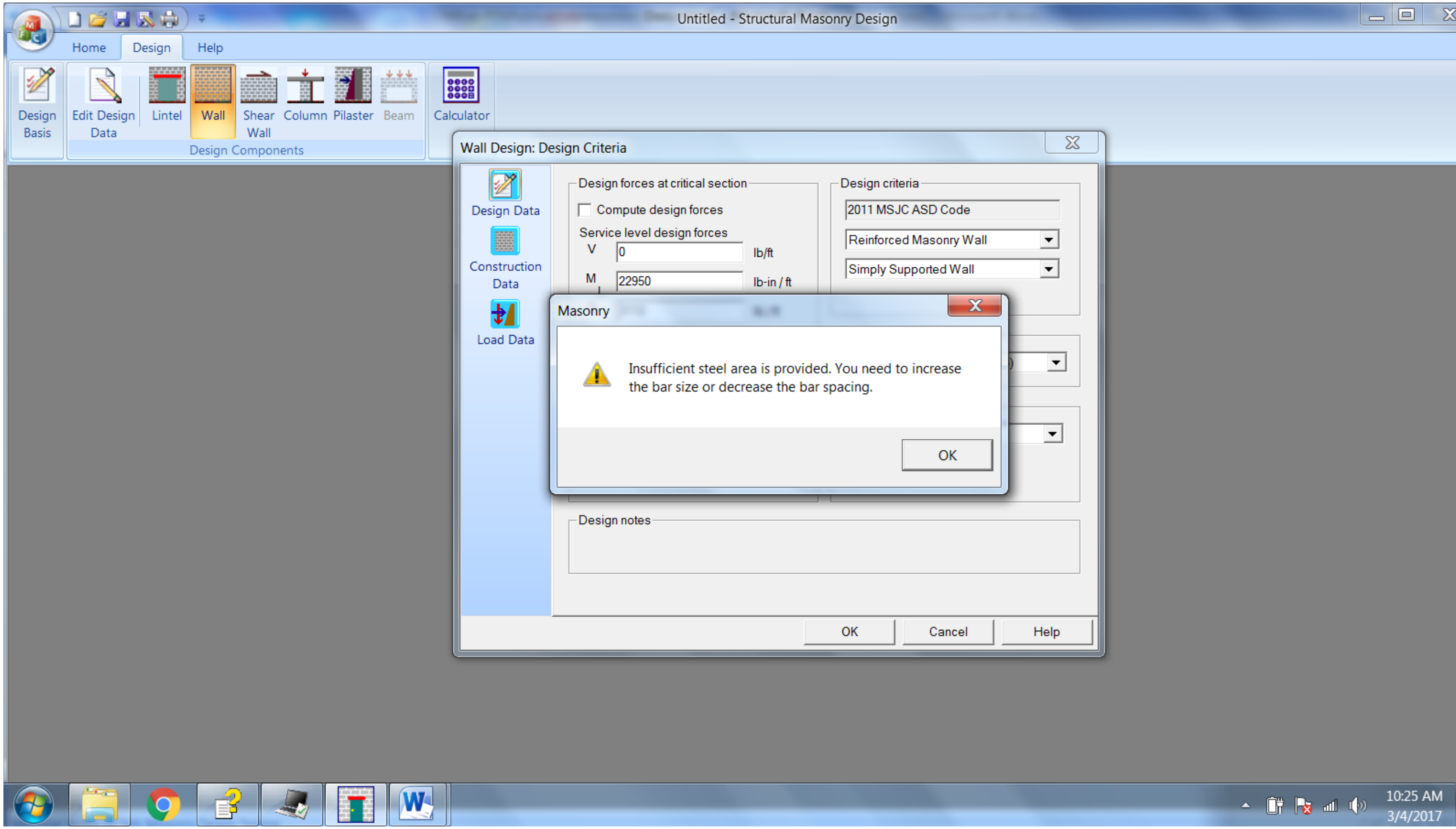
Example


Untitled - Structural Masonry Design
Home
Design
Help
Design Basis
Edit Design Data
Lintel
Wall
Shear Wall
Column
Pilaster
Beam
Calculator
Design Components
Wall Design: Design Criteria
Design Data
Construction Data
Load Data
Design forces at critical section
Compute design forces
Service level design forces
V 0 lb/ft
M L 22950 lb-in / ft
P 3112 lb / ft
e 0 in
Design criteria
2011 MSJC ASD Code
Reinforced Masonry Wall
Simply Supported Wall
Reinforcement
Use #5 Bars (A=0.31, D=0.625)
Display results
Interaction diagram for this wall
Family Grout Spacing
Design notes
OK
Cancel
Help
10:23 AM
3/4/2017

Example

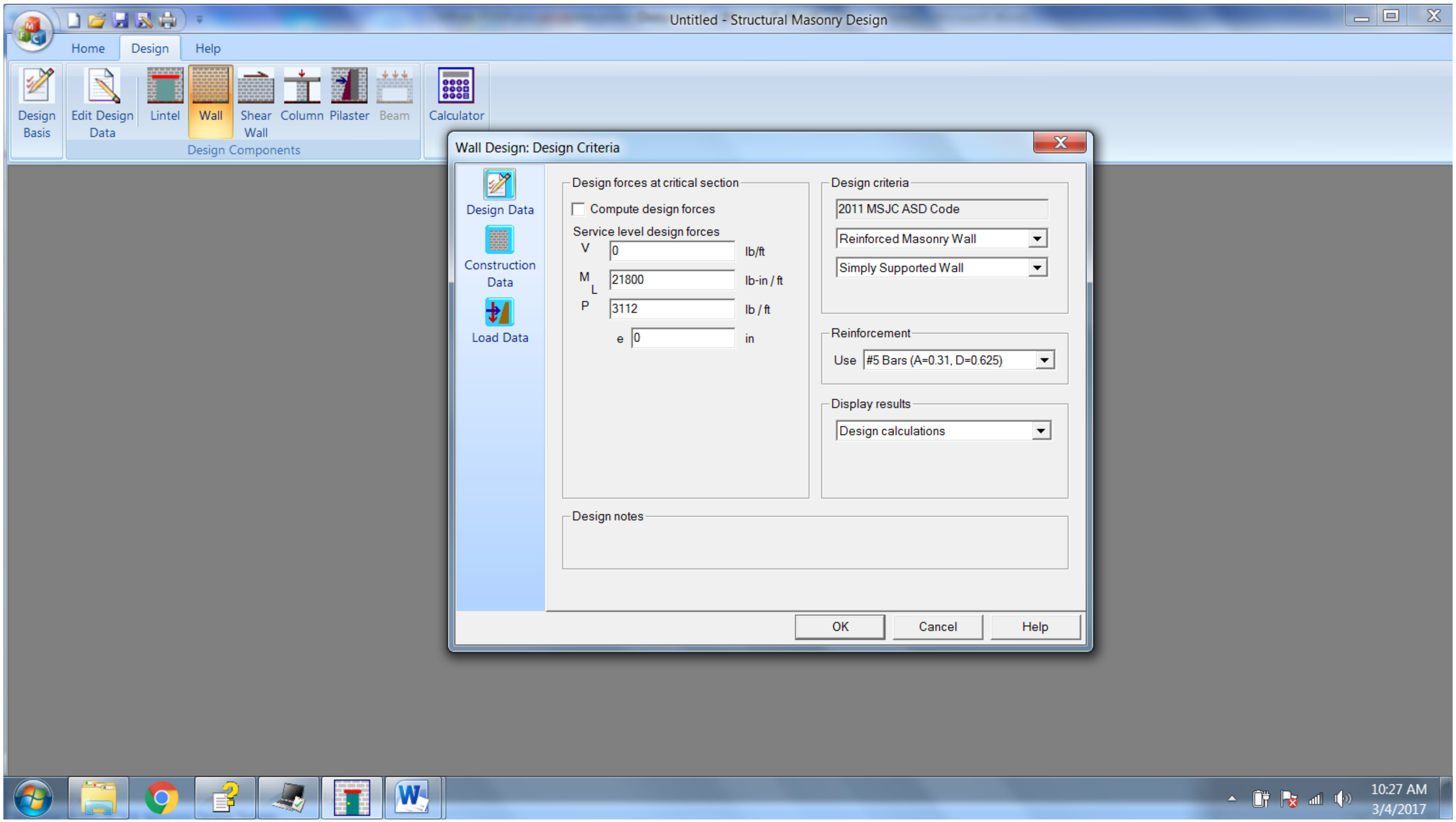

Untitled - Structural Masonry Design
Home
Design
Help
Design Basis
Edit Design Data
Lintel
Wall
Shear Wall
Column
Pilaster
Beam
Design Components
Calculator
Wall Design: Construction Data
Design Data
Construction Data
Load Data
Wall masonry units
8 in. CMU (t=7.625 in.)
Wall construction dimensions
t 7.625 in.
x 3.8125 in.
H 20 ft. 0.00 in.
L Length is not required
Wall construction and spacing
Construction
Full grout, running bond
Reinforcement spacing
16 in. on center
Wall design dimensions
H 240 in.
s
H
L
x
Exterior
Interior
t
OK
Cancel
Help
10:24 AM
3/4/2017

