P.E. Civil Exam Review: Foundation

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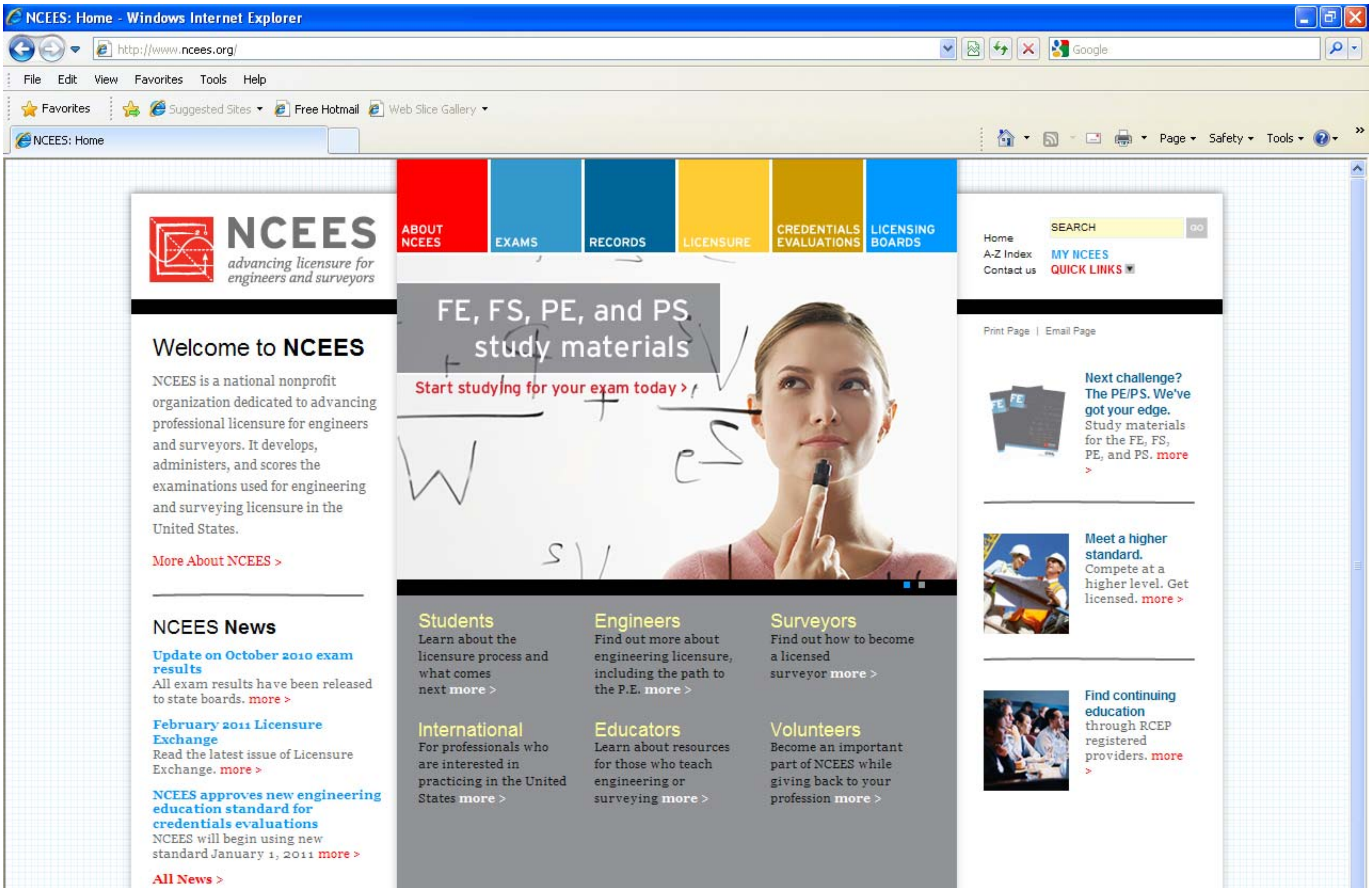
ASCE | KNOWLEDGE & LEARNING

FOUNDATION

National Council of Examiners for Engineering and Surveying
http://www.ncees.org/



STUDY REFERENCES

- Foundation Engineering; Peck Hanson & Thornburn
- Introductory Soil Mechanics and Foundations; Sowers
- NAVFAC Design Manuals DM-7.1 & 7.2
- Foundation Analysis and Design; Bowles
- Practical Foundation Engineering Handbook; Brown

Foundation Type Selection

Shallow Bearing Footings: Adequate Bearing Capacity
______________________________ Acceptable Settlement_

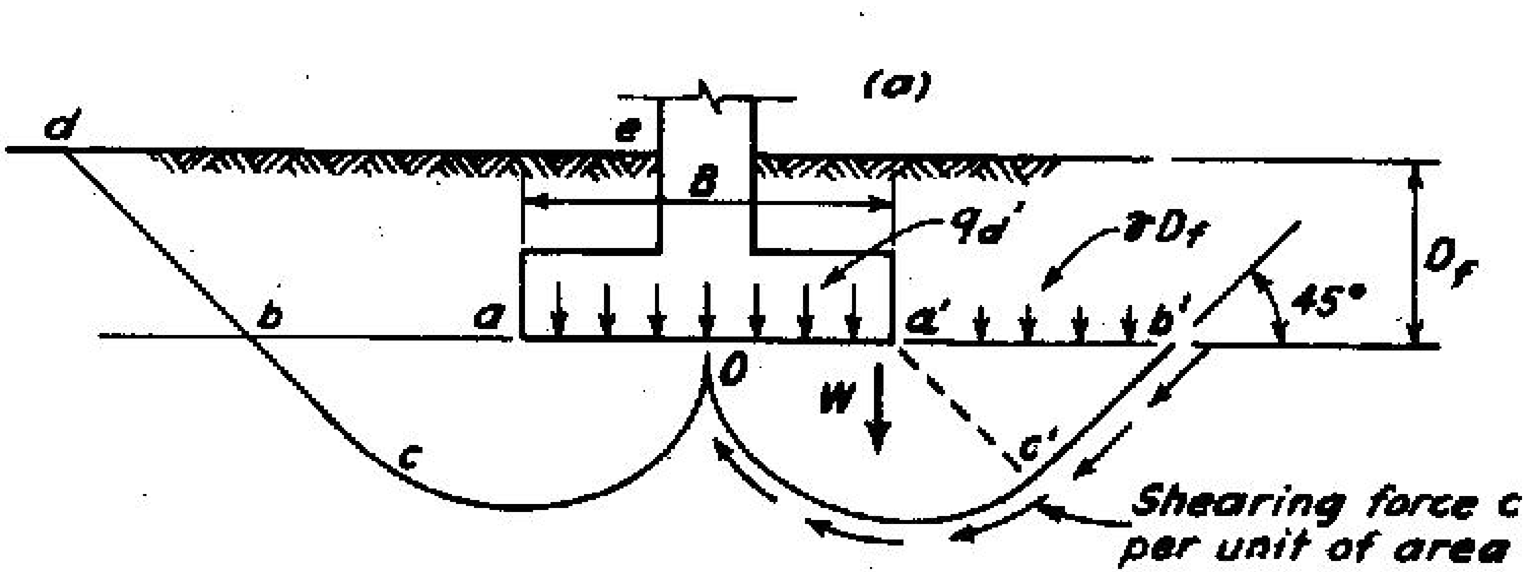
Mat or Raft: Low Bearing Pressure & Usually Minimum Settlement

Deep Foundations: Higher Load Capacity & Minimal Settlement

Drilled Piers: (Caissons) Large Load Capacity; Good Quality Control
Generally Limited to < 100’

Driven/Auger or Injected Piles: Moderate Load Capacity
Can Extend >> 100’

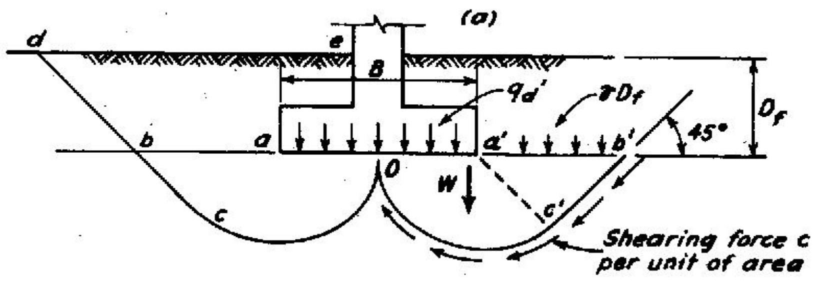
General Bearing Capacity: Terzaghi-Meyerhof



$$\frac{1}{2} \gamma \mathbf{B} N\gamma + \mathbf{C} N_c + \mathbf{q} Nq - \gamma \mathbf{Df}$$

Note: $q = \gamma Df$

General Bearing Capacity



$$Qa = \frac{Qnet}{F} = \frac{Qult - \gamma\, Df}{F} = \mathbf{(^1/_2\, \gamma\, B\, N_\gamma + C\, N_c + q\, Nq - \gamma\, Df) / F}$$

Where:

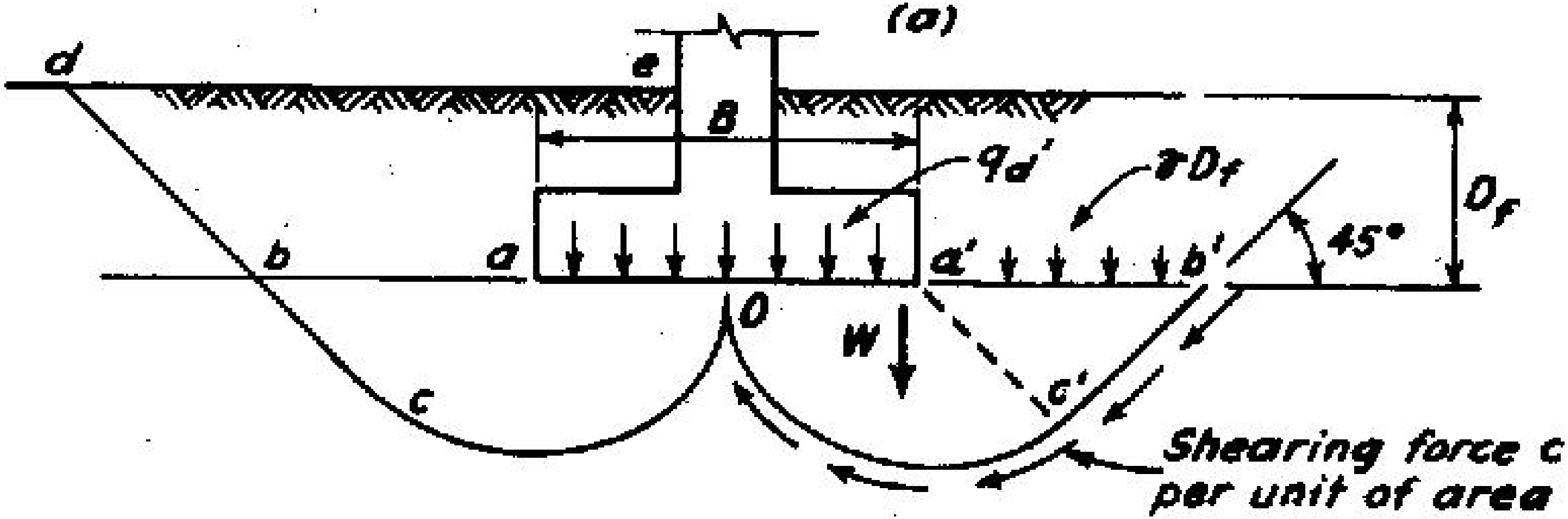
Qa = maximum net allowable bearing pressure

