

USDA Natural Resources Conservation Service Science and Technology

Conservation Webinars



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Concrete Retaining Wall Design for Animal Waste Management Structures

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and

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Objectives

- Upon completion of this presentation, the attendee will be able to:
 - Identify the industry and agency standard codes and references related to concrete design.
 - Understand structural design loads and critical load combinations for the design of NRCS retaining wall structures
 - Design basic components of typical NRCS type retaining walls.

Concrete Retaining Wall Design

Design References

Design Guidance

- **CPS 313 – Waste Storage Facility**
- **NRCS General Design Guidance**
 - National Engineering Manual (NEM)
 - Part 511, Design
 - Provides guidance and requirements for jobs requiring engineered designs
 - Part 536, Structural Engineering
 - Provides specific guidance and requirements for jobs requiring structural engineering
 - Engineering Field Manual, Chapter 6
 - NEH 6, Structural Design

NEM Part 536 Section Summaries

- **536.20 Design Criteria for Reinforced Concrete Structures**

- Design per ACI 318 if not classified as hydraulic or environmental in Section 536.21
 - Reinforcement $f_y \leq 60,000$ psi w/o CED approval
- In lieu of ACI 318, designers may use ACI 350

- **536.21 Design Criteria for Reinforced Concrete Hydraulic & Environmental Structures**

- A hydraulic structure is any structure subjected to hydrostatic or hydrodynamic pressures, either externally or internally. An environmental structure is any structure intended for conveying, storing, or treating water, wastewater, or other liquids and non-hazardous materials such as solid waste, and for secondary containment of hazardous liquids or solid waste, AND, designed to be liquid-tight, with minimal leakage under normal service conditions
- Note liquid tight isn't liquid/water proof. Water proof requires a steel tank, liner, or other material
- Design per ACI 350.
 - Flexural tension reinforcement stress due to unfactored (service) loads $\leq 20,000$ psi.
 - Reinforcement $f_y \leq 60,000$ psi w/o CED approval
 - Construction joints, if intended to be watertight, prepared per ACI 224.3R-95 Section 3.2.1, need not include integral waterstops as required in ACI 350-06, Section 6.4.7
- Plus additional guidance on Riser design (TR 30 and TR 68) and the use of PCA's Circular Concrete Tanks without Prestressing (1993)

NRCS References

- **NRCS**
 - NEH Section 6, Chapter 4 – Reinforced Concrete (Working Stress)
 - TR 67 - Reinforced Concrete Strength Design
 - TR 74, Lateral Earth Pressures
 - NEH 19, Construction Inspection
 - NEM Part 642 – Construction Specifications
 - Chapter 31 – Concrete for Major Structures
 - Various Design Notes and Technical Releases
- **National Structural Engineering Design Standards**
 - American Society of Civil Engineers (ASCE)
 - Minimum Design Loads for Buildings and Other Structures, (ASCE 7), (2016 Ed.)
 - International Code Council (ICC)
 - International Building Code (IBC) (2015)

Concrete References

- **American Concrete Institute**
 - ACI 302 Guide for Concrete Floor and Slab Construction
 - ACI 315 Detailing Manual
 - ACI 318 Building Code Requirements (new format in 2014)
 - ACI 330 Guide for the Design & Construction of
Concrete Parking Lots
 - ACI 350 Detailing Manual
 - ACI 350.4R Design Considerations for Environmental Engineering
 - ACI 360 Design of Slabs on Grade
 - ACI SP-2 Manual of Concrete Inspection

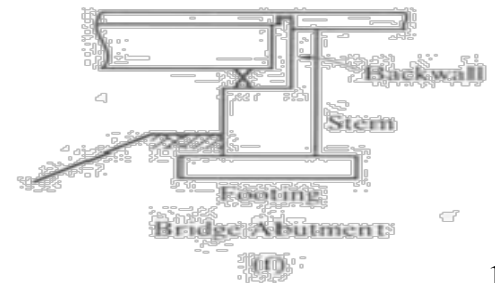
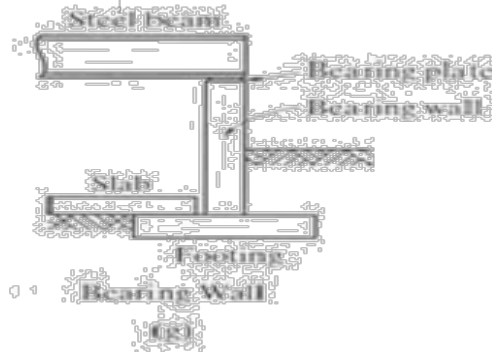
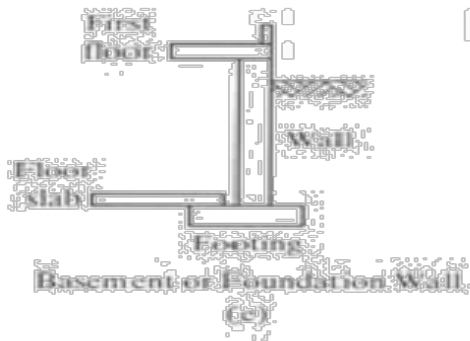
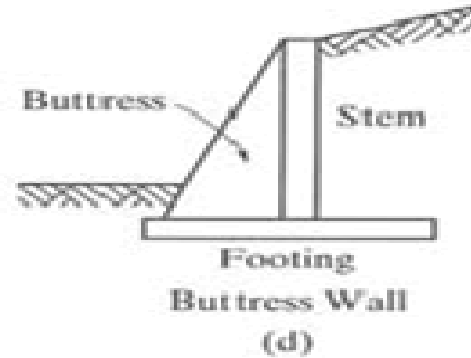
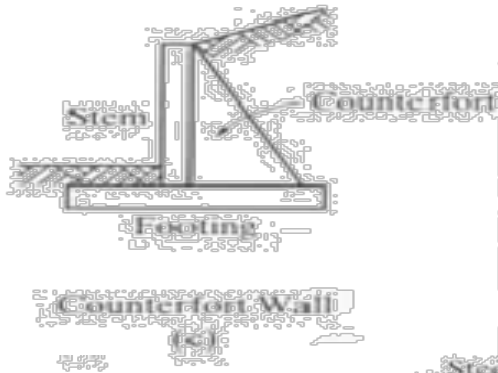
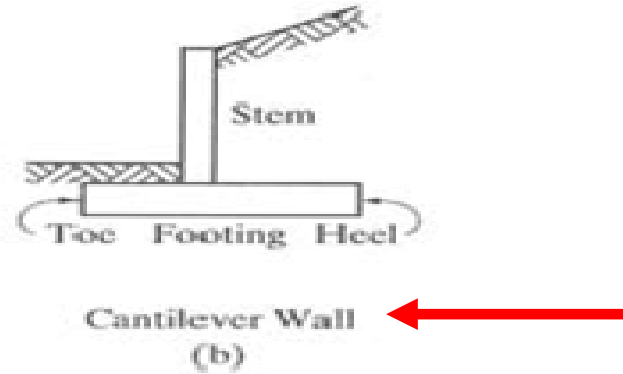
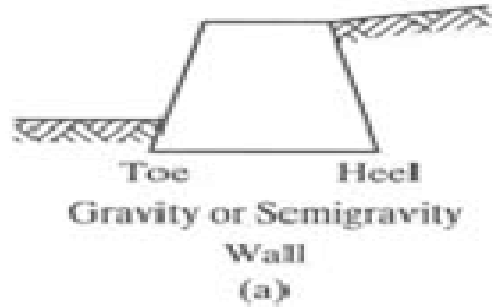
Concrete Retaining Wall Design

Wall Types

Retaining Walls

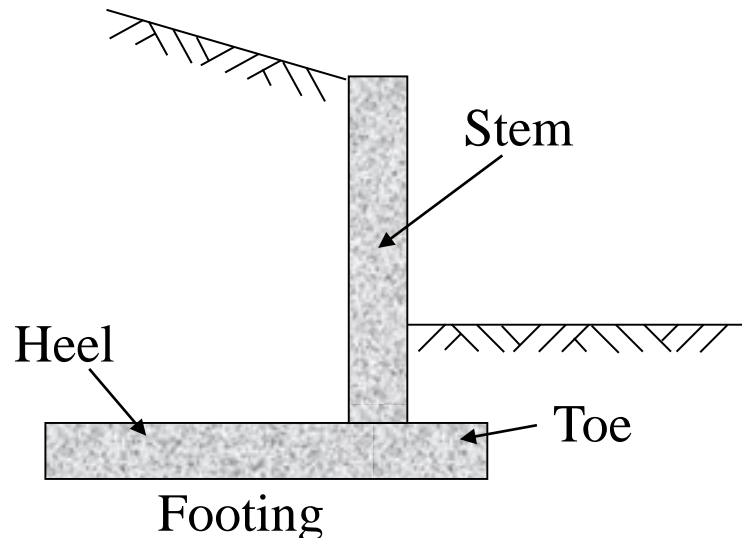
- Walls are generally used to provide lateral support for:
 - An earth fill
 - Embankment, or
 - Other materials that supports vertical loads.
- Some retaining walls also support vertical loads in addition to the lateral loads from the retained materials.
- Typically, the main purpose for these walls is to maintain a difference in elevation of the ground surface on each side of the wall by providing lateral support for the higher level backfill material.
- The following slide shows some common types of retaining walls. The various types do not necessarily behave the same under load due to different support and other conditions.

Common Types of Retaining Walls



Cantilever Retaining Walls

- The cantilever wall is the most common type of retaining structure and generally is used for walls up to 25 ft in height.
- The individual parts (toe, heel, and stem) of the wall are all designed as cantilever beams.
- Aside from its stability, the capacity of the wall is a function of the strength of its individual parts.



Cantilever Wall

Concrete Retaining Wall Design for Animal Waste Management Structure

Using ACI 350-06

Structural Design - Basic Design Approach

- **Evaluate Loading**
 - Various load cases and combinations
- **Check Stability of the Structure**
 - Evaluate the local stability
 - Sliding
 - Overturning
 - Bearing capacity
- **Perform the Structural Design of the Structure**
 - Perform the concrete design for the heel, toe, and wall stem per ACI 350-06 in accordance with NEM 536

Concrete Retaining Wall Design

Loading

Structural Design - Loading

- **Various Load Cases and Conditions**

- Backfill or stored materials ← Typically the primary loading
- Backfill moisture conditions and groundwater table level
- Surcharge loads
- Seismic Loads
- Other miscellaneous loading – columns, impact, etc.

- **Geotechnical Earth Pressure Theories**

- Coulomb and Rankine soil mechanics theories can be used
- Reference **NRCS Technical Release 74**

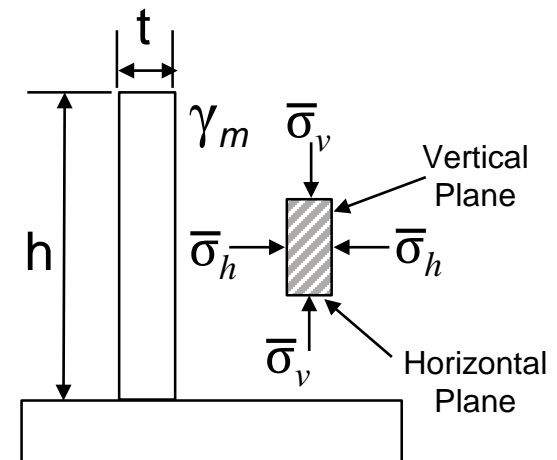
Structural Design - Loading

- **Soil Backfill Pressures Vary for Many Reasons**
 - Stiffness of wall
 - Type and slope of backfill
 - Moisture condition of backfill
 - Drainage of the backfill material
 - Level of water table
 - Proper drainage must be provided
 - Well drained, cohesionless material.
 - Maintenance of these systems is a key assumption

Structural Design – Loading Lateral Earth Pressure

• Three Lateral Earth Pressure Categories

- Active, K_a
 - Wall deflects away from backfill allowing earth to expand laterally
 - Typical movement required to fully mobilize: 1 in / 20 ft
 - For yielding walls, $t/h \leq 0.085$
- Passive, K_p
 - Wall is forced to move toward the backfill and compresses soil
 - Typical movement required to fully mobilize: 1 in / 2 ft
- At rest, K_o
 - Wall is rigid and has no movement
 - For non-yielding walls $t/h > 0.085$



K = Lateral Earth Pressure Coeff.

$$\bar{\sigma}_h = K\bar{\sigma}_v = K(H\gamma_m)$$

Structural Design - Basic Load Cases

- **Usual**

- Normal Operation

- Backfill in place (moist) and (storage side empty)
 - No backfill in place and storage side filled
 - Surcharge applied to above conditions (check with & without)

- **Unusual**

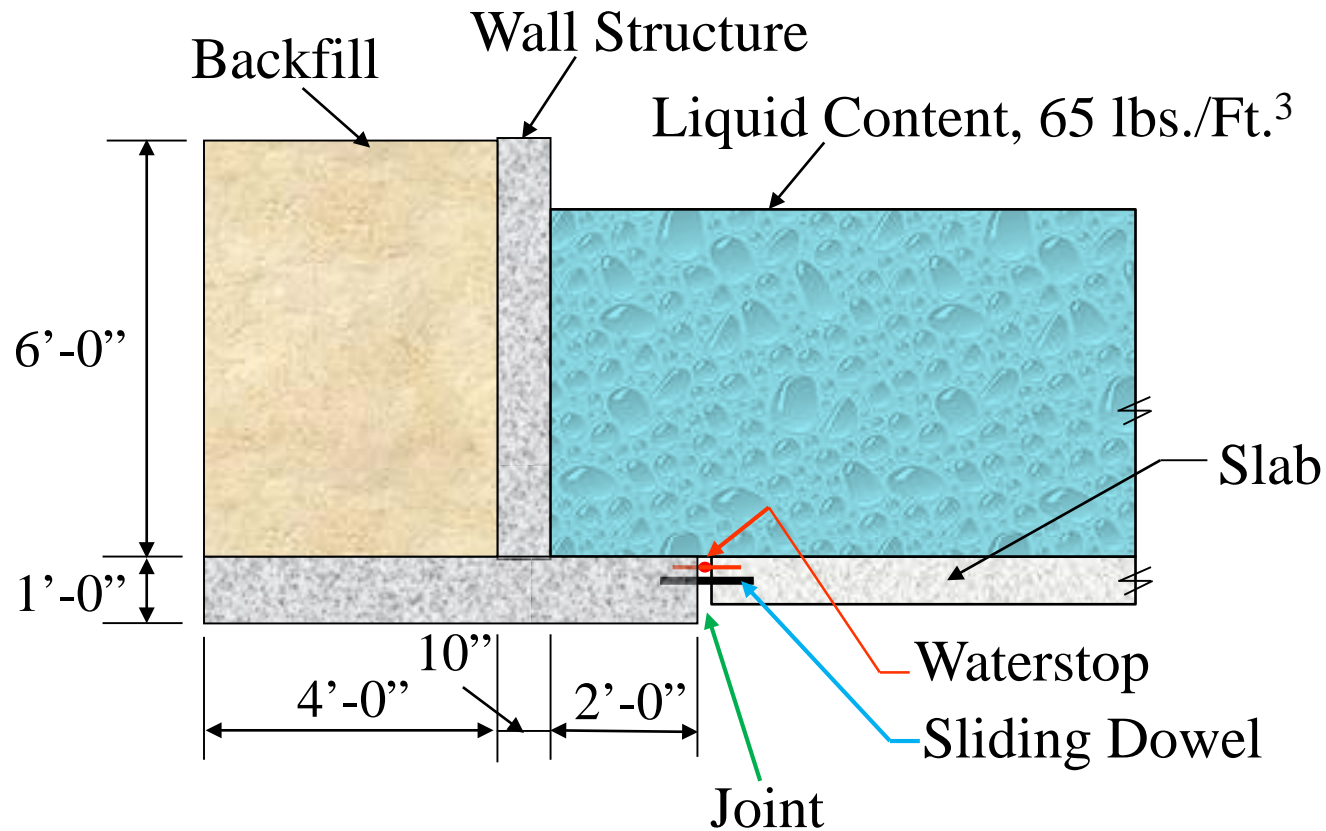
- Construction loading

- High water table

- Backfill in place and saturated (storage side empty)
 - No backfill in place and storage side filled

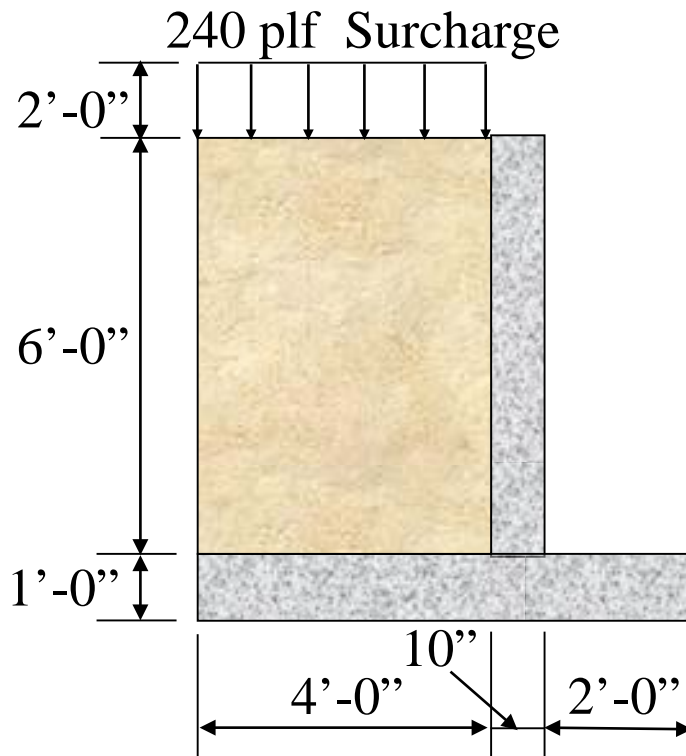
- **Earthquake / Extreme / Flood**

Dimensions and Assumptions



For this example we will look at the case with backfill in place and with/without a surcharge load. No stored waste is assumed but that condition would be checked in another case

Dimensions and Assumptions



Surcharge:

It is customary to use 2 feet of soil as a general surcharge load for retaining walls with accessible surfaces adjacent to the wall.

Assumed Soil Properties:

Classification: SP-GW

Unit Weight, γ_e : 120 pcf

Angle of Internal Friction, θ : 30°

Lateral Pressure Coefficients:

Active: $K_a = \tan^2(45^\circ - \theta/2) = 0.33$

Passive: $K_p = \tan^2(45^\circ + \theta/2) = 3.00$

At-Rest: $K_0 = 1 - \sin \theta = 0.50$

TR-74: Use K_0 when $t/h > 0.085$

$$t/h = 12 \text{ in.} / 72 \text{ in.} = 0.167$$

Concrete:

Normal weight, Portland cement

Unit Weight: 150 pcf

Compressive Strength:

4000 psi @ 28 days

Mod. of Elasticity, E_C :

3,644,150 psi

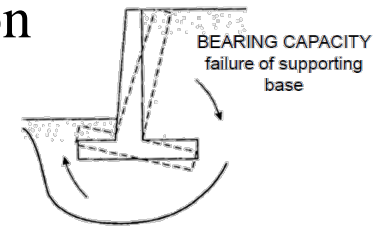
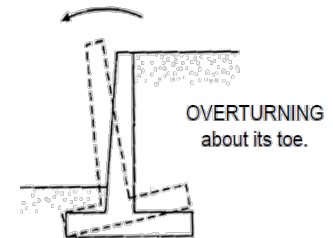
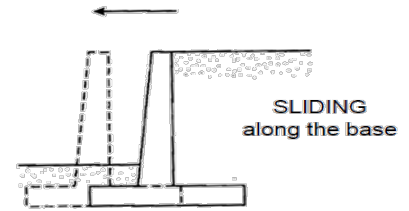
Concrete Retaining Wall Design

Stability Checks

Structural Design - Stability

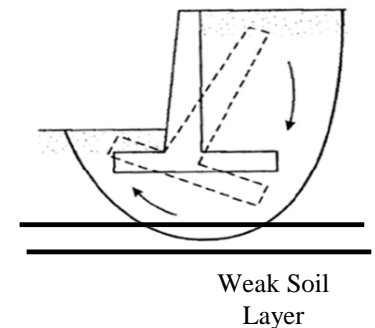
• **Check Stability of the Structure**

- Sliding - Min. FS 1.5 (2.0 w/cohesion used)
- Overturning - Min. FS 1.5 (2.0 w/cohesive backfill)
- Buoyancy (Floatation) – Min. FS 1.5
- Bearing capacity - Min. FS 1.5
 - Goal is a uniform bearing pressure distribution
- Use service loads (not factored)

