NCSEA Structural Engineering Exam Review Course

Lateral Forces Review

Concrete

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Exam Topics

- Shear Walls
  - Ordinary, Intermediate, and Special
- Moment-Resisting Frames
  - Ordinary, Intermediate, and Special
- Ductile Detailing and Anchorage
- Diaphragms
- Tilt-up Walls

Review Contents

- Overview of Codes
- Load Combinations, Prerequisite Information
- Reinforcement Development and Splices
- Shear Walls
- Moment-Resisting Frames
- Ductile Detailing and Anchorage
- Tilt-up Walls
- Diaphragms
- Final Remarks

Reference Codes for Concrete

- IBC 2012: Section 1613 and Chapter 19
- ASCE 7-2010: Excluding Chapter 14 and Appendix 11A (IBC 1613.1)
- ACI 318-11 (with appropriate amendments per IBC Section 1905)

Applicable Load Combinations

- IBC 2012, Section 1605.2
  1. $1.2D + 1.0W + f_1L + 0.5(L_r \text{ or } S \text{ or } R) + 1.6H$
  2. $1.2D + 1.0E + f_1L + f_2S + 1.6H$
  3. $0.9D + 1.0W + 1.6H$
  4. $0.9D + 1.0E + 1.6H$

f_1 = 0.5 or 1.0 for live loads
f_2 = 0.2 or 0.7 for snow loads
See IBC 1605.2

- ASCE 7-10, 12.4.2:
  – $E = E_h + 0.2S_{DS}D$ *Use in LC 2*
  – $E = E_h - 0.2S_{DS}D$ *Use in LC 4*
  – $E_h = \rho Q_E$ (Q_E => V or F_p)

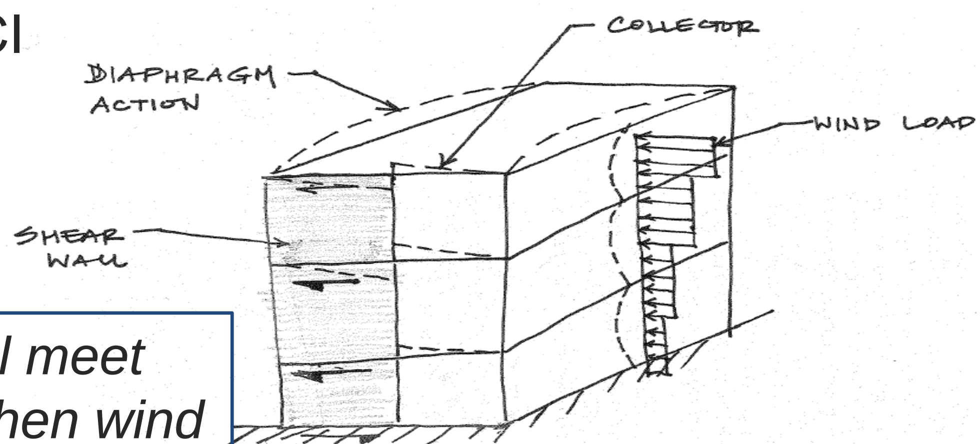
Lateral Forces on Walls

Seismic loads

- Inertial and generated due to ground shaking and structure mass
- Nonlinear behavior expected for most moderate to high seismic events
- Design load reduced by *R*
- Load path critical and defines design demands
- Concrete design based on ductility—Chapters 1 through 18 and 21 of ACI 318

Wind loads

- Design demands based on external load.
- Concrete design based on linear behavior—Chapters 1 through 18 of ACI 318
- No considerations of ductility
- Load path similar to seismic



Hydrostatic loads—similar to wind

Per IBC 1604.10, Lateral systems shall meet seismic detailing requirements even when wind load effects exceed seismic load effects.

Prerequisite Information

- Calculation wind design forces
- Seismic design category (SDC)
  - SDC determined seismic risk level and sets design and detailing requirements
- Seismic design forces
  - Base shear (Equivalent Lateral Force Procedure, ASCE 7, 12.8 or Simplified Procedure ASCE 7, 12.14)
  - Vertical distribution of base shear
  - Redundancy factor, ρ (ASCE 7, 12.3.4.2)

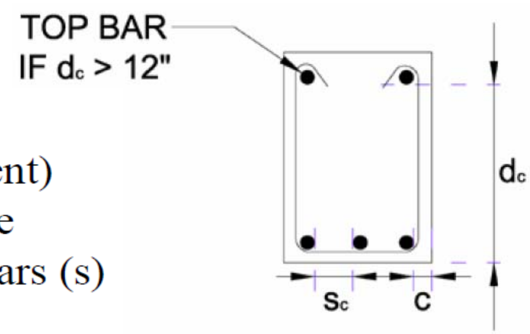
Reinforcement Development and Splices

Development Length in Tension

Development length is based on bar clear spacing, clear cover, stirrup spacing et cetera. See Chapter 12.2.

$$l_d = \left(\kappa \frac{f_y \psi_t \psi_e}{\lambda \sqrt{f'_c}} \right) d_b \;\; \& \;\; l_d \geq 12" \qquad (f_y \;\&\; f'_c \text{ in psi}) \qquad \S 12.2.2$$

$$\& \qquad \psi_t \psi_e \text{ need not exceed } 1.7 \quad \& \quad \sqrt{f'_c} \leq 100\,psi \qquad \S 12.1.2$$



where, ψ_t = 1.0 for $d_c \leq$ 12in; 1.3 for d_c >12in (i.e. top reinforcement)

ψ_e = 1.0 for uncoated reinforcement (§12.2.4 for coated reinforcement)

λ = 1.0 for normal weight concrete; 0.75 for all light weight concrete

κ = based on clear spacing, cover, stirrup or tie extent over lapped bars (s) etc.:

Case I - $s_c \geq d_b$; $c \geq d_b$; stirrups or ties over lap comply with code minimum OR $s_c \geq 2d_b$ & $c \geq d_b$:

κ = 1/25 for #6 and smaller bars; 1/20 for larger bars

Case II – Other conditions

κ = 3/50 for #6 and smaller bars; 3/40 for larger bars

See handout for hooked bar development etc.

Development Length for Joints of SMF

- Development length for standard hooked bar (ACI 318, 21.7.5.1)

$$l_{dh} = \frac{f_y d_b}{65\sqrt{f_c'}} \qquad (f_y \ \& \ f_c' \text{ in psi})$$

$$l_{dh} \geq 8db \text{ or } 6in$$

(Note: For LW concrete use 25% longer)

- Development length for straight bars (ACI 318, 21.7.5.2)

Top bars: $l_d \geq 3.25 l_{dh}$

Other bars: $l_d \geq 2.5 l_{dh}$

Lap Splices in Tension

- Tension lap splices (ACI 318, 12.15):

  Class A lap length = $1.0\ell_d$

  Class B lap length = $1.3\ell_d$

  Use Class B lap splices for seismic detailing

  ℓ_d = development length (≥ 12in)

- Compression lap splices (ACI 318, 12.16):

  For $f_y \leq 60$ksi $\quad lap\,length = 0.0005 f_y d_b \quad$ (f_y in psi)

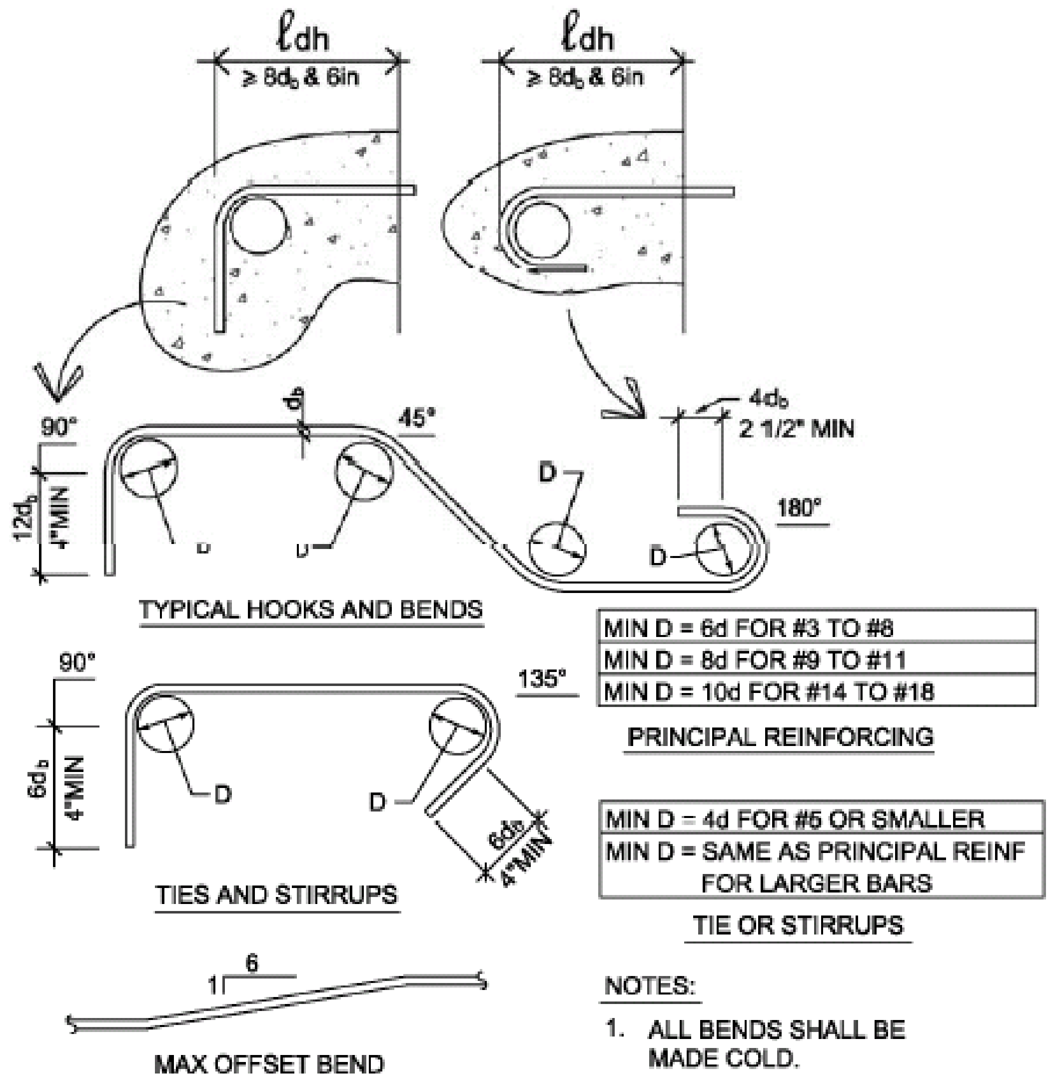
  For $f_y > 60$ksi $\quad lap\,length = (0.0009 f_y - 24) d_b \quad$ (f_y in psi)

  But not less than 12in

Note: Column lap splices (ACI 12.17) => Use Class B splice in exam

Note: No splices in regions of plastic hinging

Typical Hooks and Bends



Shear Walls

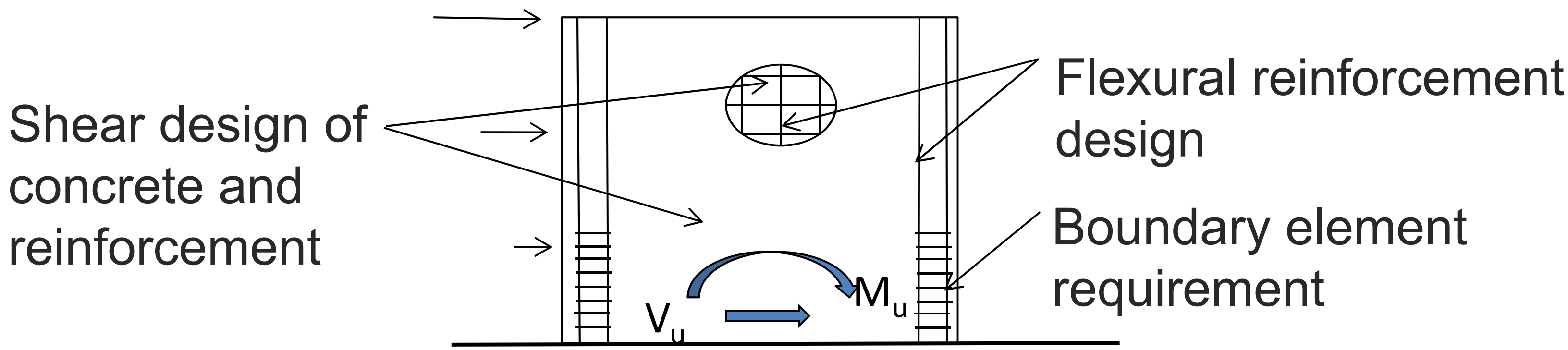
Selection of Wall Type

WALL TYPE	DESIGN REQUIREMENTS	SDC
Ordinary RC shear walls	ACI 318, Chapters 1–18	A, B, C
Special RC shear walls	ACI 318, Chapters 1–8 & Sections 21.1.3 to 21.1.7, 21.9	All
Ordinary precast shear walls	ACI 318, Chapters 1–18	A, B
Intermediate precast shear walls	ACI 318, Chapters 1–18 & Section 21.4	A, B, C
Special precast shear walls	All requirements for Special RC Walls & Section 21.10	All

Refer to ASCE 7, Table 12.2-1 for walls types, R and Ω_o, height limitations, and so on