F = factor of safety (typ. 2-3)

Qnet = maximum net bearing pressure

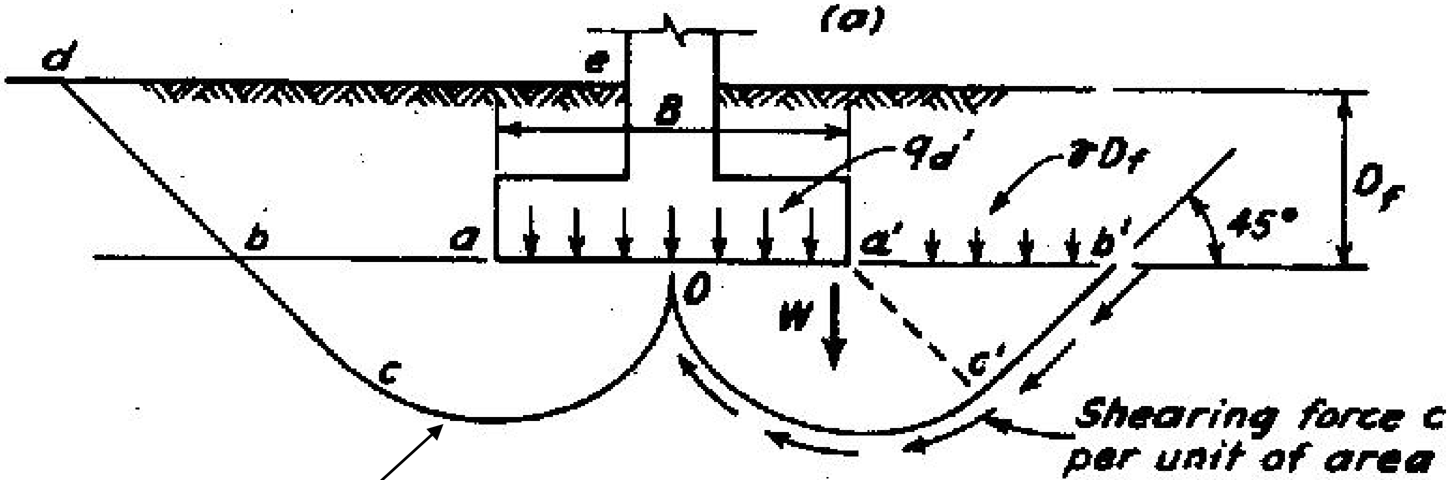
Qult = ultimate general bearing capacity

General Bearing Capacity



Qnet (at Df) = qd' = $\mathbf{(^1/_2\ \gamma\ B\ N\gamma + C\ N_c + q\ Nq - \gamma\ Df)}$

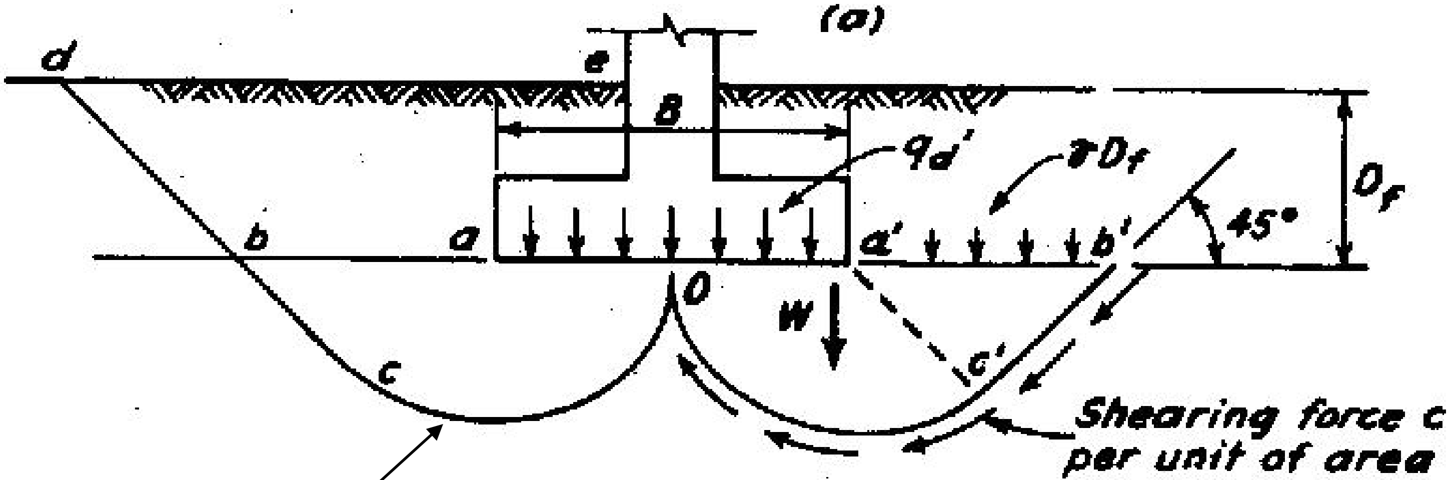
General Bearing Capacity



$$Q_{net}(\text{at } Df) = q_{d'} = \left(\tfrac{1}{2}\,\gamma\, B\, N_\gamma + C\, N_c + q\, N_q - \gamma\, D_f\right)$$

N_γ = footing width & soil weight factor, accounts for friction along bearing failure line ($\Rightarrow$1 if ϕ=0)

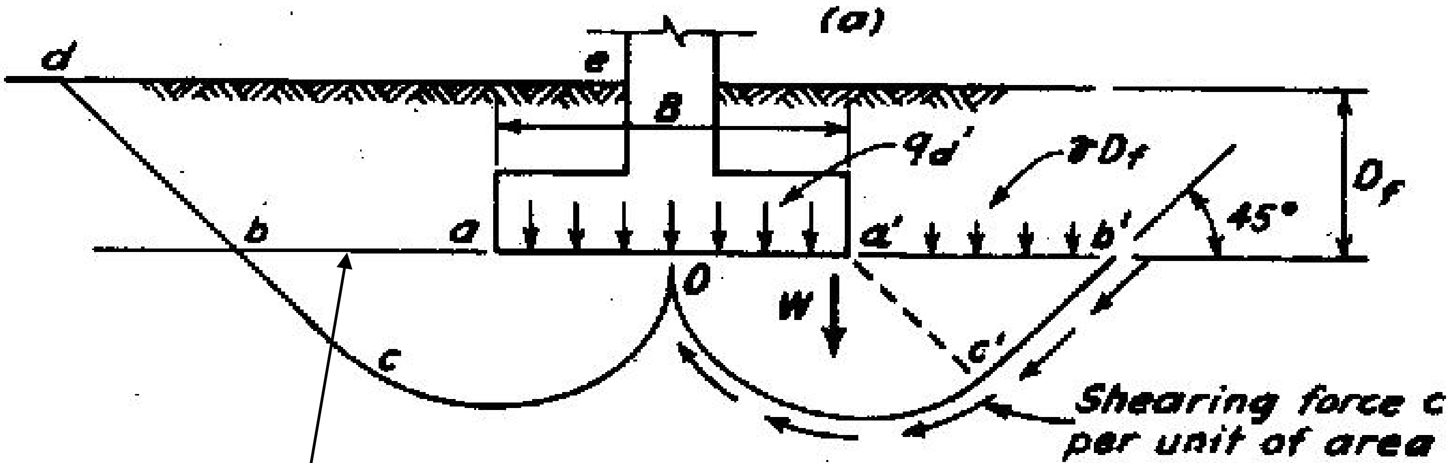
General Bearing Capacity



Q_{net}(at Df)= $q_{d'}$ = ($^1/_2\ \gamma$ B Nγ + C N_c+ q Nq- γ Df)

Nc = soil cohesion factor, accounts for cohesion along bearing failure line (⇒5.3 ± if ϕ=0)

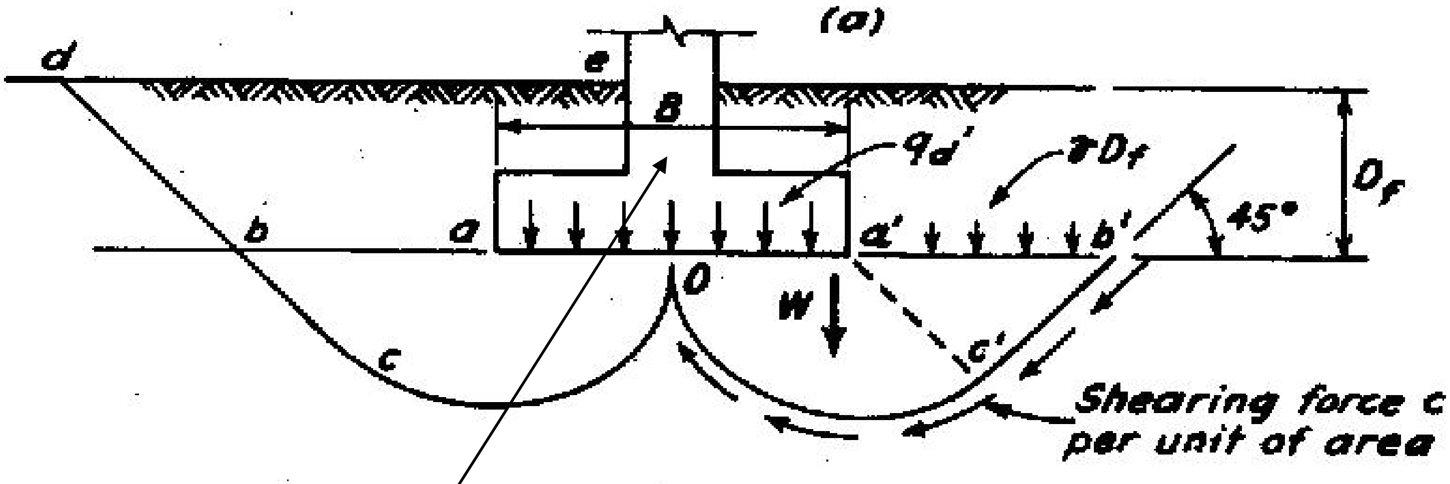
General Bearing Capacity



$$Q_{net}\ (\text{at } D_f) = q_d' = (\tfrac{1}{2}\,\gamma\, B\, N_\gamma + C\, N_c + q\, N_q - \gamma\, D_f)$$