Example



Untitled - Structural Masonry Design
Home
Design
Help
Design Basis
Edit Design Data
Lintel
Wall
Shear Wall
Column
Pilaster
Beam
Calculator
Design Components
Wall Design: Design Criteria
Design Data
Construction Data
Load Data
Design forces at critical section
Compute design forces
Service level design forces
V
0
lb/ft
M
22950
lb-in / ft
Design criteria
2011 MSJC ASD Code
Reinforced Masonry Wall
Simply Supported Wall
Masonry
Insufficient steel area is provided. You need to increase the bar size or decrease the bar spacing.
OK
Design notes
OK
Cancel
Help
10:25 AM
3/4/2017

Example


Untitled - Structural Masonry Design
Home
Design
Help
Design Basis
Edit Design Data
Lintel
Wall
Shear Wall
Column
Pilaster
Beam
Calculator
Design Components
Wall Design: Design Criteria
Design Data
Construction Data
Load Data
Design forces at critical section
Compute design forces
Service level design forces
V
0
lb/ft
M L
21800
lb-in / ft
P
3112
lb / ft
e
0
in
Design criteria
2011 MSJC ASD Code
Reinforced Masonry Wall
Simply Supported Wall
Reinforcement
Use
#5 Bars (A=0.31, D=0.625)
Display results
Design calculations
Design notes
OK
Cancel
Help
10:27 AM
3/4/2017

Example

MASONRY DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY
The National Concrete Masonry Association · Brick Industry Association · Western States Clay Product Association · International Code Council · Masonry 7.1 (Release 7.1.0)

Prjct: IMI Hawaii
Topic: Software Beta
Page:
Name: John G Tawresey
Date: March 13, 2017
Chkd: No one

Design of a Reinforced Masonry Wall with Out-of-Plane Loads
Using the 2011 MSJC ASD Code

Material and Construction Data
- 8 in. CMU, Full grout, running bond
- Wall Weight = 80.35 psf
- Type S Masonry cement / Air-entrained PCL Mortar, Coarse Grout
- CMU Density = 115.0 pcf
- f'_m = 1,500 psi (Specified)
- E_m = 900f'_m = 1,350,000 psi

Reinforcing Steel Properties
- #5 Bars, F_y = 60,000 psi
- Allowable stress F_s = 32,000 psi

Wall Design Details
- Thickness = 7.625 in.
- Height = 240.0 in. (Simply Supported \
- x = 3.813 in.
- Reinforcement Spacing = 16.00 in. On

Wall Design Section Properties
- A_o = 91.50 in^2 per foot width
- I_o = 443.3 in^4 per foot width
- S_o = 116.3 in^3 per foot width
- r_o = 2.201 in

Wall Average Section Properties
- A_{avg} = 91.50 in^2 per foot width
- I_{avg} = 443.3 in^4 per foot width
- r_{avg} = 2.201 in

Design Calculations: Specified Section Forces

Section Design Forces Used
- V = 0. lb/ft (Specified)
- M_L = 21,800 lb-in./ft (Specified)
- P = 3,112 lb/ft at e = 0. in (Specified)

Computed Design Values
- Effective Width = 16.00 in.
- Web Width = 16.00 in. on effective width
- Web Width = 12.00 in. per foot width

MASONRY DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY
The National Concrete Masonry Association · Brick Industry Association · Western States Clay Product Association · International Code Council · Masonry 7.1 (Release 7.1.0)

Prjct: IMI Hawaii
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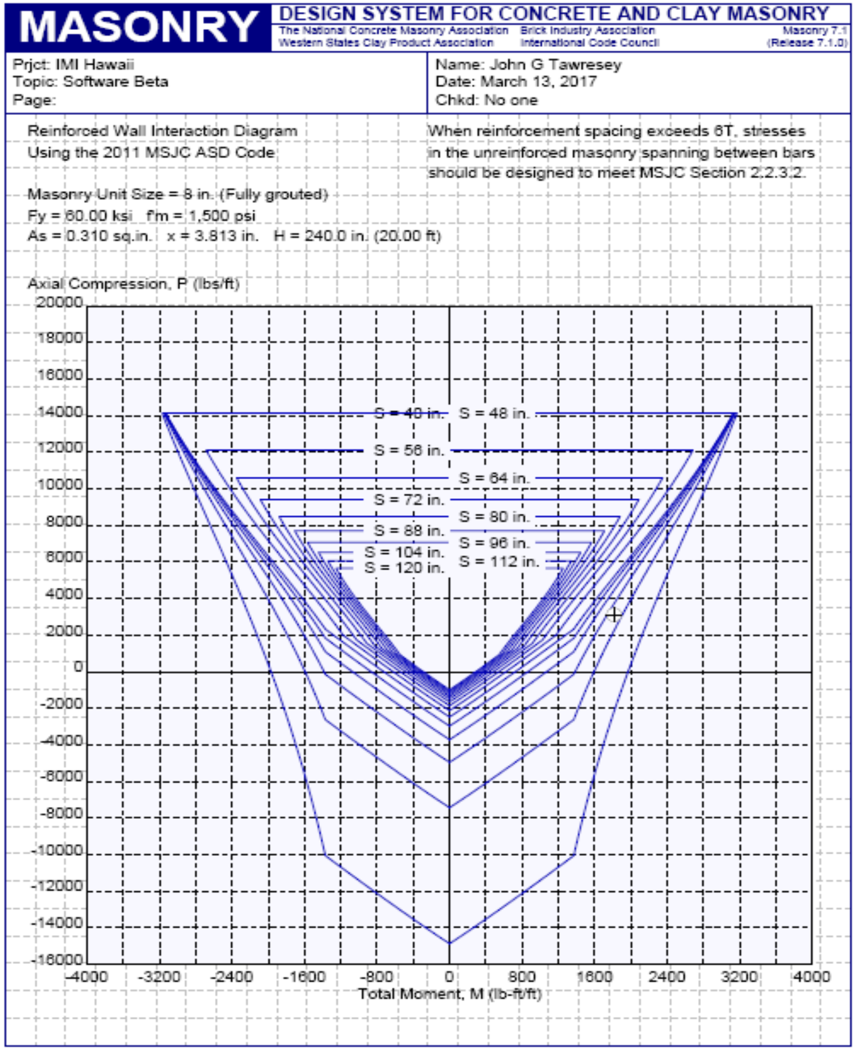
Page:
Name: John G Tawresey
Date: March 13, 2017
Chkd: No one