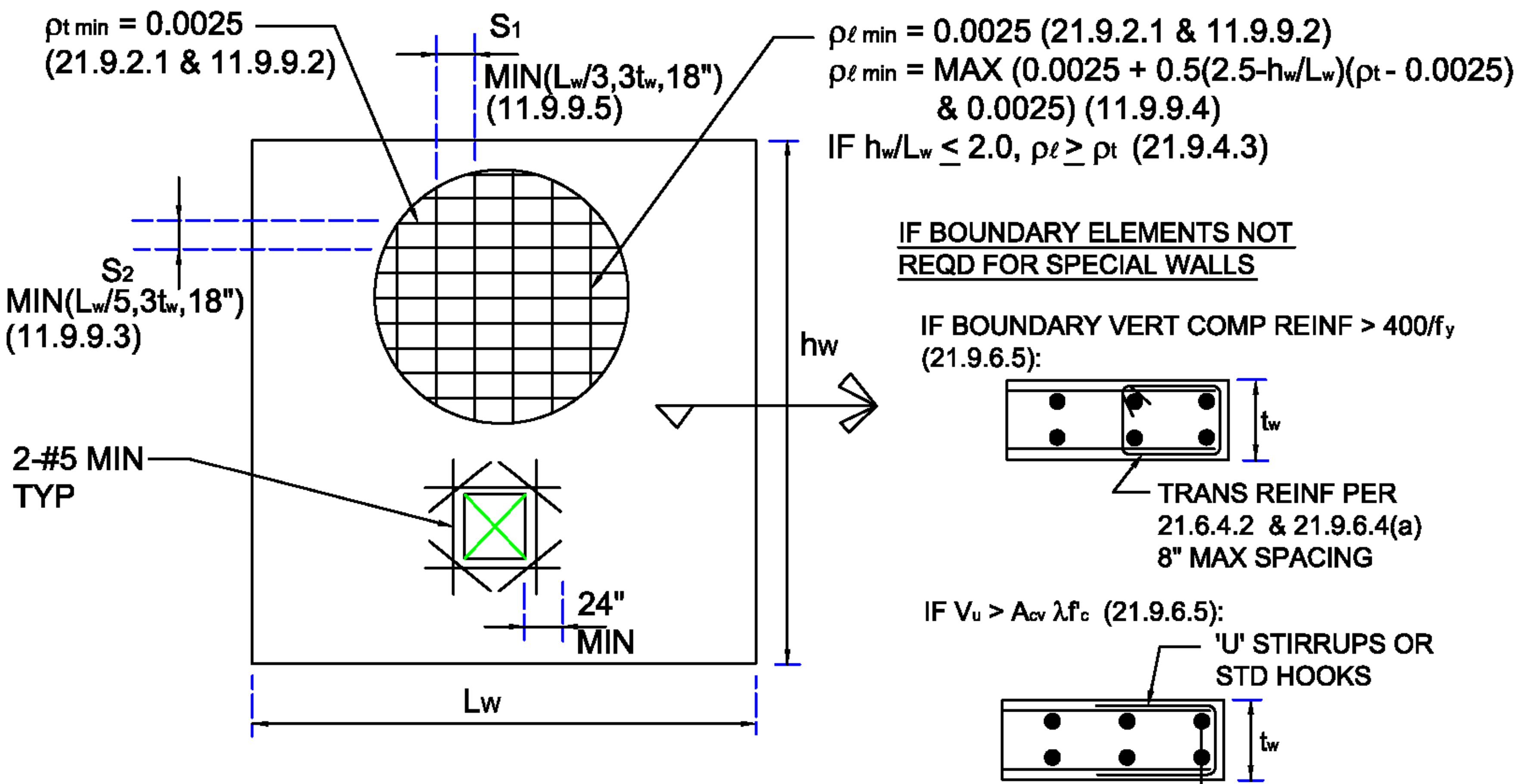
Also see IBC Section 1905 for all Modifications to ACI 318

Reinforced Concrete Shear Walls



Wall Type	SDC	Reinf Limits	Shear Design	Axial and Flexure Design	Other
Ordinary shear wall	A, B, C	14.3 11.9.8, 11.9.9	11.9	14.2, 14.3 10.2, 10.3	—
Special shear wall	ALL	21.9.2	21.9.4	21.9.5 10.2, 10.3	Boundary elements 21.9.6

Reinforcement Limits for Walls



IF t_w > 10" => 2 CURTAINS OF REBAR (14.3.4)

IF $V_u > 2A_{cv}\,\lambda\sqrt{f'_c}$ => 2 CURTAINS OF REBAR (21.9.2.2)

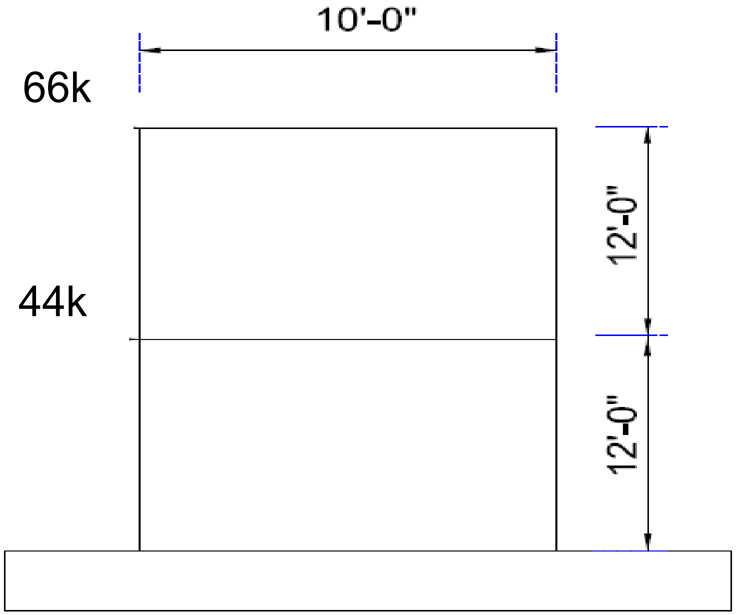
IF $V_u \le A_{cv}\,\lambda\sqrt{f'_c}$, $\rho_{\ell\,min}$ & $\rho_{t\,min}$ CAN BE REDUCED PER 14.3 (21.9.2.1)

Shear Design of Wall

<u>Two-story 12-in. thick special shear wall</u>

ρ = 1.0, S_{DS} = 1.0g, I = 1.0, δ_u = 6 in.

f'_c = 4000psi, f_y = 60ksi



Gravity loads (includes wall weight):

D_R = 20k, D_{2nd} = 20k; L_R = 10k, L_{2nd} = 30k

Seismic loads shown

Load combinations:

1. $(1.2 + 0.2S_{DS})D + 1.0\rho Q_E + 0.5L$ =>
2. $(0.9 - 0.2S_{DS})D + 1.0\rho Q_E$ =>

For first floor wall:

Pu	Vu	Mu
76k	110k	2,112k-ft
28k	110k	2,112k-ft

Shear Design of Wall

Reinforcement (ACI 21.9.2):

$$A_{cv}\lambda\sqrt{f_c'} = \frac{(12"\times 120"\times\sqrt{4000})}{1000} = 91.0k$$

λ=1.0 for Normal Weight Concrete

Since $V_u > A_{cv}\sqrt{f_c'}$ Provide Minimum Reinforcement => ρ_l & $\rho_t \geq 0.0025$

$\rho_t = 0.0025$ => 0.0025 x 12" x 12" = 0.36in^2/ft

Use #4 @ 12" o/c EF = 0.2in^2 x 2 = 0.4in^2/ft ($\rho_t = 0.00277$)

Note: Check bar spacing versus max spacing requirements => OK

Shear Design of Wall

Shear strength (§11.9.5, 11.9.9, and 21.9.4)

Ordinary shear walls (§11.9.4, 11.9.5, 11.9.9)

$$\phi V_n = \phi\left(2\lambda\sqrt{f'_c}\,(t_w)(0.8L_w) + \frac{A_v f_y\,(0.8L_w)}{s_2}\right) \qquad (\text{§11.9.5 \& 11-29})$$

where, $0.8L_w$ represents effective depth d and A_v is horizontal reinforcement.

Special shear walls (§21.9.4)

$$\phi V_n = \phi A_{cv}\left(\alpha_c\lambda\sqrt{f'_c} + \rho_t f_y\right) = \phi A_{cv}\alpha_c\lambda\sqrt{f'_c} + \phi A_v f_y \qquad (21\text{-}7)$$

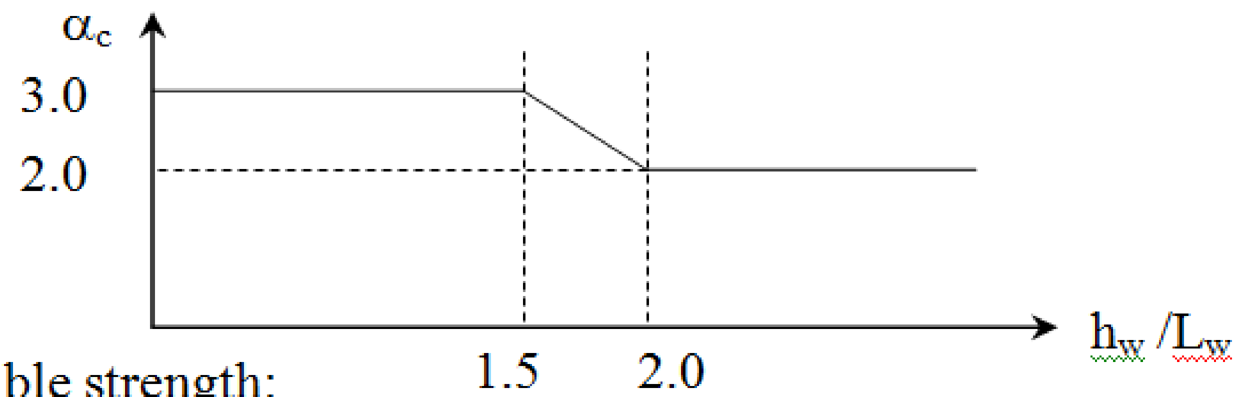
Where, A_{cv} = Concrete shear area, $L_w \times t_w$.

$\phi = 0.6$ (if certain that flexure governs, $\phi = 0.75$). §9.3.2.3 & 9.3.4

ρ_t = Horizontal shear reinforcement ratio

α_c = Factor that varies linearly between h_w/L_w of 1.5 and 2. §21.9.4.1

Shear Design of Wall



Maximum permissible strength:

$V_n \leq 10\sqrt{f'_c}\,A_{cv}$	Horizontal wall, individual piers & coupling beams.	§21.9.4.5
$V_n \leq 8\sqrt{f'_c}\,A_{cv}$	All piers sharing common lateral load.	§21.9.4.4

Shear Design of Wall

<u>Shear Capacity (ACI 21.9.4)</u>:

Wall h_w/ℓ_w = 24/10 = 2.4 > 2.0

α_c = 2.0

$$\boxed{\phi V_n = \phi A_{cv}\left(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y\right)}$$

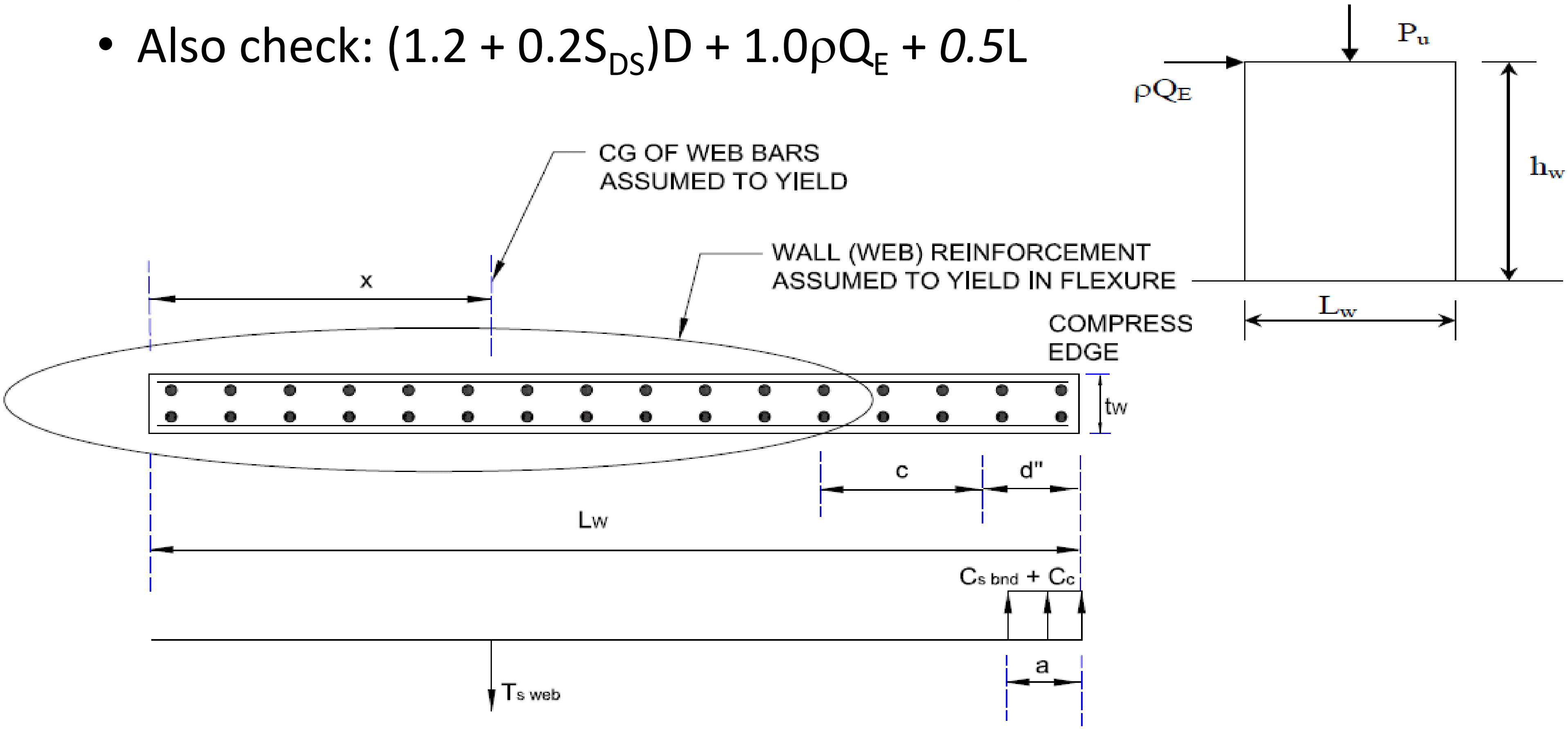
ϕV_n = 0.6(12" x 120")[2(4000)$^{0.5}$ + 0.00277 x 60,000]/1000

ϕV_n = 253.3k > 110k => OK

Use #4 @ 12"o/c Each Face of Wall

Flexural Design of Wall

- Axial load contributes towards flexural capacity
- Controlling load combo (typ): $(0.9 - 0.2S_{DS})D + 1.0\rho Q_E$
- Also check: $(1.2 + 0.2S_{DS})D + 1.0\rho Q_E + \mathit{0.5}L$



Flexural Design of Wall

- Assume all vertical wall (A_{sw}) steel yields (the two final bars at the compression edge could be neglected)
- Compute depth of compression block

$$a = \frac{A_{sw} f_y + P_u}{0.85 f_c' t_w}$$

- Generally walls are tension controlled elements, (i.e., $\varepsilon_t \geq 0.005$ at $\varepsilon_{cu} = 0.003$ and per ACI 9.3.21 & 10.3.4) $\phi = 0.9$ can be assumed

Flexural Design of Wall

$$\phi M_{n\ web+P_u} = \phi\left[\left(A_{s\,w} f_y\right) + P_u\right]\left(x - \frac{a}{2}\right)$$

Where x = $L_W/2$ if all the web steel is assumed to yield.