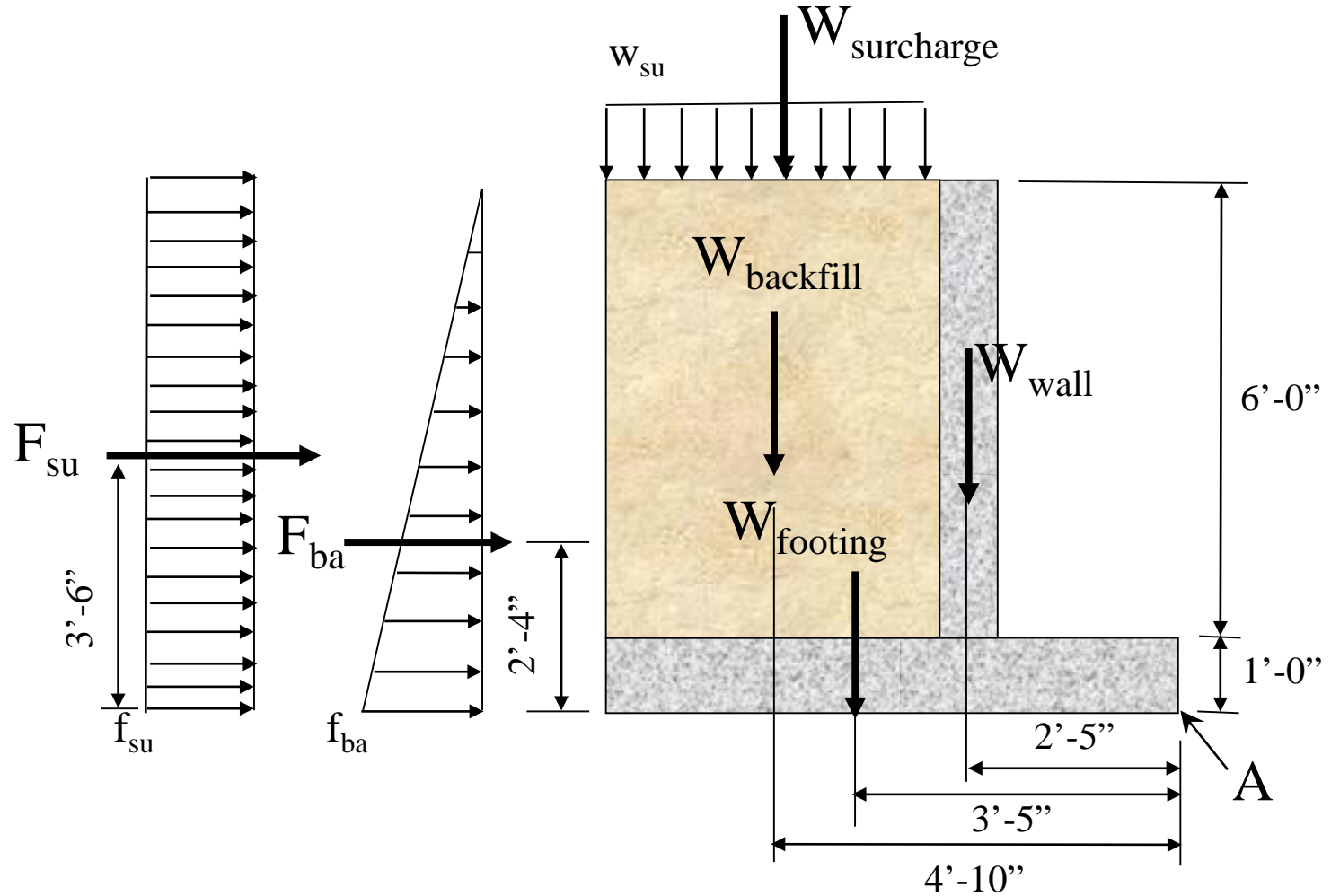


• **Other Stability Considerations**

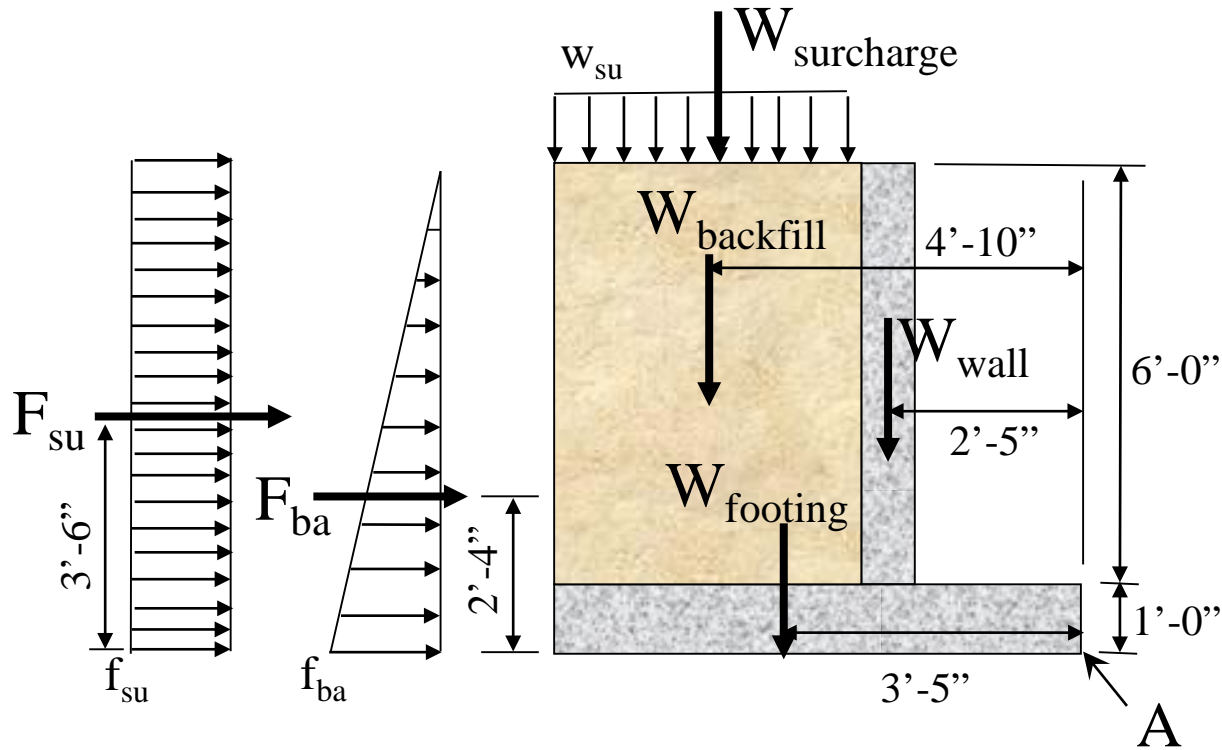
- Settlement
 - Total and differential settlement
- Global Stability
- Water Related Concerns
 - Exposure to flow and build up of pressures



Stability Analysis - Force Diagram



Load Determination - Weights



Concrete

$$W_{Wall} = 150 \text{ pcf} (10 \text{ in}) (1 \text{ ft}/12 \text{ in}) (6 \text{ ft}) (1 \text{ lf}) = 750 \text{ lbs.}$$

$$W_{Footing} = 150 \text{ pcf} (12 \text{ in}) (1 \text{ ft}/12 \text{ in}) (6.833 \text{ ft}) (1 \text{ lf}) = 1025 \text{ lbs.}$$

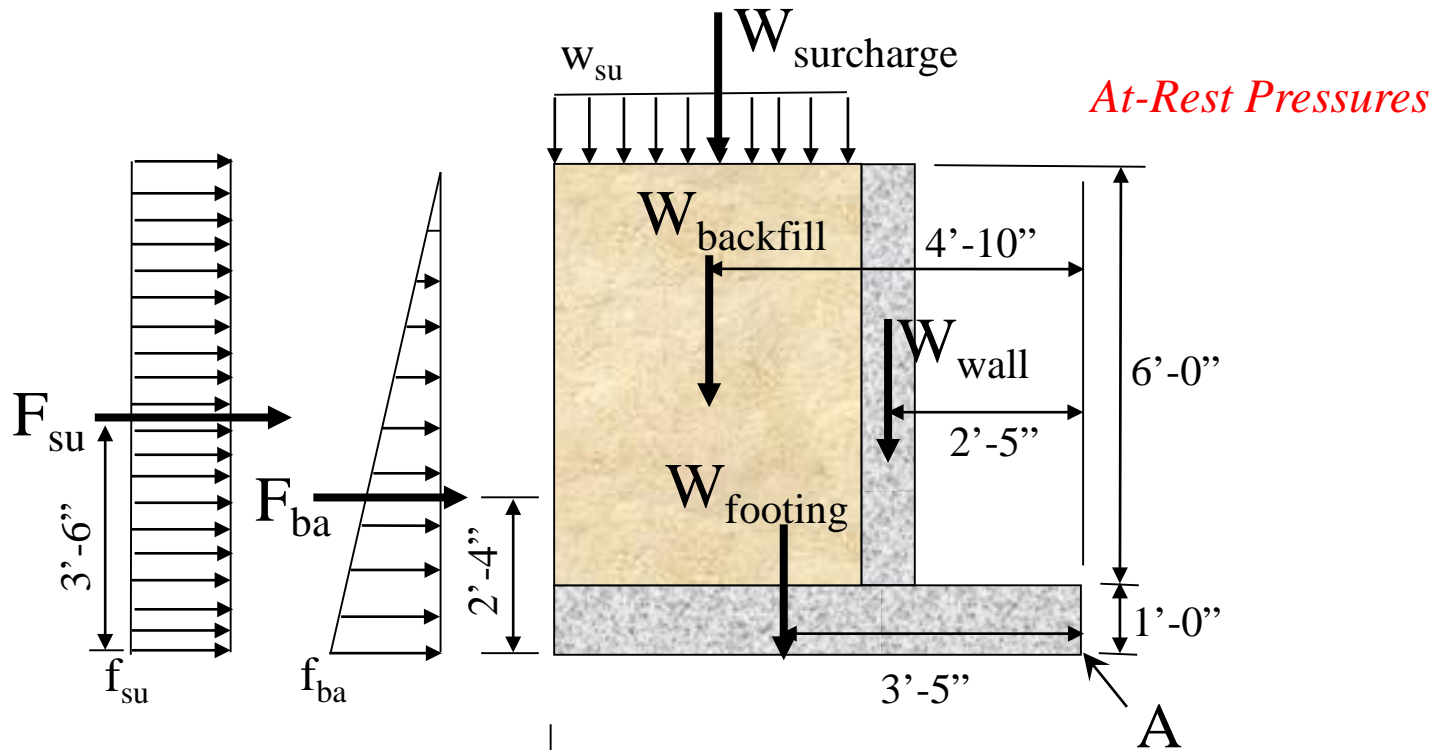
Soil

$$W_{Backfill} = 120 \text{ pcf} (6 \text{ ft}) (4 \text{ ft}) (1 \text{ lf}) = 2880 \text{ lbs.}$$

Surcharge

$$W_{surcharge} = 120 \text{ pcf} (2 \text{ ft}) (4 \text{ ft}) (1 \text{ lf}) = 960 \text{ lbs.}$$

Load Determination - Lateral Earth Pressure



Surcharge

$$f_{\text{surcharge}} = K_O (w_{\text{surcharge}})$$

$$w_{\text{surcharge}} = (\gamma_e)(\text{surcharge height})$$

$$w_{\text{surcharge}} = (120 \text{ pcf})(2 \text{ ft}) = 240 \text{ plf}$$

$$f_{\text{surcharge}} = (0.5)(240 \text{ plf}) = 120 \text{ plf}$$

$$F_{\text{surcharge}} = f_{\text{surcharge}} (\text{height})$$

$$= (120 \text{ plf})(7 \text{ ft}) = 840 \text{ lbs}$$

Backfill

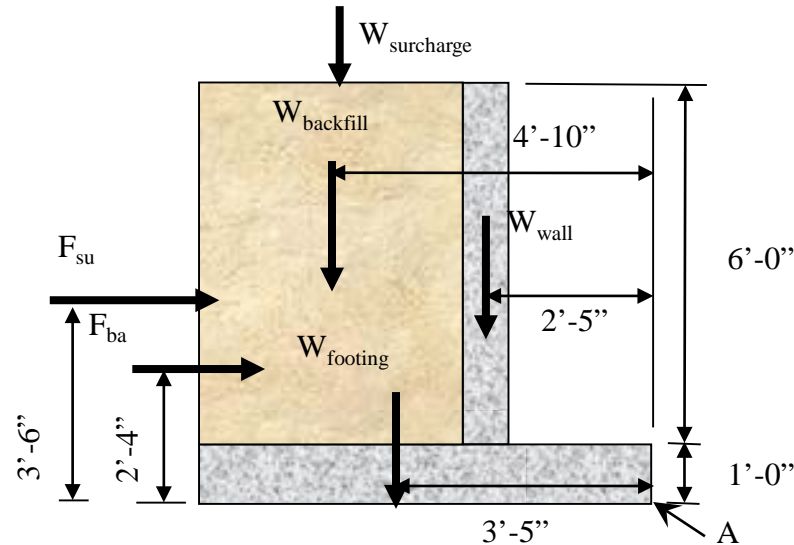
$$f_{\text{backfill}} = K_O \gamma_e (\text{height})$$

$$= 0.50 (120 \text{ pcf}) (7 \text{ ft}) (1 \text{ lf}) = 420 \text{ plf}$$

$$F_{\text{backfill}} = \frac{1}{2} f_{\text{backfill}} (\text{height})$$

$$= \frac{1}{2} (420 \text{ plf}) (7 \text{ ft}) = 1470 \text{ lbs}$$

Stability Analysis - Stabilizing Moments



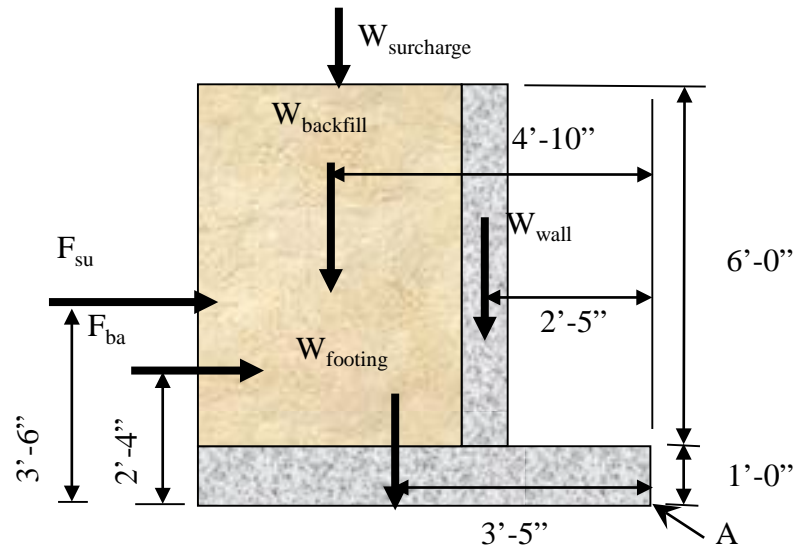
<u>Force</u>	<u>Magnitude</u>	<u>Arm</u>	<u>Moment</u>
W_{wall}	750 lbs	2.417 ft	1812.5 lb-ft
$W_{footing}$	1025 lbs	3.417 ft	3502.1 lb-ft
$W_{backfill}$	<u>2880 lbs</u>	4.833 ft	<u>13919.9 lb-ft</u>

$$\Sigma W = 4655 \text{ lbs} \qquad \Sigma M_R = 19234.6 \text{ lb-ft}$$

$W_{surcharge}$	<u>960 lbs</u>	4.833 ft	<u>4640.0 lb-ft</u>
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$$\Sigma W = 5615 \text{ lbs} \qquad \Sigma M_R = 23874.6 \text{ lb-ft}$$

Stability Analysis - Overturning Moments and Sliding Forces



<u>Force</u>	<u>Magnitude</u>	<u>Arm</u>	<u>Moment</u>
F_{backfill}	1470 lbs	2.333 ft	3429.5 lb-ft
F_{surchage}	<u>840 lbs</u>	3.500 ft	<u>2940.0 lb-ft</u>
$\Sigma F = 2310 \text{ lbs}$		$\Sigma M_O = 6369.5 \text{ lb-ft}$	

Coefficient of Sliding Friction μ

- For most NRCS structures, μ should be limited to about 0.35.
- USACE, NAVFAC, & other ref have values for various materials & soil conditions.
- The coefficient of sliding friction can be approximated as : $\mu = 0.67 \tan \phi$
 - For $\phi = 30^\circ$ get $\mu = 0.67 \tan 30^\circ = 0.387$ **Use 0.35**

Overturning

$FS_O = \text{Stabilizing Moments} / \text{Overturning Moments}$

$$= \Sigma M_R / \Sigma M_O$$

$$= 19234.6 \text{ lb-ft} / 3429.5 \text{ lb-ft}$$

$$FS_O = 5.6 \quad \text{Minimum } FS_O \text{ is } 1.5$$

Sliding

$FS_{SL} = [(\text{Soil Coefficient}) (\text{Weight}) + \text{Passive}] / \Sigma \text{Sliding Force}$

$$= [(\mu) (\Sigma W)] / \Sigma F$$

$$= [(0.35) (4655 \text{ lbs}) + 0.0] / 1470 \text{ lbs}$$

$$FS_{SL} = 1.1 \quad \text{Minimum } FS_{SL} \text{ is } 1.5 \quad \text{Additional sliding resistance is required, so use slab or resize wall}$$

Stability Analysis - Safety Factors

With Surcharge

Overturning

$$\begin{aligned}FS_O &= \text{Stabilizing Moments} / \text{Overturning Moments} \\ &= \Sigma M_R / \Sigma M_O \\ &= 23874.5 \text{ lb-ft} / 6369.5 \text{ lb-ft}\end{aligned}$$

$$FS_O = 3.7 \quad \text{Minimum } FS_O \text{ is } 1.5$$

Sliding

$$\begin{aligned}FS_{SL} &= [(\text{Soil Coefficient}) (\text{Weight}) + \text{Passive}] / \Sigma \text{Sliding Force} \\ &= [(\mu) (\Sigma W)] / \Sigma F \\ &= [(0.35) (5615 \text{ lbs}) + 0.0] / 2310 \text{ lbs}\end{aligned}$$

$$FS_{SL} = 0.8 \quad \text{Minimum } FS_{SL} \text{ is } 1.5 \quad \text{Additional sliding resistance is required, so use slab or resize wall}$$

Bearing Pressure

$$p = \frac{P}{A} \pm \frac{Mc}{I}$$

The basic stress equation for a section subject to axial and bending forces:

p = stress at section's extreme fiber or edge

P = axial force = ΣW

A = cross-sectional area of the section

M = applied bending moment = $\Sigma W(e)$

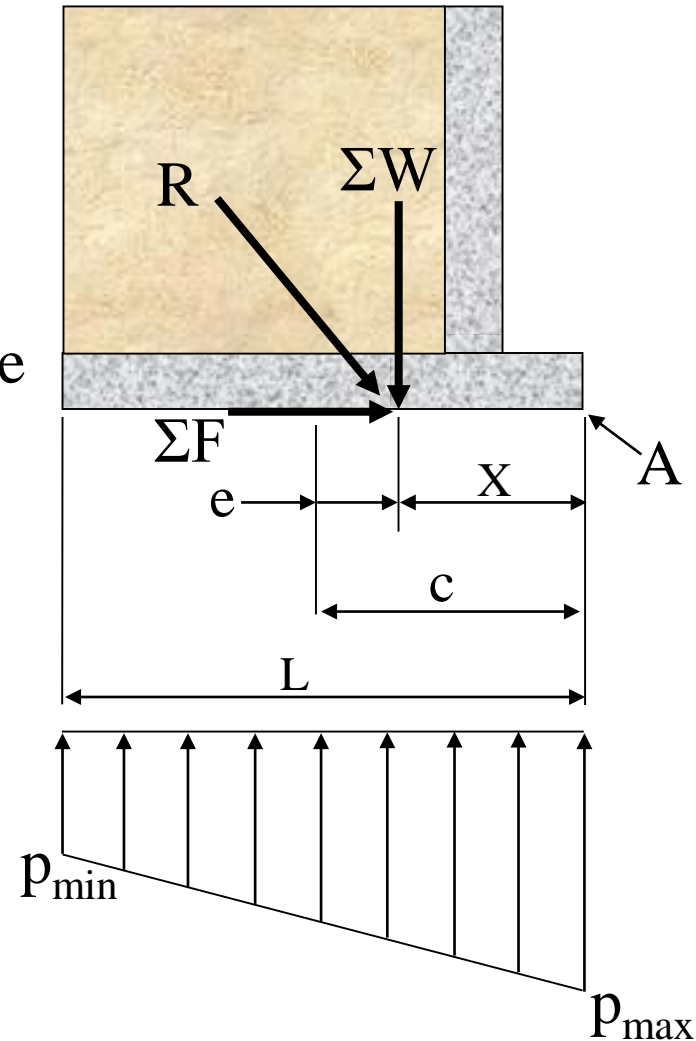
c = distance from the section's neutral axis to its extreme fiber

I = section's moment of inertia

e = eccentricity of ΣW

x = $c - e$

For a rectangular section, the general stress equation can be rewritten



$$p = \frac{\Sigma W}{A} \left(1 \pm \frac{6e}{L} \right)$$

Bearing Pressure

For a rectangular section, the general stress equation can be rewritten as follows:

$$p = \frac{P}{A} \pm \frac{Mc}{I}$$

$$I_{xx} = bL^3/12 \quad c = L/2 \quad A = bL$$

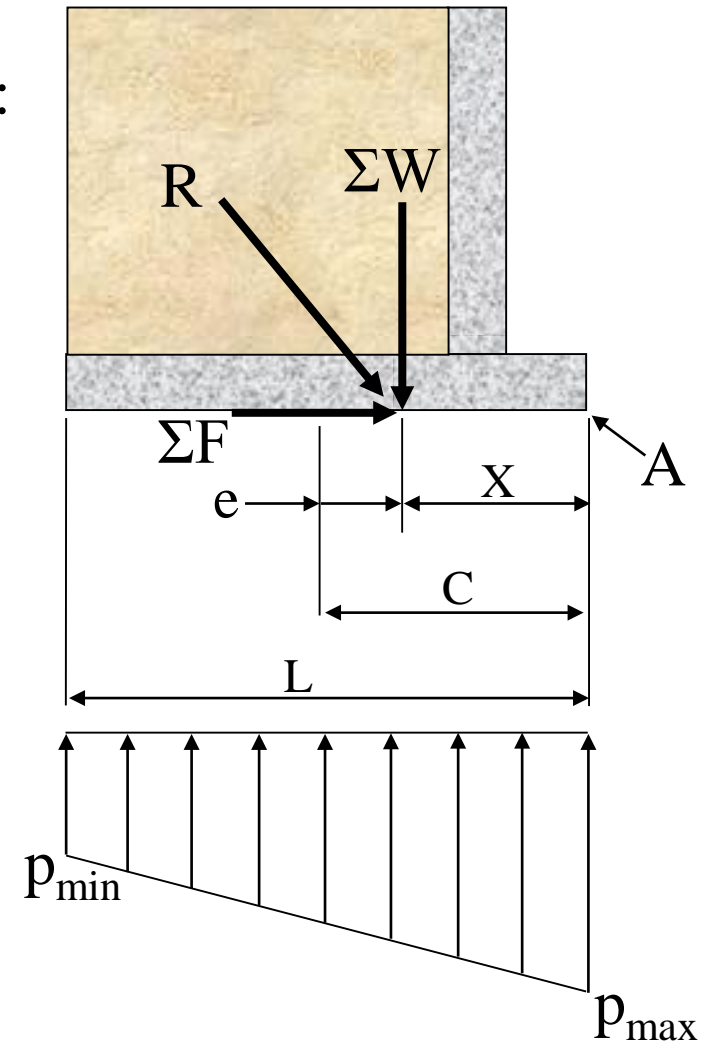
$$P = \Sigma W \quad M = \Sigma W(e)$$

$$c/I = (L/2)/(bL^3/12) = 6/(bL^2)$$

$$p = \frac{\Sigma W}{bL} \pm \frac{6M}{bL^2} = \frac{\Sigma W}{bL} \pm \frac{6M}{b(bL)}$$

$$p = \frac{\Sigma W}{bL} \pm \frac{6(\Sigma W)(e)}{b(bL)}$$

$$p = \frac{\Sigma W}{A} \left(1 \pm \frac{6e}{L} \right)$$



The maximum value for ΣW and “ e ” will produce the highest bearing stresses.