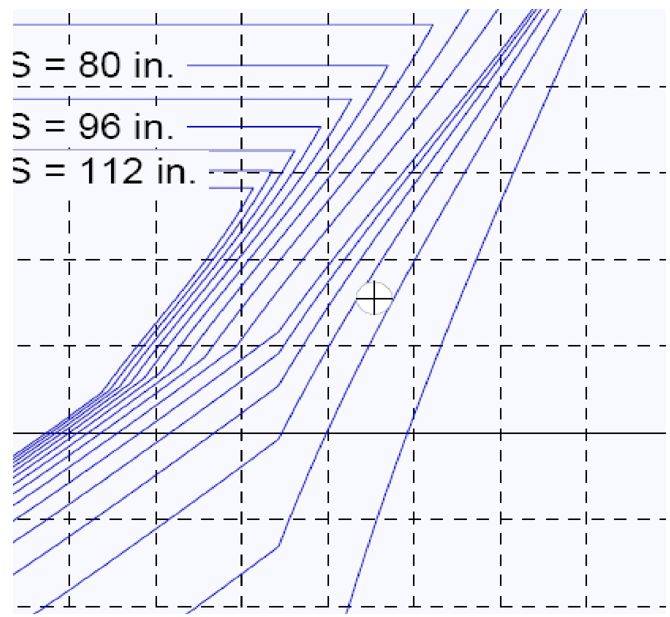
Check for Shearing Force
- Shear Area Used A_{nv} = 91.50 in^2/ft (MSJC 1.6)
- Maximum Shearing Force Permitted = 7,088 lb/ft (MSJC 2.3.6.1.2)
- Maximum Shearing Stress Permitted F_v = 77.46 psi (MSJC 2.3.6.1.2)
- Maximum Shearing Force in Masonry Permitted = 4,765 lb/ft (MSJC 2.3.6.1.3)
- Maximum Shearing Stress in Masonry Permitted F_{vm} = 52.07 psi (MSJC 2.3.6.1.3)
- Calculated Shearing Stress f_v = 0. psi (MSJC 2.3.6.1.1

)
nt and splice lengths.

Required A_s = 0.250 in^2 each reinforced cell (0.187 in^2/ft) OK
d = 3.813 in.
n = 21.48
$k_{balanced}$ = 0.311
$j_{balanced}$ = 0.896
k = 0.432
j = 0.855
P_{max} (Compression) = 18,860 lbs (14,140 lbs/ft) OK
P_{max} (Tension) = 9,920 lbs (7,440 lbs/ft) OK

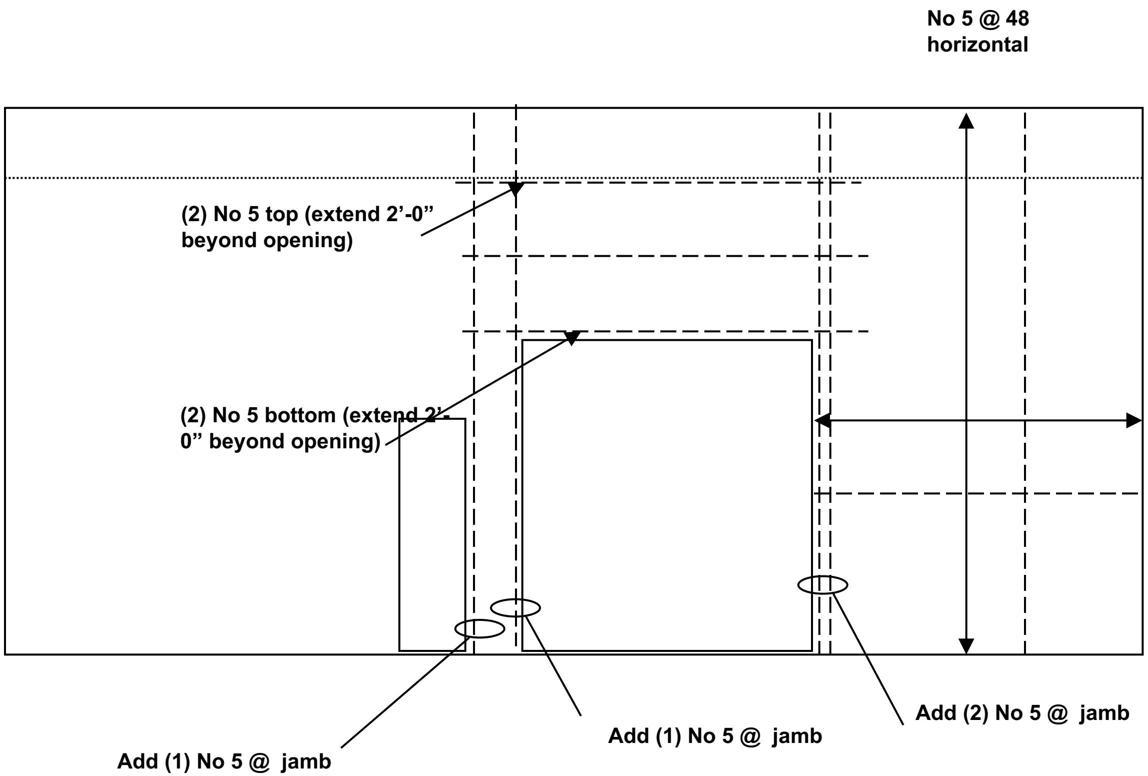
Example


MASONRY
DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY
The National Concrete Masonry Association Brick Industry Association
Western States Clay Product Association International Code Council
Masonry 7.1
(Release 7.1.0)
Prjct: IMI Hawaii
Topic: Software Beta
Page:
Name: John G Tawresey
Date: March 13, 2017
Chkd: No one
Reinforced Wall Interaction Diagram
Using the 2011 MSJC ASD Code
When reinforcement spacing exceeds 6T, stresses in the unreinforced masonry spanning between bars should be designed to meet MSJC Section 2.2.3.2.
Masonry Unit Size = 8 in. (Fully grouted)
Fy = 60.00 ksi f'm = 1,500 psi
As = 0.310 sq.in. x = 3.813 in. H = 240.0 in. (20.00 ft)
Axial Compression, P (lbs/ft)
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
-10000
-12000
-14000
-16000
S = 40 in.
S = 48 in.
S = 56 in.
S = 64 in.
S = 72 in.
S = 80 in.
S = 88 in.
S = 96 in.
S = 104 in.
S = 112 in.
S = 120 in.
-4000
-3200
-2400
-1600
-800
0
800
1600
2400
3200
4000
Total Moment, M (lb-ft/ft)


S = 80 in.
S = 96 in.
S = 112 in.

Example

One more thing: Special Reinforced Masonry Shear Walls



Example

1.18.3.2.6 *Special reinforced masonry shear walls*
— Design of special reinforced masonry shear walls shall comply with the requirements of Section 2.3 or Section 3.3. Reinforcement detailing shall also comply with the requirements of Section 1.18.3.2.3.1 and the following:

(a) The maximum spacing of vertical reinforcement shall be the smallest of one-third the length of the shear wall, one-third the height of the shear wall, and 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.

(b) (b) The maximum spacing of horizontal reinforcement required to resist in-plane shear shall be uniformly distributed, shall be the smaller of one-third the length of the shear wall and one-third the height of the shear wall, and shall be embedded in grout. The maximum spacing of horizontal reinforcement shall not exceed 48 in. (1219 mm) for masonry laid in running bond and 24 in. (610 mm) for masonry not laid in running bond.

(c) The minimum cross-sectional area of vertical reinforcement shall be one-third of the required shear reinforcement. The sum of the cross-sectional area of horizontal and vertical reinforcement shall be at least 0.002 multiplied by the gross cross-sectional area of the wall, using specified dimensions.

    1. For masonry laid in running bond, the minimum cross-sectional area of reinforcement in each direction shall be at least 0.0007 multiplied by the gross cross-sectional area of the wall, using specified dimensions.

    2. For masonry not laid in running bond, the minimum cross-sectional area of vertical reinforcement shall be at least 0.0007 multiplied by the gross cross-sectional area of the wall, using specified dimensions. The minimum cross sectional area of horizontal reinforcement shall be at least 0.0015 multiplied by the gross crosssectional area of the wall, using specified dimensions.

(d) Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook.

(e) Mechanical splices in flexural reinforcement in plastic hinge zones shall develop the specified tensile strength of the spliced bar.