Nq = surcharge factor, accounts for weight above the bearing failure line (⇒1 if ϕ=0)

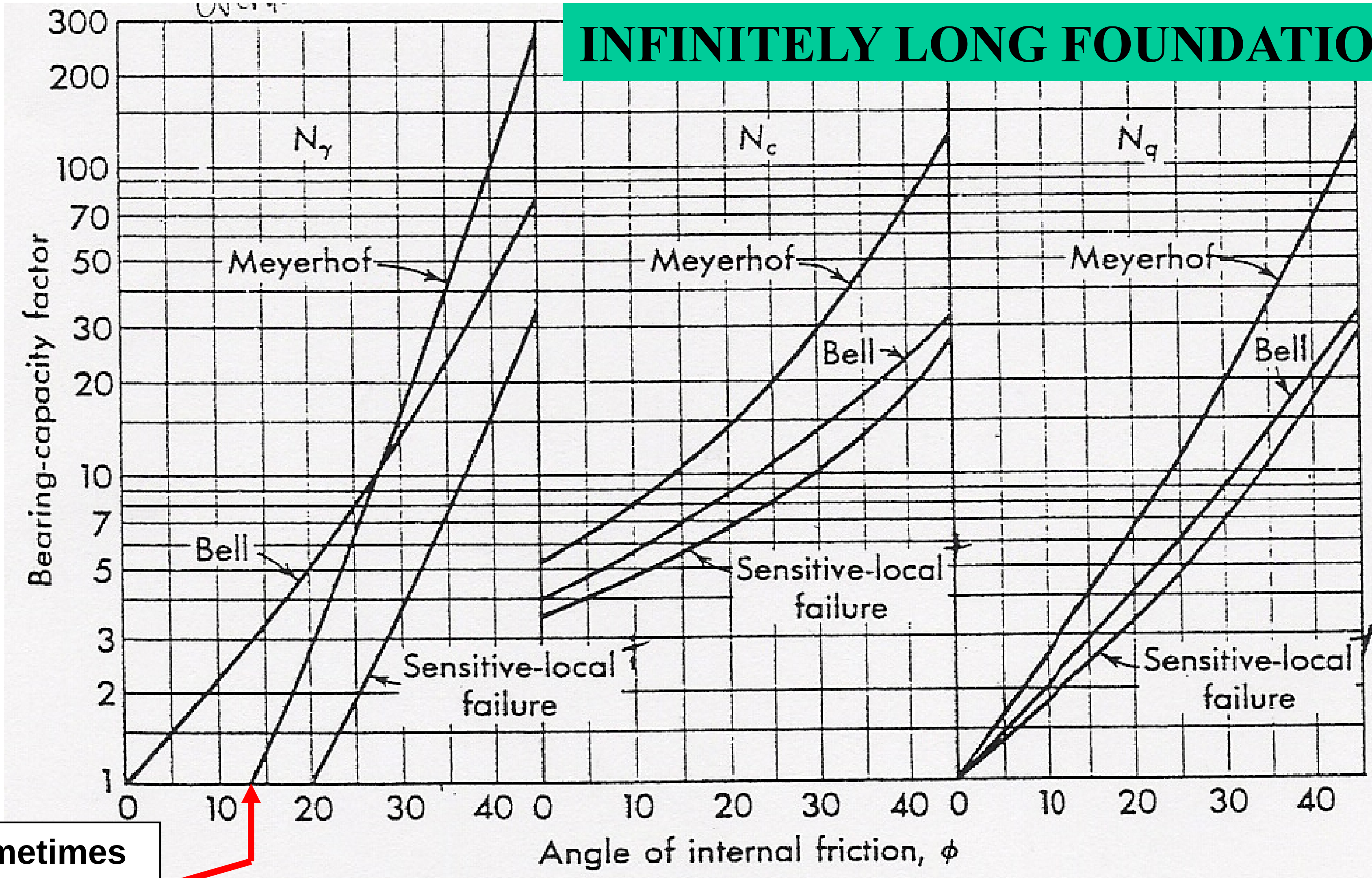
General Bearing Capacity



$$Q_{net}\ (\text{at } D_f) = q_{d'} = (\tfrac{1}{2}\gamma B N_\gamma + C N_c + q N_q - \gamma D_f)$$

Df = net bearing correction, reduces the ultimate bearing capacity by the weight of the soil and foundation above the bearing surface

INFINITELY LONG FOUNDATION



Sometimes shown as ZERO

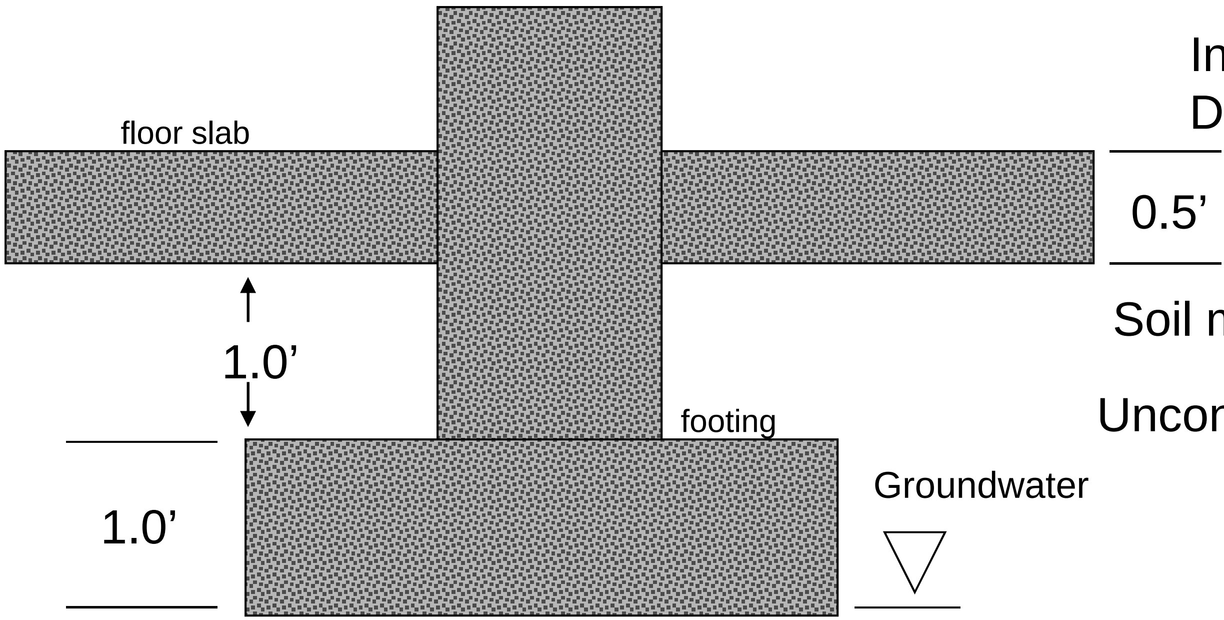
$$Q_{net} = \left(\tfrac{1}{2}\,\gamma\, B\, N\gamma + C\, N_c + q\, Nq - \gamma\, Df\right)$$

Note: multiply "N" factors by shape corrections for other shape footings

Bearing Factor Corrections for Rectangular and Circular Foundations

Shape	N_c Correction	$N\gamma$ Correction
Square	1.25	0.85
Rectangular L/B=2	1.12	0.90
L/B=5	1.05	0.95
Circular (dia. = B)	1.2	0.70