Bearing Pressure

The “section” is that of the structure’s footprint, and is viewed as if it projects up out of the ground.

Section Properties

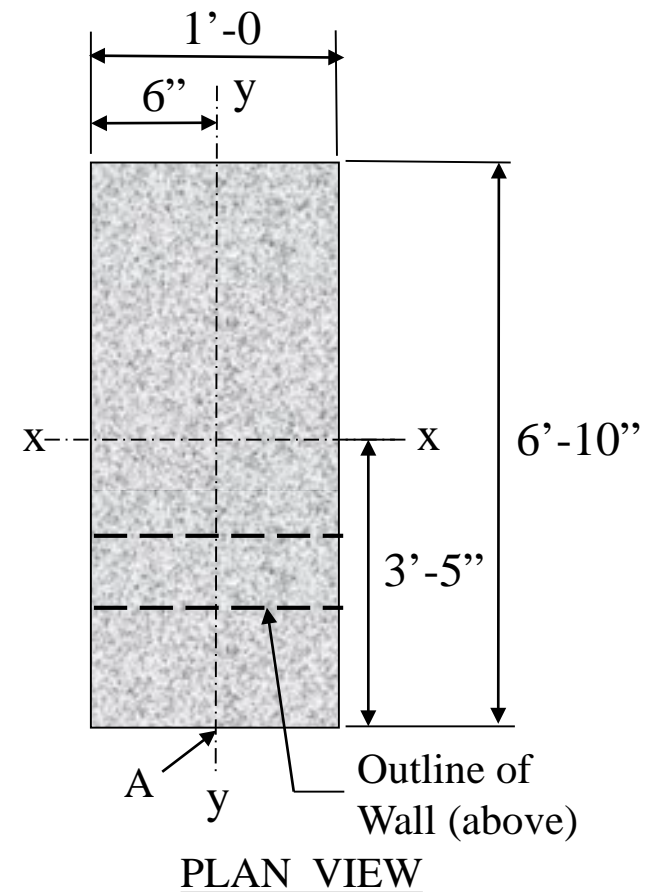
$$\text{Area} = (b)(h) = (1.0 \text{ ft})(6.83 \text{ ft}) = 6.83 \text{ ft}^2$$

Moment of Inertia, I_{xx} :

$$\begin{aligned} I_{xx} &= (b)(h^3)/12 \text{ or } (b)(L^3)/12 \\ &= (1.0 \text{ ft})(6.83 \text{ ft})^3/12 \\ &= 26.59 \text{ ft}^4 \end{aligned}$$

For a rectangular section, the neutral axes--shown as the lines x-x and y-y--project through the mid-point.

Since $L = 6' - 10''$ then $c = L/2 = 3' - 5''$



Bearing Pressure

Footing – Resultant Location

$$x = (\Sigma M_R - \Sigma M_O) / \Sigma W$$
$$= \frac{(23874.5 \text{ lb-ft} - 6369.5 \text{ lb-ft})}{5615 \text{ lbs}}$$

$$x = 3.117 \text{ ft}$$

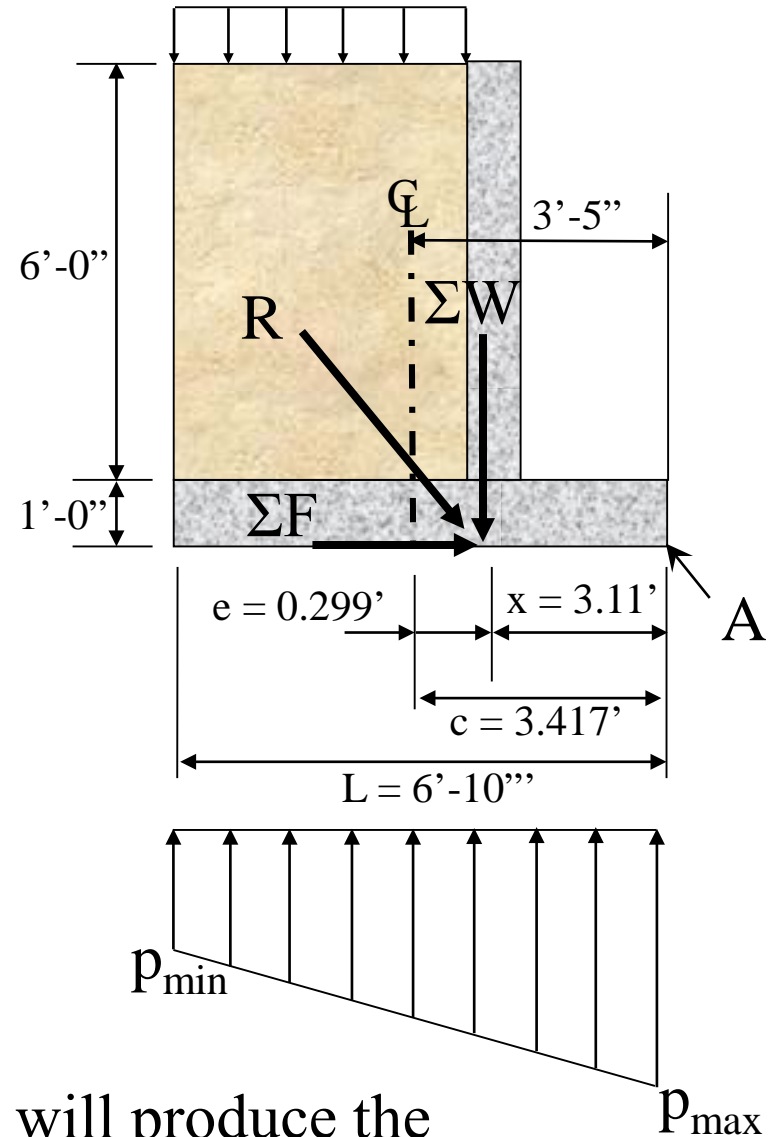
Footing – Eccentricity of ΣW

$$e = \text{Centerline} - x$$

$$= 3.417 \text{ ft} - 3.117 \text{ ft}$$

$$e = 0.299 \text{ ft}$$

The maximum value for ΣW and “e” will produce the highest bearing stresses.



Bearing Pressure

$$p = \frac{\Sigma W}{A} \left(1 \pm \frac{6e}{L} \right)$$

$$= \frac{5615}{6.83 \text{ ft}^2} \left(1 \pm \frac{6 (0.299 \text{ ft})}{6.833 \text{ ft}} \right)$$

$$= 822.11 \text{ psf} \pm 215.75 \text{ psf}$$

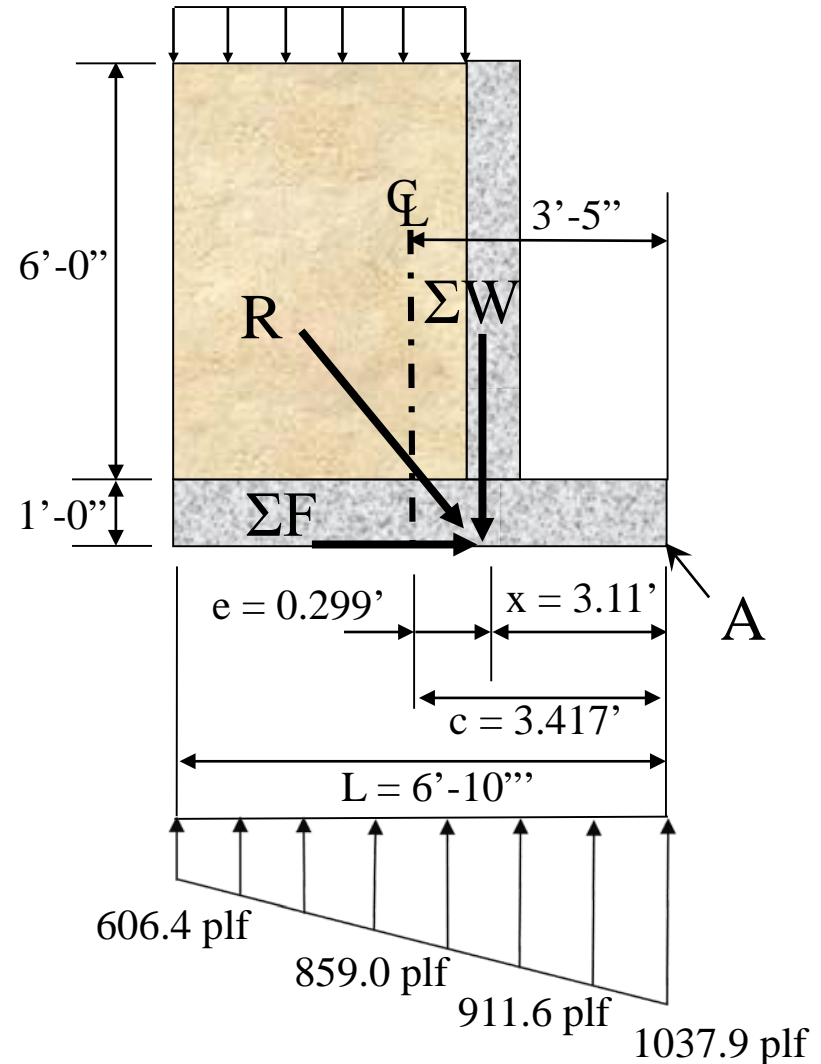
$$= 1037.9 \text{ psf}, 606.4 \text{ psf}$$

$$p_{\min} = 606.4 \text{ psf}$$

$$p_{\max} = 1037.9 \text{ psf}$$

for a 1-ft length of wall:

$$w_{\min} = 606.4 \text{ plf}, w_{\max} = 1037.9 \text{ plf}$$



Concrete Retaining Wall Design

Structural Concrete Design

ACI 350-06

Structural Design - Basic Objective & Load Factors

• **Basic Design Objective**

- Balance the compressive strength of the concrete with the tensile strength of the reinforcing steel
- Therefore, reinforcement is required where flexure, axial loads, or shrinkage effects cause tensile stresses.
- Excessive cracking and deflection provides warnings prior to failure
- Net tensile strain is a measure of both

• **Load Factors**

- ASCE 7-10 Chapter 2, Section 2.3.2 and ACI 350-06 Section 9.2.1:
 - $D = 1.2$; $F = 1.2$; $L = 1.6$; $W = 1.6$; $H = 1.6$ Earth Pressure
- ACI 350-06, Section 9.2.6 requires that internal forces other than those in compression-controlled sections be multiplied by an Environmental Durability Factor (S_d) where $S_d = \frac{\phi f_y}{\gamma f_s}$
- Shear: $V_u = 1.6 (V_s) S_d$ and Bending Moment: $M_u = 1.6 (M_s) S_d$

Structural Analysis – Environmental Durability Factor S_d

$$S_d = \frac{\phi f_y}{\gamma f_s}$$

ϕ = Strength Reduction Factor
 f_y = Reinforcing Steel Yield Stress
 f_s = Allowable Reinforcing Steel Stress
 γ = factored load / unfactored load

Reinforcing Steel Stress	Exposure	
	Normal	Severe
Flexure*, Direct tensile, Hoop	20 ksi	17 ksi
Shear	24 ksi	20 ksi

*Flexure Stress may be calculated in accordance with Section 10.6.4. Lower limits shown.

NRCS policy is to limit reinforcing steel stress to 20 ksi.

S_d for seismic loads is 1.0

Structural Analysis – Environmental Durability Factor S_d

$$S_d = \frac{\phi f_y}{\gamma f_s}$$

ϕ = Strength Reduction Factor

f_y = Reinforcing Steel Yield Stress

f_s = Allowable Reinforcing Steel Stress

γ = factored load / unfactored load

Bending Moment:

$$S_d = \frac{(0.9)(60 \text{ ksi})}{(1.6)(20 \text{ ksi})} \quad S_d = 1.69$$

$$M_U = 1.6 (M_{\text{Service}}) (S_d = 1.69)$$

$$M_U = 2.7 (M_{\text{Service}})$$

Since $M_U \leq \phi M_n$ rewrite to get:

$$\phi M_n / M_{\text{Service}} \geq 2.7 \quad (\text{normal exposure})$$

Using $f_s = 17 \text{ ksi}$:

$$\phi M_n / M_{\text{Service}} \geq 3.18 \quad (\text{severe exposure})$$

Shear:

$$S_d = \frac{(0.75)(60 \text{ ksi})}{(1.6)(24 \text{ ksi})} \quad S_d = 1.17$$

$$V_U = 1.6 (V_{\text{Service}}) (S_d = 1.17)$$

$$V_U = 1.88 (V_{\text{Service}})$$

Since $V_U \leq \phi V_n$ rewrite to get:

$$\phi V_n / V_{\text{Service}} \geq 1.88 \quad (\text{normal exposure})$$

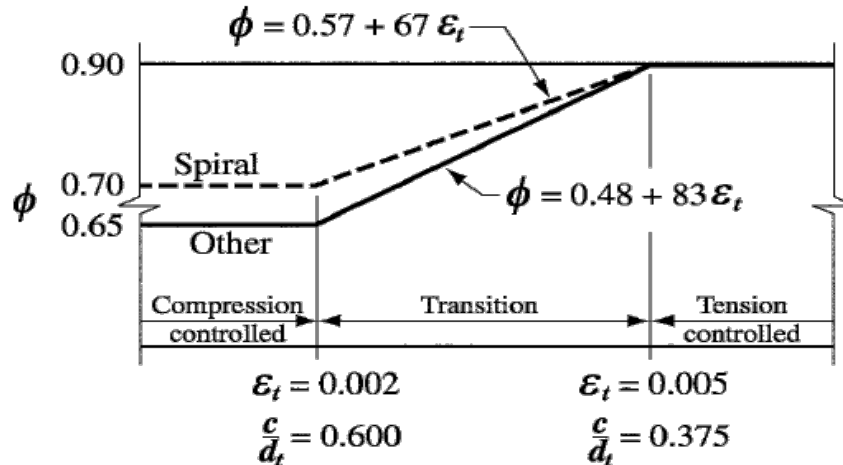
Using $f_s = 20 \text{ ksi}$:

$$\phi V_n / V_{\text{Service}} \geq 2.25 \quad (\text{severe exposure})$$

Structural Analysis – Strength Reduction Factor ϕ

• Strength Reduction Factor, Phi ϕ

- ACI 350-06, Sect. R9.3.2.2
- ϕ depends on the Net Tensile Strain
 - Tension Controlled $\phi = 0.9$ typically but must be calculated
 - $\phi_b = 0.48 + 83(\epsilon_s) \leq 0.90$
 - Compression Controlled $\phi = 0.65$
 - Shear $\phi = 0.75$

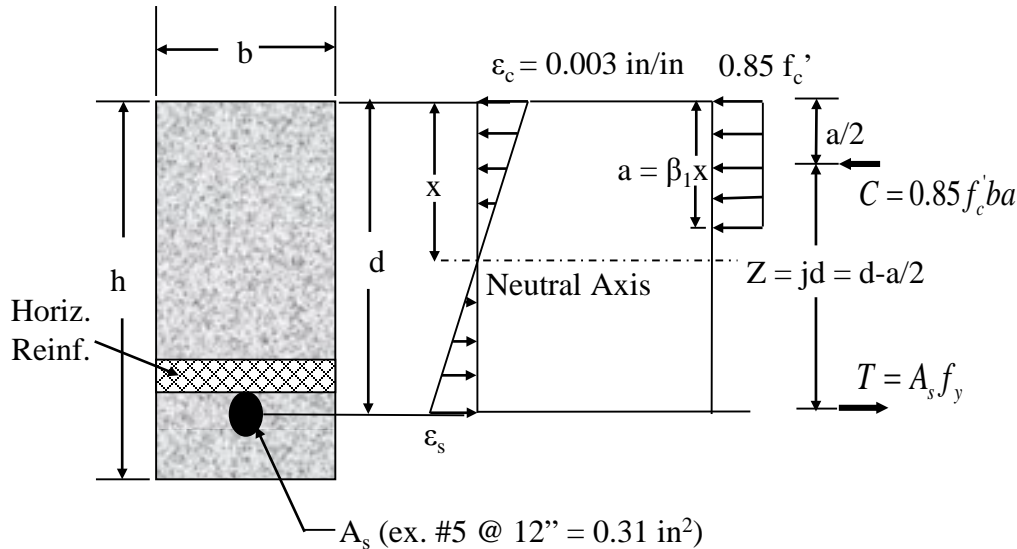


Interpolation on c/d_t :
 Spiral $\phi = 0.37 + 0.20/(c/d_t)$
 Other $\phi = 0.23 + 0.25/(c/d_t)$

Fig. R9.3.2—Variation of ϕ with net tensile ϵ_t and c/d_t for Grade 60 reinforcement and for prestressing steel.

ACI 350-06 shown; Note ACI 318-11 and ACI 318-14 have slight variations to the formula but the values are essentially equal

Structural Design - Basic Design Equations



Strain in Reinforcement

$$\epsilon_s = \epsilon_c (d-c) / c$$

by similar triangles with $\epsilon_c = 0.003 \text{ in/in}$

Force Equilibrium

$$C = T \quad C = 0.85 f'_c b a$$

$$T = A_s f_y$$

$$0.85 f'_c b a = A_s f_y$$

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

Moment Equilibrium

$$M_u = \phi M_n = \phi (C \text{ or } T) \left(d - \frac{a}{2} \right)$$

$$\phi M_n = \phi \left[0.85 f'_c b a \left(d - \frac{a}{2} \right) \right]$$

$$\phi M_n = \phi \left[A_s f_y \left(d - \frac{a}{2} \right) \right]$$

Structural Design – Design Considerations for Concrete

- **Basic Design and Reinforcement Considerations**

- Flexure, Shear, Axial Loads
- Control of Cracking
 - Temperature and shrinkage or other due to restraint
- Exposure Category - ACI guidance on mix design
- Concrete Cover, Spacing, Development, and Lap Splices
- Max. and Min. Amounts
- Temperature and Shrinkage Reinf.