Example

Vertical Reinforcement:

Maximum Spacing = L/3 = 32/3 = 10.6 in

Maximum Spacing = H/3 = 240/3 = 80 in

Maximum Spacing = 48 in

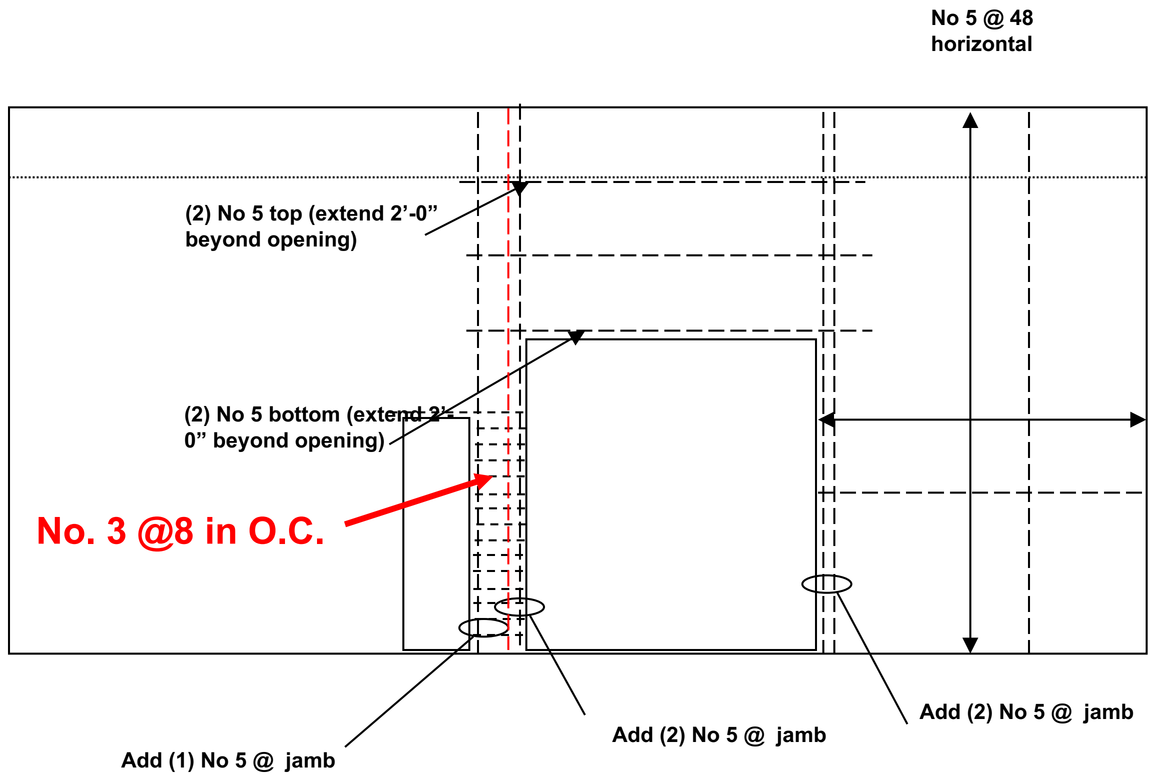
Horizontal Reinforcement:

Maximum Spacing = L/3 = 32/3 = 10.6 in

Maximum Spacing = H/3 = 240/3 = 80 in

Maximum Spacing = 48 in

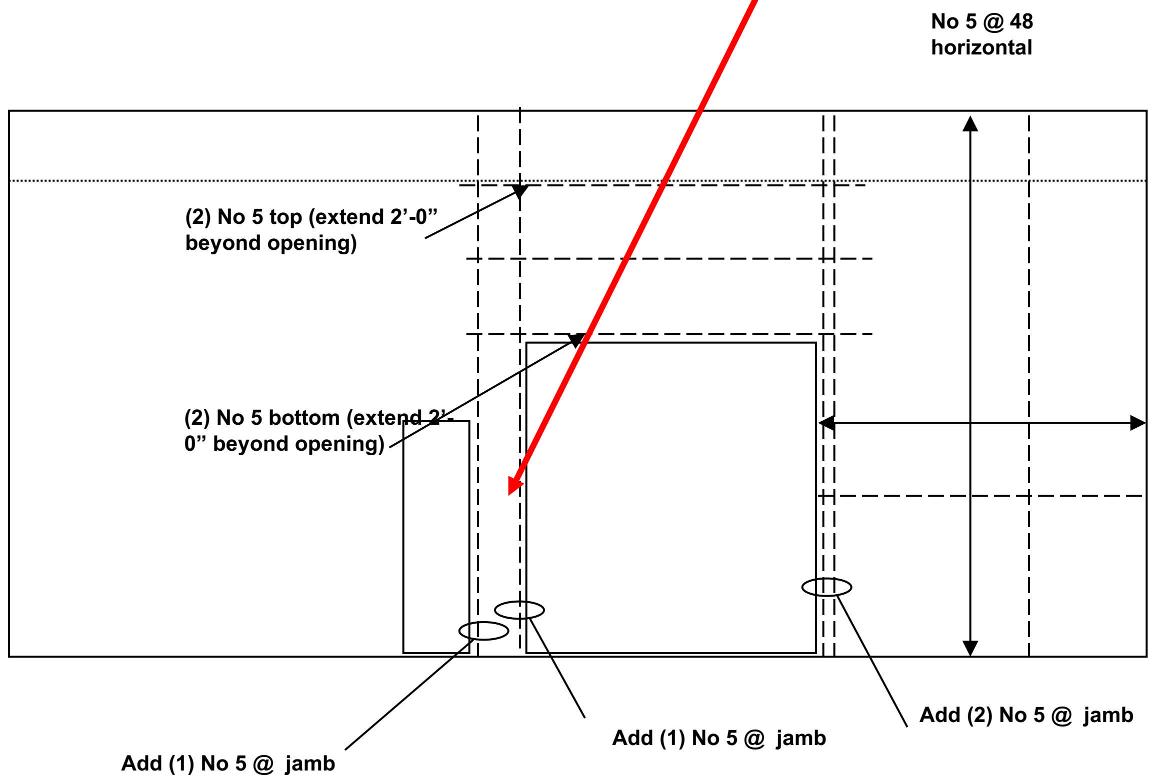
Example



No 5 @ 48
horizontal
(2) No 5 top (extend 2'-0" beyond opening)
(2) No 5 bottom (extend 2'-0" beyond opening)
No. 3 @8 in O.C.
Add (1) No 5 @ jamb
Add (2) No 5 @ jamb
Add (2) No 5 @ jamb

Example

Not counted as part of the lateral force resisting system



Example

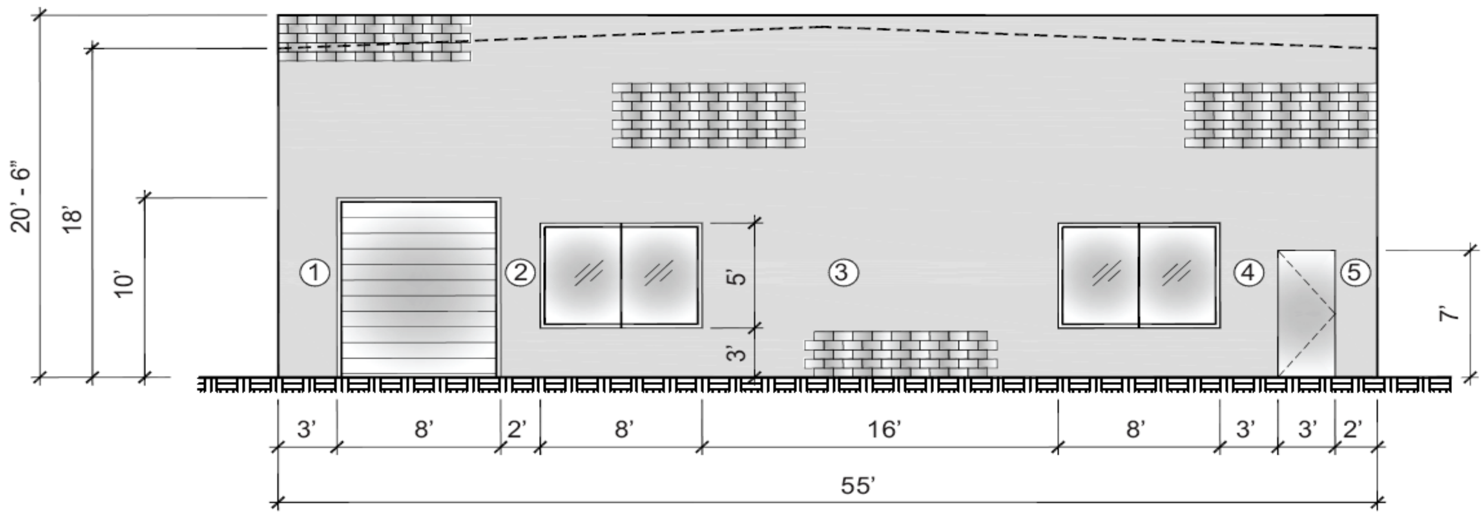


TABLE 11.2 Relative Rigidities of Piers – West Wall

Pier No.	Height ***h*** (ft)	Length ***d*** (ft)	***h/d*** Ratio (all piers fixed)	Relative Rigidity Tables GN-89 Fixed Piers	Percentage Lateral Force to Each Pier	Force ***V*** to each Pier (pounds)	Unit Shear f_v, in each pier $= \frac{V}{tl}$ (psi)
1	10	3	3.33	0.213	1.5	209	0.8
2	5	2	2.50	0.432	3.1	432	2.4
3	5	18	0.28	11.602	82.8	11,550	7.0
4	4	3	1.33	1.577	11.20	1,562	5.7
5	7	2	3.50	0.187	1.3	181	1.0
				Σ = 14.011	99.9%	Σ = 13,934 pounds	