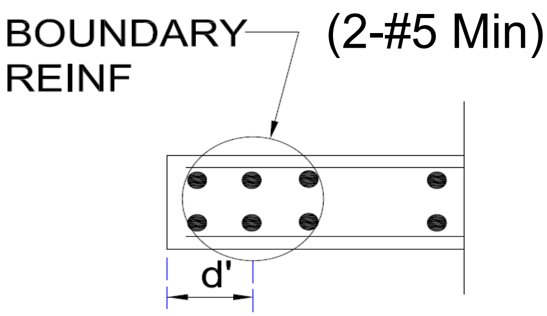
$\phi M_{n\ web+Pu} \geq M_u$ => Wall flexural capacity does not require boundary reinforcement.

$\phi M_{n\ web+Pu} \leq M_u$ => Boundary reinforcement required.

$$\phi M_{n\ bnd} = M_u - \phi M_{n\ web+Pu}$$

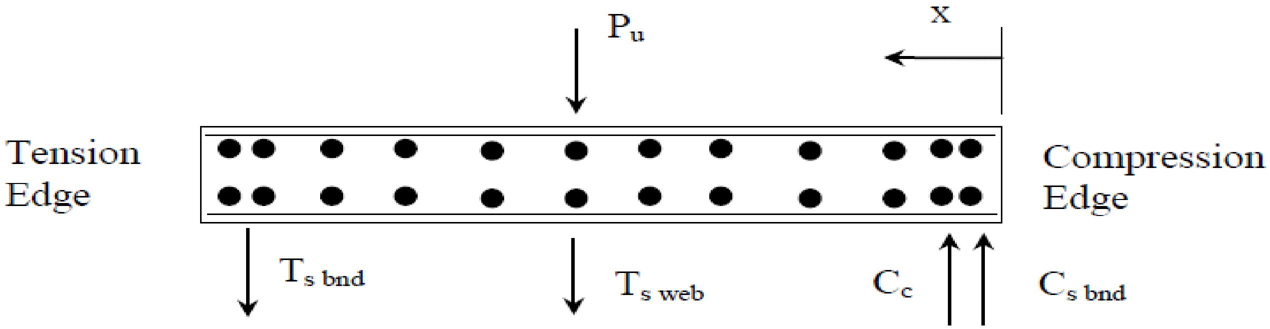
Assume d'.

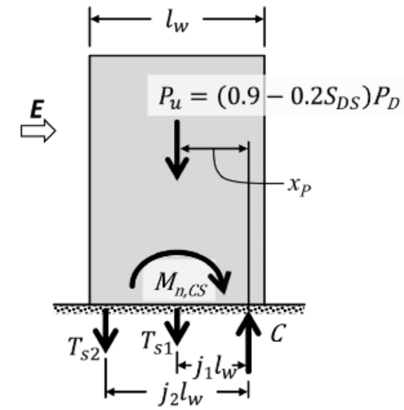
$$A_{s\ bnd} = \frac{M_{n\ bnd}}{f_y\left(L_w - 2d'\right)}$$



Note: Boundary element (i.e., edge confinement requirement is checked later)

Flexural Design of Wall





(Ref 4)

Force Component	A_s	C or T (C –ve, T +ve)	x	C*x or T*x
$C_{s\ bnd}$	$A_{s\ bnd}$	$- A_{s\ bnd} f_y$	d'	$(- A_{s\ bnd} f_y)\ d'$
$T_{s\ web}$	A_{sw}	$A_{sw} f_y$	$L_w/2$	$(A_{sw} f_y)\ (L_w/2)$
$T_{s\ bnd}$	$A_{s\ bnd}$	$A_{s\ bnd} f_y$	$L_w - d'$	$(A_{s\ bnd} f_y)\ (L_w - d')$
P_u	-	P_u	$L_w/2$	$(P_u)(L_w/2)$
C_c	-	$-C_{c\ reqd}$ See Note	$a/2$	$(-C_{c\ reqd})(a/2)$
				$\Sigma = M_n$

Note: 1. $C_{c\ reqd} = (-C_{s\ bnd} + T_{s\ web} + T_{s\ bnd} + P_u)$
2. Check $a = C_c/0.85 f'_c t_w$ –> Depth of compression block.
3. $c = a/0.85$ –> Depth to neutral axis.

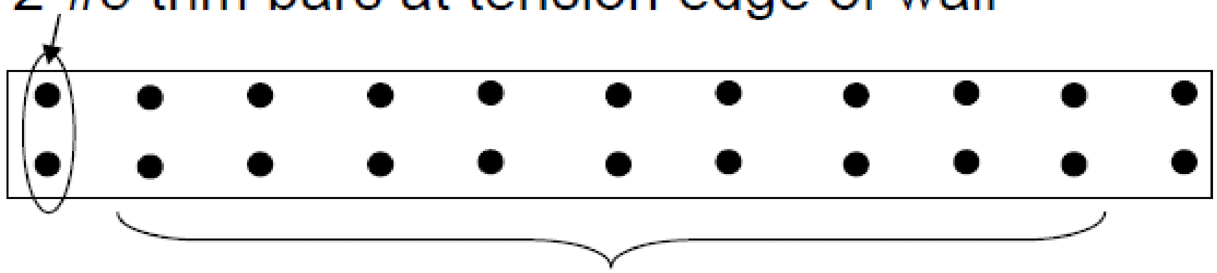
Flexural Design of Wall

Minimum Distributed Vertical Reinforcement in wall:

$\rho_\ell = 0.0025 \Rightarrow 0.0025 \times 12" \times 12" = 0.36\text{in}^2/\text{ft}$

Use #4 Bars @ 12" o/c EF = $0.20\text{in}^2 \times 2 = 0.4\text{in}^2/\text{ft}$

Also use 2-#5 trim bars at tension edge of wall



18-#4

$A_{sw} = 2 \times 0.31\text{in}^2 + 18 \times 0.20\text{in}^2 = 4.22\text{in}^2$

$a = \{4.22\text{in}^2 \times 60\text{ksi} + 28\text{k}\} / 0.85(4\text{ksi})(12") = 6.89"$

Flexural Design of Wall

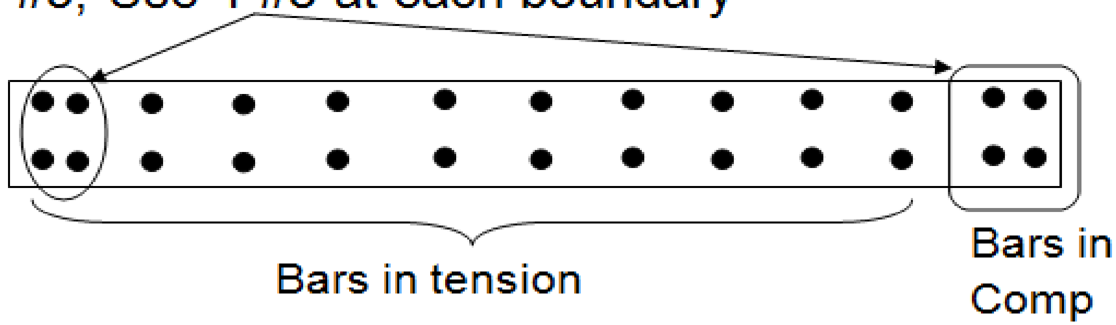
Quick check of capacity: $\phi M_n = 0.9 \times A_{sw} \times f_y \times (d - a/2)$

Assume $d = L_w / 2 = 60"$

$\phi M_n = 0.9 \times 4.22in^2 \times 60ksi \times (60" - 6.89"/2)/12 = 1{,}074k\text{-}ft$

This is less than $M_u = 2{,}112k\text{-}ft$ => Add boundary reinforcing

In lieu of 2-#5, Use 4-#8 at each boundary



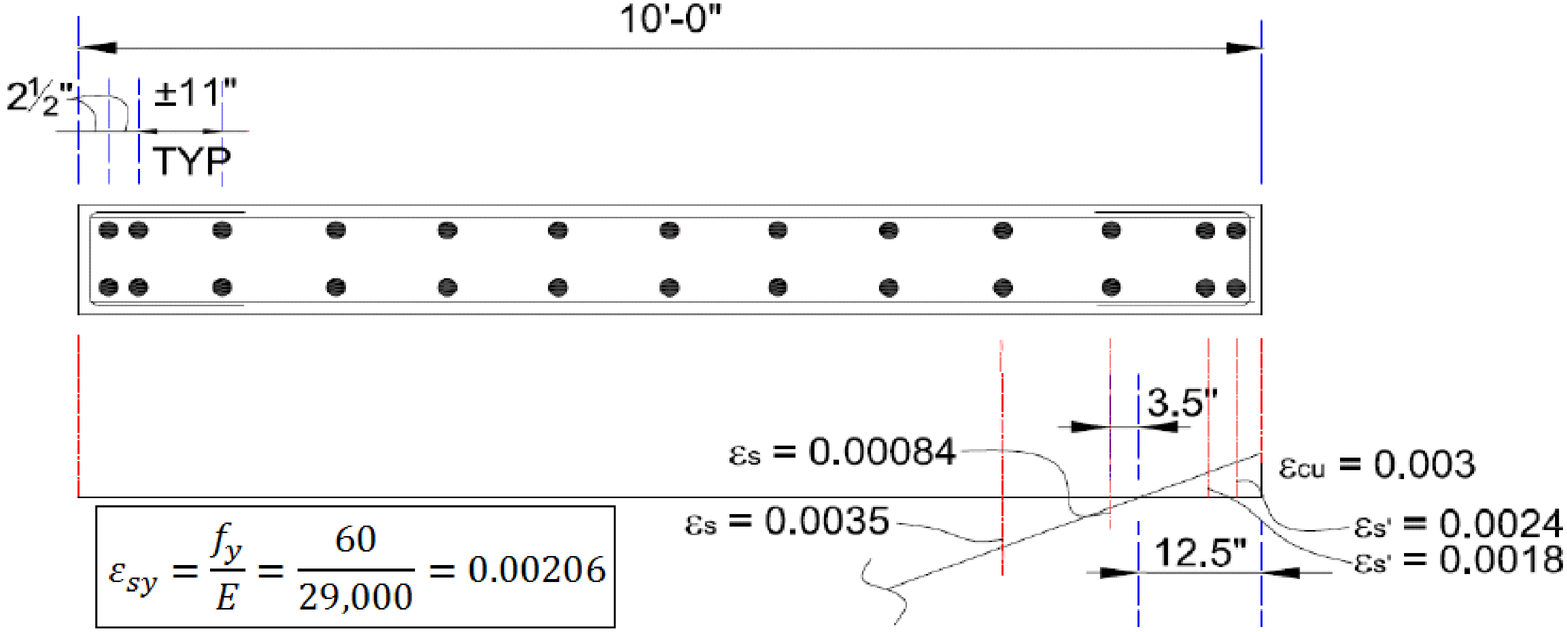
$A_{sw} = 4 \times 0.79in^2 + 18 \times 0.20in^2 = 6.76in^2$ (Tension bars only)

$a = \{6.76in^2 \times 60ksi + 28k\} / 0.85(4ksi)(12") = 10.6"$

Depth to neutral axis, $c = a/0.85 = 12.5"$

Flexural Design of Wall

Check assumption of web reinforcement yield:



$$\varepsilon_{sy} = \frac{f_y}{E} = \frac{60}{29{,}000} = 0.00206$$

Where, ε_{cu} = maximum comp stress in concrete = 0.003

ε_s = strain in first vertical bars in tension zone

= (3.5/12.5) x 0.003 = 0.00084

First bars in tension are below yield. Similarly, for first bars in compression.