- **Other Considerations - Attention to the details**

- Wall thickness (constructability)
- Joint types, location, and details
 - Reduced joint numbers due to higher reinforcing %
 - Need for waterstops vs proper joint preparation
- Attachments and embedded items
- Construction capabilities and constraints
 - Contractor experience and level of QA/QC
 - Project Location
- Material quantities are only a part of the cost
 - Constructability & simplification of design are also key considerations

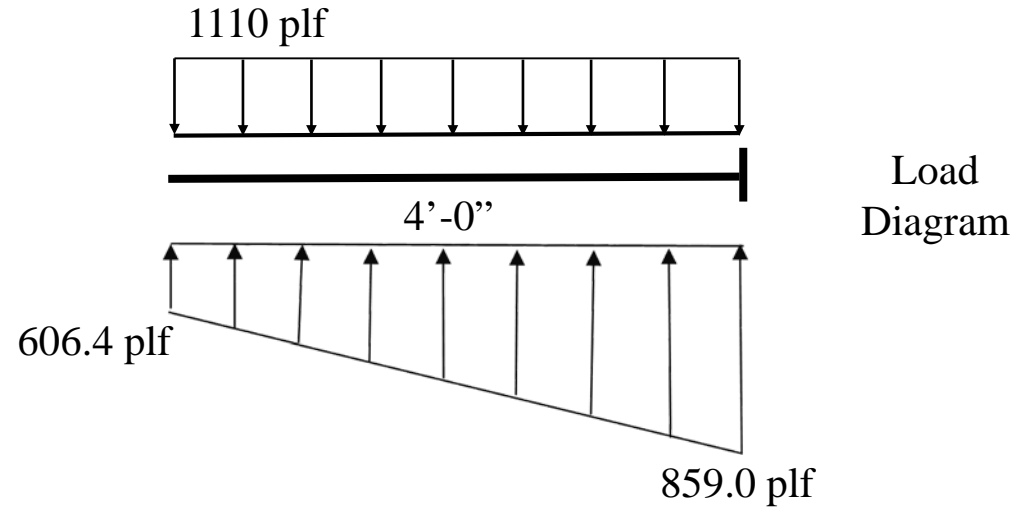
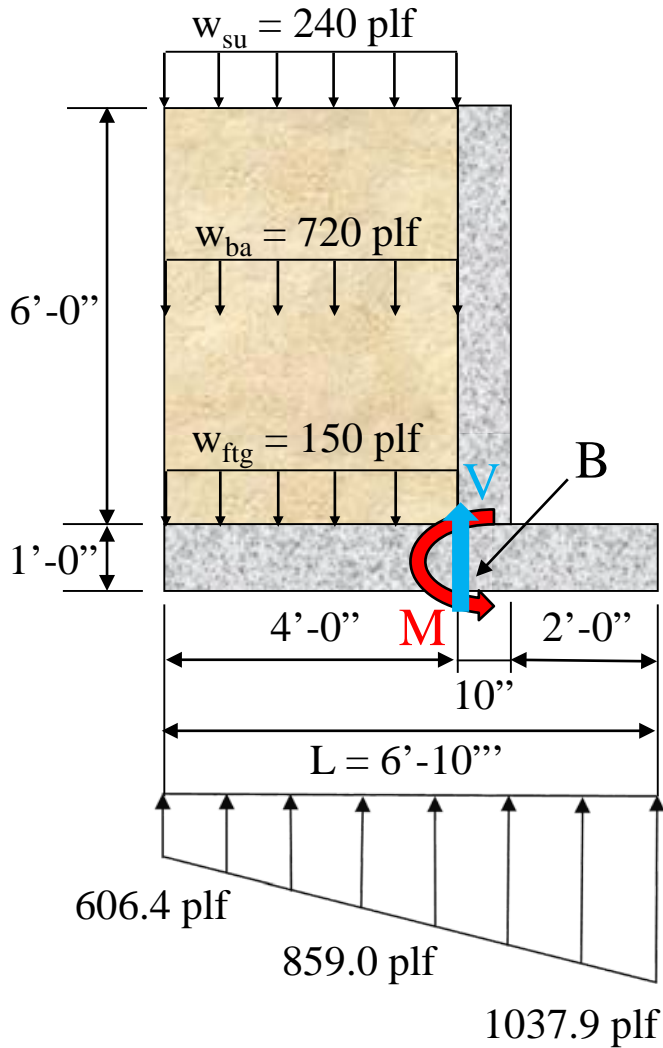
Concrete Retaining Wall Design

Structural Concrete Design

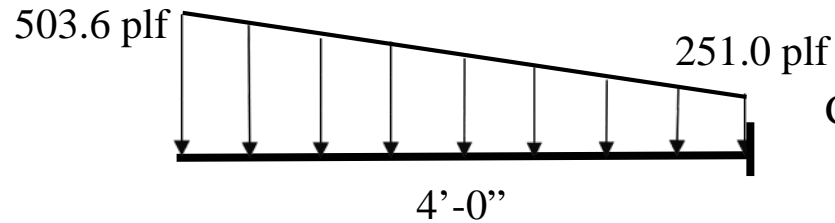
ACI 350-06

Heel Design

Structural Analysis - Heel



Load Diagram



Combined Load Diagram

Shear:

$$V_B = 4 \text{ ft} (503.6 \text{ lb} + 251.0 \text{ lb})/2$$

$$V_B = 1510 \text{ lb} = 1.51 \text{ kips}$$

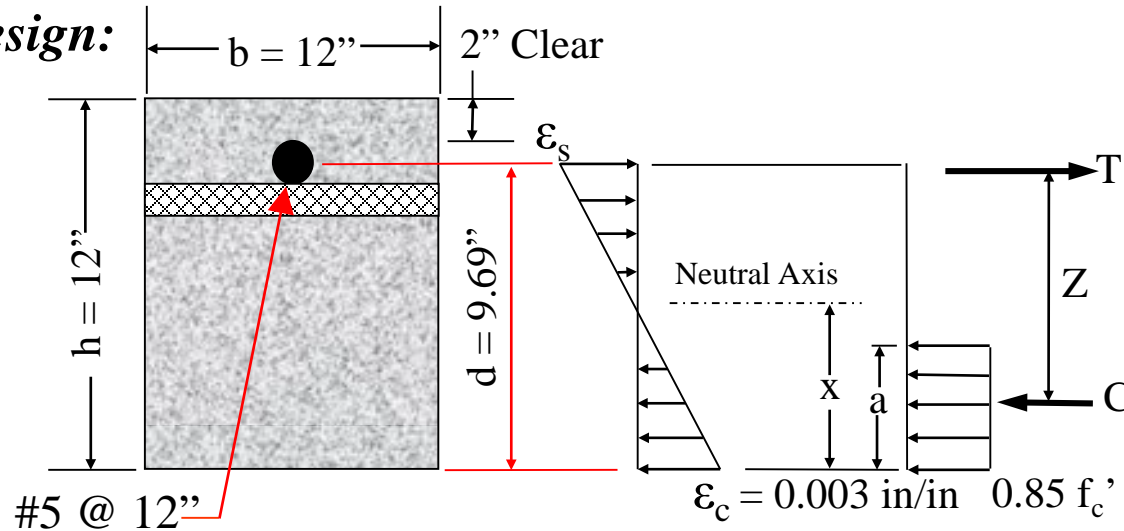
Moment:

$$M_B = (1510 \text{ lb}) \left[(4 \text{ ft}) - \frac{(4 \text{ ft}/3)(2(251.0 \text{ plf}) + 503.6 \text{ plf})}{(251.0 \text{ plf} + 503.6 \text{ plf})} \right]$$

$$M_B = 3357 \text{ lb-ft} = 3.36 \text{ kip-ft} \quad \text{Service Load}$$

Structural Design – Heel Reinf. Bending Moment Capacity ϕM_n

Preliminary Design:



Estimate reinforcement and check versus needed capacity and code required minimums

Use rule of thumb for $f_y = 60$ ksi and $f'_c = 4$ ksi : $A_S = M_u / 4d$

$A_S = M_u / 4d$ where $M_u = M_s * \text{load factors including durability factor, and}$
 $d = \text{depth of reinf.} = h - \text{cover} - \text{bar dia.}/2$

$$A_S = M_u / 4d = 2.7 (3.36^{\text{k-ft}}) / 4(9.69 \text{ in.}) = 0.234 \text{ in.}^2$$

#4 @ 12 in. $A_S = 0.20 \text{ in.}^2$

#4 @ 10 in. $A_S = 0.24 \text{ in.}^2$

#5 @ 12 in. $A_S = 0.31 \text{ in.}^2$

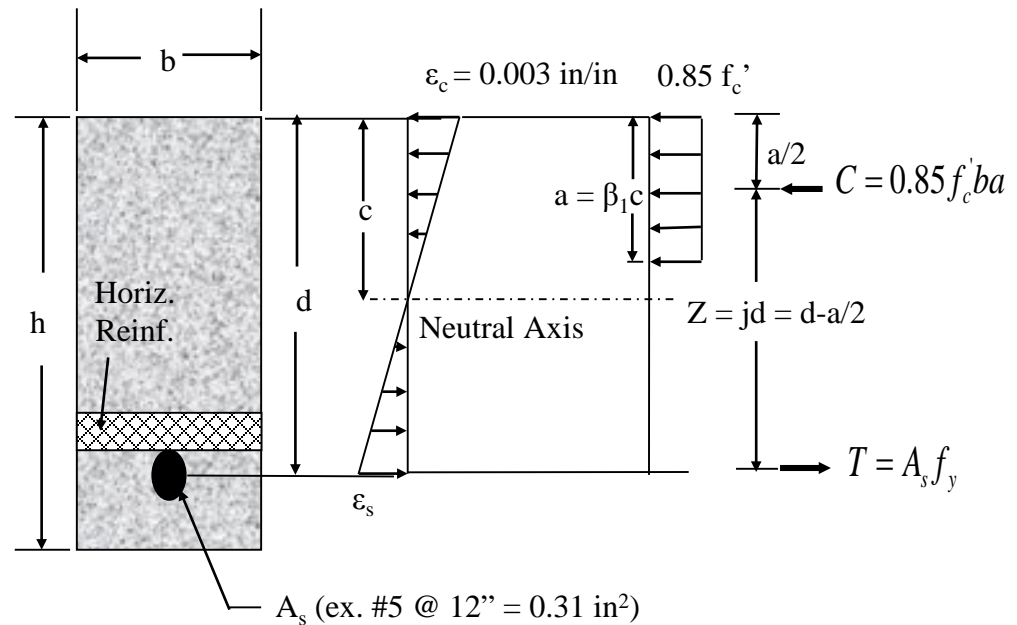
Try #5 @ 12 in.

Structural Design - Design Equations – Rule of Thumb

Rule of thumb: $A_s = M_u / 4d$

With some assumptions, traditional equations can be simplified to solve for A_s if M_u is known:

1. Section is tension controlled so use $\phi = 0.9$
2. $f_y = 60$ ksi
3. The internal moment arm ($d - a/2$) can be approximated at $0.9d$



Units:

M_n & M_u kip-ft.

A_s in.²

f_y kips/in²

d in.²

$$\phi M_n = M_u = \phi [A_s f_y (d - a/2)] / 12$$

$$M_u = 0.9 [A_s (60) (0.9d)] / 12$$

$$M_u = 4.05 A_s d \approx 4 A_s d$$

$$A_s = M_u / 4d$$

Structural Design - Heel Reinf. Bending Moment Capacity ϕM_n

Minimum Flexural Reinforcement for Beams & Retaining Walls – **ACI 350-06 10.5.1**

The larger of $A_{s,\min} = \left(\frac{3\sqrt{f'_c}}{f_y}\right)(b)(d)$ and $A_{s,\min} = \left(\frac{200}{f_y}\right)(b)(d)$

$$A_{s,\min} = 0.368 \text{ in.}^2 \qquad A_{s,\min} = 0.389 \text{ in.}^2$$

Also need to check $A_{s,\min}$ against ACI 350-06, Section 10.5.3

$$A_{s,\min} = \frac{4}{3} A_s$$

$$M_U = 1.6 M_{\text{Service}} \quad (M_U \text{ calculated without } S_d)$$

$$A'_s = M_U / 4(d) = 1.6 (3.36 \text{ k-ft}) / 4(9.69 \text{ in.}) = 0.139 \text{ in.}^2$$

$$A_{s,\min} = \frac{4}{3} (0.139 \text{ in.}^2) = 0.185 \text{ in.}^2$$

If $A_{s,\text{provided}} \geq \frac{4}{3} A_s$ then the requirements of 10.5.1 don't apply, so use the smaller of $\frac{4}{3} A_s$ and the larger section 10.5.1 provided they have adequate capacity for the loading.

$$\#5 @ 12 \text{ in.} \quad A_s = 0.31 \text{ in.}^2$$

Compare with ACI 350 Section 10.5.4

Structural Design - Heel Reinf. Bending Moment Capacity ϕM_n

ACI 350-06 Section 10.5.4 stipulates that minimum primary reinforcement for slabs, mats, and footings be equivalent to that required by Section 7.12 (Shrinkage and Temperature Reinforcement). For walls it refers to 14.3.2 and 14.3.3. Section 7.12 says account for restraint and for environmental exposure conditions or members required to be liquid tight, refer to Table 7.12.2.1.

Section 7.12.2.1 stipulates that the reinforcement in the bottom of base slabs in contact with soil may be reduced to 50% of that required in Table 7.12.2.1

Section 7.12.2.2 stipulates that the maximum spacing of the bars at 12" and the minimum bars size as a #4. It also states that at least 1/3 of the reinforcing must be in each face

Use #5 @ 12 in.

TABLE 7.12.2.1—MINIMUM SHRINKAGE AND TEMPERATURE REINFORCEMENT

Length between movement joints, ft	Minimum shrinkage and temperature reinforcement ratio	
	Grade 40	Grade 60
Less than 20	0.0030	0.0030
20 to less than 30	0.0040	0.0030
30 to less than 40	0.0050	0.0040
40 and greater	0.0060*	0.0050*

*Maximum shrinkage and temperature reinforcement where movement joints are not provided.

Note: This table applies to spacing between expansion joints and full contraction joints. When used with partial contraction joints, the minimum reinforcement ratio shall be determined by multiplying the actual length between partial contraction joints by 1.5.

Table 7.12.2.1 : $A_{s,\min} = 0.004 A_{\text{gross}}$

Based on joints at 30 to 40 feet

$$A_{s,\min} = 0.004 (12'')(12'') = 0.58 \text{ in}^2$$

#4 @ 8" each face $A_s = 0.60 \text{ in}^2$

#5 @ 12" each face $A_s = 0.62 \text{ in}^2$

#5 @ 10" each face $A_s = 0.74 \text{ in}^2$

Structural Design - Heel Reinf. Bending Moment Capacity ϕM_n

Spacing of primary reinforcement

Section 7.6.5: Spacing shall be not exceed 2 times the slab thickness, or 12 inches.

$$12 \text{ in.} < 2 (12 \text{ in.}) = 24'' \text{ OK}$$

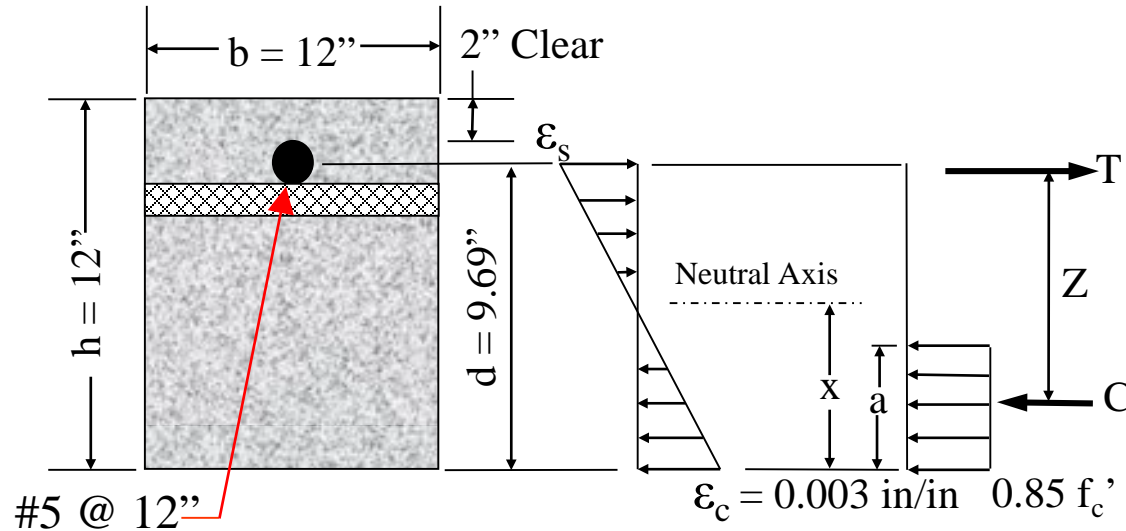
$$12 \text{ in.} \leq 12 \text{ in.} \text{ OK}$$

Section 7.12.2.2 stipulates that the maximum spacing of the bars at 12'' and the minimum bars size as a #4. It also states that at least 1/3 of the reinforcing must be in each face

$$12 \text{ in.} \leq 12 \text{ in.} \text{ OK}$$

Section 14.3.5: For walls, spacing of vertical and horizontal reinforcing shall be not exceed 12 inches.

Structural Design - Heel Reinf. Bending Moment Capacity ϕM_n



Compressive Stress Block Depth:

$$A_s = 0.31 \text{ in.}^2 \text{ for } \#5 @ 12''$$

$$a = A_s F_y / 0.85 f'_c b$$

$$= \frac{(0.31 \text{ in.}^2)(60 \text{ ksi})}{0.85(4 \text{ ksi})(12 \text{ in})}$$

$$= 0.456 \text{ in}$$

Neutral Axis Distance:

$$x = \frac{a}{0.85} = 0.536 \text{ in.}$$

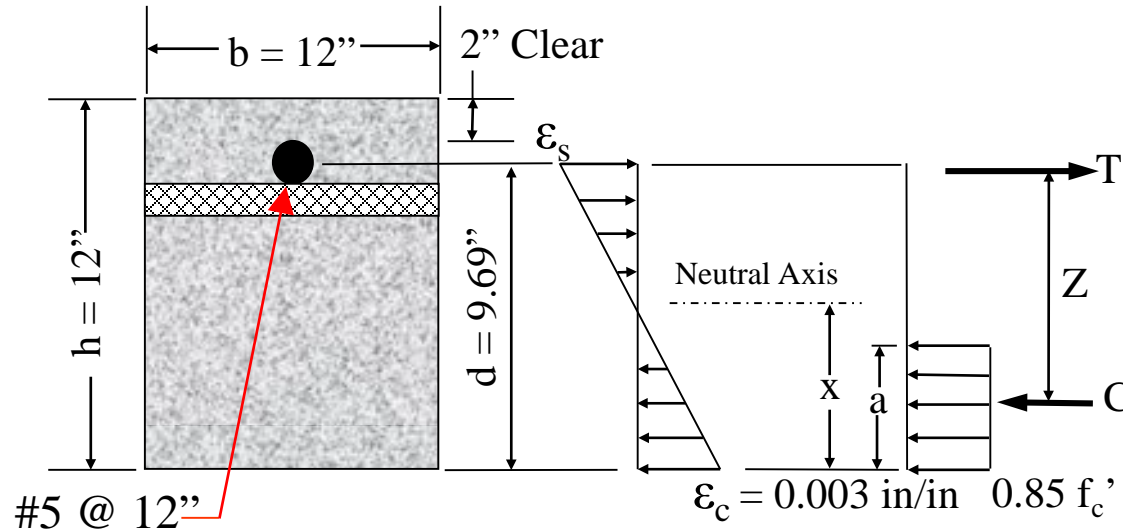
Force-Couple Distance (Z):

$$Z = d - \frac{a}{2}$$

$$Z = 9.69 \text{ in.} - \left(\frac{0.456 \text{ in.}}{2} \right)$$

$$Z = 9.46 \text{ in.}$$

Structural Design - Heel Reinf. Bending Moment Capacity ϕM_n



Strain in Reinforcing Steel:

$$\epsilon_s = \frac{(d-x)}{x} \epsilon_c$$

$$\epsilon_s = \left(\frac{(9.69 \text{ in.} - 0.536 \text{ in.})}{0.536 \text{ in.}} \right) (0.003 \text{ in./in.})$$

$$\epsilon_s = 0.0512 \text{ in./in.} \quad \epsilon_s \geq \epsilon_s \text{ min.}$$

Reinforcing Steel Strain Limits:

ACI 350-06, Section 10.3.4:

("tension controlled" with $\phi_b = 0.9$)

$$\epsilon_s \text{ min.} = 0.005 \text{ in./in.}$$

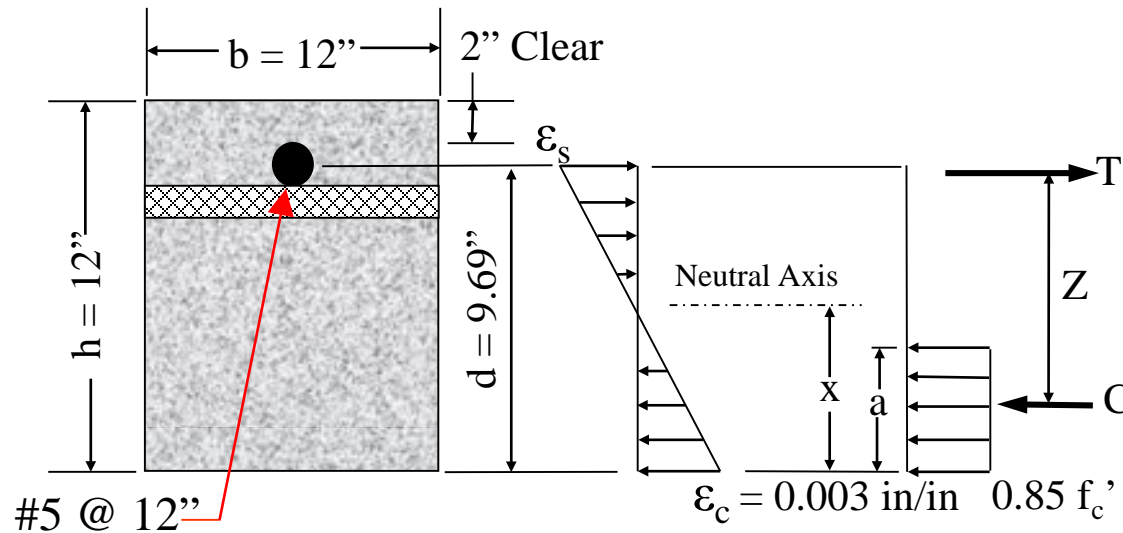
ACI 350-06, Section 10.3.5

(w/axial load ; approx $0.75 \rho_{bal}$)

$$\epsilon_s \text{ min.} = 0.004 \text{ in./in.}$$

Use $\phi_b = 0.90$

Structural Design - Heel Reinf. Bending Moment Capacity ϕM_n



Flexural Capacity (M_n):

$$\begin{aligned}
 M_n &= [(0.85f'_c)(b)(a)](Z) \\
 &= [(0.85(4 \text{ ksi})(12 \text{ in})(0.456 \text{ in})](9.46 \text{ in}) \\
 &= 176.0 \text{ k-in} = 14.67 \text{ k-ft}
 \end{aligned}$$

$$\begin{aligned}
 M_n &= (A_s f_y)(Z) \\
 M_n &= (0.31 \text{ in}^2)(60 \text{ ksi})(9.46 \text{ in}) \\
 &= 176.0 \text{ k-in} = 14.67 \text{ k-ft}
 \end{aligned}$$

$$\phi M_n = 0.9M_n = 0.9(14.67) = 13.20 \text{ k-ft} \geq M_u$$

Check Flexural Capacity & Durability Factor:

$$M_s = 3.36 \text{ k-ft}$$

$$M_u = 2.7 M_s \quad \text{from Durability Factor calculation}$$

$$\frac{\phi M_n}{M_s} = \frac{13.20}{3.36} \geq 2.7$$

$$\frac{\phi M_n}{M_s} = 3.93 \geq 2.7 \quad \text{OK}$$

Structural Design - Heel Reinf. Bending Moment Capacity ϕM_n

Actual Stress in Reinforcement due to Service Loads

$$f_s = \frac{M_s}{z A_s} = \frac{(3.36 \text{ k-ft})(12,000)}{(9.46 \text{ in.})(0.31 \text{ in}^2)} \quad \text{ACI 350-06} \quad M_s = 3.36 \text{ k-ft}$$

$Z = 9.46 \text{ in.}$

Sect.10.6.4.6

$f_s = 13,749 \text{ psi}$ **NRCS NEM 536 and NEH-6: $f_{s,max} \leq 20,000 \text{ psi}$**

Allowable Stress in Reinforcement due to Service Loads

$$\beta = \frac{h - x}{d - x} = \frac{(12 - 0.536)}{(9.69 - 0.536)} = 1.252 \quad d_b = 0.625 \text{ in.}$$

$s = 12''$ **ACI 350-06, Section 10.6.4.4**

Normal Exposure: **ACI 350-06, Sect. 10.6.4.1** Severe Exposure: **ACI 350-06, Sect. 10.6.4.2**

$$f_{s,max} = \frac{320}{\beta \sqrt{s^2 + 4 \left(2 + \frac{d_b}{2}\right)^2}} \geq 20 \text{ ksi.}$$

$$f_{s,max} = \frac{260}{\beta \sqrt{s^2 + 4 \left(2 + \frac{d_b}{2}\right)^2}} \geq 17 \text{ ksi.}$$

$$f_{s,max} = 19,874 \text{ psi}$$

$$f_{s,max} = 16,147 \text{ psi}$$

$f_s = 13,749 \text{ psi}$ $f_s \leq f_{s,max}$ **OK**

$f_s = 13,749 \text{ psi}$ $f_s \leq f_{s,max}$ **OK**

ACI 350-06 Sect. 10.6.4.5: *Normal exposure is defined as exposure to liquids with a pH > 5 or exposure to sulfate solutions of 1000 ppm or less*

Structural Design - Heel Reinf. Shrinkage & Temperature

ACI 350-06 Section 7.12 (Shrinkage and Temperature Reinforcement) says account for restraint and for environmental exposure conditions or members required to be liquid tight, refer to Table 7.12.2.1. For walls it refers to 14.3.2 and 14.3.3.