Contents:

1. The Theory [ASD and SD]

2. The Code [2012 IBC, ASCE 7 –10 and TMS 402-11]

3. The Examples

Example: In-Plane Shear Wall Design - Seismic

Example: In-Plane Shear Wall Design - Seismic

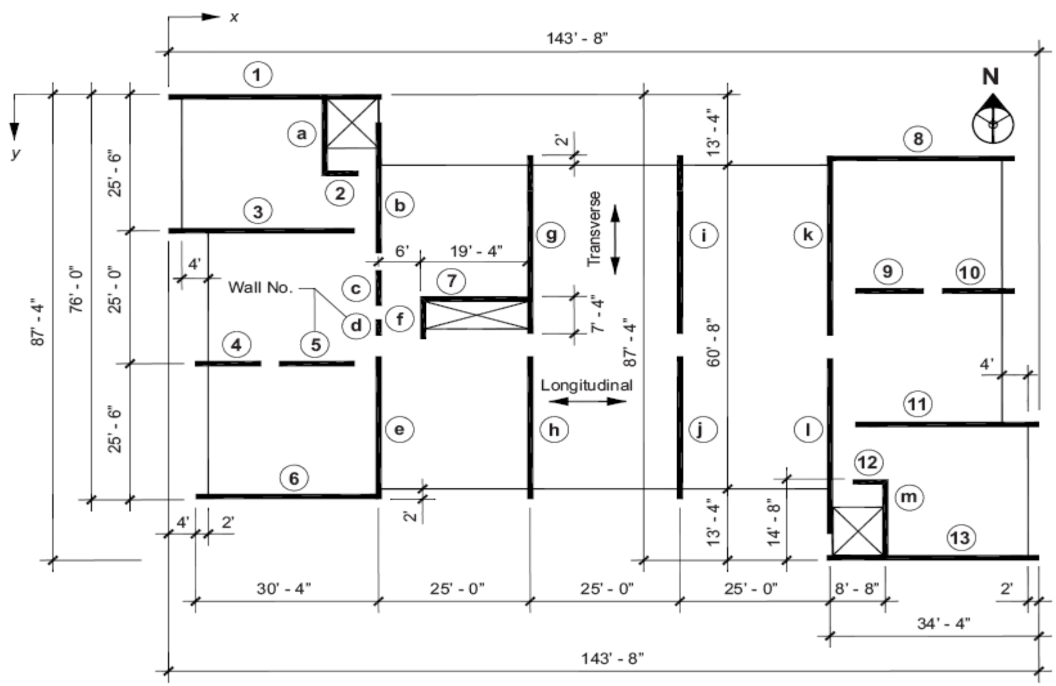
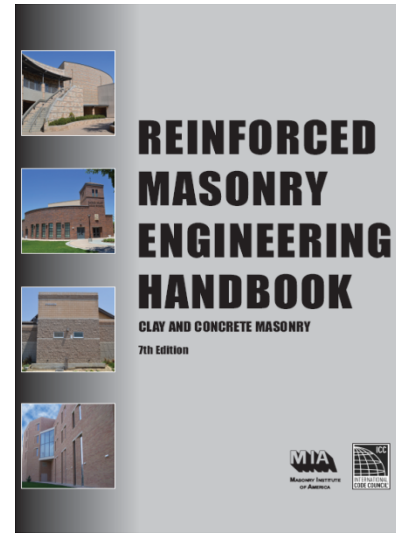


FIGURE 12.4 Typical structural floor plan.



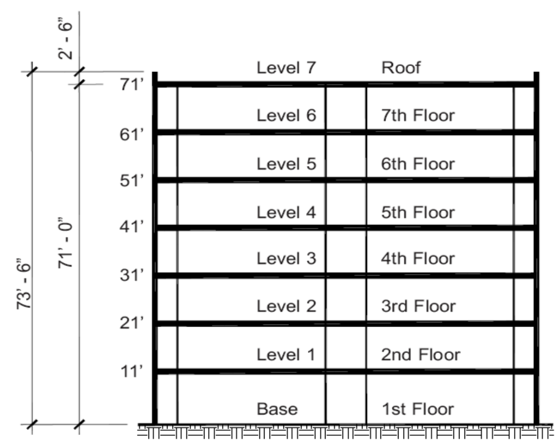


FIGURE 12.3 Transverse cross-section.

Example: In-Plane Shear Wall Design - Seismic

TABLE 12.2 Gravity Load Distribution for Wall j

FLOOR	Trib. Area	Wall Height	Floor D	Wall D	Live L	Sum Area	LL Reduction	Sum D	Sum L
R	655.2		35.1		13.1	655.2	40.0		
		10.0		20.1				55.2	7.9
7	655.2		60.9		30.7	655.2	40.4		
		10.0		20.1				136.3	26.2
6	655.2		60.9		30.7	1,310.4	60.0		
		10.0		20.1				217.4	32.4
5	655.2		60.9		30.7	1,965.6	60.0		
		10.0		20.1				298.4	44.7
4	655.2		60.9		30.7	2,620.8	60.0		
		10.0		20.1				379.5	57.0
3	655.2		60.9		30.7	3,276.0	60.0		
		10.0		20.1				460.6	69.3
2	655.2		60.9		30.7	3,931.2	60.0		
		11.0		22.2				543.7	81.6
Ground									