Calculate the Bearing Capacity



Use factor of safety = 3

Interior Square Footing
DL=75 kips LL=50 kips

Soil moist unit weight=120 pcf

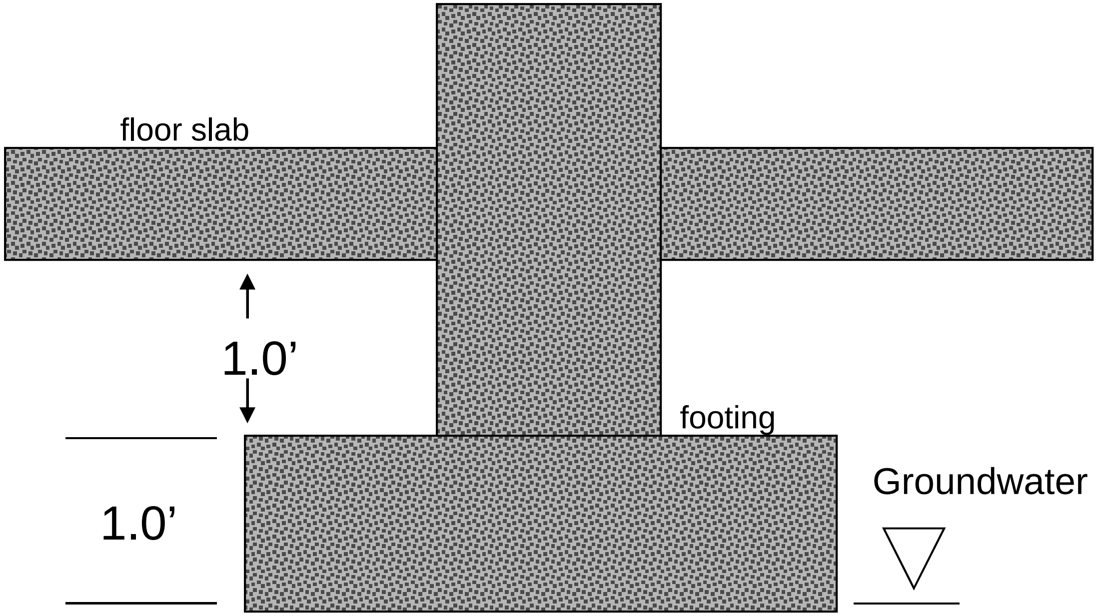
Unconfined Strength q_u=1.8 ksf

A) 2000 psf

B) 2500 psf

C) 3000 psf

Bearing Capacity – Shallow Footing



Interior Column Footing
DL=75 kips   LL=50 kips

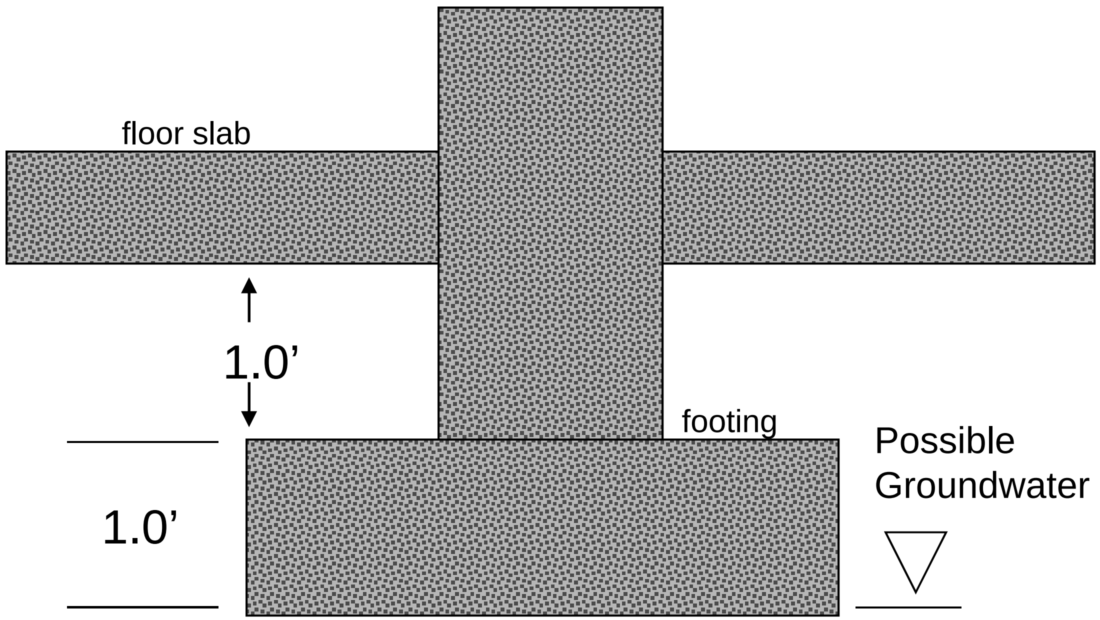
Soil moist unit weight=120 pcf

Unconfined Strength q_u=1.8 ksf

$$Q_a = \frac{Q_{ult} - \gamma D_f}{F} = \left(\tfrac{1}{2}\gamma B N_\gamma + C N_c + q N_q - \gamma D_f\right) / F$$

since $\emptyset=0$ $N_\gamma=1$ $N_q=1$

Bearing Capacity – Shallow Footing



Interior Column Footing
DL=75 kips LL=50 kips

Soil moist unit weight=120 pcf

Unconfined Strength qu=1.8 ksf

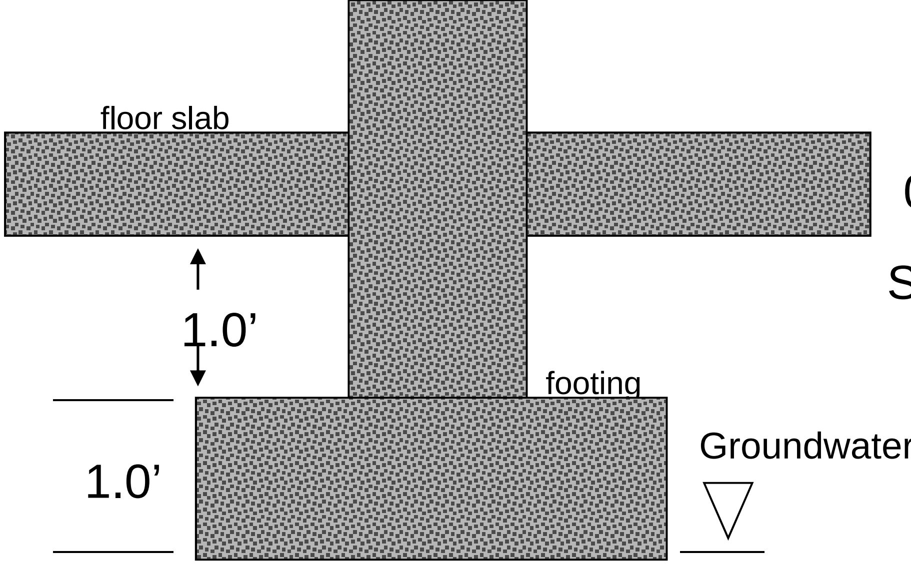
$$\mathbf{Qa} = \frac{\mathbf{Qult} - \gamma\,\mathbf{Df}}{F} = (\tfrac{1}{2}\gamma B N_\gamma + C N_c + q\,Nq - \gamma\,Df)/F$$

Assume a reasonable footing width B = 7.9 feet

From Tables N_c=5.3; for square footing shape factor=1.25

N_γ= 1; shape factor = 0.85

Nq = 1; no surcharge shape factor



Interior Column Footing

DL=75 kips   LL=50 kips

Soil moist unit weight=120 pcf

q_u=1.8 ksf

$C = {}^1/_2\, q_u = 1.8 / 2$

= 0.9 ksf or 900 psf

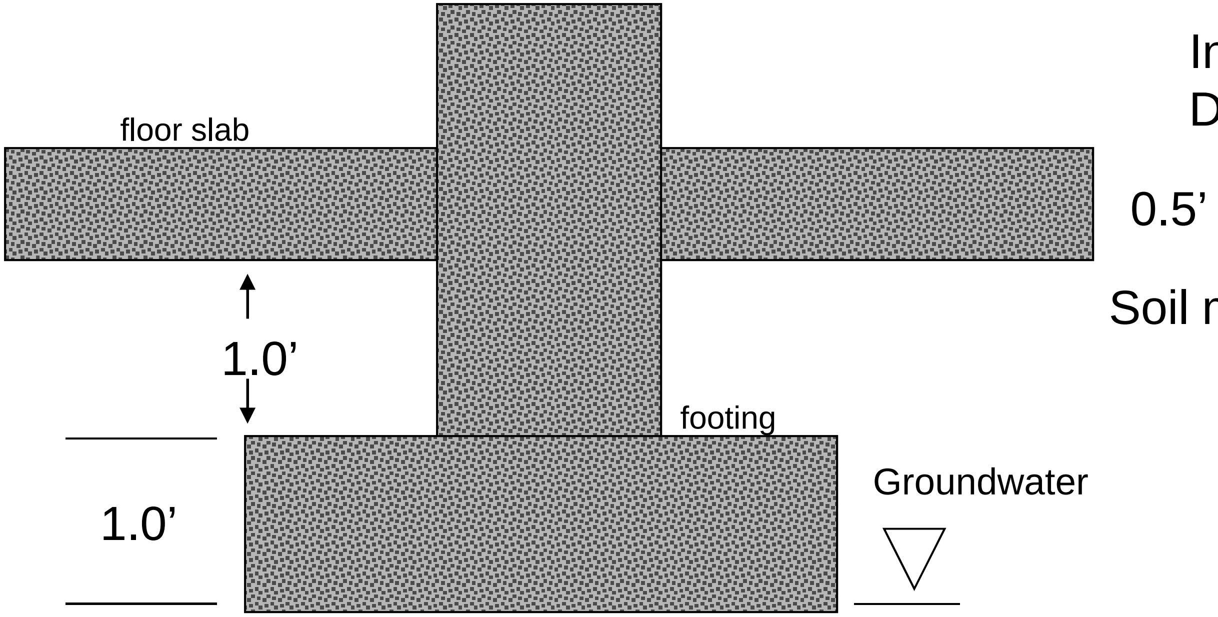
$$Qa = \frac{(0.85)1/2(120-62.4)7.9(1)+1.25(900(5.3)+[(2.5)120(1)-(2.5)120]}{3}$$

$$Qa = \frac{193+5963}{3} = 2052 \text{ psf net allowable}$$

Answer is "A"

Footing Size= (75,000+50,000)/2000 = 62.5 ft^2 $\sqrt{\ }$ = 7.9'

say 7' – 11"

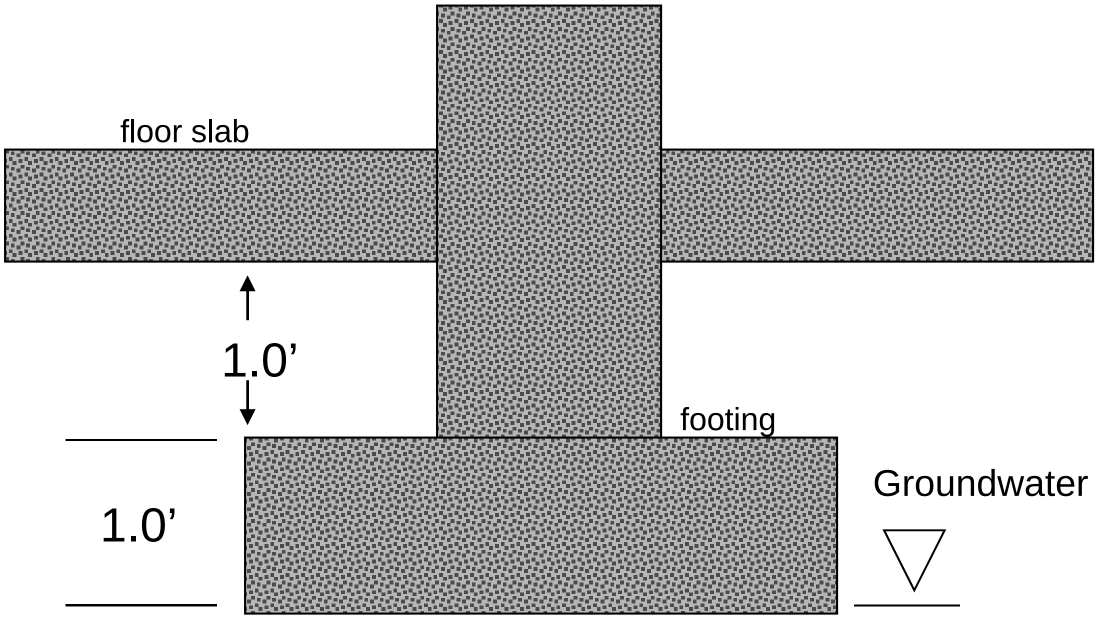


Interior Column Footing
DL=75 kips LL=50 kips

Soil moist unit weight=120 pcf

Recalculate for sand with water at bearing level:

$\phi=30^0$



Interior Column Footing
DL=75 kips LL=50 kips

Soil moist unit weight=120 pcf

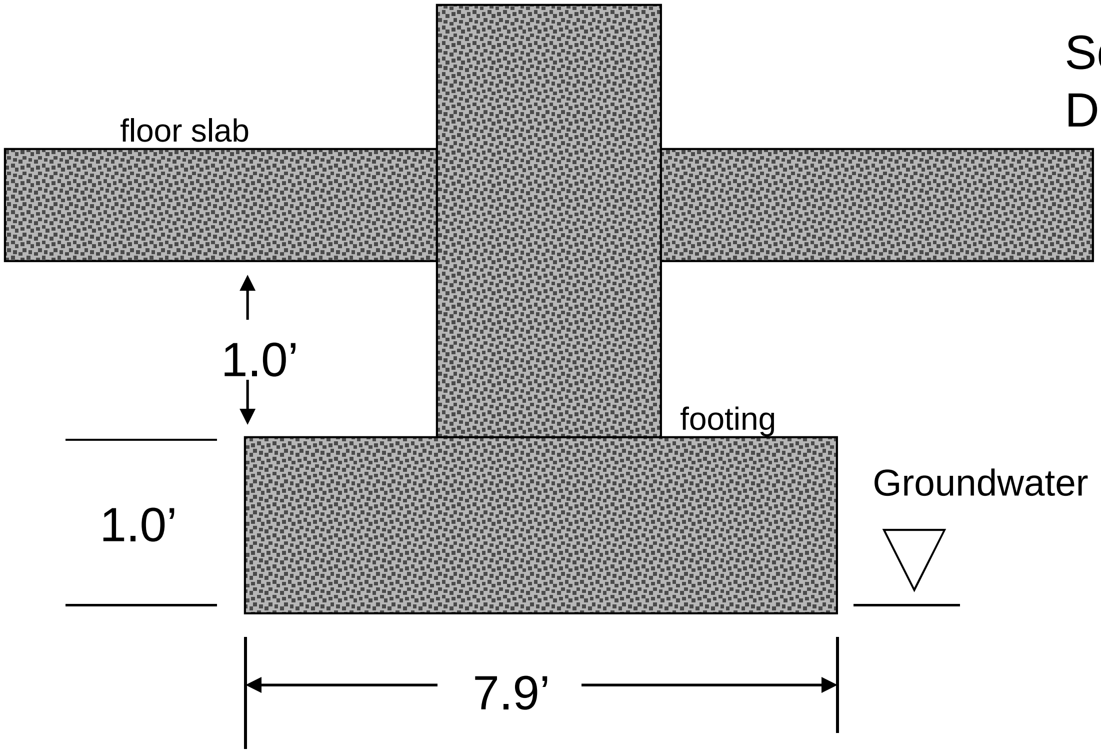
$\phi=30^0$

Overburden above water- moist unit weight

$$Q_{ult} = \frac{1}{2}\gamma_b B N_\gamma + \gamma D_f N_q$$

Base failure submerged unit weight

Square Interior Column Footing
DL=75 kips   LL=50 kips



Soil moist unit weight=120 pcf

$\phi = 30^0$

$$Q_{ult} = \tfrac{1}{2}\,\gamma_b\, B N_\gamma + \gamma\, D_f\, N_q$$

$$= \tfrac{1}{2}\,(120-62.4)7.9(17)0.85 + 120(2.5)20 = 9{,}288 \text{ psf}$$

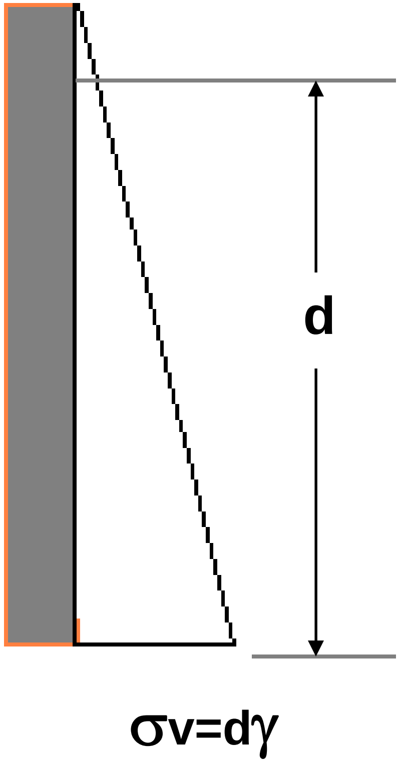
shape factor

net=9,288-2.5(120)= 8,988

for FS= 3; 8,988/3= 2,996 psf

say = 3000 psf net allowable

Lateral Earth Pressure



$\sigma_h = \sigma_v k = d\gamma k$

"k" depends on loading condition

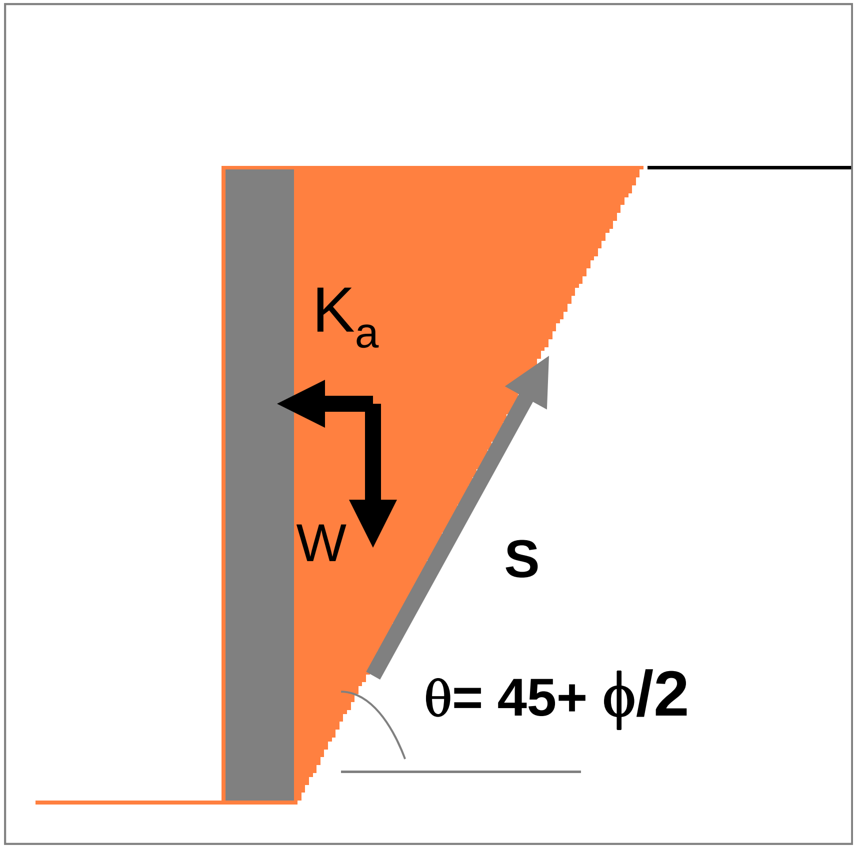
Active

At Rest

Passive

equivalent fluid pressure = γ k

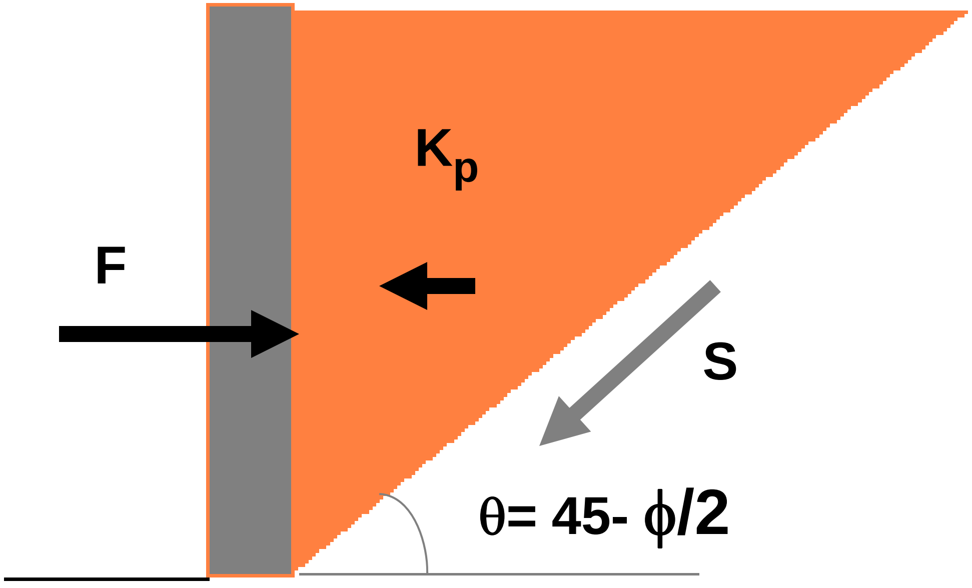
Active Lateral Earth Pressure
(load primarily gravity or surcharge - vertical)