Section 7.12.2.1 stipulates that the reinforcement in the bottom of base slabs in contact with soil may be reduced to 50% of that required in Table 7.12.2.1

Section 7.12.2.2 stipulates that the maximum spacing of the bars at 12" and the minimum bars size as a #4. It also states that at least 1/3 of the reinforcing must be in each face

Table 7.12.2.1 : $A_{s,min} = 0.004 A_{gross}$

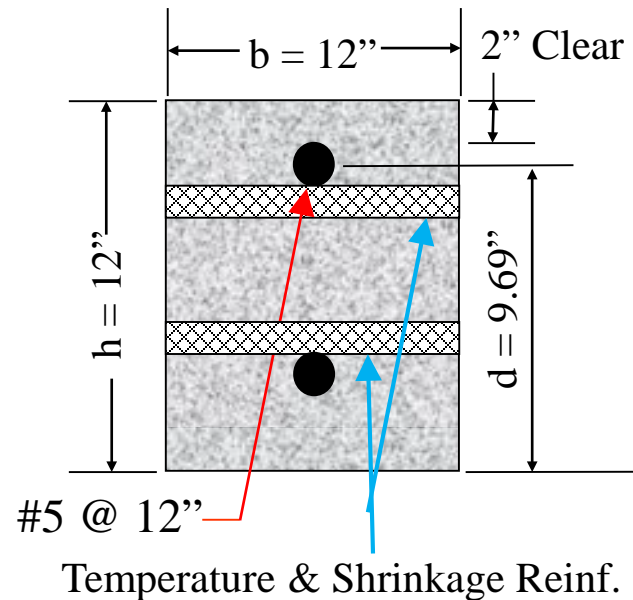
Based on joints at 30 to 40 feet

$$A_{s,min} = 0.004 (12'') (12'') = 0.58 \text{ in}^2$$

$$\#4 @ 8'' \text{ each face } A_s = 0.60 \text{ in}^2$$

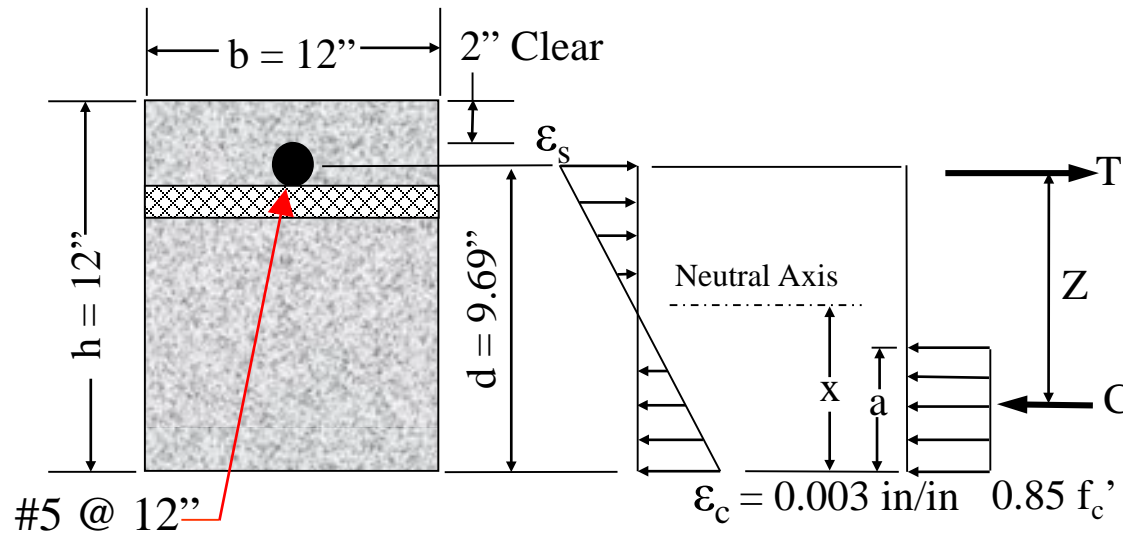
$$\#5 @ 12'' \text{ each face } A_s = 0.62 \text{ in}^2$$

$$\#5 @ 10'' \text{ each face } A_s = 0.74 \text{ in}^2$$



Use #5 @ 12 in.

Structural Design - Heel Reinforcement Shear Capacity ϕV_n



$$V_c = 2 \sqrt{f'_c} (b) (d)$$

$$V_c = 2 \sqrt{4000 \text{ psi}} (12 \text{ in}) (9.69 \text{ in})$$

$$= 14,708 \text{ lbs}$$

$$\phi V_n = \phi (V_c + V_s) \text{ where } V_s \text{ is shear reinf.} = 0$$

$$\phi V_c = 11,031 \text{ lbs} \quad \phi = 0.75$$

$$\phi V_n / V_{\text{Service}} \geq 1.88 \text{ from durability factor calc.}$$

$$\phi V_n / V_{\text{Service}} = 11031/1510 = 7.31 \geq 1.88 \text{ OK}$$

ACI 350-06, Section 9.3.2.3 :

For shear capacity, the value for the strength reduction factor, ϕ , is 0.75

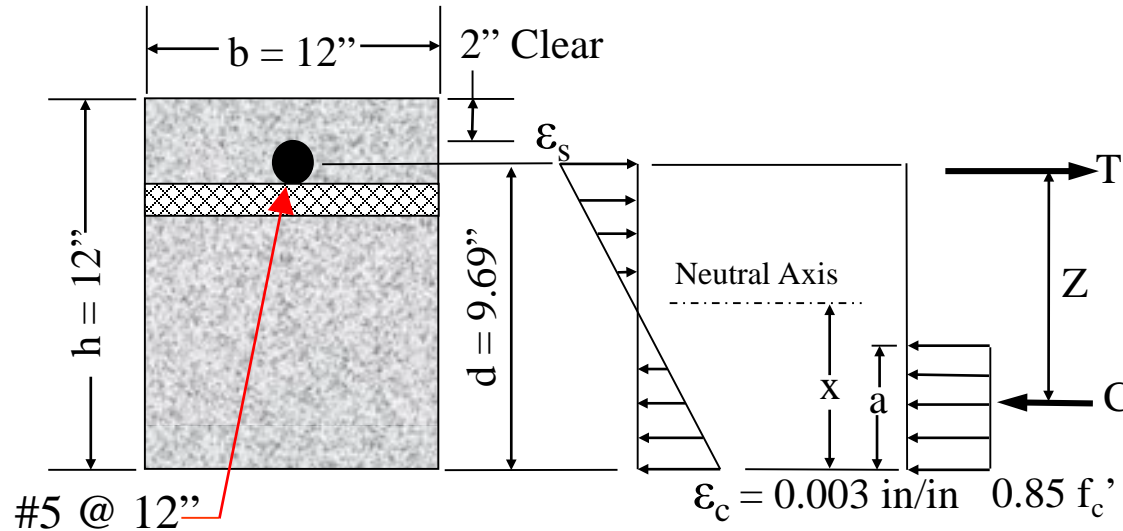
ACI 350-06, Section 11.5.5.1 :

If $V_U < \frac{1}{2} \phi V_C$ then the shear capacity of the reinforcement may be neglected

$$\phi V_n / V_{\text{Service}} \geq 2 \text{ (1.88)}$$

$$\phi V_n / V_{\text{Service}} \geq 3.76 \quad 7.31 \geq 3.76 \text{ ok}$$

Structural Design - Heel Reinforcement Shear Capacity ϕV_n



ACI 350-06, Section 11.5.5 :

$$A_v \text{ min.} = 0.75 \sqrt{f'_c} \frac{b s}{f_y}$$

Minimum shear reinforcement required where $V_U \geq \frac{1}{2} \phi V_C$

ACI 350-06, Section 11.5.5.1 :

$$A_v \text{ min.} \geq \frac{50 b s}{f_y}$$

Minimum shear reinforcement not required for:
Slabs and footings

s is spacing of shear reinforcement

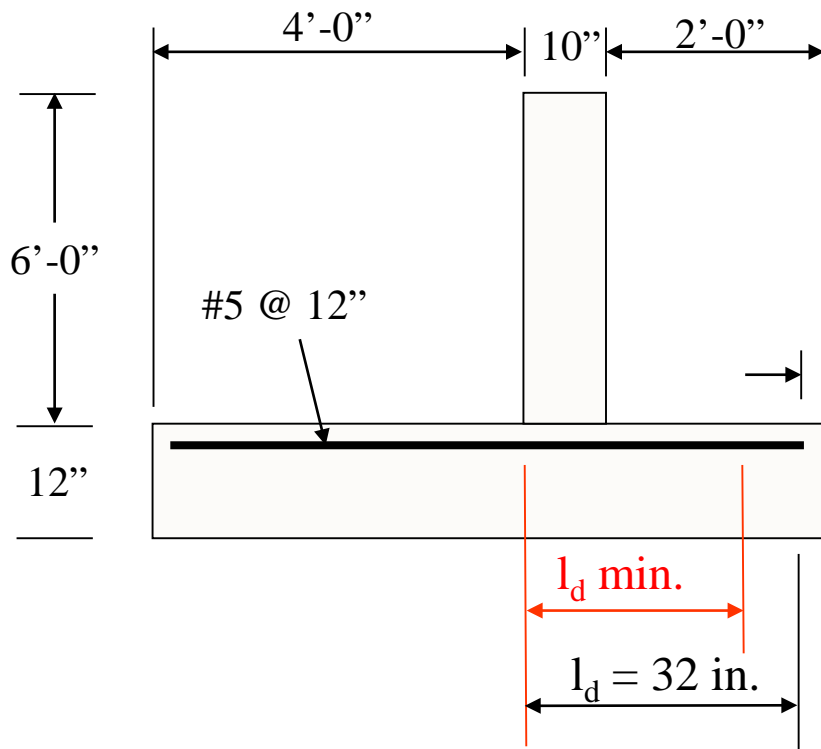
Concrete joist construction (Section 8.11)

b and s are in inches

Beams with depth, d , less than:

10 inches, 2.5x flange thickness, or $\frac{1}{2}$ web width

Structural Design – Heel Reinforcement – Development Length



ACI 350-06, Chapter 12

ACI 350-06, Section 12.2.4:

Horizontal bars with more than 12" of fresh concrete placed beneath them can develop voids underneath the bar due to settling, resulting in a reduced development length.

$$t = 9.50 \text{ in.} \quad t \leq 12 \text{ in.}, \quad \alpha = 1.0$$

Note: It is good design practice to extend bars to achieve maximum development length available

$$l_d \text{ min.} = 0.04 \left(\frac{F_y}{\sqrt{f'_c}} \right) \alpha \beta \lambda d_b$$

$\alpha = 1.3$ bars with more than 12" fresh concrete below

$\beta = 1.0$ bars not epoxy coated

$\lambda = 1.0$ normal weight concrete

ACI 350-06: Table 12.2.2

Bar size \leq No. 6

Clear cover $\geq d_b$

Bar spacing $\geq 2 (d_b)$

$$l_d \text{ min.} = 0.04 \left(\frac{60,000}{\sqrt{4000}} \right) (1.0)(1.0) (1.0) (0.625 \text{ in.}) \quad l_d \text{ min.} = 24 \text{ in.}$$

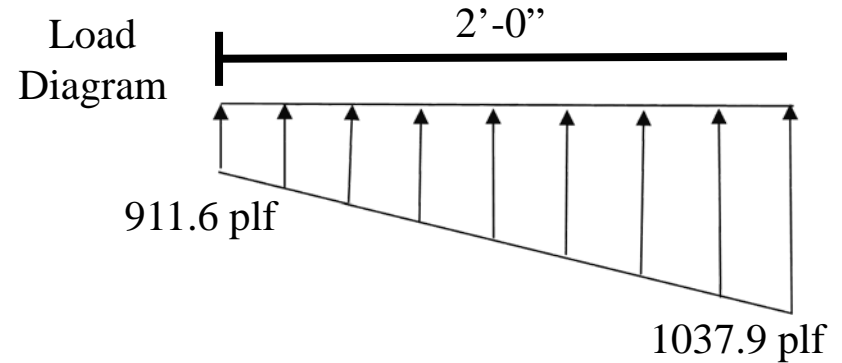
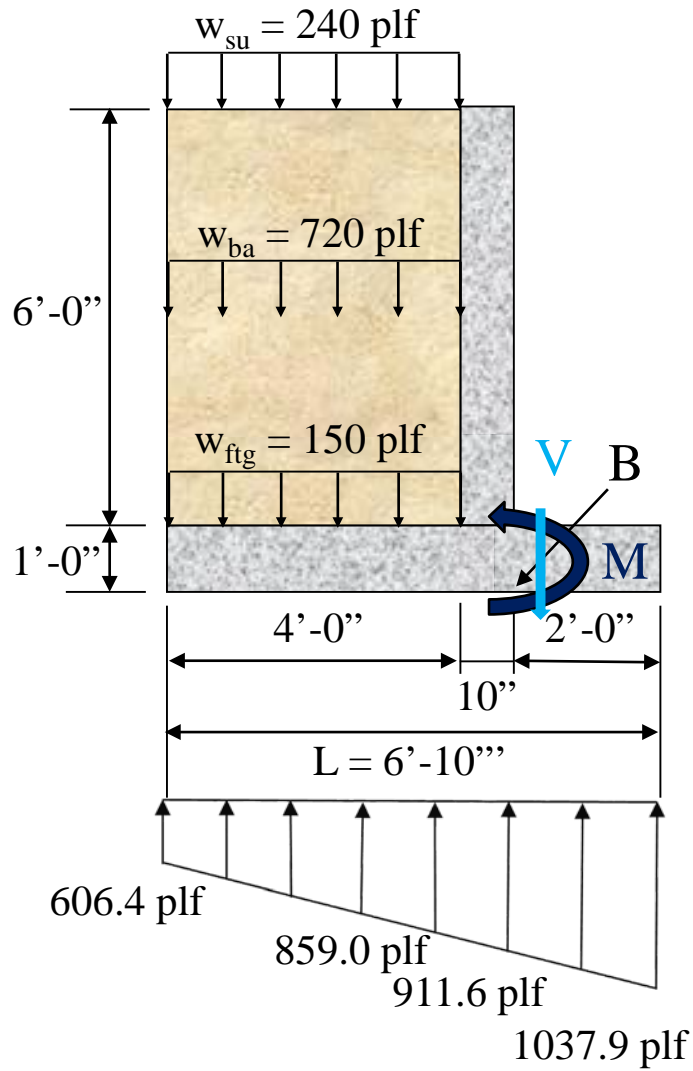
Concrete Retaining Wall Design

Structural Concrete Design

ACI 350-06

Toe Design

Structural Analysis - Toe



Shear:

$$V_B = 2 \text{ ft} (911.6 \text{ lb} + 1037.9 \text{ lb})/2$$

$$V_B = 1950 \text{ lb} = 1.95 \text{ kips}$$

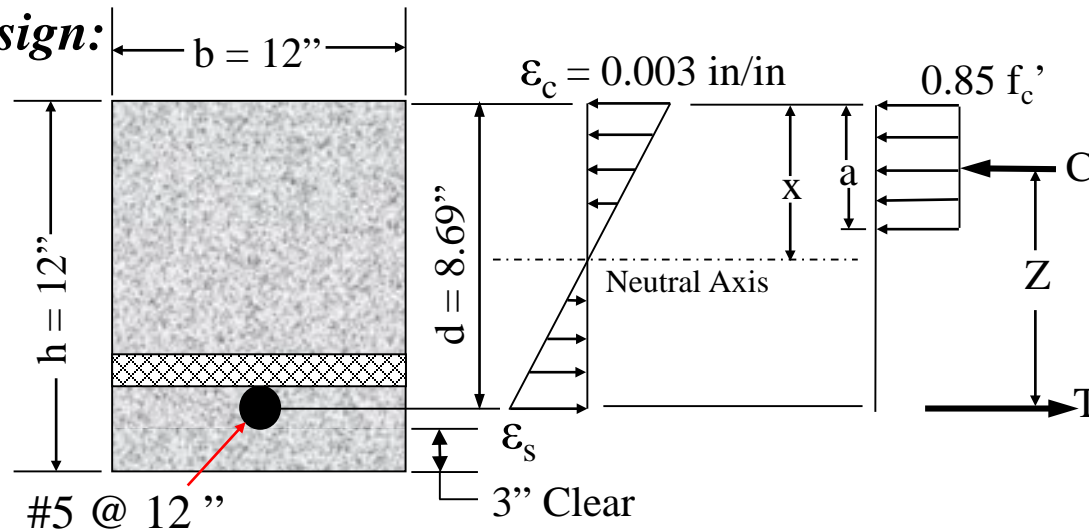
Moment:

$$M_B = (1950 \text{ lb}) \left[(2 \text{ ft}) - \frac{(2 \text{ ft}/3)(2(911.6 \text{ plf}) + 1037.9 \text{ plf})}{(911.6 \text{ plf} + 1037.9 \text{ plf})} \right]$$

$$M_B = 1992 \text{ lb-ft} = 1.99 \text{ kip-ft}$$

Structural Design – Toe Reinf. Bending Moment Capacity ϕM_n

Preliminary Design:



Estimate reinforcement and check versus needed capacity and code required minimums

Use rule of thumb for $f_y = 60$ ksi and $f'_c = 4$ ksi : $A_S = M_u / 4d$

$A_S = M_u / 4d$ where $M_u = M_s * \text{load factors including durability factor}$, and
 $d = \text{depth of reinf.} = h - \text{cover} - \text{bar dia.}/2$

$$A_S = M_U / 4d = 2.7 (1.99^{\text{k-ft}}) / 4(8.69 \text{ in.}) = 0.155 \text{ in.}^2$$

#4 @ 12 in. $A_S = 0.20 \text{ in.}^2$

#4 @ 10 in. $A_S = 0.24 \text{ in.}^2$

#5 @ 12 in. $A_S = 0.31 \text{ in.}^2$

Try #5 @ 12 in.