Example: In-Plane Shear Wall Design - Seismic

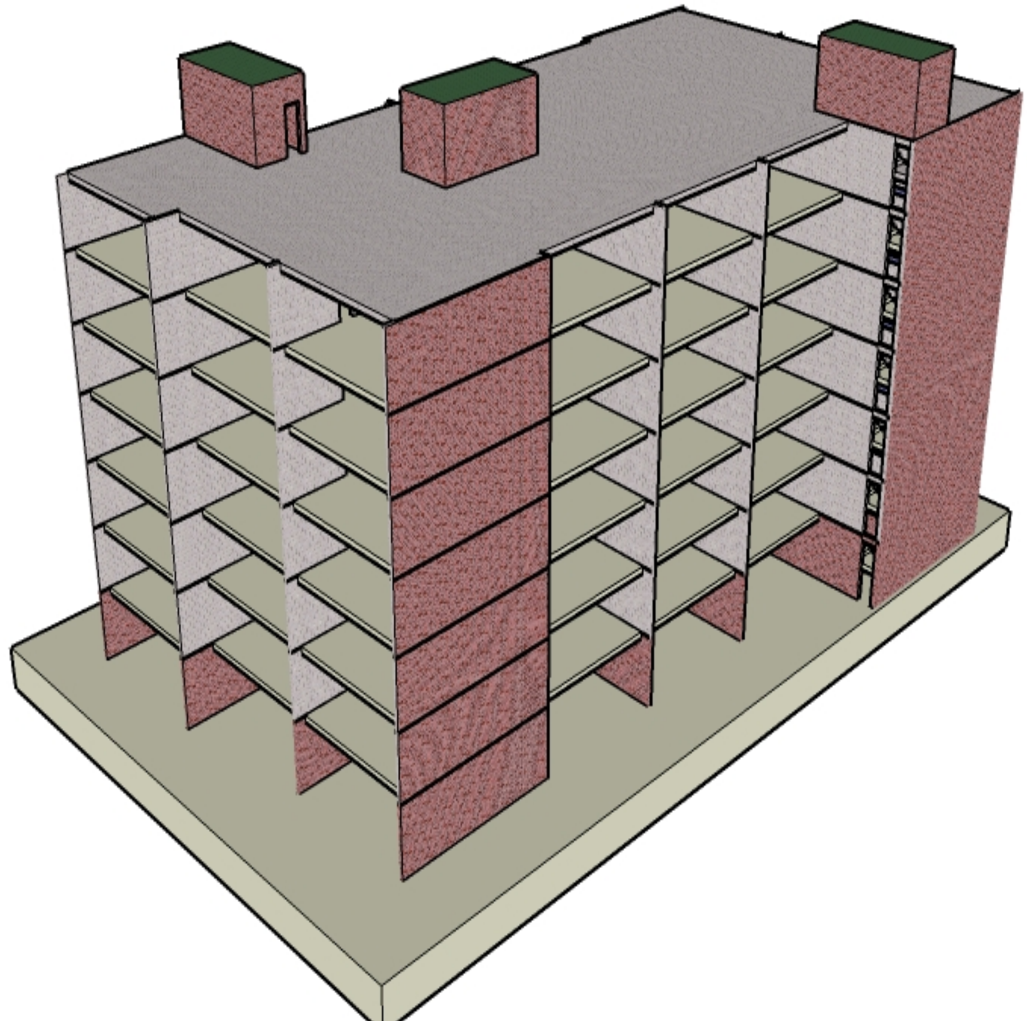
	Class 2012	Class 2012	Class 2014	Class 2014	Class 2015	Class 2015	Class 2016	Class 2016
	DL	LL	DL	LL	DL	LL	DL	LL
	745	114	560	84	1235	210	481	110
	589	119	600	84	667	127	598	85
	589	97	600	84	36	14	549	110
	581	96	914	181	576	112	648	198
	751	114	600	207	685	162	636	124
	581	96	664	106	541	184	633	103
	521	113	630	176	440	140	716	200
	583	96	664	106	550	103	712	200
	533	131	600	132	948	166	502	123
	608	123	600	132	699	141	594	96
	589	119	600	94	556	160	605	86
	592	120	600	182	700	110	633	173
	601	106	585	80	599	96	582	84
	624	78	587	80			438	183
	542	421	723	102			631	212
	602	99	569	173			947	473
	641	140					589	91
	744	193					631	89
	594	97					638	173
	683	138						
	593	120						
	745	114						
	592	120						
	626	211						
	658	98						
	577	113						
	589	98						
	600	99						
	521	114						
AVE	614	128	631	125	633	133	619	153
STD	64	63	86	44	273	49	116	90