Minimum pressure achieved when soil strains toward wall & mobilizes shear resistance

$$k_a = \tan^2(45 - \phi/2)$$

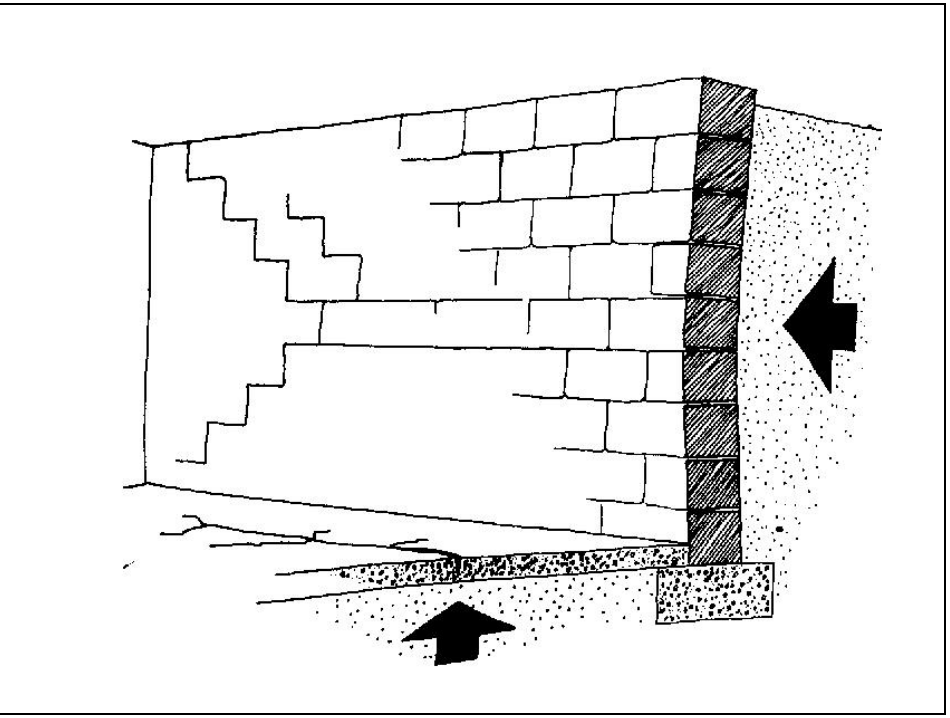
Passive Lateral Earth Pressure



$$k_p = \tan^2(45+\phi/2) = 1/k_a$$

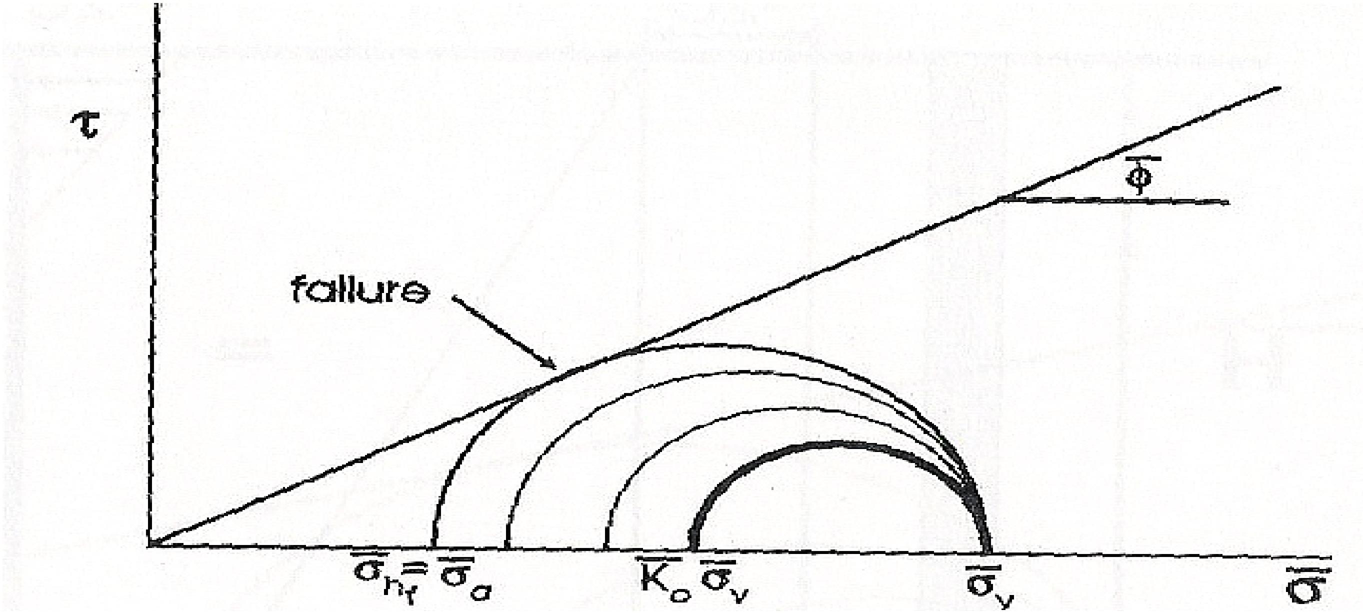
The maximum pressure achieved when structure is pushed toward soil - lateral bearing failure at limit

At Rest Lateral Earth Pressures



The pressure maintained when no movement or relief occurs

At Rest Earth Pressure



$\sigma h = \sigma v\ k_0 = d\gamma k_0$

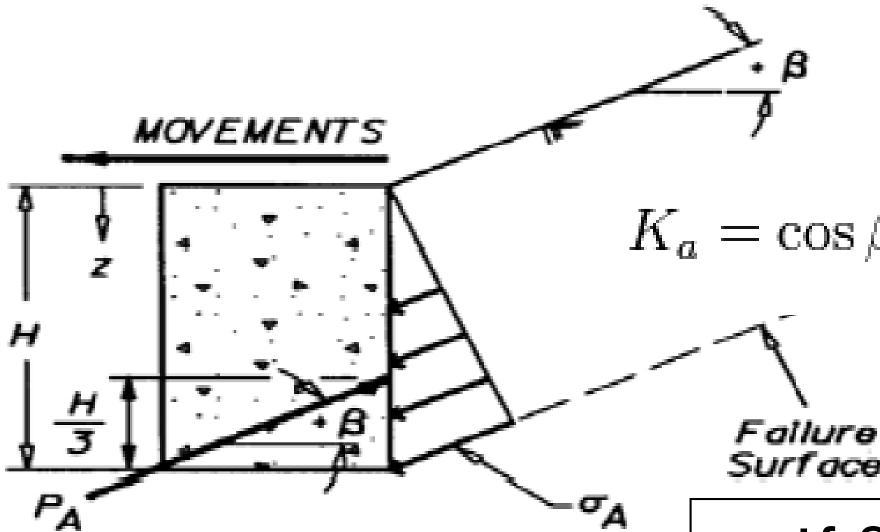
strain in elastic zone; stress at equilibrium (not in failure)

$k_0 = \mu/(1-\mu)$ where μ = Poisson's ratio

or $k_0 = 1 - \sin\phi$

(for sand and normally consolidated clay)

Rankine Analysis

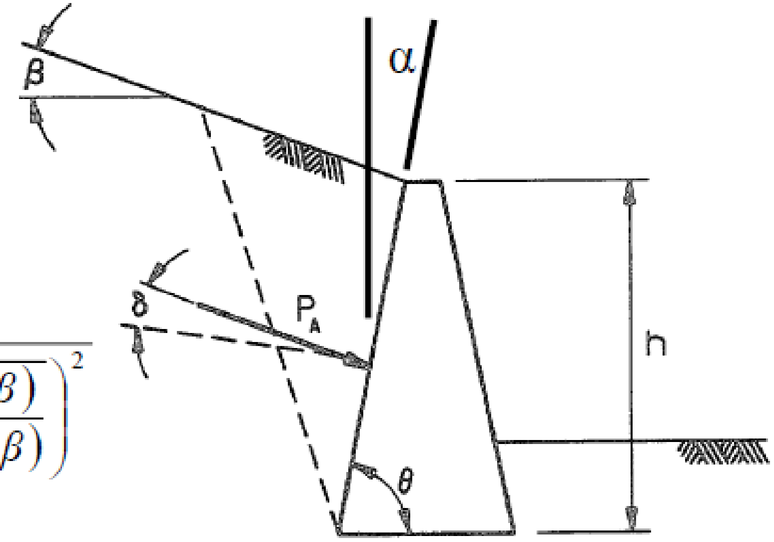


$$K_a = \cos\beta \frac{\cos\beta - \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}{\cos\beta + \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}$$

If ß= 0; $k_a = \tan^2(45 - \phi/2)$

$$K_p = \cos\beta \frac{\cos\beta + \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}{\cos\beta - \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}$$

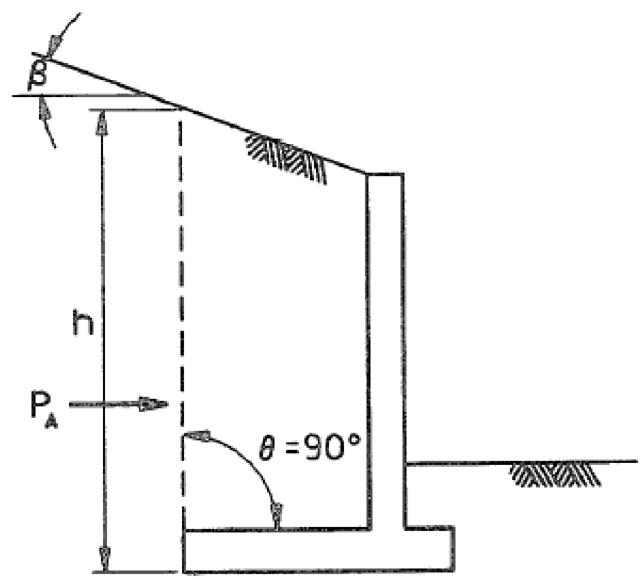
Coulomb Theory



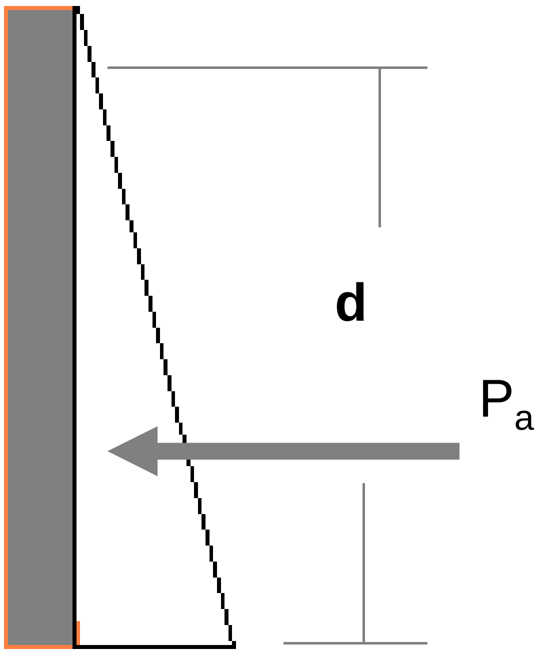
$$K_a = \frac{\cos^2(\varphi-\alpha)}{\cos^2\alpha\cos(\delta+\alpha)\left(1+\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi-\beta)}{\cos(\delta+\alpha)\cos(\alpha-\beta)}}\right)^2}$$

$$K_p = \frac{\cos^2(\varphi+\alpha)}{\cos^2\alpha\cos(\delta-\alpha)\left(1-\sqrt{\frac{\sin(\varphi+\alpha)\sin(\varphi+\beta)}{\cos(\delta-\alpha)\cos(\beta-\alpha)}}\right)^2}$$