Structural Design - Toe Reinf. Bending Moment Capacity ϕM_n

Minimum Flexural Reinforcement for Beams & Retaining Walls – **ACI 350-06 10.5.1**

The larger of

$$A_{s,\min} = \left(\frac{3\sqrt{f'_c}}{f_y}\right)(b)(d) \quad \text{and} \quad A_{s,\min} = \left(\frac{200}{f_y}\right)(b)(d)$$
$$A_{s,\min} = 0.330 \text{ in.}^2 \qquad A_{s,\min} = 0.348 \text{ in.}^2$$

Also need to check $A_{s,\min}$ against ACI 350-06, Section 10.5.3

$$A_{s,\min} = \frac{4}{3} A_s$$

$$M_U = 1.6 M_{\text{Service}} \quad (M_U \text{ calculated without } S_d)$$

$$A'_s = M_U / 4(d) = 1.6 (1.99 \text{ k-ft}) / 4(8.769 \text{ in.}) = 0.092 \text{ in.}^2$$

$$A_{s,\min} = \frac{4}{3} A_s = \frac{4}{3} (0.092 \text{ in.}^2) = 0.123 \text{ in.}^2$$

If $A_{s,\text{provided}} \geq \frac{4}{3} A_s$ then the requirements of 10.5.1 don't apply, so use the smaller of $\frac{4}{3} A_s$ and the larger section 10.5.1 provided they have adequate capacity for the loading.

$$\#5 @ 12 \text{ in.} \quad A_s = 0.31 \text{ in.}^2$$

Compare with ACI 350 Section 10.5.4

Structural Design - Toe Reinf. Bending Moment Capacity ϕM_n

ACI 350-06 Section 10.5.4 stipulates that minimum primary reinforcement for slabs, mats, and footings be equivalent to that required by Section 7.12 (Shrinkage and Temperature Reinforcement). For walls it refers to 14.3.2 and 14.3.3. Section 7.12 says account for restraint and for environmental exposure conditions or members required to be liquid tight, refer to Table 7.12.2.1.

Section 7.12.2.1 stipulates that the reinforcement in the bottom of base slabs in contact with soil may be reduced to 50% of that required in Table 7.12.2.1

Section 7.12.2.2 stipulates that the maximum spacing of the bars at 12" and the minimum bars size as a #4. It also states that at least 1/3 of the reinforcing must be in each face

To simplify construction, coordinate with heel reinforcing sizing

Use #5 @ 12 in.

TABLE 7.12.2.1—MINIMUM SHRINKAGE AND TEMPERATURE REINFORCEMENT

Length between movement joints, ft	Minimum shrinkage and temperature reinforcement ratio	
	Grade 40	Grade 60
Less than 20	0.0030	0.0030
20 to less than 30	0.0040	0.0030
30 to less than 40	0.0050	0.0040
40 and greater	0.0060*	0.0050*

*Maximum shrinkage and temperature reinforcement where movement joints are not provided.

Note: This table applies to spacing between expansion joints and full contraction joints. When used with partial contraction joints, the minimum reinforcement ratio shall be determined by multiplying the actual length between partial contraction joints by 1.5.

Table 7.12.2.1 : $A_{s,min} = 0.004 A_{gross}$

Based on joints at 30 to 40 feet

$$A_{s,min} = \frac{1}{2} (0.004)(12'')(12'')$$

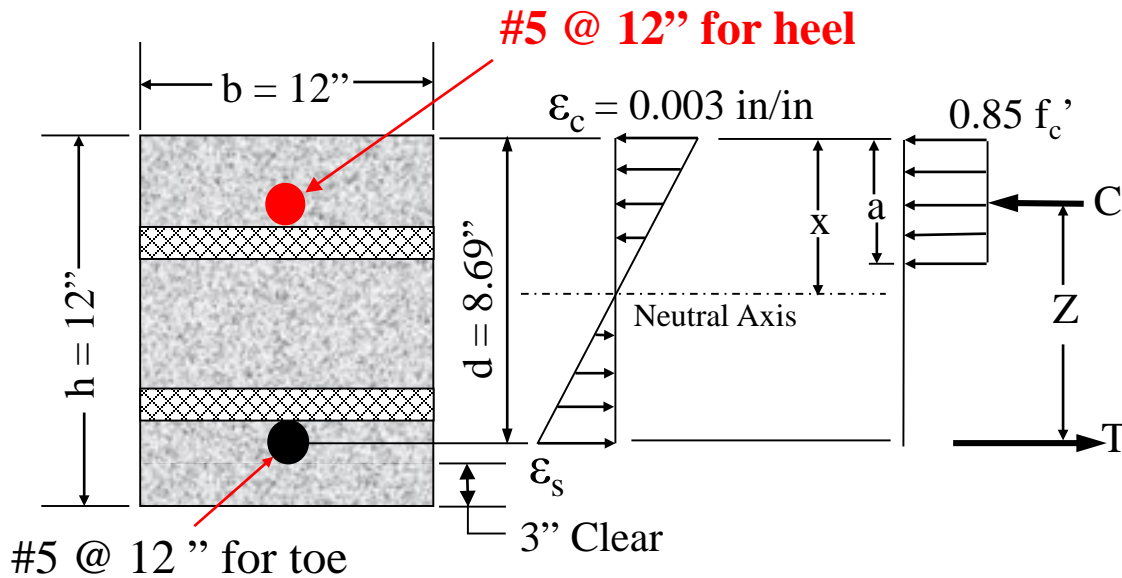
$$A_{s,min} = 0.29 \text{ in}^2.$$

$$\#5 @ 12'' \text{ each face } A_s = 0.62 \text{ in}^2$$

$$\#4 @ 8'' \text{ each face } A_s = 0.60 \text{ in}^2$$

$$\#4 @ 12'' \text{ each face } A_s = 0.40 \text{ in}^2$$

Structural Design - Toe Reinf. Bending Moment Capacity ϕM_n



Preliminary Design: 0.155 in^2 approx. #4 @ 15''

Minimum Required: 0.123 in^2 approx. #4 @ 18''

Per Section 10.5.4: 0.290 in^2 approx. #4 @ 12'' at each face

Since top bar for heel is #5 @ 12'' use same bar size and spacing for bottom bars:

Use #5 @ 12''

Structural Design - Toe Reinf. Bending Moment Capacity ϕM_n

Spacing of primary reinforcement

Section 7.6.5: Spacing shall be not exceed 2 times the slab thickness, or 12 inches.

$$12 \text{ in.} < 2 (12 \text{ in.}) = 24'' \text{ OK}$$

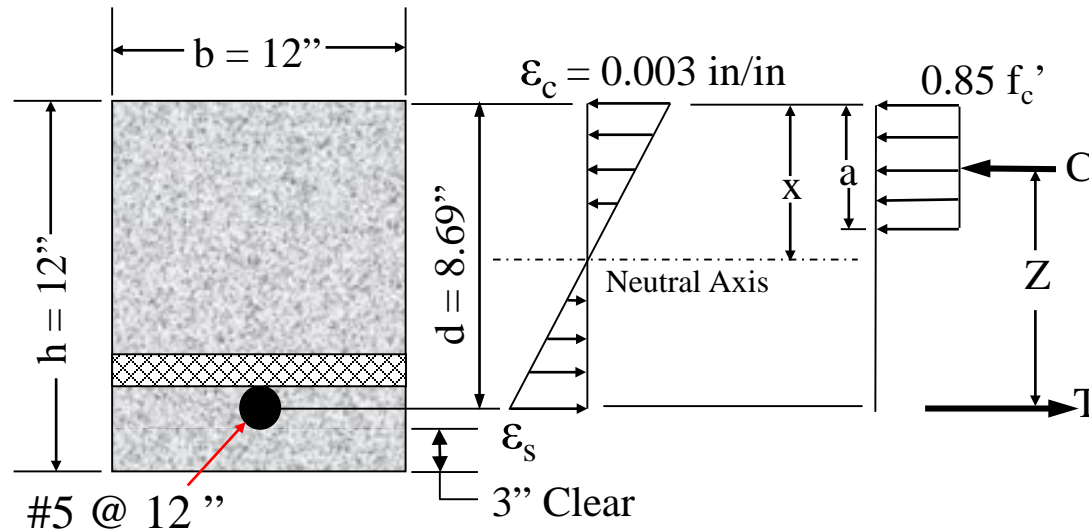
$$12 \text{ in.} < 12 \text{ in.} \text{ OK}$$

Section 7.12.2.2 stipulates that the maximum spacing of the bars at 12” and the minimum bars size as a #4. It also states that at least 1/3 of the reinforcing must be in each face

$$12 \text{ in.} < 12 \text{ in.} \text{ OK}$$

Section 14.3.5: For walls, spacing of vertical and horizontal reinforcing shall be not exceed 12 inches.

Structural Design - Toe Reinf. Bending Moment Capacity ϕM_n



Compressive Stress Block Depth:

$$A_s = 0.31 \text{ in.}^2 \text{ for } \#5 @ 12''$$

$$a = A_s F_y / 0.85 f'_c b$$

$$= \frac{(0.31 \text{ in.}^2)(60 \text{ ksi})}{0.85(4 \text{ ksi})(12 \text{ in})}$$

$$= 0.456 \text{ in}$$

Neutral Axis Distance:

$$x = \frac{a}{0.85} = 0.536 \text{ in.}$$

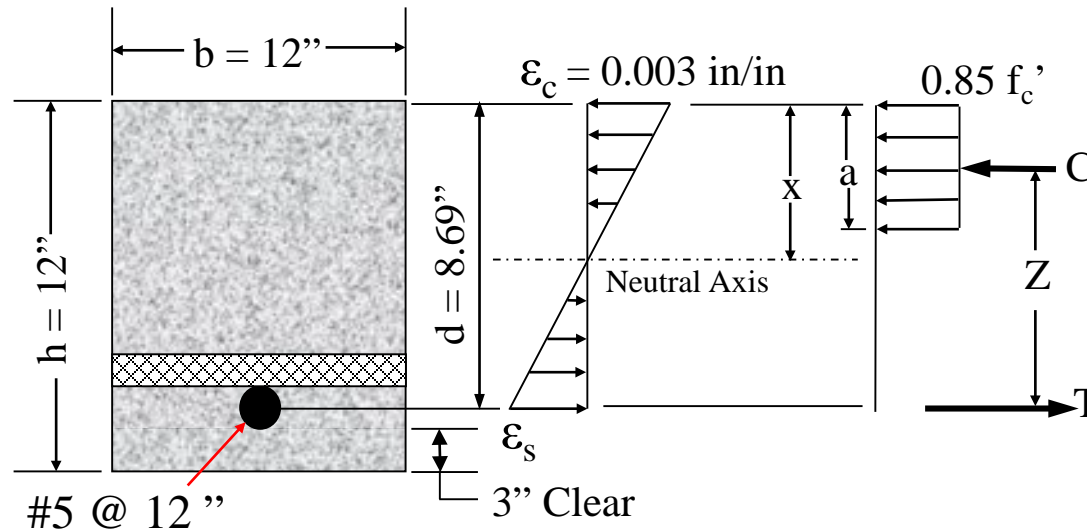
Force-Couple Distance (Z):

$$Z = d - \frac{a}{2}$$

$$Z = 8.69 \text{ in.} - \left(\frac{0.456 \text{ in.}}{2} \right)$$

$$Z = 8.46 \text{ in.}$$

Structural Design - Toe Reinf. Bending Moment Capacity ϕM_n



Strain in Reinforcing Steel:

$$\epsilon_s = \frac{(d-x)}{X} \epsilon_c = \left(\frac{(8.69 \text{ in.} - 0.536 \text{ in.})}{0.536 \text{ in.}} \right) (0.003 \text{ in./in.})$$

$$\epsilon_s = 0.0456 \text{ in./in.} \quad \epsilon_s \geq \epsilon_s \text{ min.}$$

Use $\phi_b = 0.90$

Reinforcing Steel Strain Limits:

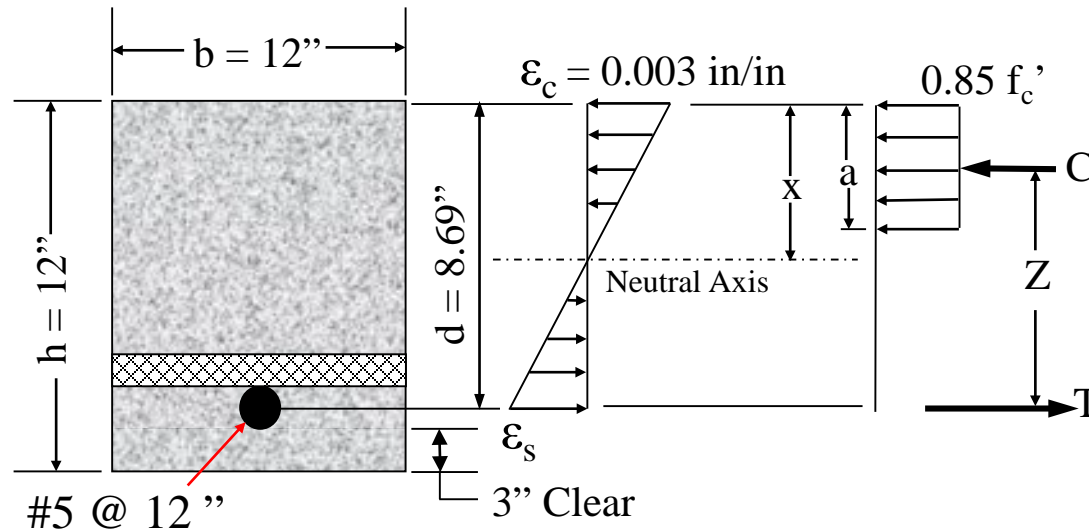
ACI 350-06, Section 10.3.4

$$\epsilon_s \text{ min.} = 0.005 \text{ in./in.}$$

ACI 350-06, Section 10.3.5

$$\epsilon_s \text{ min.} = 0.004 \text{ in./in.}$$

Structural Design - Toe Reinf. Bending Moment Capacity ϕM_n



Flexural Capacity (M_n):

$$\begin{aligned} M_n &= [(0.85f'_c)(b)(a)](Z) \\ &= [(0.85(4 \text{ ksi})(12 \text{ in})(0.456 \text{ in})](8.46 \text{ in}) \\ &= 157.4 \text{ k-in} = 13.12 \text{ k-ft} \end{aligned}$$

$$\begin{aligned} M_n &= (A_s f_y)(Z) \\ M_n &= (0.31 \text{ in}^2)(60 \text{ ksi})(8.46 \text{ in}) \\ &= 157.4 \text{ k-in} = 13.12 \text{ k-ft} \end{aligned}$$

$$\phi M_n = 0.9M_n = 11.81 \text{ k-ft} \geq M_u$$

Check Flexural Capacity & Durability Factor:

$$M_s = 1.99 \text{ k-ft}$$

$$M_u = 2.7 M_s \quad \text{From Durability Factor calculation}$$

$$\frac{\phi M_n}{M_s} = \frac{11.81}{1.99} \geq 2.7$$

$$\frac{\phi M_n}{M_s} = 5.93 \geq 2.7 \quad \text{OK}$$

Structural Design - Toe Reinf. Bending Moment Capacity ϕM_n

Actual Stress in Reinforcement due to Service Loads

$$f_s = \frac{M_s}{z A_s} = \frac{(1.99 \text{ k-ft})(12,000)}{(8.46 \text{ in.})(0.31 \text{ in}^2)} \quad \text{ACI 350-06} \quad M_s = 1.99 \text{ k-ft}$$

Z = 8.46 in.

Sect.10.6.4.6

$$f_s = 9,105 \text{ psi}$$

Allowable Stress in Reinforcement due to Service Loads

$$\beta = \frac{h - x}{d - x} = \frac{(12 - 0.536)}{(8.69 - 0.536)} = 1.406 \quad d_b = 0.625 \text{ in.}$$

ACI 350-06, Section 10.6.4.4

s = 12"

Normal Exposure: *ACI 350-06, Sect. 10.6.4.1* Severe Exposure: *ACI 350-06, Sect. 10.6.4.2*

$$f_{s, \max} = \frac{320}{\beta \sqrt{s^2 + 4 \left(2 + \frac{d_b}{2}\right)^2}} \geq 20 \text{ ksi.}$$

$$f_{s, \max} = \frac{260}{\beta \sqrt{s^2 + 4 \left(2 + \frac{d_b}{2}\right)^2}} \geq 17 \text{ ksi.}$$

$$f_{s, \max} = 17,697 \text{ psi}$$

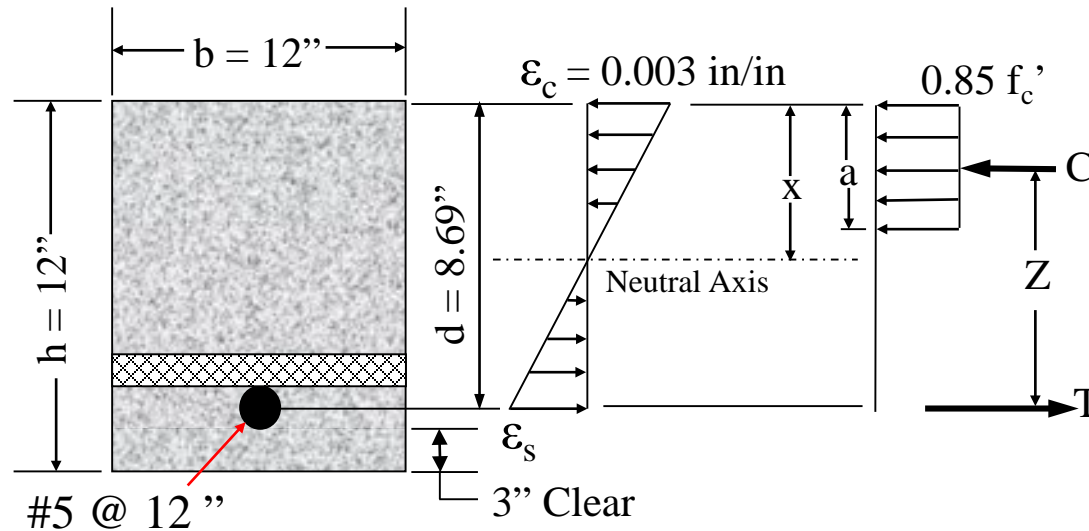
$$f_{s, \max} = 14,379 \text{ psi}$$

$$f_s = 9,105 \text{ psi} \quad f_s \leq f_{s, \max} \quad \text{OK}$$

$$f_s = 9,105 \text{ psi} \quad f_s \leq f_{s, \max} \quad \text{OK}$$

NRCS NEM 536 and NEH-6: $f_{s, \max} \leq 20,000 \text{ psi}$

Structural Design - Toe Reinforcement Shear Capacity ϕV_n



$$V_c = 2 \sqrt{f'_c} (b) (d)$$

$$V_c = 2 \sqrt{4000 \text{ psi}} (12 \text{ in}) (8.69 \text{ in})$$

$$= 13,190 \text{ lbs}$$

$$\phi V_n = \phi (V_c + V_s) \text{ where } V_s \text{ is shear reinf.} = 0$$

$$\phi V_c = 9,893 \text{ lbs} \quad \phi = 0.75$$

$$\phi V_n / V_{\text{Service}} \geq 1.88 \text{ from durability factor calc.}$$

$$\phi V_n / V_{\text{Service}} = 9893 / 1950 = 5.07 \geq 1.88 \text{ OK}$$

ACI 350-06, Section 9.3.2.3 :

For shear capacity, the value for the strength reduction factor, ϕ , is 0.75

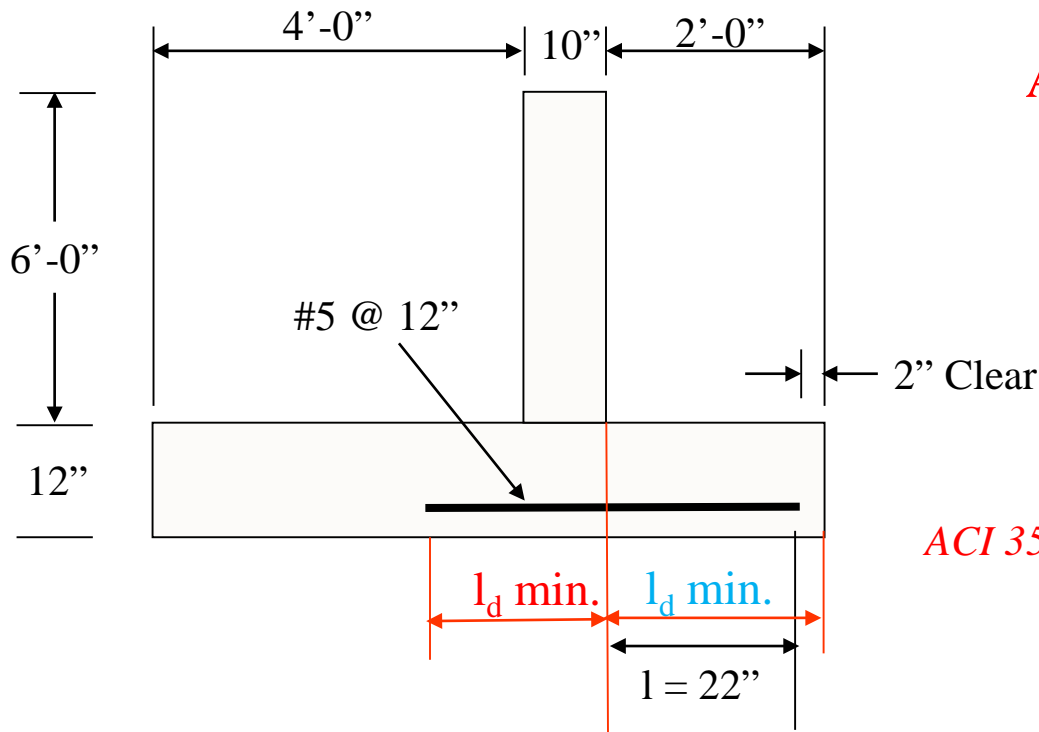
ACI 350-06, Section 11.5.5.1 :