Flexural Design of Wall

Stress in 1st Tension Bars = 0.00084 x 29,000ksi = 24.4ksi
Stress in 1st Comp. Bars = 0.0018 x 29,000ksi = 52.2ksi

Wall Flexural capacity:

Force Component	**A_s In^2**	**C or T kips**	**X in**	**C*x or T*x Kip-ft**
$C_{s\ bnd}$	1.58 1.58	-82.5 -94.8	5.0 2.5	-34.4 -19.8
$T_{s\ web}$	3.2 0.4	192.0 9.76	65.5 16.0	1048.0 13.01
$T_{s\ bnd}$	1.58 1.58	94.8 94.8	117.5 115.0	928.3 908.5
P_u	-	28	60.0	140.0
C_c	-	-242.1	5.3	-106.9
				2,876.7

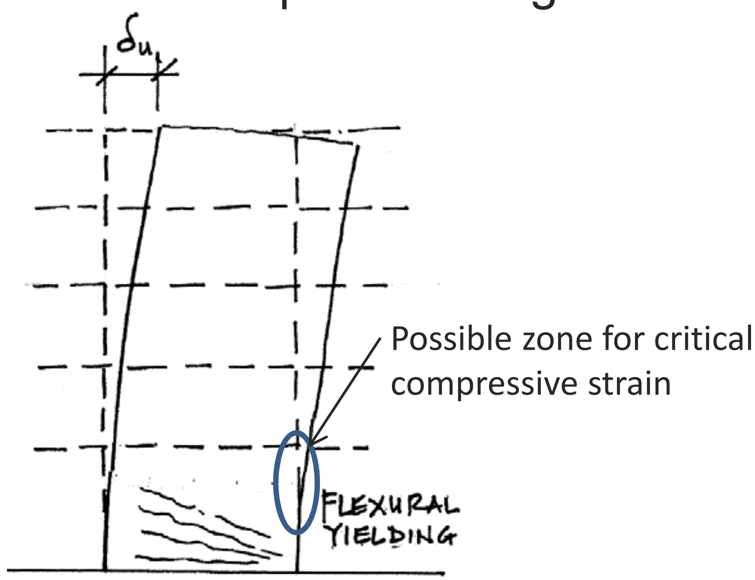
Net tensile strain in tension steel > 0.005, ϕ=0.9 OK

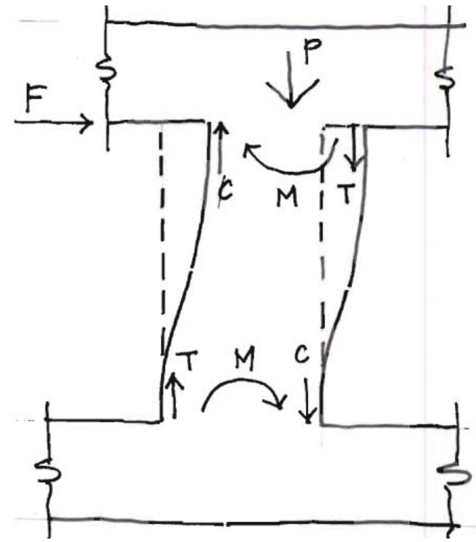
ϕM_n = 2,589k-ft > M_u Reinforcement Provided is OK

Flexural Design of Wall

Boundary Element for Special Walls

Confinement of compressed edges of walls—two approaches in ACI 318





Displacement-based design (Section 21.9.6.2)

- Maximum expected displacement, δ_u
- Critical Section with axial loads & flexural yielding
- Typically applies to cantilevered walls

Force-based design (Section 21.9.6.3)

- Apply all axial and lateral loads
- Compute maximum compressive stress (axial + flexure)
- Applicable to all walls

Flexural Design of Wall

Displacement-Based Design (21.9.6.2)

Boundary elements shall be provided where

$c \geq \ell_w / 600(\delta_u/h_w)$ Eqn 21-8, &

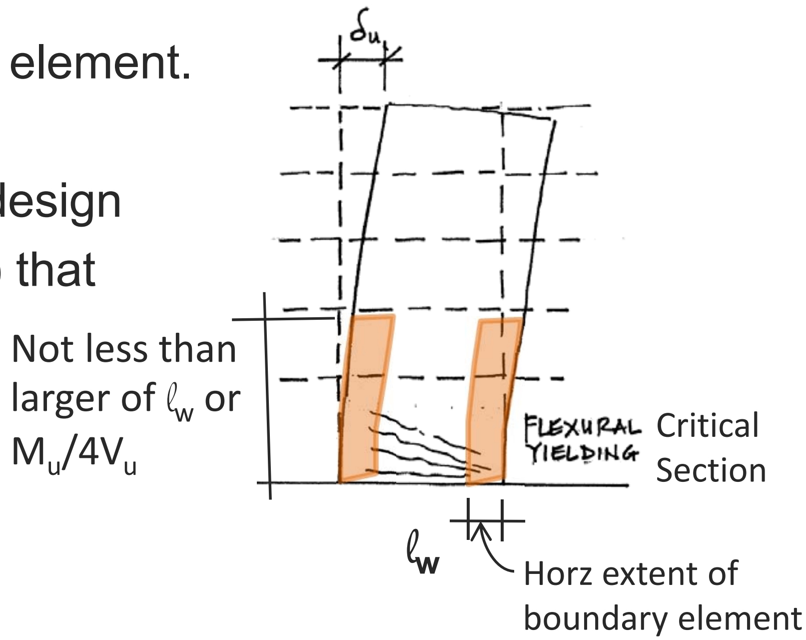
$\delta_u/h_w \geq 0.007$

For $\delta_u/h_w = 0.007$, the limit become $c \geq 0.24\,\ell_w$.

Higher the δ_u, lower the threshold for a boundary element.

Neutral axis depth shall be calculated using the design Load combination (especially factored axial load) that produces the design displacement, δ_u;

Most likely: $(1.2 + 0.2S_{DS})D + \rho Q_E + f_1 L$



Flexural Design of Wall

Boundary Elements (ACI 21.9.6): Using Displacement-Based Design:

Boundary element requirement dictated by largest depth of neutral axis.

Use Load Combo: $1.2D + 1.0(\rho Q_E + 0.2S_{DS}D) + f_1L + f_2S$

Using P_u = 76k in the earlier table,

$$C_{c\ reqd} = (-C_{s\ bnd} + T_{s\ web} + T_{s\ bnd} + P_u)$$

Then calculate, depth of compression block and depth to NA

$$a = C_c/0.85f'_c t_w \quad c = a/0.85$$

For the example: C_c = 290.1k a = 7.11” c = 8.37”

Flexural Design of Wall

Special boundary elements are required where:

$c \geq \ell_w / 600(\delta_u/h_w)$

δ_u/h_w = 6"/288" = 0.0208 ≥ 0.007 OK (ACI 21.9.6.2)
$\ell_w / 600(\delta_u/h_w)$ = 120"/600(6"/288") = 9.6"

c = 8.37" < 9.6" => Special Boundary Elems not reqd.

Check other requirements of ACI 21.9.6.5:

Reinforcement ratio at boundary,
$\rho = A_s/(a \times t_w) = 4 \times 0.79 in^2 / (7.5" \times 12") = 0.035$

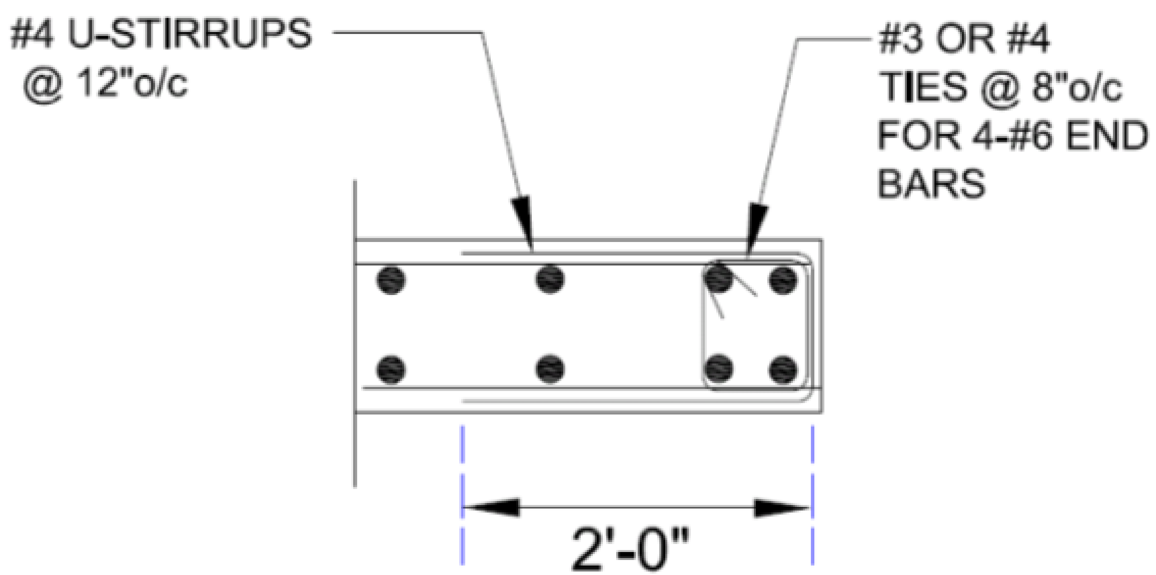
$400/f_y = 400/60{,}000 = 0.007$

Since $\rho > 400/f_y$, provide confinement to the boundary reinf.
(21.6.4.2 & 21.9.6.4(a))
Provide #3 or #4 closed ties around the 4 end bars.

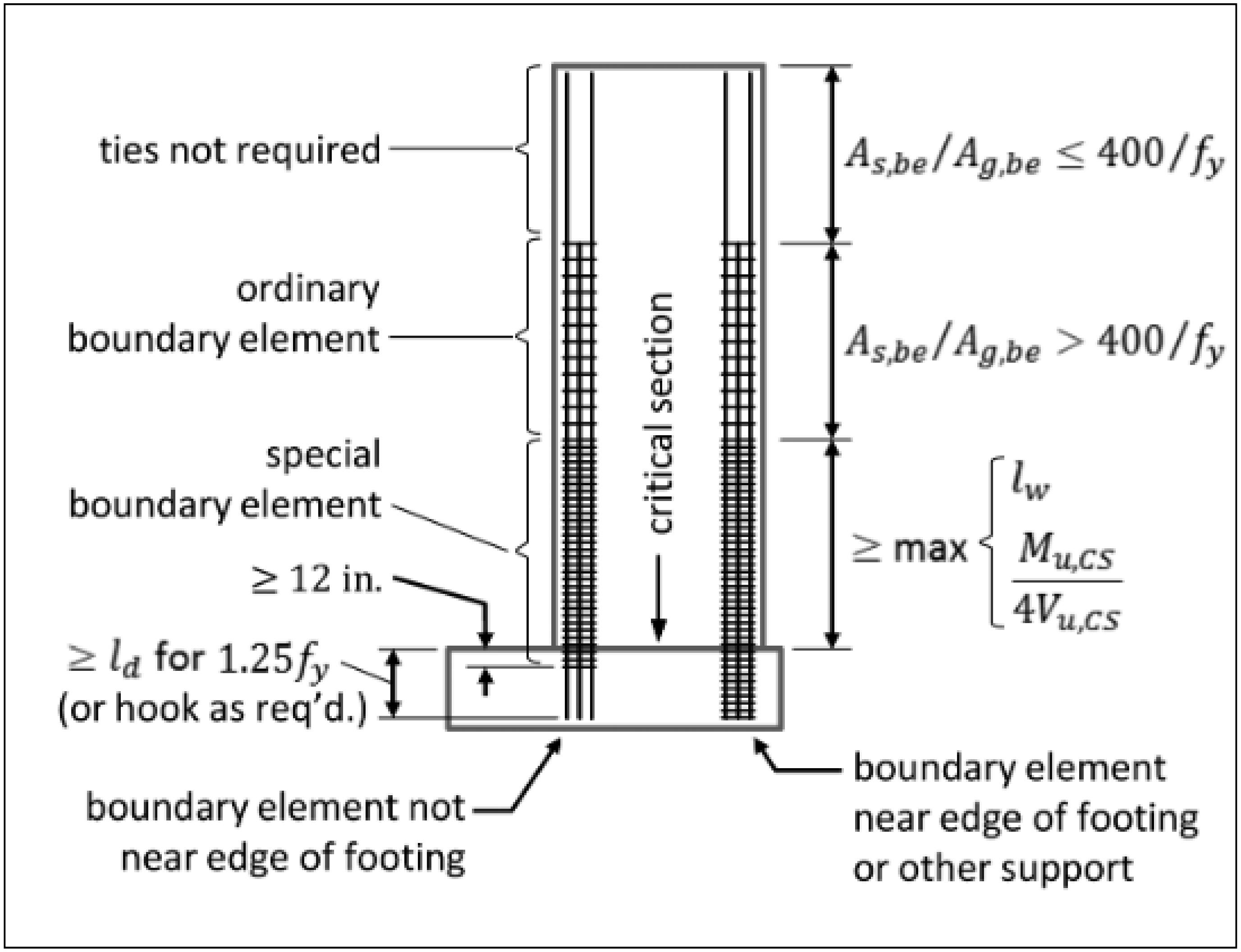
Flexural Design of Wall

Since $V_u > A_{cv}\lambda\sqrt{f_c'}$ provide one of the following:

- Standard hooks at the ends of the horizontal bars, OR
- U-stirrups of the same size & spacing and spliced to the horizontal bars.



Flexural Design of Wall



(Ref 4)

Flexural Design of Wall

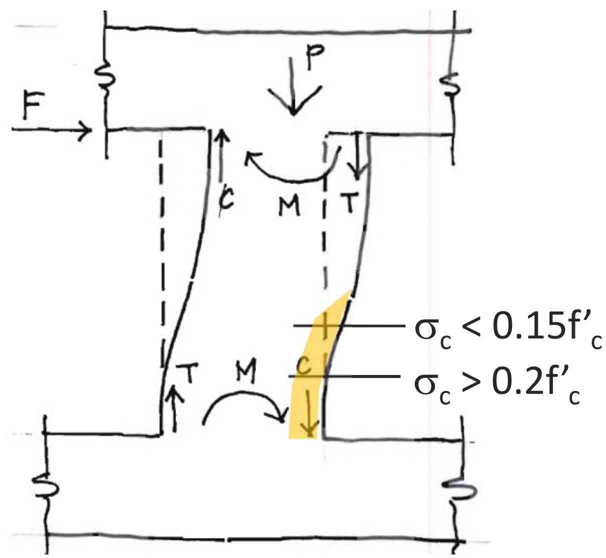
Force/Demand-Based Design (21.9.6.3):

More conservative approach.

Boundary elements shall be provided where the compressive stress due to axial and flexural demands exceeds $0.2f'_c$.

Discontinue boundary element where compressive stress
is less than $0.15f'_c$.

Stress calculated for factored loads from linear elastic analysis and gross section properties of wall



Flexural Design of Wall

Does not apply to example wall. Shown for illustration purposes only.

Using Force/Demand Based Design:

Area = 1,440in^2, I = 1,728,000in^4, c = 60in

$P_u/A + M_u c/I$ => 933psi

933psi > 0.2f'$_c$ (800psi) => Special Boundary Elems reqd.