- Includes wall friction
- Passive earth coefficients can be UNCONSERVATIVE (too high)



Active Lateral Earth Pressure



$$p_a = d\,\gamma\,k_a$$

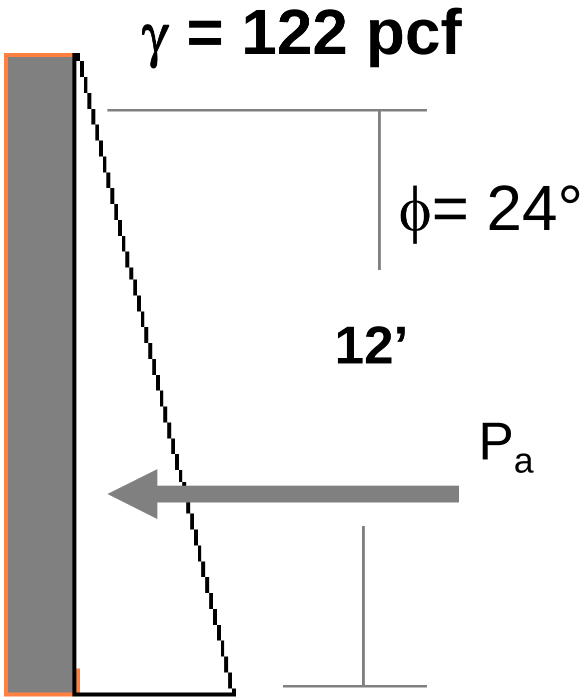
$$p_a = d\gamma k_a$$

$$k_a = \tan^2(45 - \phi/2)$$

$$p_a = d\,\gamma\,\tan^2(45 - \phi/2)$$

$$P_a = 1/2d\ p_a = \gamma\,k_a\,d^2 / 2$$

Calculate Active Pressure
Total Force per Foot of Wall

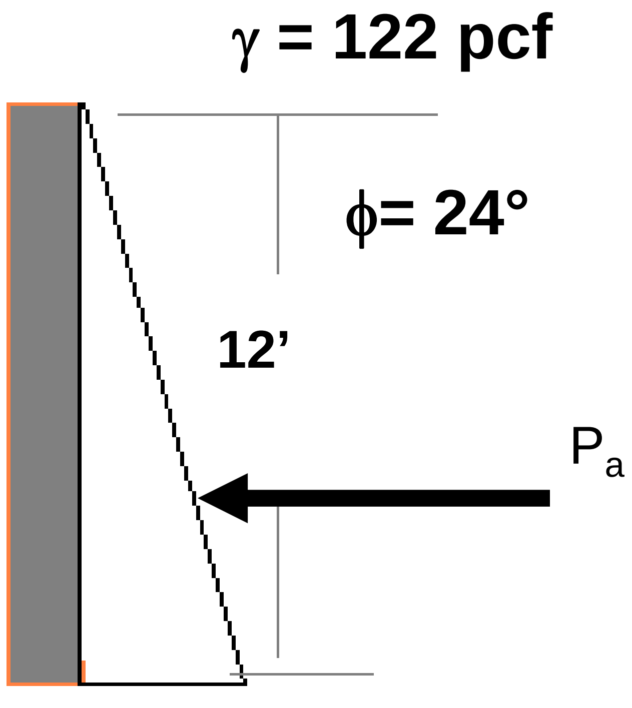


A) 3690 pounds

B) 615 pounds/ft^2

C) 307 pounds

Calculate Active Pressure Total Force per Foot of Wall



$p_a = d\gamma k_a$

$k_a = \tan^2(45-24/2) = 0.42$

$p_a = 12 \times 122 \times 0.42$

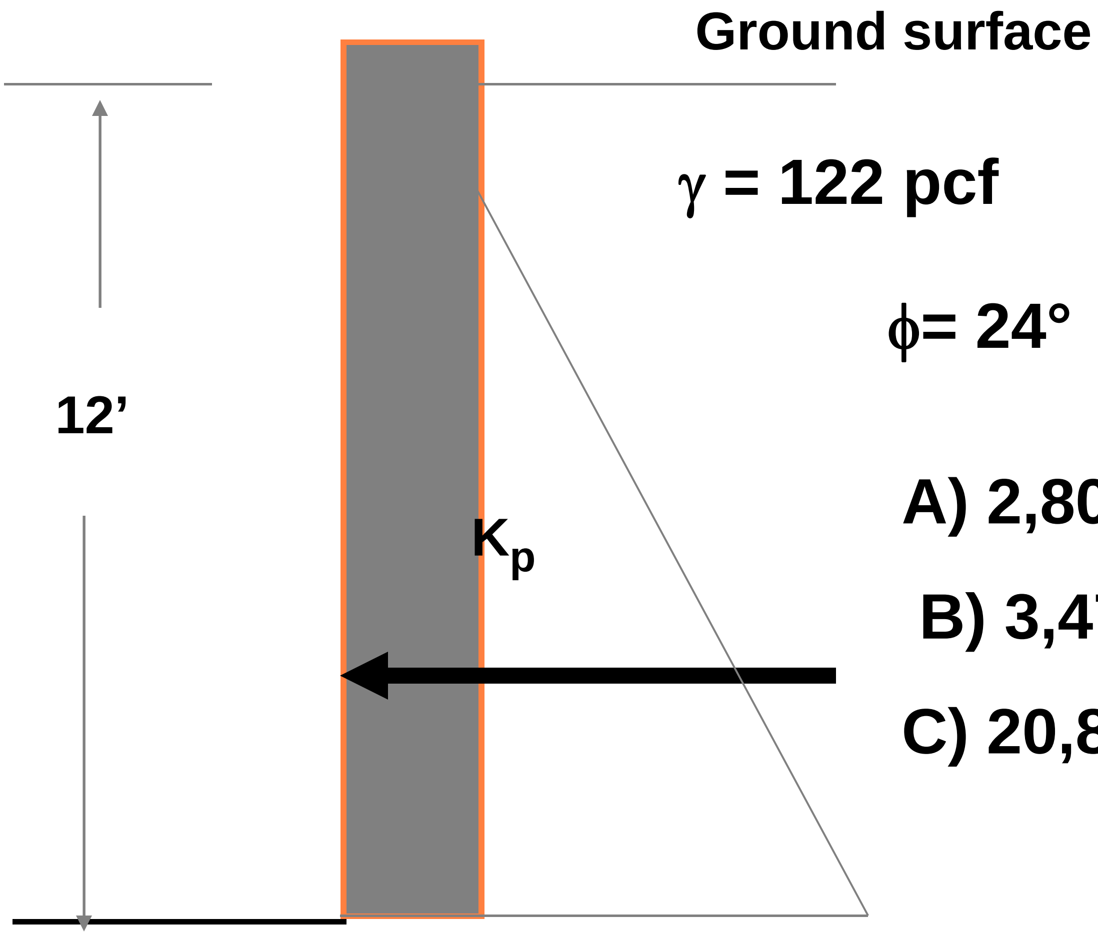
$= 615$ psf

$P_a = 122(0.42)12^2/2$

$= 3689$ pounds force

Answer is "A"

Calculate Total Force Due to Passive Earth Pressure



γ = 122 pcf

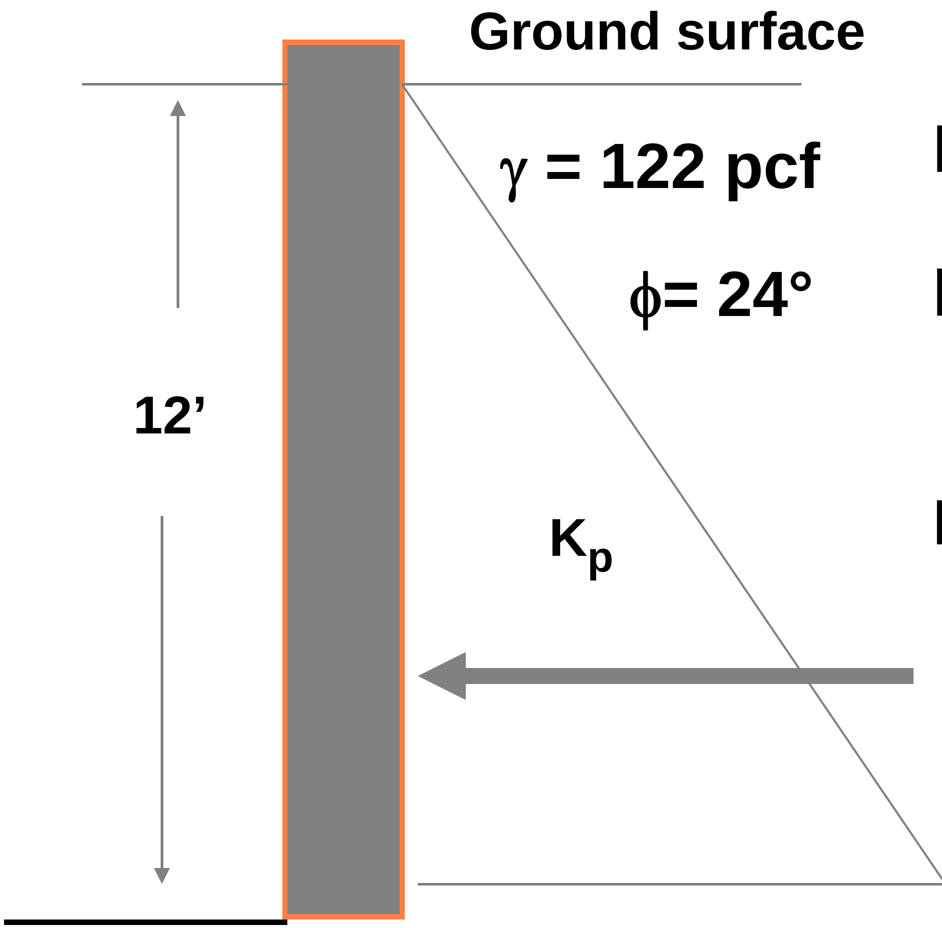
ϕ= 24°

A) 2,800 pounds force

B) 3,470 pounds force

C) 20,818 pounds force

Calculate Total Force Due to Passive Earth Pressure



$k_p = \tan^2(45+24/2) = 2.37$

$p_p = 12 \times 122 \times 2.37$

$= 3470$ psf

$P_a = 122(2.37)12^2/2$

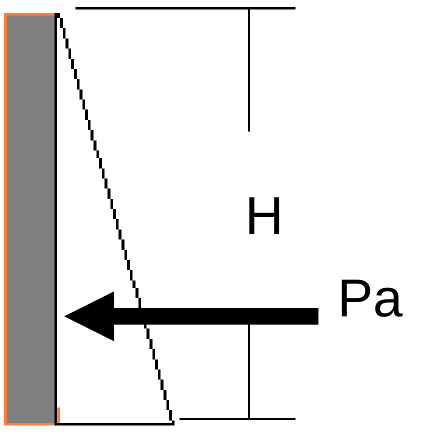
$= 20{,}818$ pounds force

Answer is “C”