If $V_U < \frac{1}{2} \phi V_c$ then the shear capacity of the reinforcement may be neglected

$$\phi V_n / V_{\text{Service}} \geq 2 \text{ (1.88)}$$

$$\phi V_n / V_{\text{Service}} \geq 3.76 \quad 5.07 \geq 3.76 \text{ ok}$$

Structural Design – Toe Reinforcement – Development Length



ACI 350-06, Section 12.2

ACI 350-06: Table 12.2.2 *Bar size* \leq No. 6
Clear cover $\geq d_b$
Bar spacing $\geq 2 (d_b)$

$\lambda = 1.0$ for normal weight concrete

$\beta = 1.0$ bars not epoxy coated

$\alpha = 1.0$ bars have less than 12" fresh concrete below

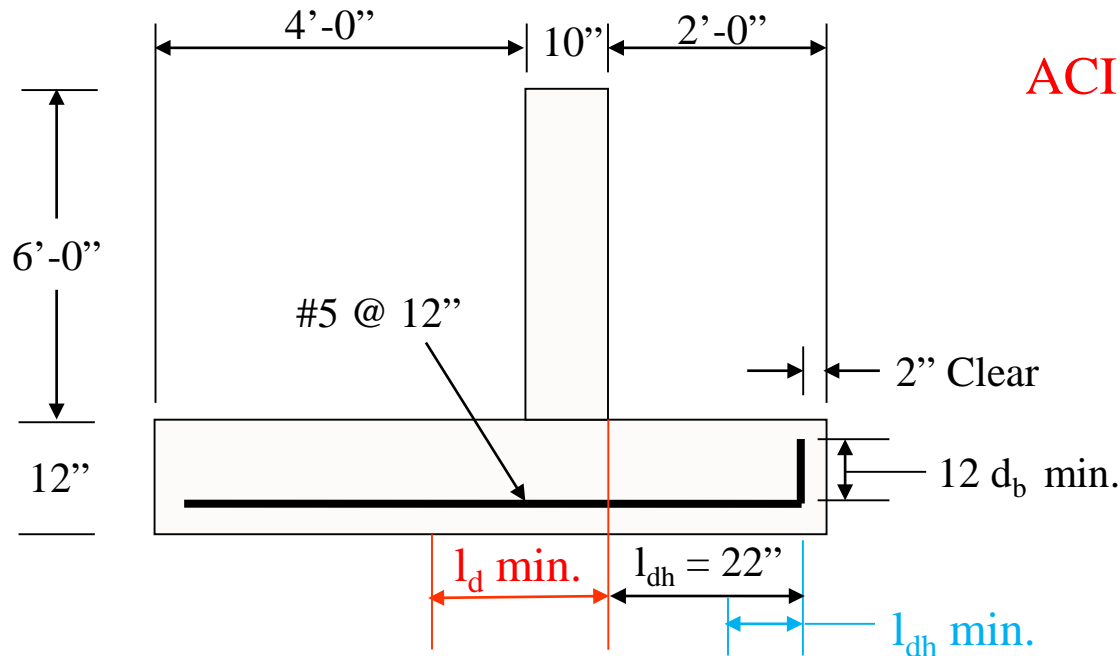
$$l_d \text{ min.} = 0.04 \left(\frac{F_y}{\sqrt{f'_c}} \right) \alpha \beta \lambda d_b$$

$$l_d \text{ min.} = 0.04 \left(\frac{60,000}{\sqrt{4000}} \right) (1.0)(1.0) (1.0)(0.625 \text{ in.})$$

$$l_d \text{ min.} = 24 \text{ in.} > l_{d \text{ prov}}$$

Because the toe reinforcing bar length available (22") is less than that required to fully develop the bar strength (24"), a hooked end should be used

Structural Design – Toe Reinf. – Development Length - Hooked Bar



ACI 350-06, Section 12.5

Note: It is good design practice to extend bars to achieve maximum development length available

ACI 350-06, Section 12.5.2

$$l_{dh} \text{ min.} = 0.02 \left(\frac{F_y}{\sqrt{f'_c}} \right) \beta \lambda d_b$$

$\beta = 1.0$ bars not epoxy coated
 $\lambda = 1.0$ for normal weight concrete

ACI 350-06, Section 12.5.3 0.7 for side cover ≥ 2.5 in. & 90° hook cover ≥ 2 in.

Other factors: 1.0 for no confinement along l_{dh} & no confinement along hook tail length

$$l_{dh} \text{ min.} = 0.02 \left(\frac{60,000}{\sqrt{4000}} \right) (1.0) (1.0) (d_b = 0.625) (0.7) (1.0) = 8.3 \text{ in. use } 9 \text{ in.}$$

ACI 350-06, Section 12.5.1

$$l_{dh} \geq l_{dh} \text{ min.} = 8.3 \text{ in.} \quad l_{dh} \geq 8 d_b \quad l_{dh} \geq 6 \text{ in.}$$

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THE END

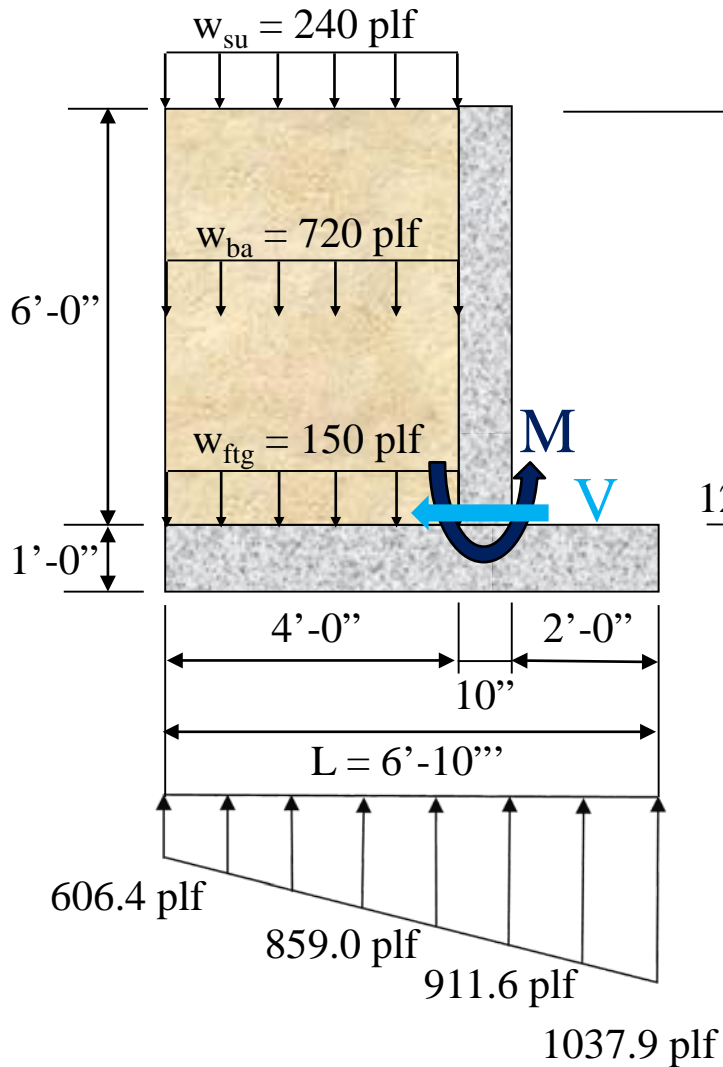
Concrete Retaining Wall Design

Structural Concrete Design

ACI 350-06

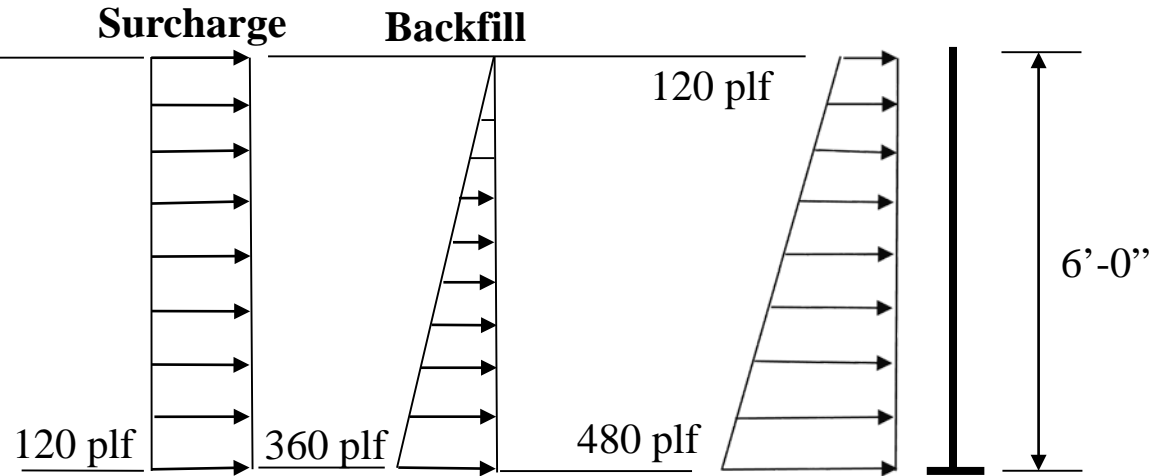
Stem Design

Structural Analysis - Wall



Load Diagram

Combined Load Diagram



Shear:

$$V_B = 6 \text{ ft} (480 \text{ lb} + 120 \text{ lb})/2$$

$$V_B = 1800 \text{ lb} = 1.80 \text{ kips}$$

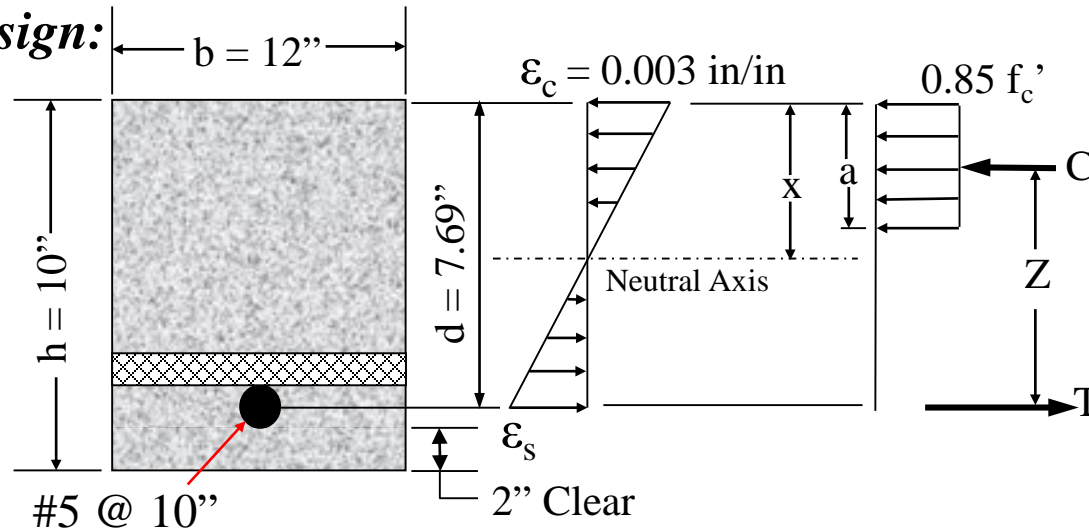
Moment:

$$M_B = (1800 \text{ lb}) \left[(6 \text{ ft}) - \frac{(6 \text{ ft}/3)(2(120 \text{ plf}) + 480 \text{ plf})}{(120 \text{ plf} + 480 \text{ plf})} \right]$$

$$M_B = 4320 \text{ lb-ft} = 4.32 \text{ kip-ft}$$

Structural Design – Wall Reinf. Bending Moment Capacity ϕM_n

Preliminary Design:



Estimate reinforcement and check versus needed capacity and code required minimums

Use rule of thumb for $f_y = 60$ ksi and $f'_c = 4$ ksi : $A_S = M_u / 4d$

$A_S = M_u / 4d$ where $M_u = M_s$ * load factors including durability factor, and
 d = depth of reinf. = h - cover - bar dia./2

$$A_S = M_U / 4d = 2.7 (4.32^{\text{k-ft}}) / 4(7.69 \text{ in.}) = 0.379 \text{ in.}^2$$

#5 @ 12 in. $A_S = 0.31 \text{ in.}^2$

#5 @ 10 in. $A_S = 0.37 \text{ in.}^2$

#5 @ 8 in. $A_S = 0.46 \text{ in.}^2$

Try #5 @ 10 in.

Structural Design - Wall Reinf. Bending Moment Capacity ϕM_n

Minimum Flexural Reinforcement for Beams & Retaining Walls – **ACI 350-06 10.5.1**

The larger of

$$A_{s,\min} = \left(\frac{3\sqrt{f'_c}}{f_y}\right)(b)(d) \quad \text{and} \quad A_{s,\min} = \left(\frac{200}{f_y}\right)(b)(d)$$
$$A_{s,\min} = 0.292 \text{ in.}^2 \qquad A_{s,\min} = 0.308 \text{ in.}^2$$

Also need to check $A_{s,\min}$ against ACI 350-06, Section 10.5.3

$$A_{s,\min} = \frac{4}{3} A_s$$

$$M_U = 1.6 M_{\text{Service}} \quad (M_U \text{ calculated without } S_d)$$

$$A'_s = M_U / 4(d) = 1.6 (4.32 \text{ k-ft}) / 4(7.69 \text{ in.}) = 0.225 \text{ in.}^2$$

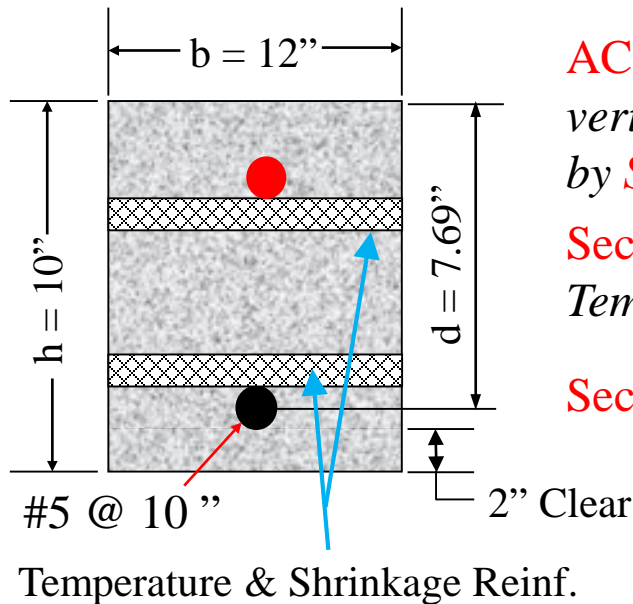
$$A_{s,\min} = \frac{4}{3} A_s = \frac{4}{3} (0.225 \text{ in.}^2) = 0.300 \text{ in.}^2$$

If $A_{s,\text{provided}} \geq \frac{4}{3} A_s$ then the requirements of 10.5.1 don't apply, so use the smaller of $\frac{4}{3} A_s$ and the larger section 10.5.1 provided they have adequate capacity for the loading.

$$\#5 @ 10 \text{ in.} \quad A_s = 0.372 \text{ in.}^2$$

Compare with ACI 350 Section 10.5.4

Structural Design - Wall Reinf. Bending Moment Capacity ϕM_n



ACI 350-06 Section 10.5.4 stipulates that minimum primary vertical reinforcement for walls be equivalent to that required by **Sections 14.3.2 and 14.3.3**.

Section 14.3.3 states horizontal reinforcement (Shrinkage and Temperature Reinforcement) should conform to **Table 7.12.2.1**.

Section 14.3.2:

$$A_{vert \ min} = 0.003 A_{Gross}$$

$$= 0.003 (120 \text{ in.}^2) = 0.360 \text{ in.}^2$$

Use #5 @ 10" back face, $A_s = 0.372 \text{ in.}^2$

ACI 350-06, Section 14.3.5: *Max. bar spacing is 12 in.*

ACI 350-06, Section 14.3.4: *Walls > 10" thick shall have two layers of reinforcement, parallel to the wall faces. No less than 1/3 of the required reinforcing steel area shall be placed in each face.*

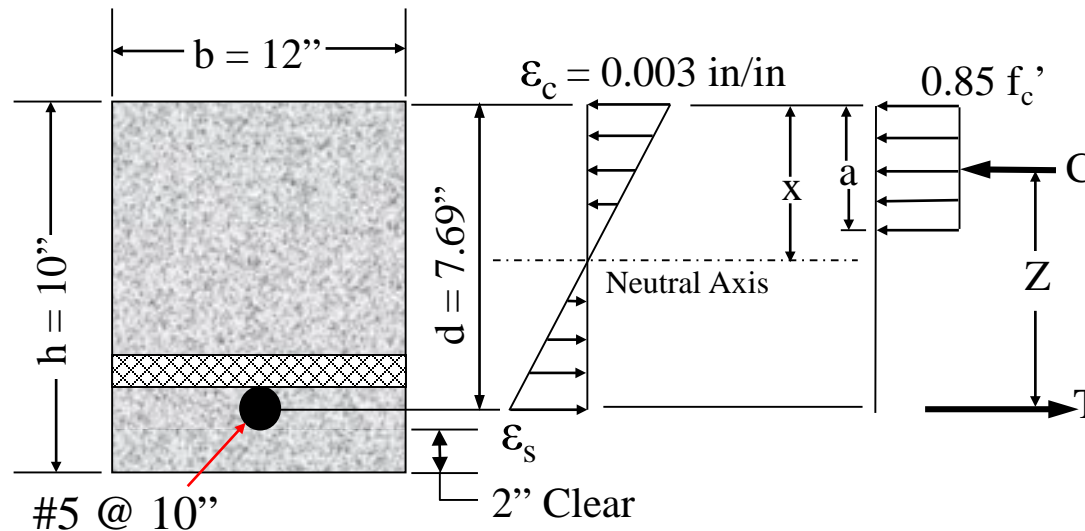
Front Face Reinforcing:

For smaller structures, some designers prefer to use the same bar size and spacing as the back face, in an effort to reduce construction errors.

Some designers use temperature and shrinkage requirements to size front face vertical bars for cantilevered walls.

Use #5 @ 10" front face $A_s = 0.372 \text{ in.}^2$ to match back face

Structural Design - Wall Reinf. Bending Moment Capacity ϕM_n



Compressive Stress Block Depth:

$$A_s = 0.372 \text{ in.}^2 \text{ for } \#5 @ 10''$$

$$a = A_s F_y / 0.85 f'_c b$$

$$= \frac{(0.372 \text{ in.}^2)(60 \text{ ksi})}{0.85(4 \text{ ksi})(12 \text{ in})}$$

$$= 0.547 \text{ in}$$

Neutral Axis Distance:

$$x = \frac{a}{0.85} = 0.644 \text{ in.}$$

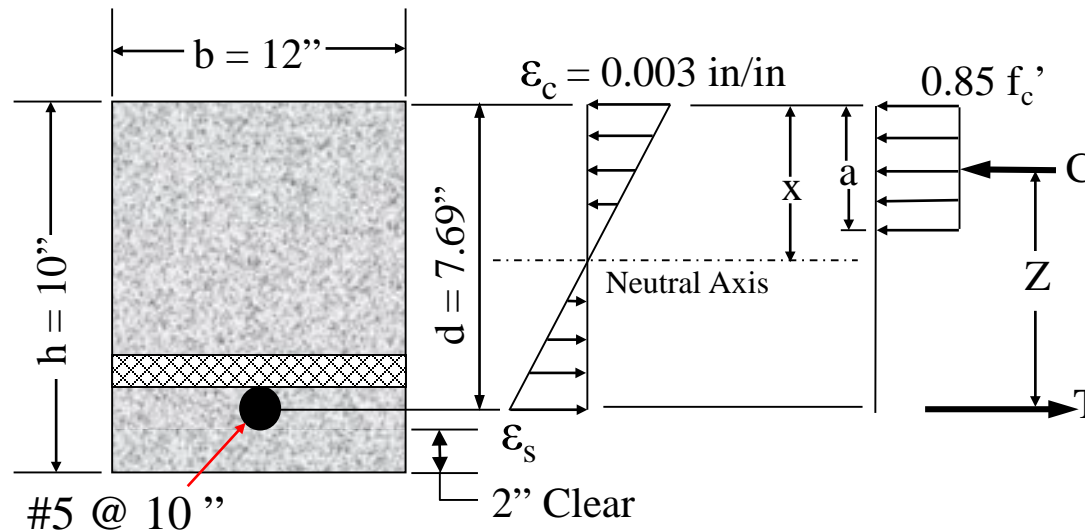
Force-Couple Distance (Z):

$$Z = d - \frac{a}{2}$$

$$Z = 7.69 \text{ in.} - \left(\frac{0.547 \text{ in.}}{2} \right)$$

$$Z = 7.42 \text{ in.}$$

Structural Design - Wall Reinf. Bending Moment Capacity ϕM_n



Strain in Reinforcing Steel:

$$\epsilon_s = \frac{(d-x)}{X} \epsilon_c = \left(\frac{(7.69 \text{ in.} - 0.644 \text{ in.})}{0.644 \text{ in.}} \right) (0.003 \text{ in./in.})$$

$$\epsilon_s = 0.0328 \text{ in./in.} \quad \epsilon_s \geq \epsilon_s \text{ min.}$$

Use $\phi_b = 0.90$

Reinforcing Steel Strain Limits:

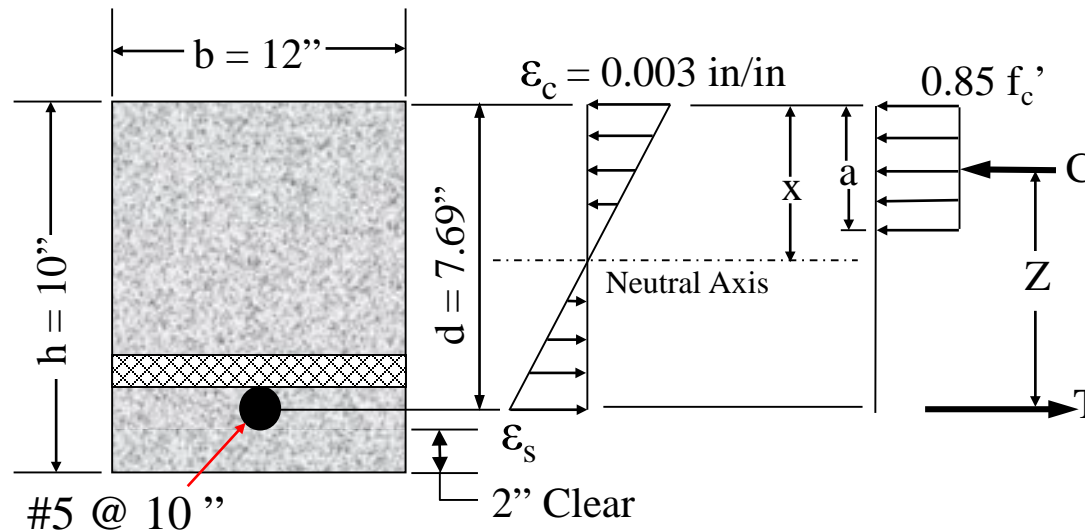
ACI 350-06, Section 10.3.4

$$\epsilon_s \text{ min.} = 0.005 \text{ in./in.}$$

ACI 350-06, Section 10.3.5

$$\epsilon_s \text{ min.} = 0.004 \text{ in./in.}$$

Structural Design - Wall Reinf. Bending Moment Capacity ϕM_n



Flexural Capacity (M_n):

$$\begin{aligned} M_n &= [(0.85f'_c)(b)(a)](Z) \\ &= [(0.85(4 \text{ ksi})(12 \text{ in})(0.547 \text{ in})](7.42 \text{ in}) \\ &= 165.6 \text{ k-in} = 13.80 \text{ k-ft} \end{aligned}$$

$$\begin{aligned} M_n &= (A_s f_y)(Z) \\ M_n &= (0.372 \text{ in}^2)(60 \text{ ksi})(7.42 \text{ in}) \\ &= 165.6 \text{ k-in} = 13.80 \text{ k-ft} \end{aligned}$$

$$\phi M_n = 0.9M_n = 12.42 \text{ k-ft} \geq M_u$$

Check Flexural Capacity & Durability Factor:

$$M_s = 4.32 \text{ k-ft}$$

$$M_u = 2.7 M_s \quad \text{From Durability Factor calculation}$$

$$\frac{\phi M_n}{M_s} = \frac{12.42}{4.32} \geq 2.7$$

$$\frac{\phi M_n}{M_s} = 2.88 \geq 2.7 \quad \text{OK}$$

Structural Design - Wall Reinf. Bending Moment Capacity ϕM_n

Actual Stress in Reinforcement due to Service Loads

$$f_s = \frac{M_s}{z A_s} = \frac{(1.99 \text{ k-ft})(12,000)}{(7.42 \text{ in.})(0.372 \text{ in}^2)} \quad \text{ACI 350-06 Sect.10.6.4.6} \quad \begin{matrix} M_s = 4.32 \text{ k-ft} \\ Z = 7.42 \text{ in.} \end{matrix}$$

$$f_s = 18,781 \text{ psi}$$

Allowable Stress in Reinforcement due to Service Loads

$$\beta = \frac{h - x}{d - x} = \frac{(10 - 0.644)}{(7.69 - 0.644)} = 1.328 \quad \begin{matrix} d_b = 0.625 \text{ in.} \\ s = 10'' \end{matrix} \quad \text{ACI 350-06, Section 10.6.4.4}$$

Normal Exposure: **ACI 350-06, Sect. 10.6.4.1** Severe Exposure: **ACI 350-06, Sect. 10.6.4.2**

$$f_{s, \max} = \frac{320}{\beta \sqrt{s^2 + 4 \left(2 + \frac{d_b}{2}\right)^2}} \geq 20 \text{ ksi.}$$

$$f_{s, \max} = \frac{260}{\beta \sqrt{s^2 + 4 \left(2 + \frac{d_b}{2}\right)^2}} \geq 17 \text{ ksi.}$$

$$f_{s, \max} = 21,871 \text{ psi}$$

$$f_{s, \max} = 17,770 \text{ psi}$$

$$f_s = 18,781 \text{ psi} \quad f_s \leq f_{s, \max} \quad \text{OK}$$

$$f_s = 18,781 \text{ psi} \quad f_s \geq f_{s, \max} \\ \text{Not OK for Severe Exposure}$$

NRCS NEM 536 and NEH-6: $f_{s, \max} \leq 20,000 \text{ psi}$

Structural Design - Wall Reinf. Shrinkage & Temperature

ACI 350-06 Section 10.5.4 stipulates that minimum primary vertical reinforcement for walls be equivalent to that required by **Sections 14.3.2 and 14.3.3**. **Section 14.3.3** noted horizontal reinforcement (Shrinkage and Temperature Reinforcement) should conform to Table 7.12.2.1.

Section 7.12.2.2 stipulates that the maximum spacing of the bars at 12" and the minimum bars size as a #4. It also states that at least 1/3 of the reinforcing must be in each face

Table 7.12.2.1 : $A_{s,min} = 0.004 A_{gross}$

Based on joints at 30 to 40 feet

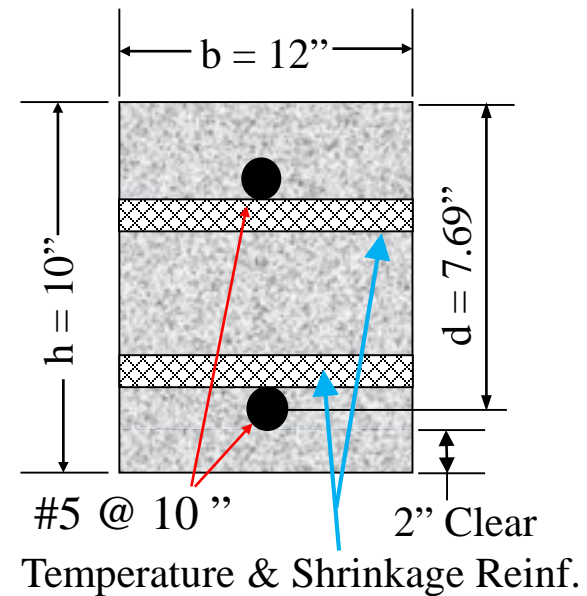
$$A_{s,min} = 0.004 (12'')(10'') = 0.48 \text{ in}^2$$

$$\#4 @ 12'' \text{ each face } A_s = 0.40 \text{ in}^2$$

$$\#4 @ 10'' \text{ each face } A_s = 0.48 \text{ in}^2$$

$$\#5 @ 12'' \text{ each face } A_s = 0.62 \text{ in}^2$$

Use #5 @ 12 in.



Structural Design - Wall Reinf. Bending Moment Capacity ϕM_n

Spacing of primary reinforcement

Section 7.6.5: Spacing shall be not exceed 2 times the slab thickness, or 12 inches.

$$10 \text{ in.} < 2 (10 \text{ in.}) = 20'' \text{ OK}$$

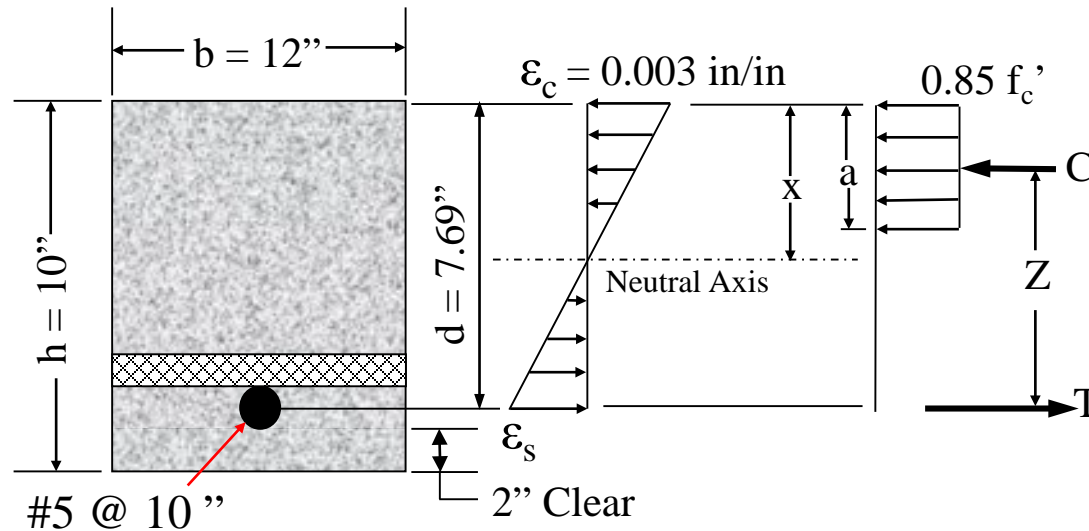
$$10 \text{ in.} < 12 \text{ in.} \text{ OK}$$

Section 7.12.2.2 stipulates that the maximum spacing of the bars at 12'' and the minimum bars size as a #4. It also states that at least 1/3 of the reinforcing must be in each face

$$10 \text{ in.} < 12 \text{ in.} \text{ OK}$$

Section 14.3.5: For walls, spacing of vertical and horizontal reinforcing shall be not exceed 12 inches.

Structural Design - Wall Reinforcement Shear Capacity ϕV_n



$$V_c = 2 \sqrt{f'_c} (b) (d)$$

$$V_c = 2 \sqrt{4000 \text{ psi}} (12 \text{ in}) (7.69 \text{ in})$$

$$= 11,673 \text{ lbs}$$

$$\phi V_n = \phi (V_c + V_s) \text{ where } V_s \text{ is shear reinf.} = 0$$

$$\phi V_c = 8,755 \text{ lbs} \quad \phi = 0.75$$

$$\phi V_n / V_{\text{Service}} \geq 1.88 \text{ from durability factor calc.}$$

$$\phi V_n / V_{\text{Service}} = 8755 / 1800 = 4.86 \geq 1.88 \text{ OK}$$

ACI 350-06, Section 9.3.2.3 :

For shear capacity, the value for the strength reduction factor, ϕ , is 0.75

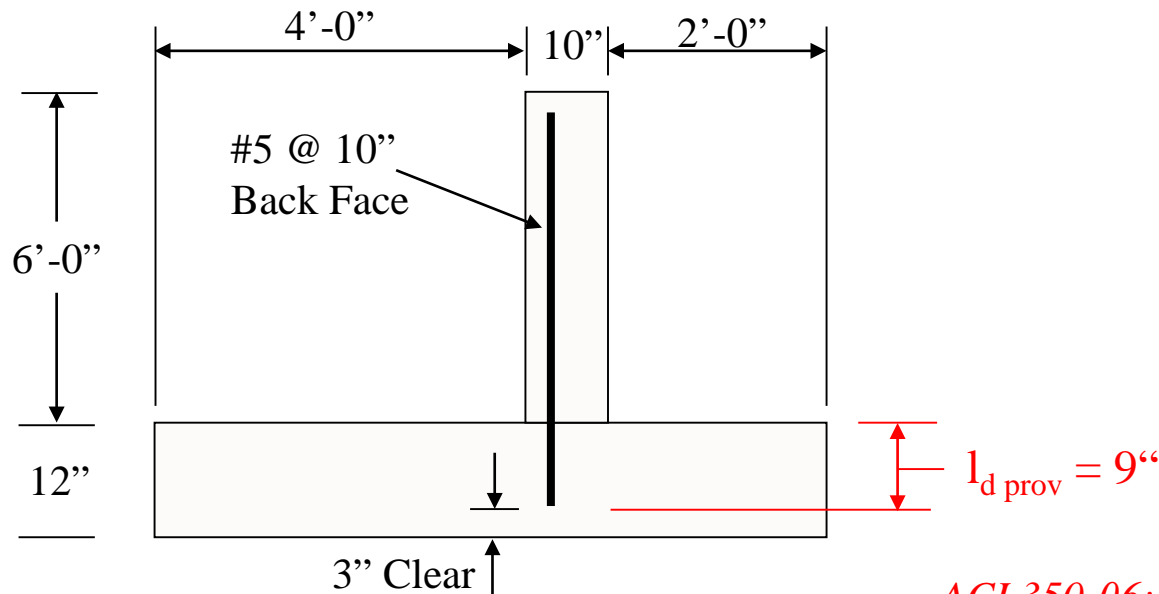
ACI 350-06, Section 11.5.5.1 :

If $V_U < \frac{1}{2} \phi V_c$ then the shear capacity of the reinforcement may be neglected

$$\phi V_n / V_{\text{Service}} \geq 2 \text{ (1.88)}$$

$$\phi V_n / V_{\text{Service}} \geq 3.76 \quad 4.86 \geq 3.76 \text{ ok}$$

Structural Design – Wall Reinf. – Development Length



ACI 350-06: Table 12.2.2 *Bar size ≤ No. 6*
Clear cover ≥ d_b
Bar spacing ≥ 2 (d_b)

$\lambda = 1.0$ for normal weight concrete

$\beta = 1.0$ bars not epoxy coated

$\alpha = 1.0$ bars have less than 12" fresh concrete below

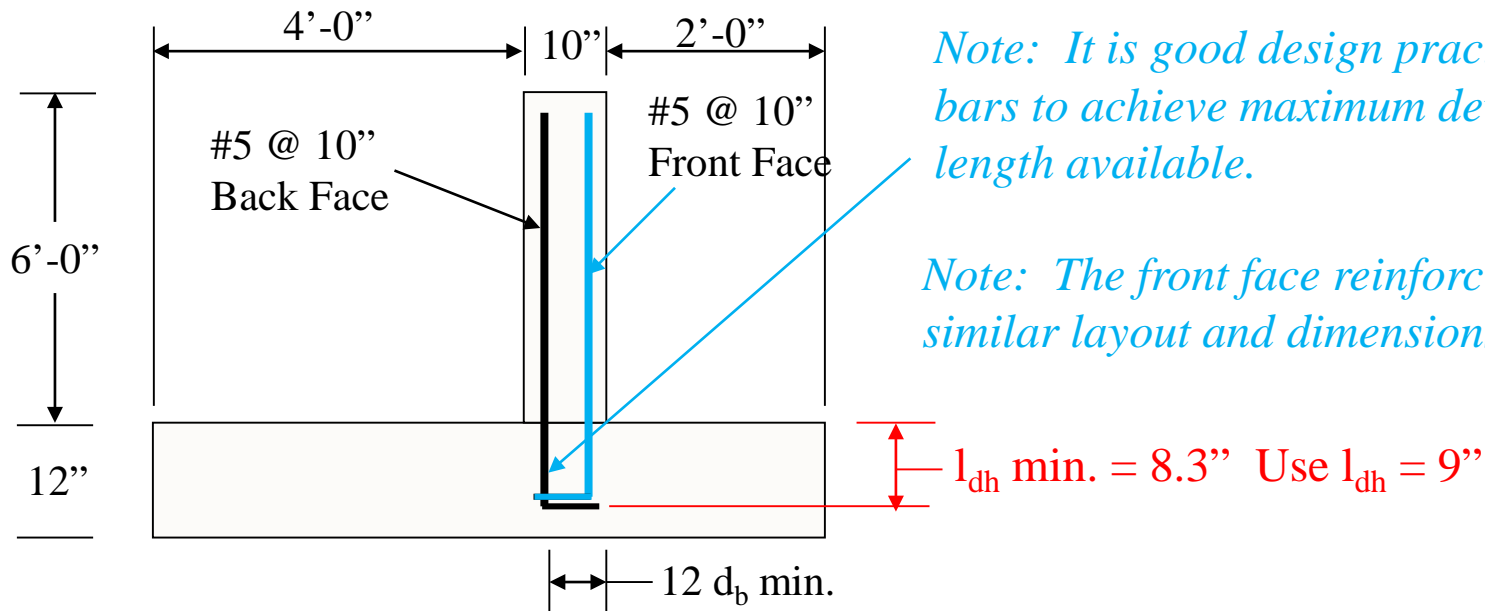
$$l_d \text{ min.} = 0.04 \left(\frac{F_y}{\sqrt{f'_c}} \right) \alpha \beta \lambda d_b$$

$$l_d \text{ min.} = 0.04 \left(\frac{60,000}{\sqrt{4000}} \right) (1.0)(1.0) (1.0)(0.625 \text{ in.})$$

$$l_d \text{ min.} = 24 \text{ in.} > l_{d \text{ prov}}$$

Because the stem reinforcing bar length available (9") is less than that required to fully develop the bar strength (24"), a hooked end should be used

Structural Design – Wall Reinf. – Development Length, Hooked Bar



Note: It is good design practice to extend bars to achieve maximum development length available.

Note: The front face reinforcing will be a similar layout and dimensions

ACI 350-06, Section 12.5.2

$$l_{dh} \text{ min.} = 0.02 \left(\frac{F_y}{\sqrt{f'_c}} \right) \beta \lambda d_b$$

$\beta = 1.0$ bars not epoxy coated

$\lambda = 1.0$ for normal weight concrete

ACI 350-06, Section 12.5.3 0.7 for side cover ≥ 2.5 in. & 90° hook cover ≥ 2 in.

Other factors:

1.0 for no confinement along l_{dh} & no confinement along hook tail length

$$l_{dh} \text{ min.} = 0.02 \left(\frac{60,000}{\sqrt{4000}} \right) (1.0) (1.0) (d_b = 0.625) (0.7) (1.0) = 8.3 \text{ in.} \quad \text{use } 9 \text{ in.}$$

ACI 350-06, Section 12.5.1

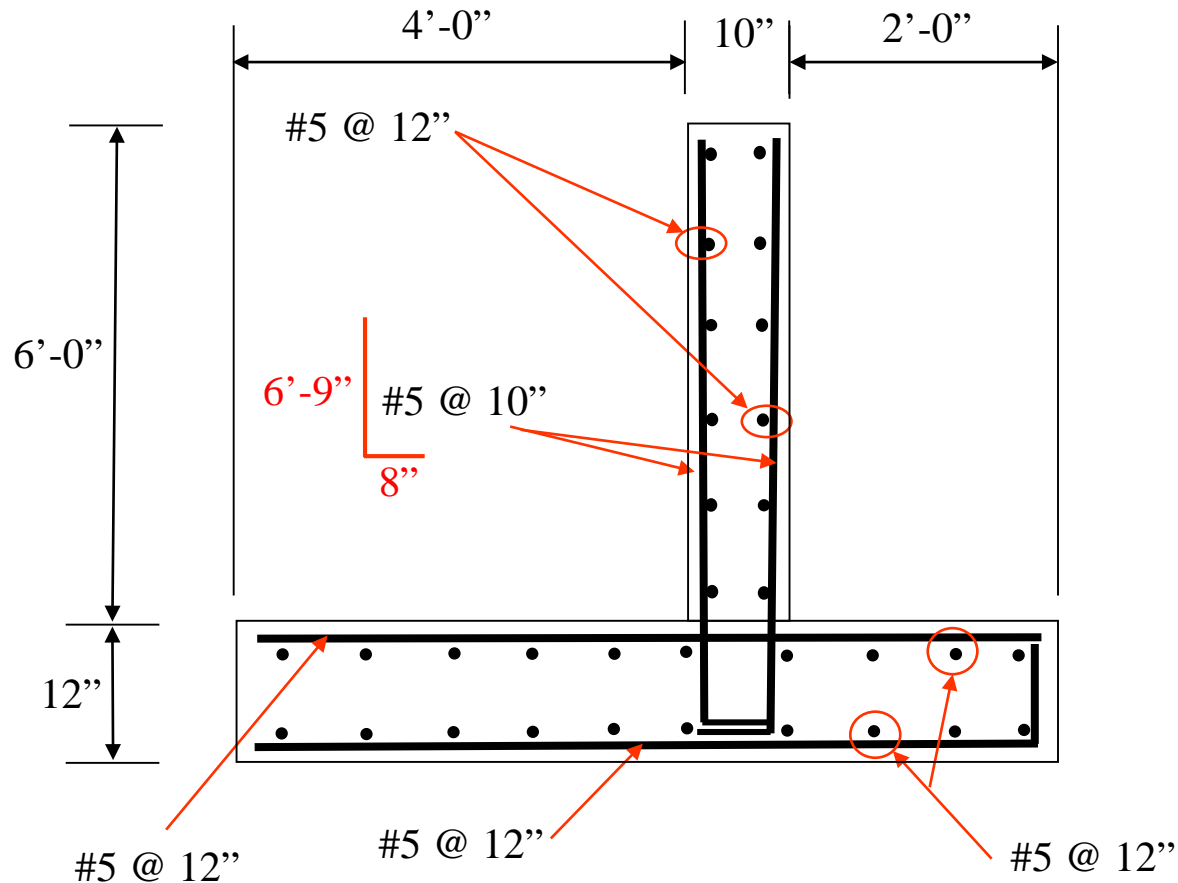
$$l_{dh} \geq l_{dh} \text{ min.} = 8.3 \text{ in.}$$

$$l_{dh} \geq 8 d_b$$

$$l_{dh} \geq 6 \text{ in.}$$

Structural Design - Cantilevered Retaining Wall ACI 350 - 06

Reinforcement Layout



Note: It is good design practice to extend bars to achieve maximum development length available. It is also good to include hooks at the ends to provide full development of the reinforcing steel for the full length per Sections 7.12.2.3 & 10.5.4.