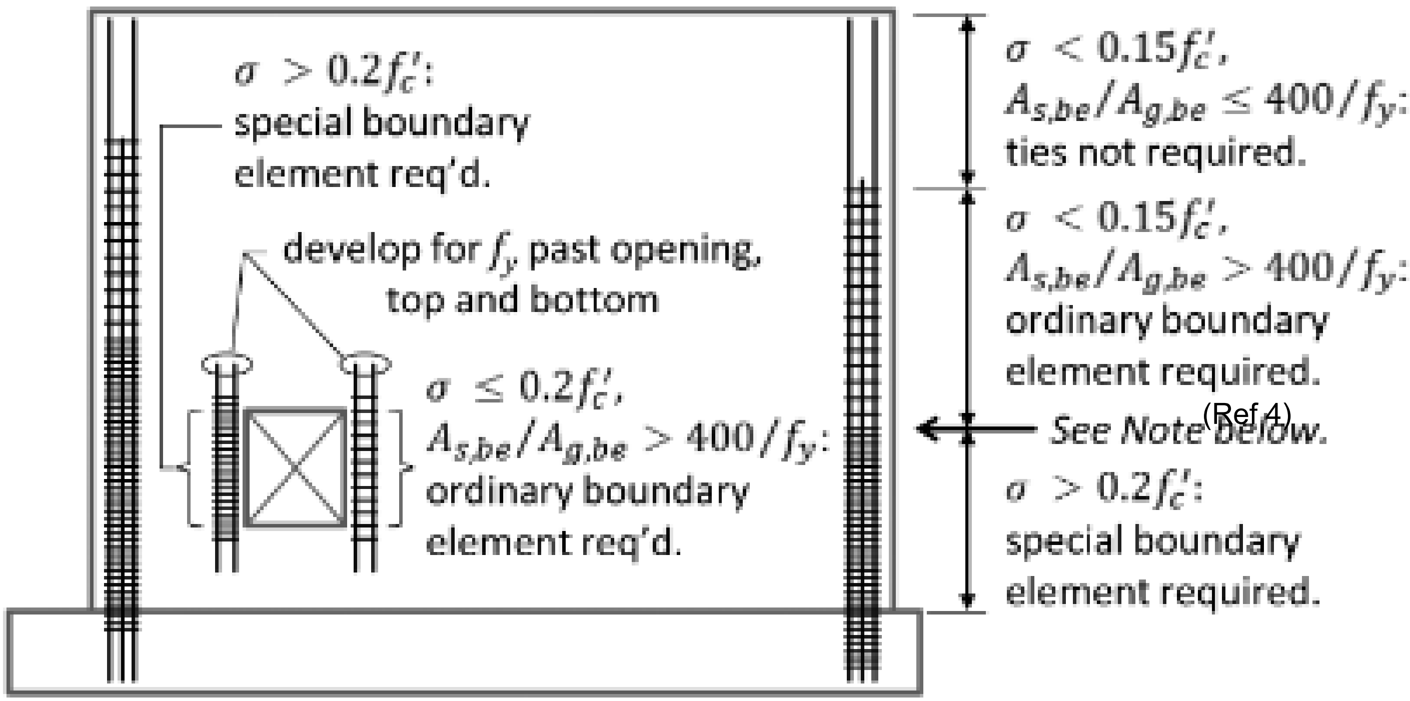
Boundary element shall extend horizontally (ACI 21.9.6.4):

$$\text{Max}\begin{cases} c - 0.1\ell_w = 8.37'' - 0.1 \times 120'' \Rightarrow \text{N.A.} \\ c/2 = 8.37''/2 = 4.18'' \end{cases}$$

Boundary element extends to the 4-#8 each end of wall.

For detailing of Boundary elements see handout.

Flexural Design of Wall



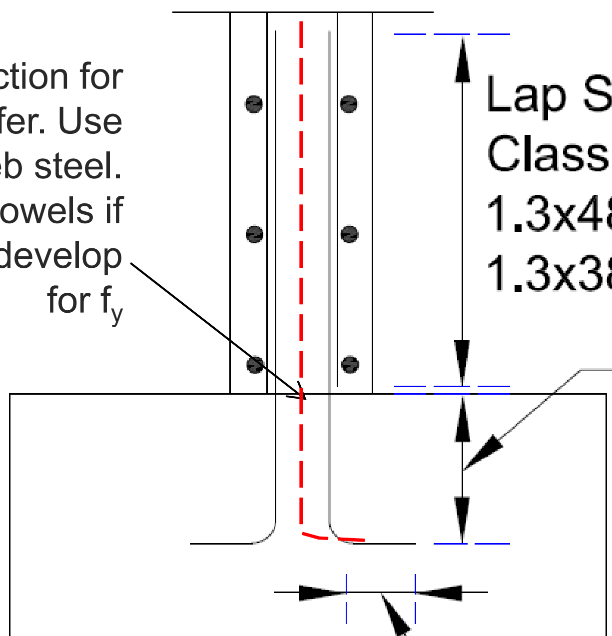
Note: Requirement for special boundary element is triggered if $\sigma > 0.2f_c'$.
Once triggered, the special boundary element extends until $\sigma < 0.15f_c'$.

Design of Wall

Other detailing considerations:

§21.9.2.3 refers to Chapter 12 for bar development & splices

§21.9.2.3(c) Where reinforcement will yield provide laps and development based on $1.25f_y$



Note: If footing is deep enough, use straight bar development

Shear Friction

Shear Friction (11.6)

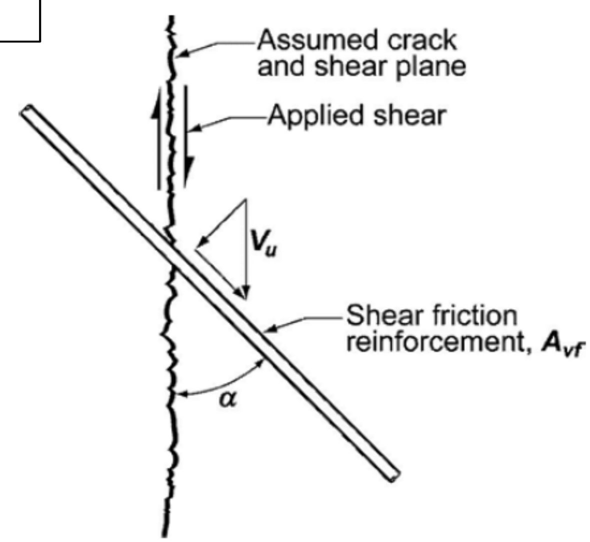
Applies where force is transferred across any shear plane (potential crack, concrete placed at different times, and concrete-to-steel interface).

$$V_n = A_{vf} f_y \mu \qquad (11\text{-}25)$$

$$V_n = A_{vf} f_y (\mu \sin \alpha + \cos \alpha) \qquad (11\text{-}26)$$

See ACI 318 11.6.5 for $V_{n\ max}$.

Contact surface condition	Coefficient of friction μ[1]
Concrete placed monolithically	1.4λ
Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in.	1.0λ
Concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened	0.6λ
Concrete placed against as-rolled structural steel that is clean, free of paint, and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.	0.7λ



Typical applications:

- Wall-to-foundation (Also see ACI 318 11.6.7)
- Slab-to-wall
- Collector-slab or collector-to-wall/frame

Moment-Resisting Frames

Selection of Frame Type

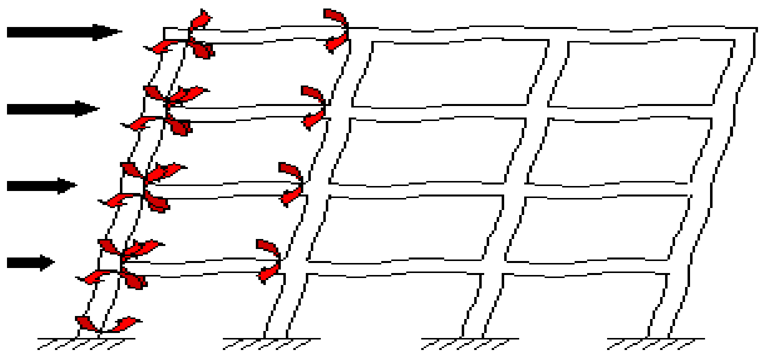
FRAME TYPE	DESIGN REQUIREMENTS	SDC
Ordinary MF	ACI 318, Chapters 1–18 & Section 21.2	A, B
Intermediate MF	ACI 318, Chapters 1–18 & Section 21.3	A, B, C
Special MF	ACI 318, Chapters 1–18 & Section 21-5 to 21-8	ALL

Refer to ASCE 7, Table 12.2-1 for frame types, R and Ω_o, height limitations, and so on.

Moment Frames

Moment frames resist lateral forces by virtue of rigid joints

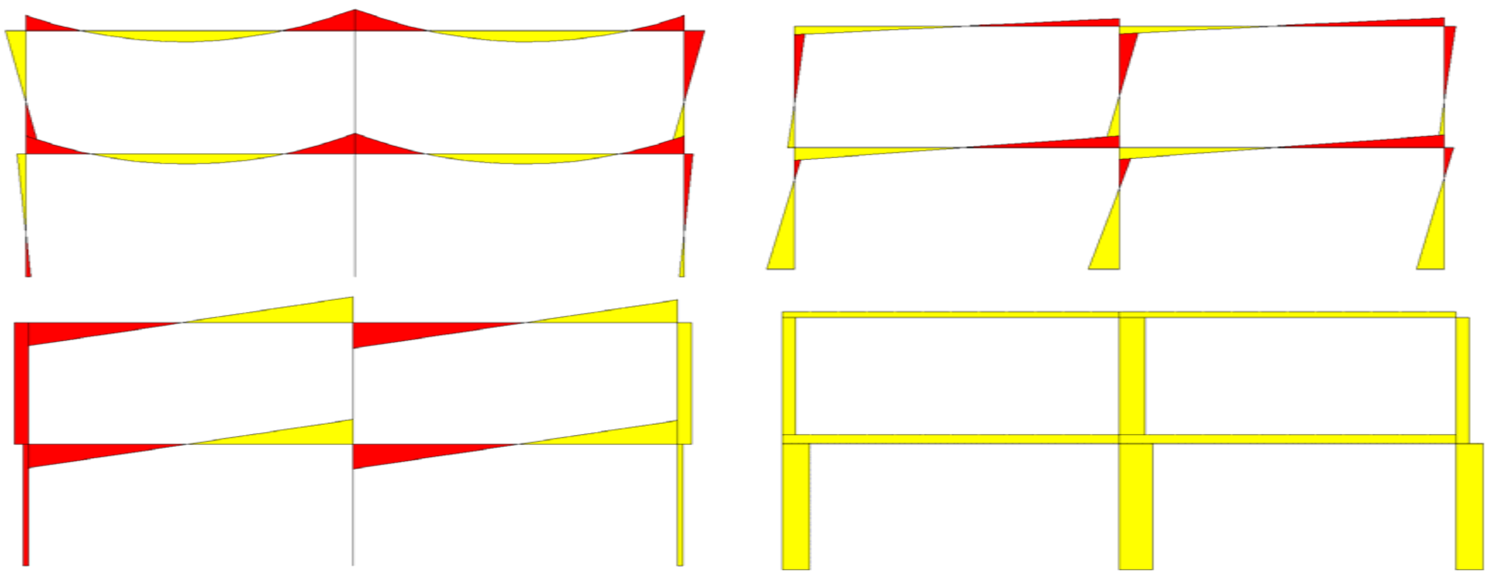
Beams and columns at the joint are subjected to moments and shears



Goal: Limit damage to frame by appropriate detailing of members and joints

Moment Frames

<u>Demands on frames:</u>



GRAVITY LOADS LATERAL LOADS

Combine with applicable load combinations for design

Ordinary Moment Frames

- Design for factored loads using provisions of Chapters 1 through 18
- No special detailing requirements, except for SDC B
  - §21.2.2: Provide at least two main continuous flexural bars top and bottom. Continue through columns or develop these bars at face of support.
  - §21.2.3: Columns of OMFs with clear height-to-maximum-plan-dimension ratio ≤ 5 shall be designed for shear per 21.3.3.2.

Part of the IMF provisions

Intermediate Moment Frames

- Design for factored loads using provisions of Chapters 1 through 18 and Section 21.3
- Shear design of beams and columns (21.3.3.1)
  - ϕV_n shall not be less than smaller of
    - shear at nominal moment capacity and shear due to factored gravity loads, or
    - shear from load combinations using two times the code prescribed seismic loads.
- Beams (21.3.4)
  - Lower limits on beam flexural strength

Intermediate Moment Frames

- Hoops provided over 2 times beam depth from face of support with spacing being the smallest of d/4, 8 times the smallest longitudinal bar diameter, 24 times diameter of hoops, or 12 inches
- Stirrup spacing along beam length ≤ d/2

- Columns (21.3.5)
  - Columns spirally reinforced per 7.10.4, OR
  - Hoops provided over length l_o from joint face where ℓ_o shall be the largest of one-sixth column clear span, maximum dimension of column or 18 inches

Intermediate Moment Frames

- Spacing of hoops, s_o, shall not exceed smallest of 8 times the smallest longitudinal bar diameter, 24 times diameter of hoops, half the smallest column dimension, or 12 inches
- First hoop shall be at $s_o/2$ from joint face
- Other joint reinforcement and slab force transfer requirements

Intermediate moment frame detailing is intended to reduce the risk of shear failure in an earthquake.