Example: In-Plane Shear Wall Design - Seismic

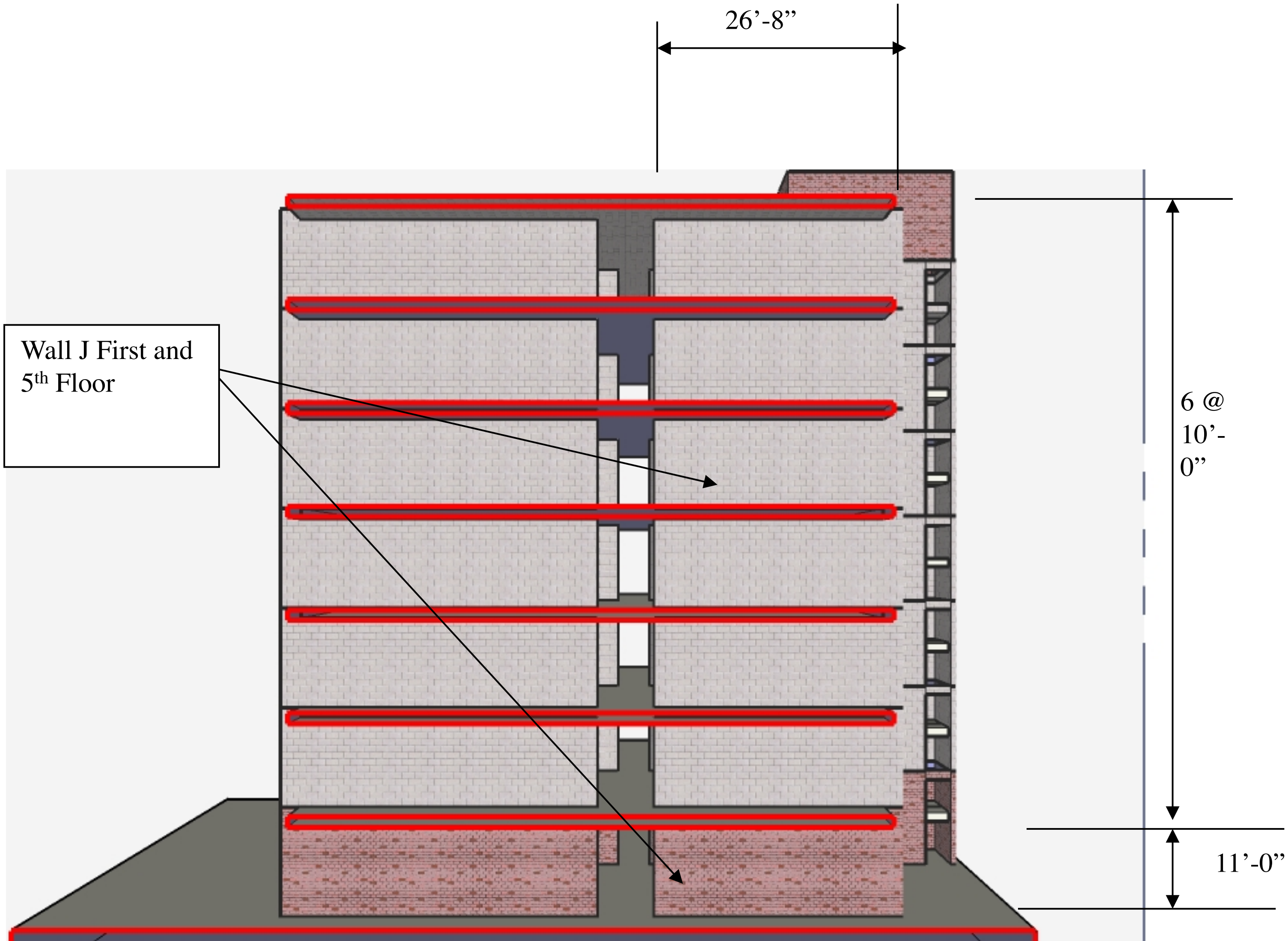
The Problem

Size: 76,500 S.F

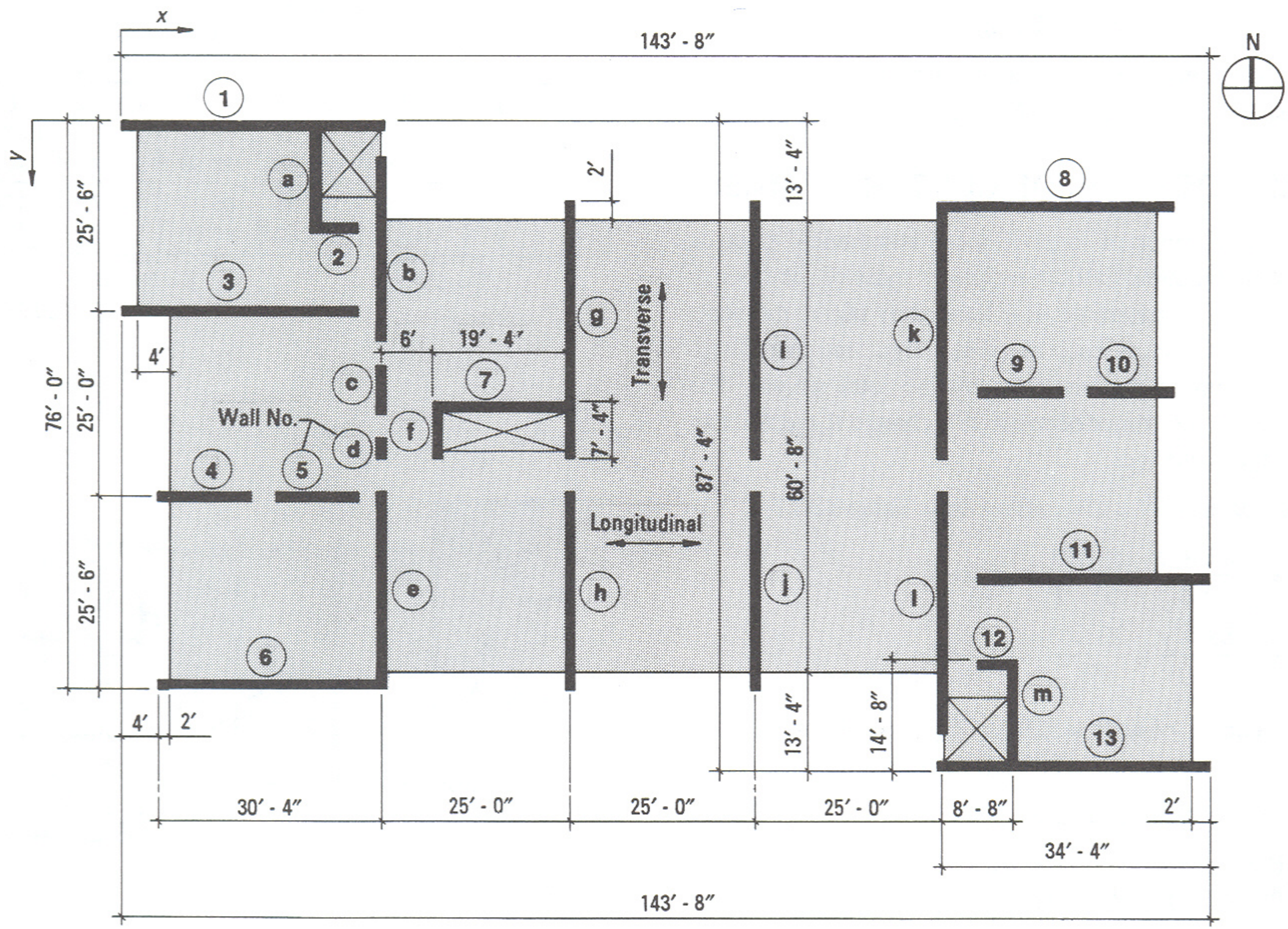
Seven Story Apartment Building



Example: In-Plane Shear Wall Design - Seismic



Example: In-Plane Shear Wall Design - Seismic



M

Example: In-Plane Shear Wall Design - Seismic

Spectral Response

$D_S = 1.5$

$D_1 = 1.0$

Soil Class D [IBC Tables]

$F_a = 1.0$

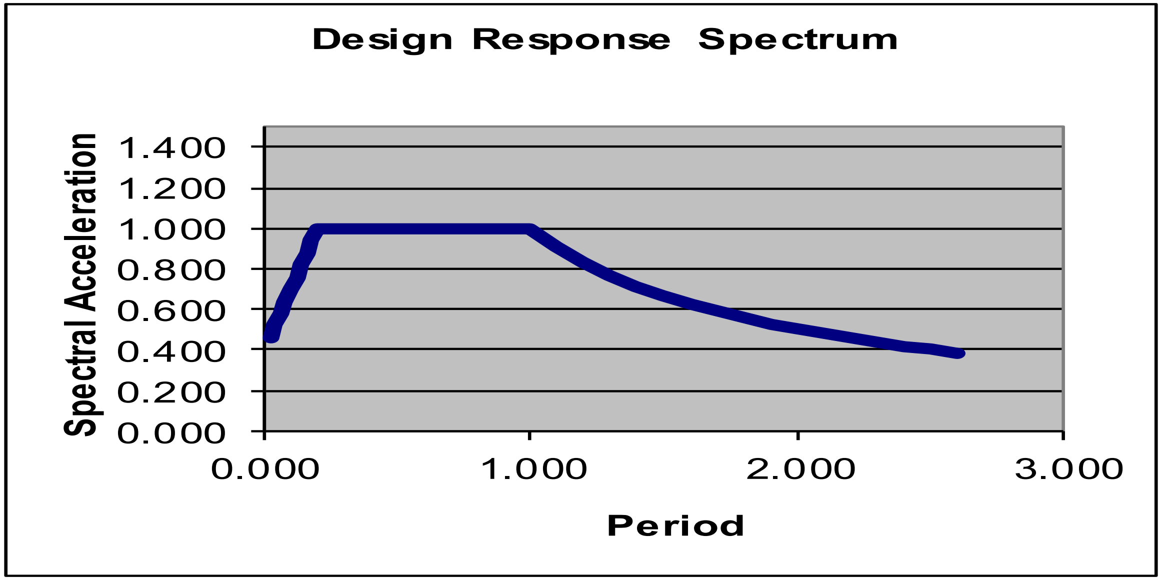
$F_b = 1.5$

Example: In-Plane Shear Wall Design - Seismic

Design values

$$S_{DS} = 2/3*1.0*1.5 = 1.0$$

$$S_{D1} = 2/3*1.5*1.0 = 1.0$$



Example: In-Plane Shear Wall Design - Seismic

From the IBC - 2012

Seismic Design Category = D

From ASCE 7 - 10

Response modification factor = 5.0

System overstrength factor = 2.5

Defection amplification factor = 3.5

Example: In-Plane Shear Wall Design - Seismic

Using ASCE 7 – 10 Equivalent Lateral Force Procedure for Scaling Model Results

Eq 12.8-7 $$T_a = C_t H_n^x$$

Table 12.8-2 Classification is other

$$C_t = .02$$

H_n = 71 ft

X = .75 Table 12.8-2 Classification is other

$$T_a = .02 * 71^{.75} = .49 \text{ sec}$$

Example: In-Plane Shear Wall Design - Seismic

$$C_s = \frac{S_{DS}}{\frac{R}{I_e}} = \frac{1.0}{5} = .2$$

$$C_s = \frac{S_{D1}}{T_a \frac{R}{I_e}} = \frac{1.0}{.49 * 5} = .41$$

The lower value applies. $C_s = .2$

Example: In-Plane Shear Wall Design - Seismic

E-TABS model produce elastic seismic response [sum of the squares all modes]:

$V_{E\text{-}W} = 7{,}407\ k$

$V_{N\text{-}S} = 7{,}299\ k$

TABLE 12.7 E-TABS Output

Level	Wall	Load	Loc	Axial Load P	V2 (kips)	V3 (kips)	T (kips)	M2 (kips)	M3 (kip-in.)
STORY 2	J	X	Top	0	336.9	1.0	39.9	75	104,889
STORY 2	J	X	Bottom	0	336.9	1.0	39.9	184	144,646
STORY 2	J	Y	Top	0	498.9	0.7	61.3	52	145,717
STORY 2	J	Y	Bottom	0	498.9	0.7	61.3	129	202,843
STORY 1	J	ATEQX	Top	0	15.8	0.1	4.2	-13	5,334
STORY 1	J	ATEQX	Bottom	0	15.8	0.1	4.2	0	7,427
STORY 1	J	ATEQY	Top	0	26.5	0.2	7.0	-22	8,927
STORY 1	J	ATEQY	Bottom	0	26.5	0.2	7.0	0	12,420
STORY 1	J	X	Top	0	404.8	1.4	10.5	184	144,646
STORY 1	J	X	Bottom	0	404.8	1.4	10.5	0	197,227
STORY 1	J	Y	Top	0	614.6	1.0	15.3	129	202,843
STORY 1	J	Y	Bottom	0	614.6	1.0	15.3	0	281,360

Example: In-Plane Shear Wall Design - Seismic

Building weight

TABLE 12.6 Vertical Distribution of Building Mass

FLOOR	Trib. Area	Wall Height	Floor D	Wall D	Sum D
R	8,627		462		
		12.5		533	994
7	8,627		802		
		10		425	2,221
6	8,627		802		
		10		425	3,448
5	8,627		802		
		10		425	4,676
4	8,627		802		
		10		425	5,903
3	8,627		802		
		10		425	7,130
2	8,627		802		
		11		467	8,399
Ground					

Some do not include ½ the wall weight on the first floor. Makes a big difference for a 1 or 2 story building.