Special Moment Frames

- Intended for enhanced ductile behavior under lateral load reversals (i.e., strong-column/weak-beam, confinement of plastic hinges, and prevention of shear failure)
- Beams

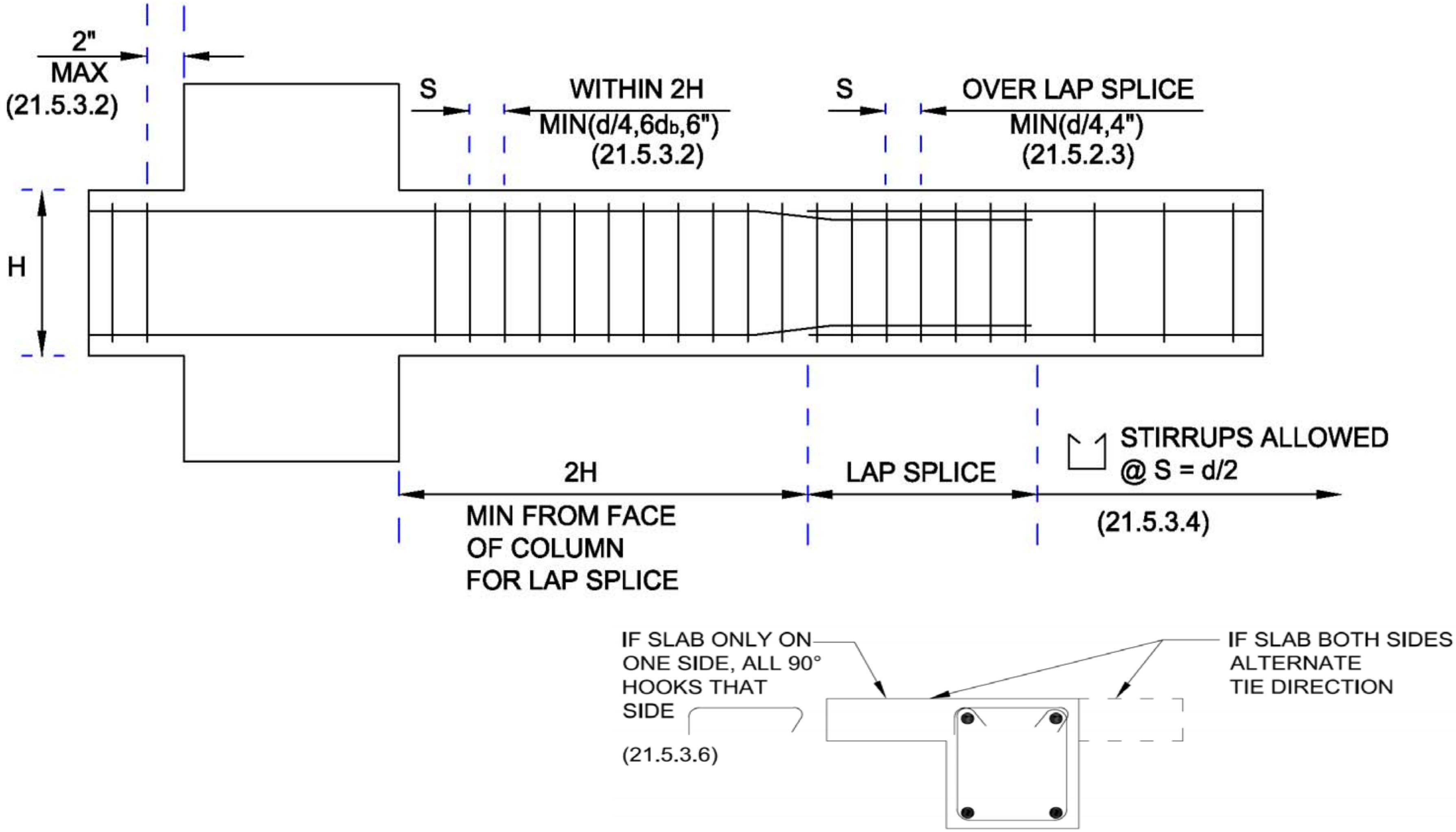
$$A_{s\ top} \text{ and } A_{s\ bot} \geq \frac{3\sqrt{f'_c}}{f_y} b_w d \text{ and } \geq \frac{200 b_w d}{f_y} \qquad \S 10.5 \,\&\, 21.5.2.1$$

$$\rho_{\max} = 0.025, \text{ where } \rho = \frac{A_s}{b_w d}$$

- Lap splices permitted only if confined over full length by hoop or spiral reinforcement (see figure below). No lap splices within joints, within 2x depth from face of support or locations of flexural yielding (§21.5.2.3)

Special Moment Frames

Beam Detailing



Special Moment Frames

Beams

- Beam shear demand, $V_e = \frac{(M_{prA} + M_{prB})}{L_{clr}} + \frac{w_g L_{clr}}{2}$ §21.5.4.1

Where, M_{prA} & M_{prB} = Moment capacities @ beam ends using $1.25f_y$ & ϕ=1.0
W_g = Factored gravity load
L_{clr} = Clear span

M_{pr} = Probable moment strength of section, §21.6.5

- If $V_e \geq \frac{V_u}{2}$ & $P_u < 0.05 A_g f'_c$ §21.5.4.2

V_e = seismic shear demand
Assume $V_c = 0$ & design stirrups to carry entire shear demand, V_e within a distance of 2H (see 21.5.4.2(b) & 21.5.3.1).

Special Moment Frames

- Columns
- Flexural strength of columns shall satisfy (21.6.2.2) (Eqn 21-1)

$$\Sigma M_{nc} \geq \frac{6}{5} \Sigma M_{nb}$$

Where, $\Sigma M_{nc} =$ Sum of nominal flexural strengths of the columns

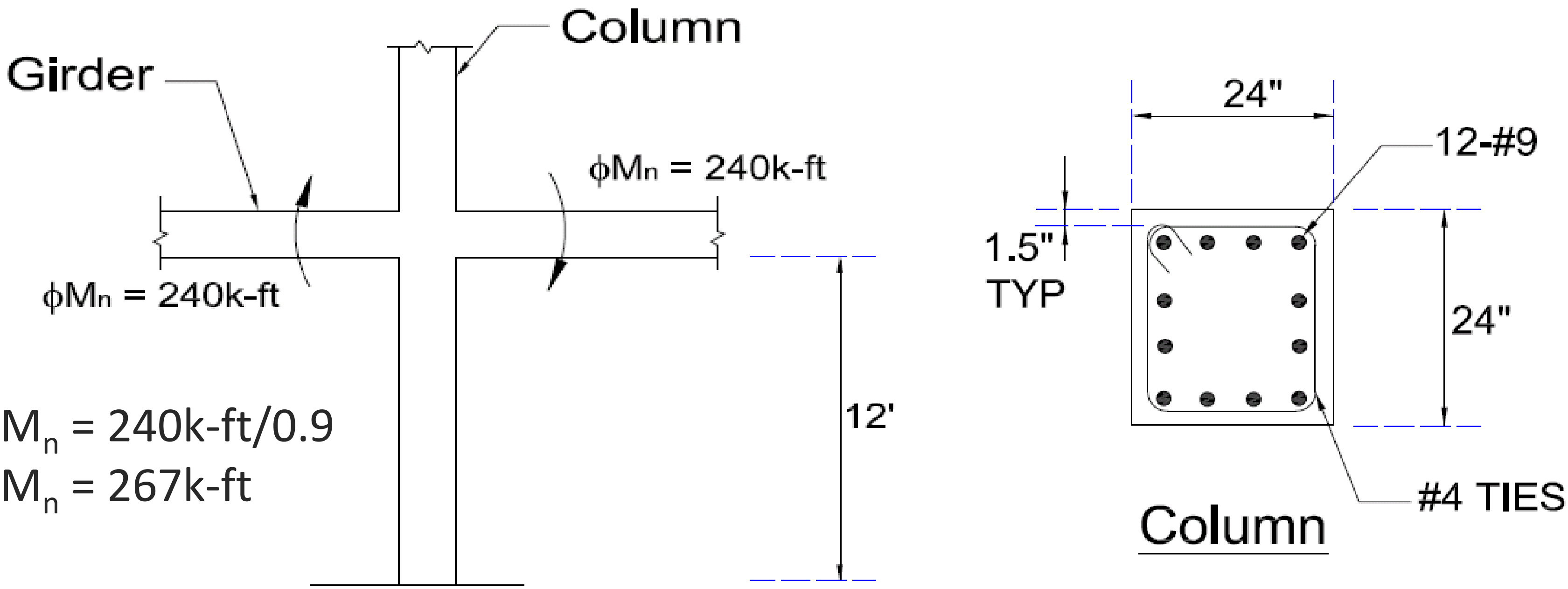
$\Sigma M_{nb} =$ Sum of nominal flexural strengths of the girders

- Longitudinal reinforcement (21.6.3)

$0.01 \leq \rho_g \leq 0.06$ (Applies to splice as well)

Lap splices only within center half of column

Special Moment Frames



M_n = 240k-ft/0.9
M_n = 267k-ft

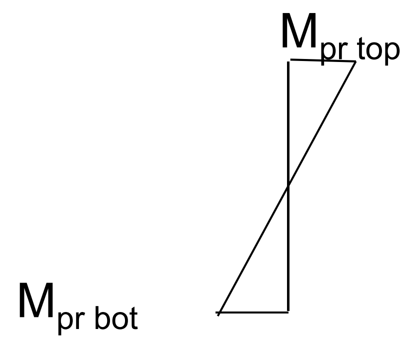
Minimum column flexural strength to comply with Eqn 21-1

$$\Sigma M_{nc} \geq \frac{6}{5} \Sigma M_{nb} \Rightarrow M_{nc} = 1.2(267 + 267) / 2 = 320\text{k-ft}$$

Assume nominal moment capacity of column, M_n = 400k-ft

Special Moment Frames

If flexural strength of column is less than 320k-ft, provide confinement transverse reinforcement over full height of column.



<u>Compute column design shear, V_e (ACI 21.5.4)</u>:

V_e based on column M_{pr}

$$V_e = \frac{(M_{pr\ top} + M_{pr\ bot})}{H} = (400 \times 1.25) \times 2 / 12' = 83.3\text{k}$$

(Use of $1.25f_y$ to compute M_{pr} is per R21.5.4.1)

Special Moment Frames

V_e need not exceed shear based on girder M_{pr}

The beam nominal moment can be distributed equally to the column above and below the joint.

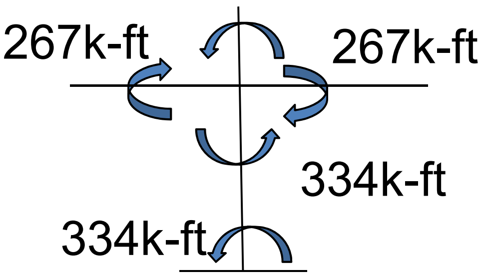
Col moment below joint => $M_{c\ top}$ = 2 × 267k-ft × 1.25 / 2 = 334k-ft

Assume same moment transferred to column base ($M_{c\ bot}$).

$$V_e = \frac{(M_{c\ top} + M_{c\ bot})}{H}$$

= 334 × 2 / 12 ft

= 55.7k => Controls



Special Moment Frames

Shear capacity of concrete is to be neglected (i.e., $V_c = 0$) when:

1. $V_e > 0.5V_u$
2. $P_u < A_g f'_c/20$

If V_c can be used, it can be computed as

$$\phi V_c = \phi\left(2\sqrt{f'_c}\, b_w d\right)$$ where, $\phi = 0.75$ (ACI 9.3.4)

Shear reinforcement can be designed per ACI 11.4.7.2.

Spacing of ties shall be minimum of $6d_b$ or 6 inches (ACI 21.6.4.5).

For confinement of plastic hinges other transverse reinforcement requirements apply.

Special Moment Frames

Column hinge confinement and main rebar support

- Transverse reinforcement (21.6.4.4)

Spirals: Volumetric ratio shall be maximum of:

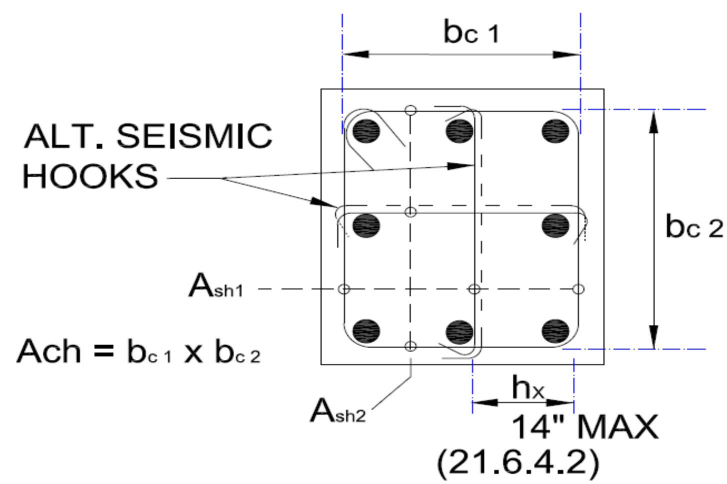
$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}} \qquad (10\text{-}5)$$

$$\rho_s = 0.12\left(\frac{f'_c}{f_{yt}}\right) \qquad (21\text{-}3)$$

Hoops: Total area shall be maximum of:

$$A_{sh} = \left(0.3 s b_c \frac{f'_c}{f_{yt}}\right)\left(\frac{A_g}{A_{ch}} - 1\right) \qquad (21\text{-}4)$$

$$A_{sh} = \left(0.09 s b_c \frac{f'_c}{f_{yt}}\right) \qquad (21\text{-}5)$$



Special Moment Frames

§21.6.4.1 The ties/spirals computed above are required within L_o as shown in the Figure:

$$L_o \text{ is the greater of } \begin{cases} h \\ H/6 \\ 18'' \end{cases}$$

§21.6.4.3 Spacing of transverse reinforcement within L_o shall be the minimum of:

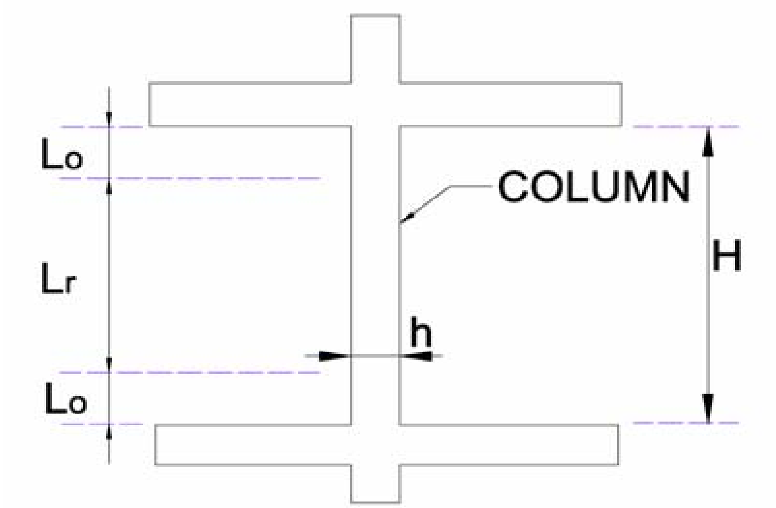
- $h/4$
- $6d_b$
- $s_o = 4 + \left(\dfrac{14 - h_x}{3}\right)$

$s_o \le 6$ in. and need not be less than 4 in.

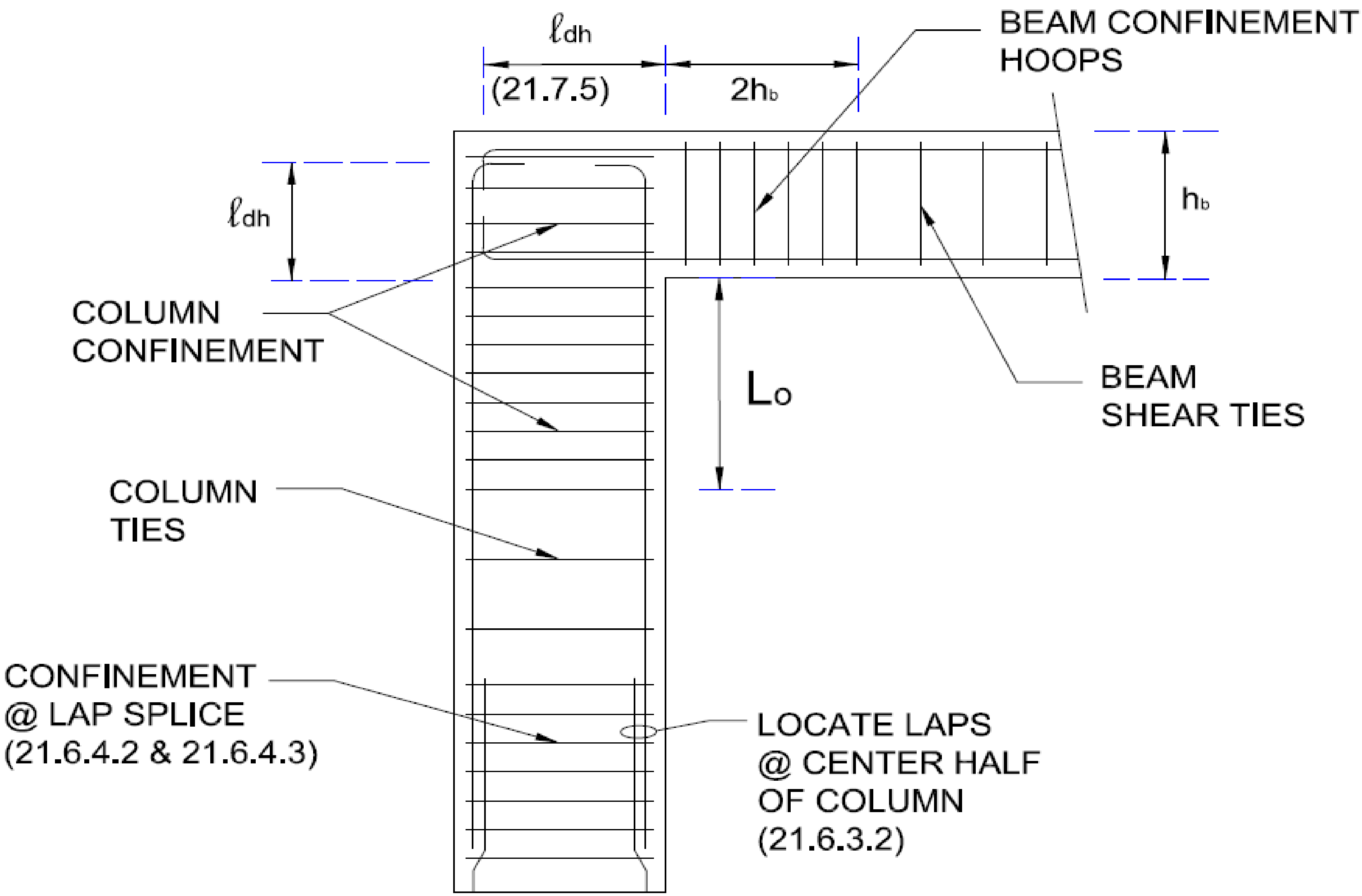
where, h_x = horizontal spacing of crossties or legs of overlapping hoops (14 in. max).

Two zones are defined for column lateral reinforcement:

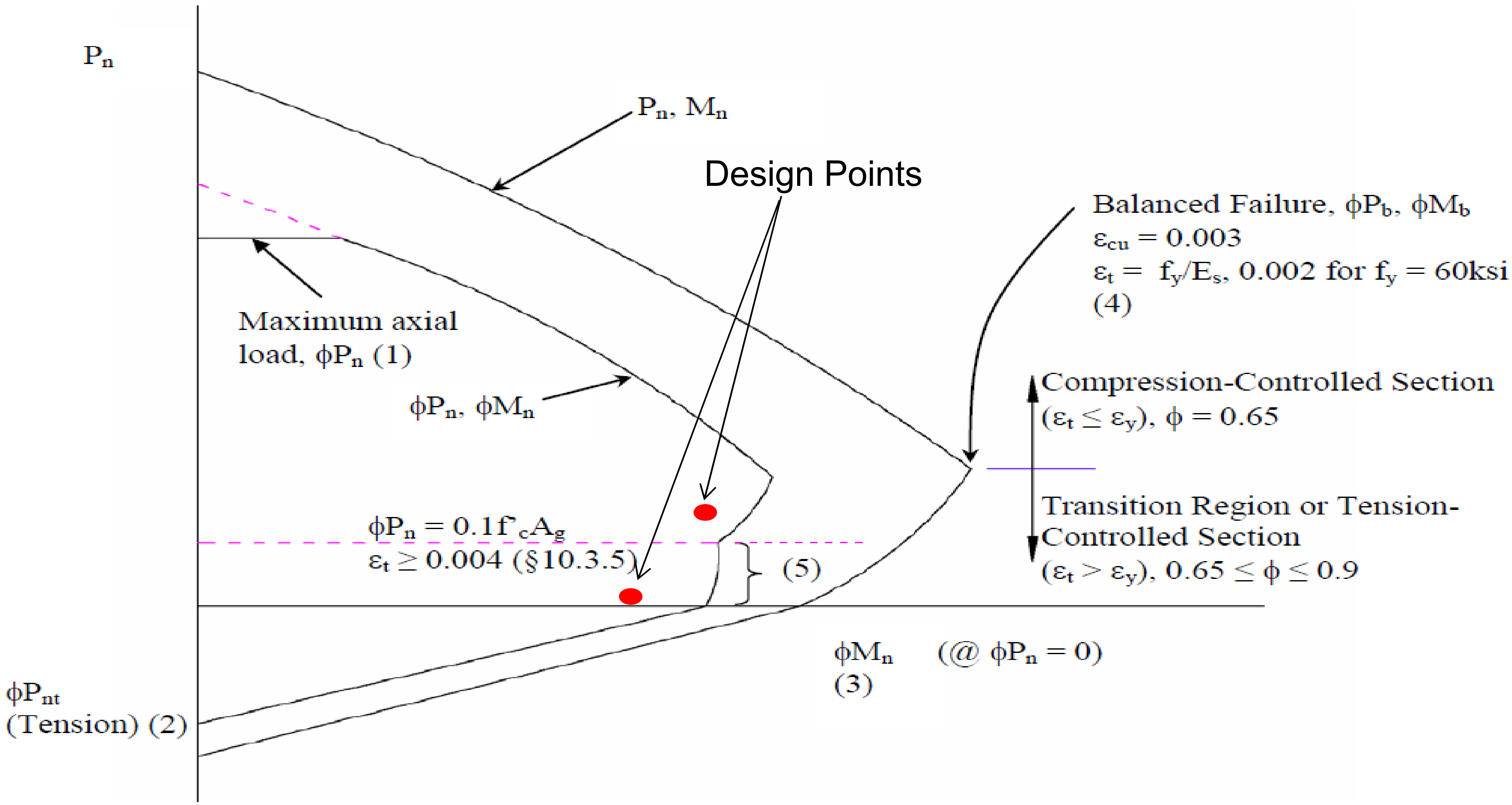
- L_o for confinement reinforcement
- L_r for ties based on shear demand



Special Moment Frames



Column P-M Interaction



Tilt-Up Walls

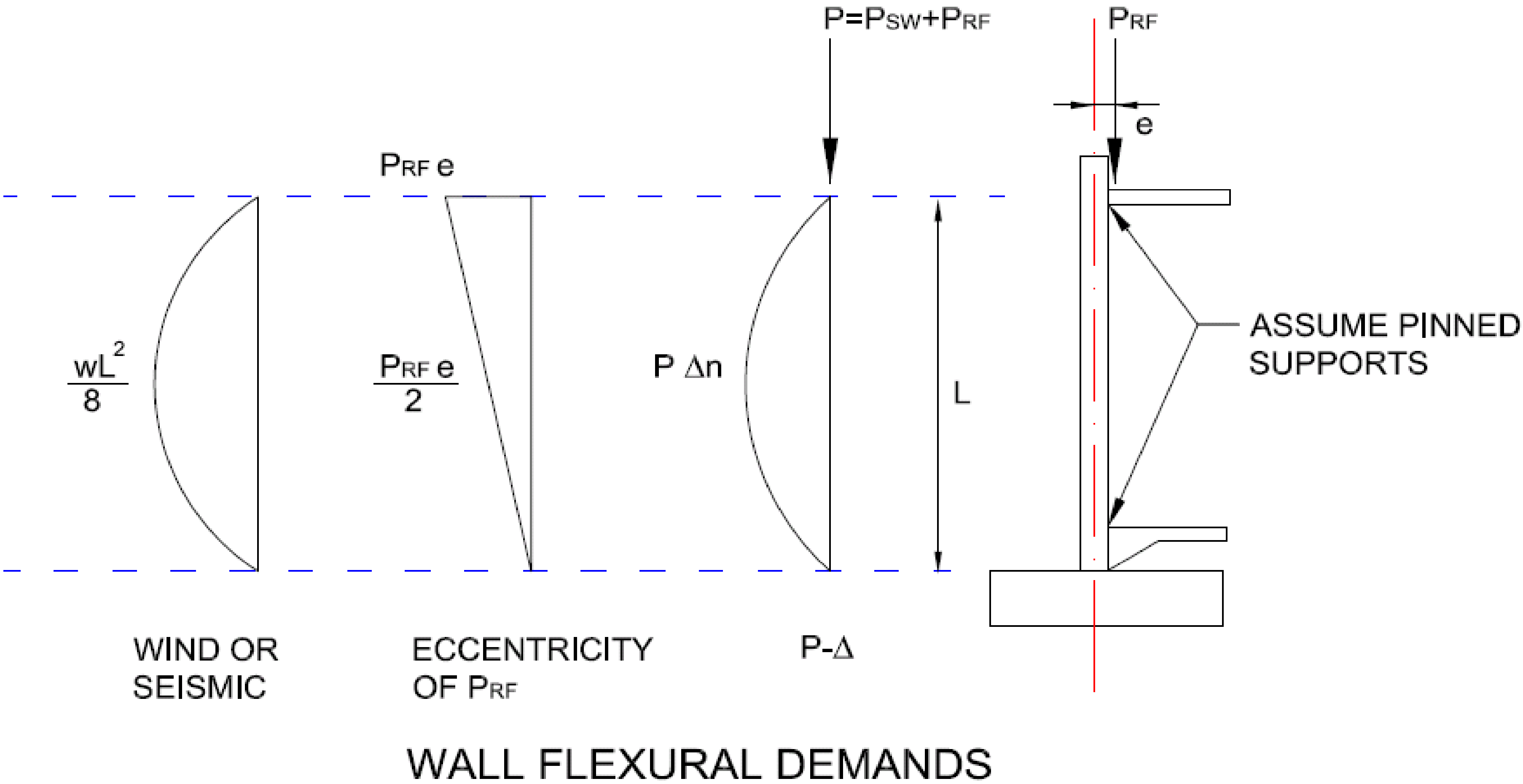
Tilt-Up Walls

- Tilt-up and precast walls are designed similar to cast-in-place concrete walls for in-plane seismic forces
- For intermediate and special walls, yielding at the connections between walls or between walls and foundations shall be restricted to steel elements (plates, etc.) or reinforcement (ACI 21.4.2)
- Elements of the connection not designed to yield shall develop at least 1½ times the yield strength of the connection (ACI 21.4.3)

Tilt-Up Walls

- Tilt-up walls tend to be slender and need to be designed for out-of-plane forces
- Wall thickness limits
  - Bearing walls: $t_w \geq$ max (supported length/25, 4 inches)
  - Non-bearing walls: $t_w \geq$ max (supported length/30, 4 inches)
- A typical slender wall has to resist combined axial and out-of-plane flexural demands
- ACI Section 14.8 (Alternative Design of Slender Walls) provisions can be used

Tilt-Up Walls



Tilt-Up Walls

Wall seismic load

For Structural Walls (ASCE 7 §12.11.1):

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

$$F_p \geq 0.2 k_a I_e W_p$$

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

k_a = amplification factor for diaphragm flexibility
L_f = Span of flexible diaphragm providing support to the wall between lateral system elements; Use zero for rigid diaphragms.

Tilt-Up Walls

Wall seismic load

For Non-Structural Walls (ASCE 7 §13.3):

$$F_p = \frac{0.4 a_p S_{DS} I_p}{R_p}\left(1 + 2\frac{z}{h_r}\right)W_p \qquad (13.3\text{-}1)$$

$$F_p \leq 1.6 S_{DS} I_p W_p \qquad \& \qquad F_p \geq 0.3 S_{DS} I_p W_p \qquad (13.3\text{-}2 \ \& \ 13.3\text{-}3)$$

Per Table 13.5-1, $\boldsymbol{a_p = 1.0}$ & $\boldsymbol{R_p = 2.5}$.