Example: In-Plane Shear Wall Design - Seismic

$$V = .2 * 8399 = 1{,}680\ k$$

$$E - W = \frac{1680}{7407} = .227$$

$$N - S = \frac{1680}{7299} = .230$$

Example: In-Plane Shear Wall Design - Seismic

Combine primary scaled shear with 30% of the shear in the other direction with the torsion for the primary shear:

V_y = 614.6*.230 + .30*404.8*.223 + ABS[26.5]*2.30 = 229.4 k

M_y = 281,360*.230 + .30*197,227*.223 + ABS[12,420]*.230 = 80,760 k-in

Example: In-Plane Shear Wall Design - Seismic

These are the in-plane loads

Load Summary:

	Axial Load (Kip)	Shear Load (Kip)	Moment (Kip-in)
Dead Load	599.5	0	0
Live Load	99.5	0	0
Seismic Load (N-S)	0	229.4	80,760

Example: In-Plane Shear Wall Design - Seismic

Masonry dimensions of the wall are:

b = 7.625 inches (CMU is laid with a 3/8 mortar joing)
L = 25 feet – 10 inches = 310 inches

Check the shear capacity first. This will usually determine the thickness of the masonry required. [IBC 2006 Section 2106.5.1 requires the seismic shear force to be increase by 1.5.]

Don’t think it exist in 2012

Example: In-Plane Shear Wall Design - Seismic

ASD Design

$[.9D - .14S_{DS}] + 0.7E$ Special exception for masonry.

$[D + .14S_{DS}] + 0.75L + 0.75*.7E$

P = [.9-.14]*599.5 = 455.6

P = [1.0 + .14]*599.5 = 683.4

V = .7*229.4 = 160.6 k

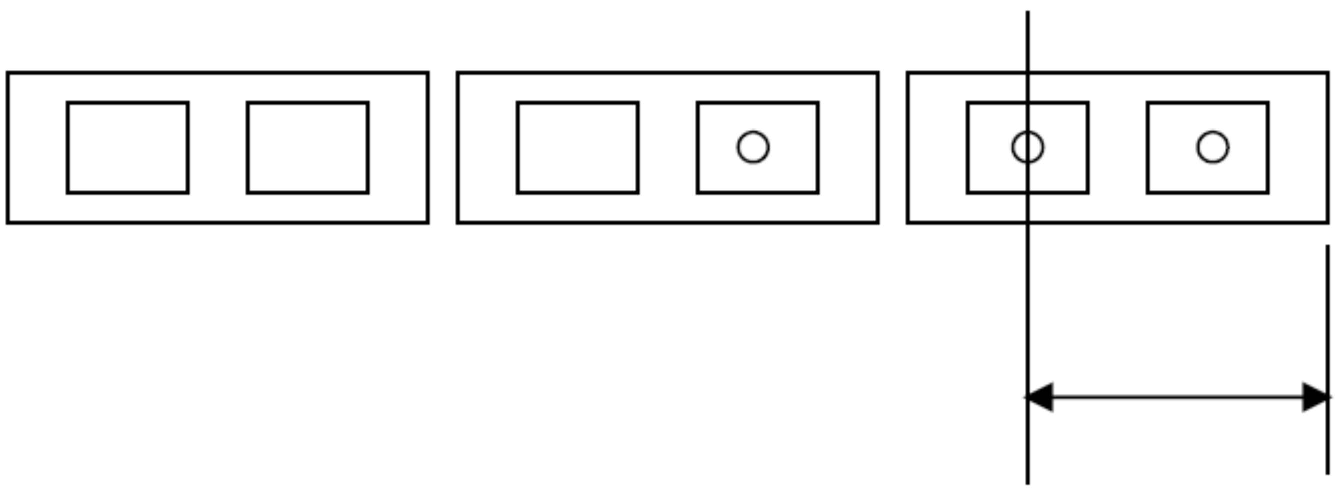
M = .7*80,760 = 56,532 k-ft

Example: In-Plane Shear Wall Design - Seismic

The maximum shear loading on the wall is:

V = .7*229.4 = 160.6 Kip

Assume 3 bars at the end of the wall. CMU typically uses a 16 inch module



Example: In-Plane Shear Wall Design - Seismic

The computed shear stress is:

d = 310 – 12 = 298 in

$$f_v = \frac{V}{bd} = \frac{160{,}600}{7.625x298} = 71\,psi$$

MSJC Equation 2-19

The allowable shear stress is:

$$\frac{M}{Vd} = \frac{56{,}532{,}000}{160{,}600x298} = 1.18 \geq 1.0$$

MSJC Equation 2-27

$$F_v = 2.0\sqrt{f_m^{'}} = 2.0\sqrt{1500} = 77.4\,psi$$

$$f_v \prec F_v \;\; OK$$

Example: In-Plane Shear Wall Design - Seismic

$$F_v = F_{vm} + F_{vs}$$

$$F_{vm} = \left(\frac{1}{4}\right)\left[4.0 - 1.75\frac{M}{Vd}\right]\sqrt{f_m^{'}} + .25\frac{P}{A_n}\ psi$$

MSJC Equation 2-29

$$F_{vm} = \left(\frac{1}{4}\right)[4.0 - 1.75 * 1.18]\sqrt{1500} + .25\frac{455{,}600}{7.625 * 298} = 18.7 + 50.1 = 68.9\ psi$$

Not much horizontal reinforcement required.
Special reinforced shear wall ρ = .0007. Use (2) No. 4 at 4'-0" ρ = .62/[7.625*48] = 0011. L/3 = 103 inches OK

Example: In-Plane Shear Wall Design - Seismic

MSJC Equation 2-30

$$F_{va} = 0.5\frac{A_v F_s d}{A_n s} = 0.5\frac{.4 * 32{,}000 * 298}{7.625 * 298 * 48} = 35\ psi$$

$$F_v = F_{vm} + F_{vs} = 68.9 + 35 = 104\ psi$$

Example: In-Plane Shear Wall Design - Seismic

Compression First

$M/Pd = 56{,}600{,}000/(683{,}400*298) = .28$

$2/3\text{-Delta} = 2/3 - (298/2 - 12)/298 = .21$

So M/Pd is in region 3

Compression limited f'_m needs to be increased. Say 2000 psi per new code.

(3) No. 7 $A_s = 1.8\ in^2$

$M_t = 67{,}600{,}000$ lb-in

$M_c = 74{,}100{,}000$ lb-in

OK

Example: In-Plane Shear Wall Design - Seismic

Tension

$M/Pd = 56{,}600{,}000/(455{,}600*298) = .42$

(3) No. 7 $A_s = 1.8$ in^2

$M_t = 63{,}600{,}000$ lb-in

$M_c = 56{,}300{,}000$ lb-in

OK

Example: In-Plane Shear Wall Design - Seismic

The MSJC 2011 section 2107.8 limits the amount of reinforcement for special reinforced masonry shear walls to the following:

$$\rho_{\max} = \frac{nf'_m}{2f_y\left(n + \frac{f_y}{f'_m}\right)} = \frac{16.1x2000}{2x60{,}000\left(16.1 + \frac{60{,}000}{2000}\right)} = .0058$$

$$\rho_{\max} = \frac{1.8}{7.625 * 298} = .0008 \le .0058\ OK$$

Example: In-Plane Shear Wall Design - Seismic

Strength Design Limit:

$$\rho_{\max} = \frac{A_S}{bd} = \frac{.64 f_m^{'}\left(\frac{\varepsilon_0}{\varepsilon_0 + \alpha\varepsilon_y}\right) - \frac{P}{bd}}{f_y}$$

$$\rho_{\max} = \frac{A_S}{bd} = \frac{.64 * 2000\left(\frac{.0025}{.0025 + 4.0 * .00207}\right) - \frac{683,400}{7.625 * 298}}{60,000} = .00495 - .0050 = -.00005$$

Negative steel required.