Compute F_p at $z = 0$ and $z = h_r$ and use average or use $z = h_r/2$

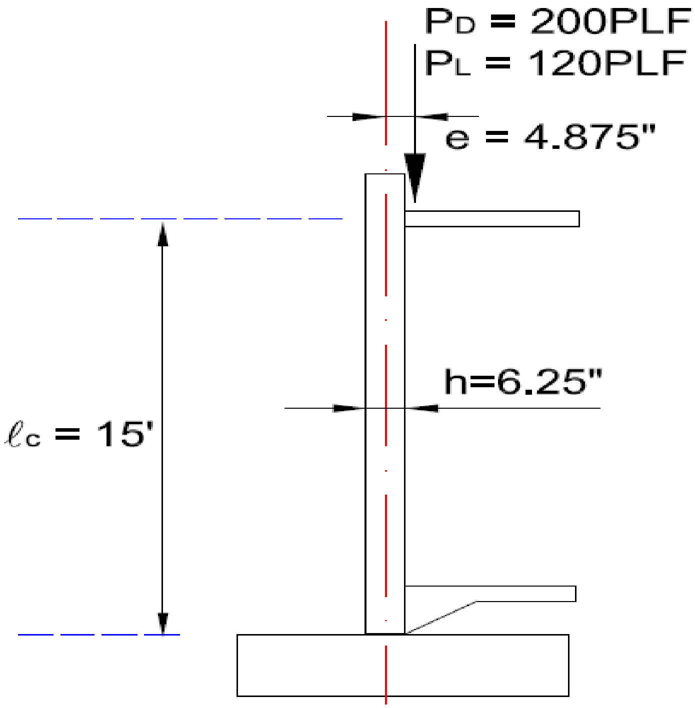
Tilt-Up Walls

Consider the 6.25-inch thick bearing wall shown

S_{DS} = 0.9g, I = 1.0, ρ = 1.0

f'_c = 3,000psi, f_y = 60ksi

E_s = 29,000ksi, E_c = 3,100ksi



Load combination

$$(1.2+0.2S_{DS})D + 1.0E_h + f_1L$$

Axial load at mid-height, $P_{u\ total}$

$P_{u\ total}$= (1.2 + 0.18)(200 + 78psf × 7.5 ft) + 0.5(120) = 1,144plf

Tilt-Up Walls

Seismic out-of-plane force

F_p = 0.4(0.9)(1.0)(78psf) = 28psf (>0.1W_p)

Moment at wall mid-height (per foot width)

Factored roof load = (1.2 + 0.18)(200) + 0.5(120) = 336plf

$M_{ua} = F_p L^2/8 + P_u R_{fe}/2$

M_{ua} = 28 × 15^2/8 + 336×4.875 in./12in / 2 = 858.8lb-ft = 10.3k-in

Per ACI 14.8.3, $M_u = M_{ua} + P_{u\,total}\,\Delta_u$

Tilt-Up Walls

M_u can be obtained from iteration of deflections or from the direct calculation (Eqn 14-5)

$$M_u = \frac{M_{ua}}{1 - \frac{5 P_{u\ total} l_c^2}{(0.75) 48 E_c I_{cr}}}$$

To calculate I_{cr}, the section reinforcement has to be known. This can be done by assuming $M_u \approx 1.2 M_{ua}$

Knowing A_{se} and depth to the neutral axis, c

$$A_{se} = \left(A_s + \frac{P_u}{f_y} \frac{h}{2d} \right)$$

$$I_{cr} = \frac{E_s}{E_c} A_{se} (d - c)^2 + \frac{l_w c^3}{3}$$

For single reinforcement curtain: d = h/2
For uniform wall load: l_w = 12 inches

Or assume I_{cr} = 25%I_g (Clearly state all assumptions!)

Tilt-Up Walls

If the wall has #4 at 10 inches o/c vertical reinforcement at center

A_{se} = As + Pu $_{total}/f_y$ = 0.24in^2 + 1.144k/60ksi

A_{se} = 0.259in^2 (per foot of wall)

a = 0.51 in., c = 0.6 in., I_{cr} = 16.31in^4 => $M_u = M_{ua}/0.898$ = 11.5k-in/foot of wall

Assuming I_{cr} = 25% of $I_g = 0.25 \times 12 \times 6.25^3/12$ = 61.03in^4 => $M_u = M_{ua}/0.97$ = 10.6k-in/foot of wall

The P-Δ effect adds 10–20% to M_u

Other checks like service deflection ≤ ℓ /150 ensure limit to P-Δ

Check maximum axial stress in wall

P_u/A_g = 1,144lb/(6.25 in. × 12 in.) = 15.2psi (< 0.06f'$_c$, satisfies ACI 14.8.2.6)

Diaphragms

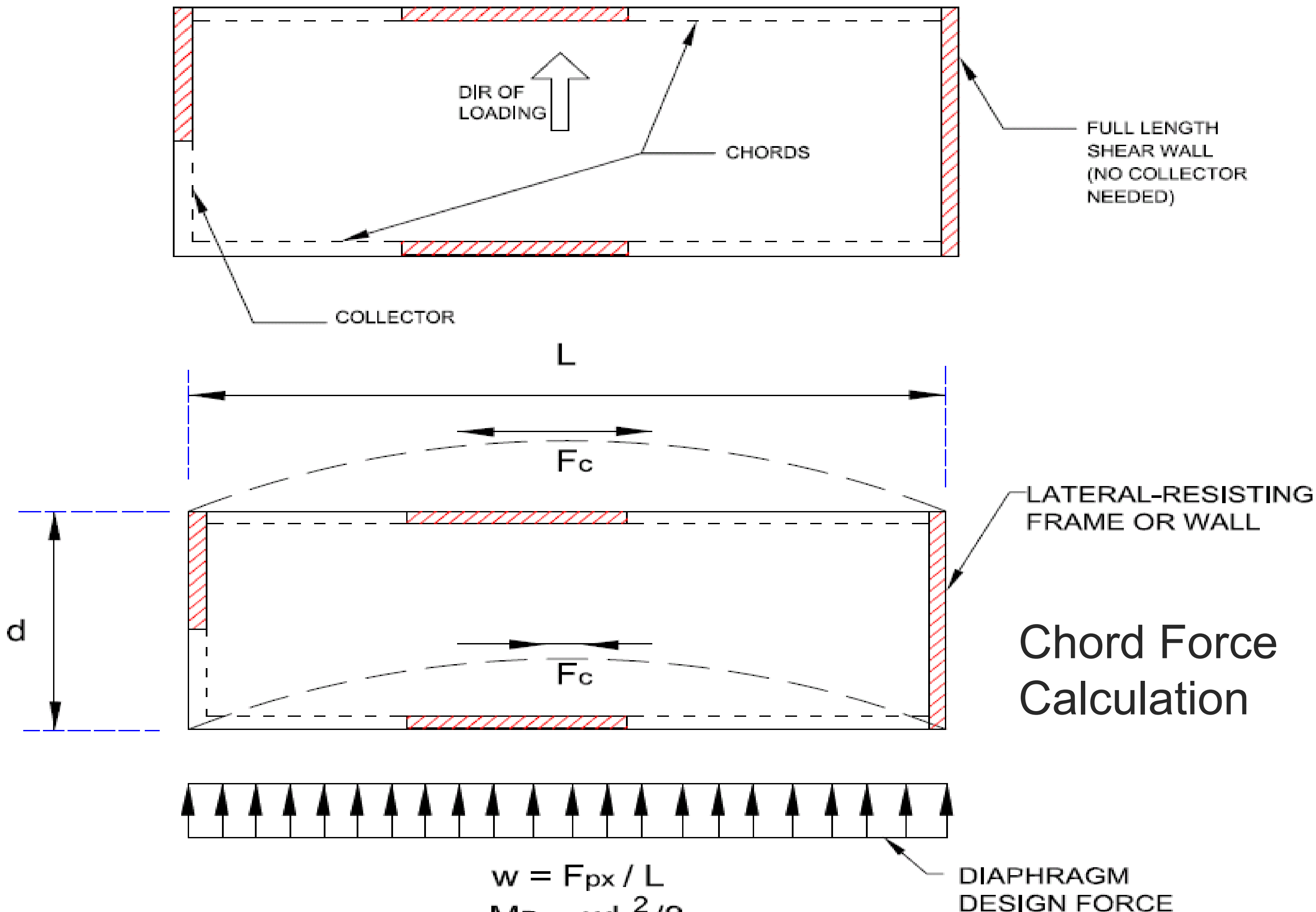
Diaphragms

- Diaphragms transfer floor inertial loads and other demands to the lateral resisting system
- For SDC A, B, and C, diaphragms are designed to ACI Chapters 1 through 18
- For SDC D, E, F also use sections 21.11
- Diaphragm design demands shall be per ASCE 7, 12.10
- In SDC C, D, E, and F, collectors, splices, and connections shall use special load combinations

$$1.\ (1.2 + 0.2S_{DS})D + \Omega_O Q_E + f_1 L$$

$$2.\ (0.9 - 0.2S_{DS})D \pm \Omega_O Q_E$$

Diaphragms



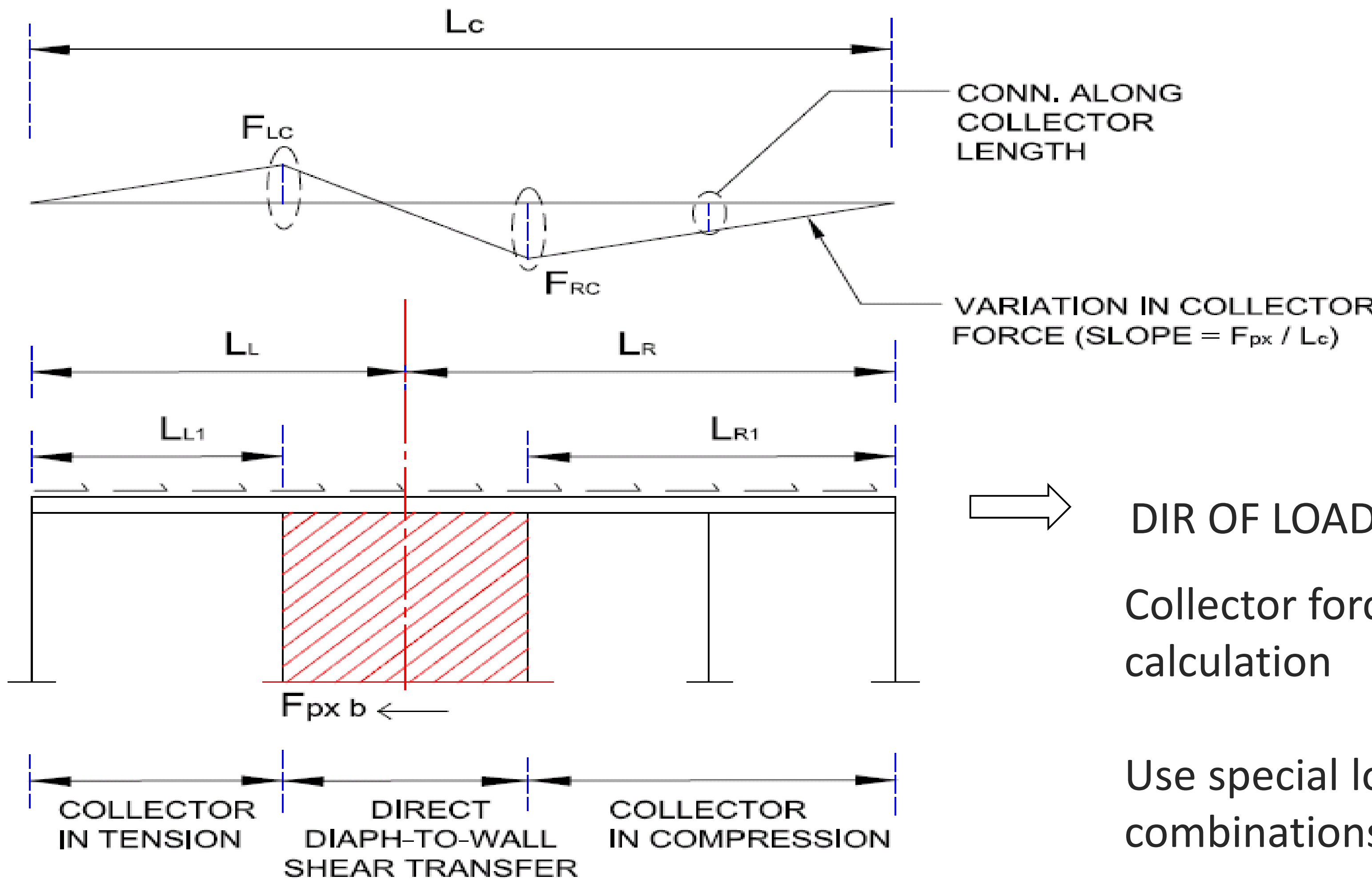
Chord Force Calculation

$w = F_{px} / L$

$M_D = wL^2/8$

$F_c = M_D / d$

Diaphragms



DIR OF LOADING

Collector force calculation

Use special load combinations

Diaphragms

- Other requirements
  - Shear capacity of diaphragm (ACI 21.11.9)

$$V_n = A_{cv}(2\lambda\sqrt{f'_c} + \rho_t f_y) \le 8A_{cv}\sqrt{f'_c}$$

Where, A_{cv} = Gross area of diaphragm section

  - In chords, collectors, and so on where compressive stress exceeds $0.2f'_c$, provide confinement reinforcement per 21.9.6.4(c). Discontinue where stress < $0.15f'_c$ (ACI 21.11.7.5).
  - Shear transfer between diaphragm and lateral force-resisting elements is typically via shear friction (ACI 11.6.4).

Final Remarks

- The review course intends to give a quick oversight of the material. Further in-depth study is essential.
- The S.E. Exam is not necessarily an exam to test specific code knowledge. However, code references are important.
- In the essay questions, assumptions are okay so long as the basis is clearly explained.
- Time-management is critical. Solve problems in a systematic manner: easiest first, most-difficult last.

Structural Design Standards Relevant for Concrete Design

- International Building Code (IBC 2012 Edition)
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-10) (forces only)
- ACI 318-11 Building Code Requirements for Structural Concrete

Recommended References/Additional Study Materials

1. Alan Williams, *Structural Engineering PE License Review Problems & Solutions*, 6th Ed.
2. Handout accompanying this review course
3. S.E. Exam Manual, Ravi Kanitkar
4. Seismic Design of CIP Concrete Special Structural Walls & Coupling Beams, NEHRP Seismic Design Tec Brief No. 6 (NIST GCR 11-9-17-11REV-1)
5. Seismic Design of CIP Concrete Diaphragms, Chords, Collectors, NEHRP Seismic Design Tech Brief No. 3 (NIST GCR 10-917-4)

Final Remarks

- Create your own a reference that you can get to know well.
- Practice solving examples; time yourself.
- Be rested.

GOOD LUCK!

GO PASS THE EXAM

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