C-ϕ Soils Lateral Earth Pressure

Active Case

Cohesionless



$p_a = H\gamma k_a$

$p_a = H\gamma \tan^2(45-\phi/2)$

For C & ϕ Soils

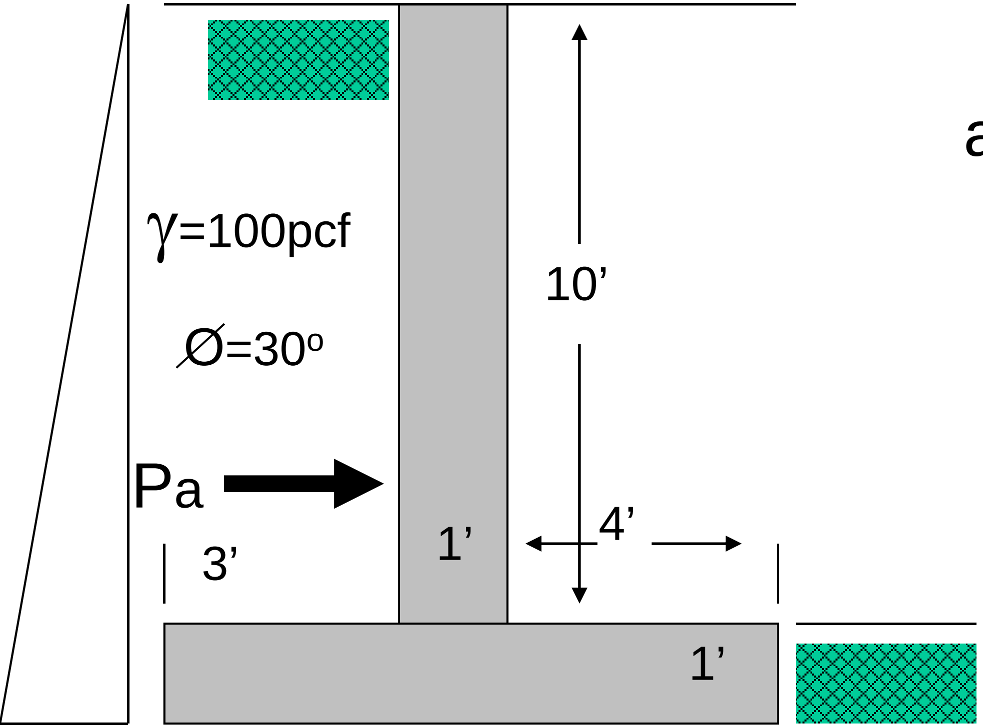
$p_a = H\gamma \tan^2(45-\phi/2) - 2C \tan(45-\phi/2)$

$P_a = 1/2 H^2 \gamma \tan^2(45 - \phi/2) - 2C H \tan(45 - \phi/2)$

$P_a = \sigma_h = K_a \sigma_v - 2c\sqrt{K_a}$

For Passive case change negative signs to positive

Lateral Earth Pressures



active coefficient $= k_a = \tan^2(45 - {}^{Ø}/_2)$

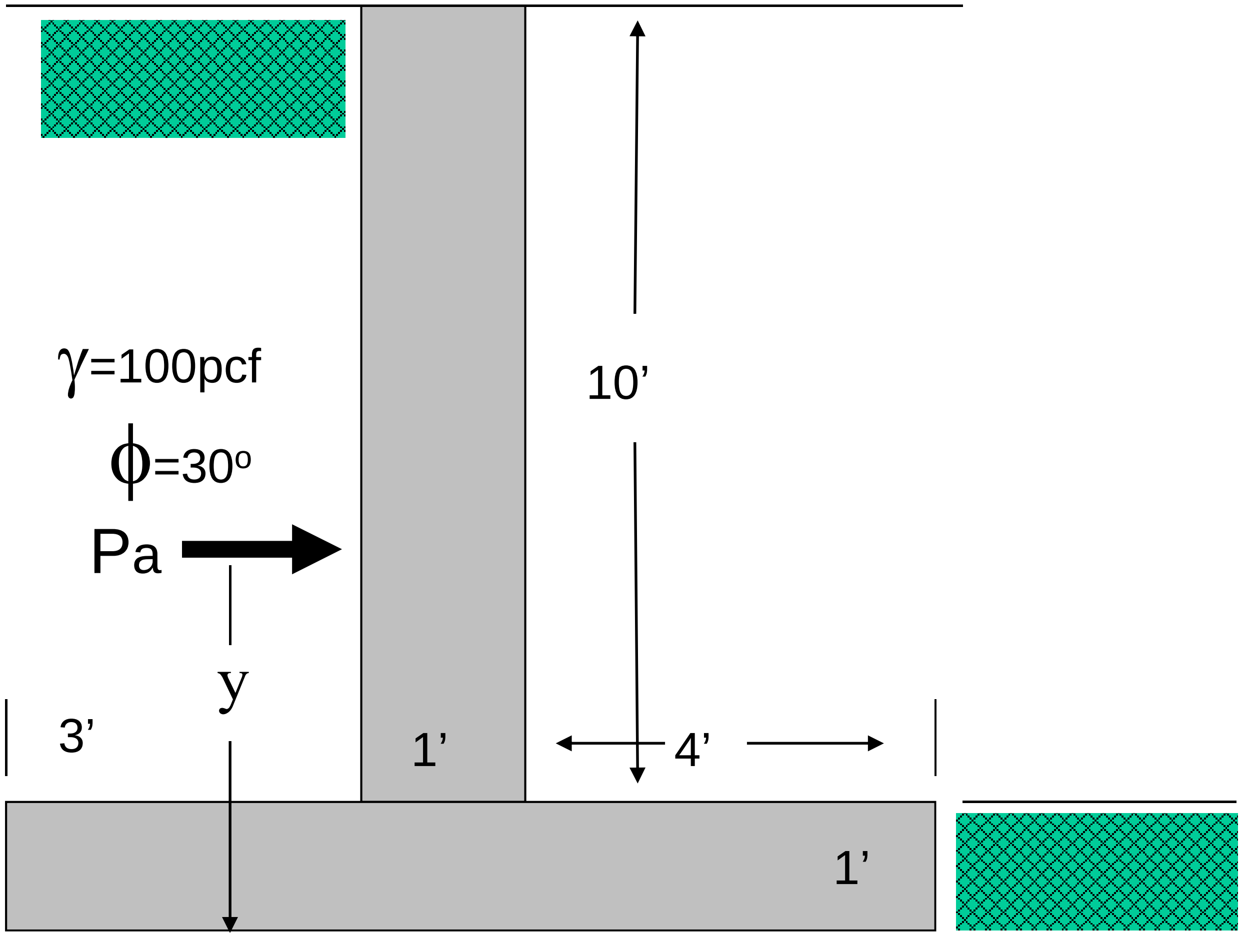
or $\frac{1 - \sin Ø}{1 + \sin Ø} = 0.33$

max pressure at base of wall

$p = \gamma K_a H = 100(0.33)11 = 363$ psf

P_a = Resultant Force $= {}^1/_2 \, K_a \, \gamma H^2 = {}^1/_2 \, \gamma \, K_a H(H)$

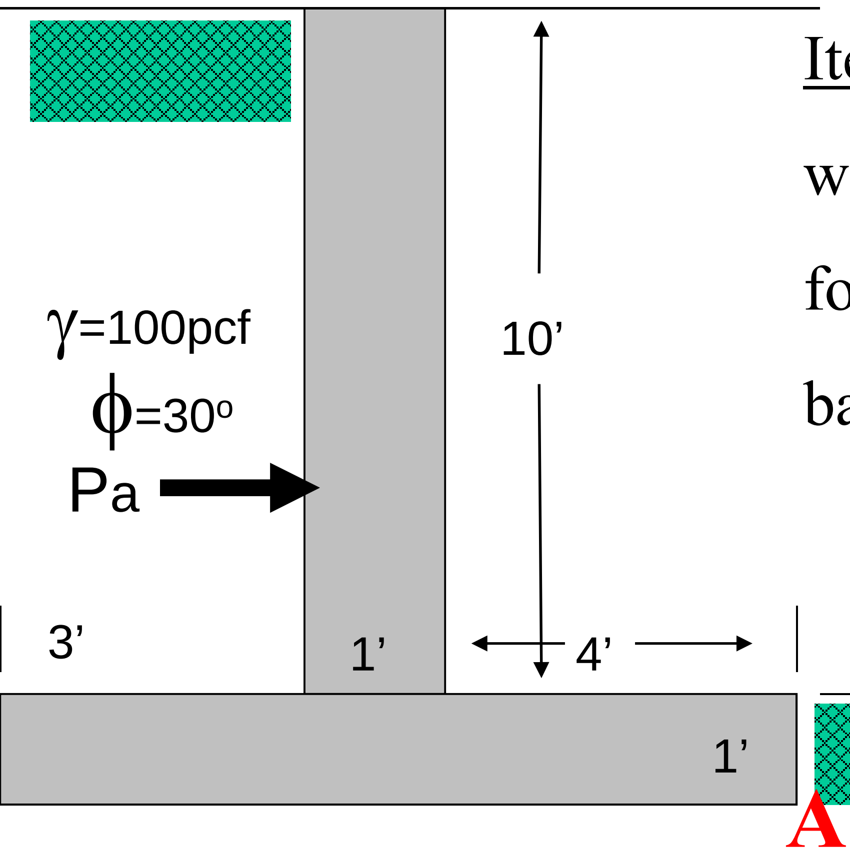
$= {}^1/_2 \; 363(11) = 1997$ pounds/ft



Moment due to force acts at $^1/_3$ wall height

y= 11/3 = 3.67’ P_ay= 3.67(1997)=7329 ft–lbs/ft

Overturning- Take Moments About "A"



Item	Force	Arm	Moment
wall	10x150	-4.5	-6750
footing	8x150	-4	-4800
backfill	30x100	-6.5	-19500
	Σ 5700		Σ -31050
	Pa 1997	3.67	7329

Overturning FS = Resisting/Driving = 31050/7329 = 4.2

Sliding Resisting Forces:

Base Friction + Passive Pressure at toe

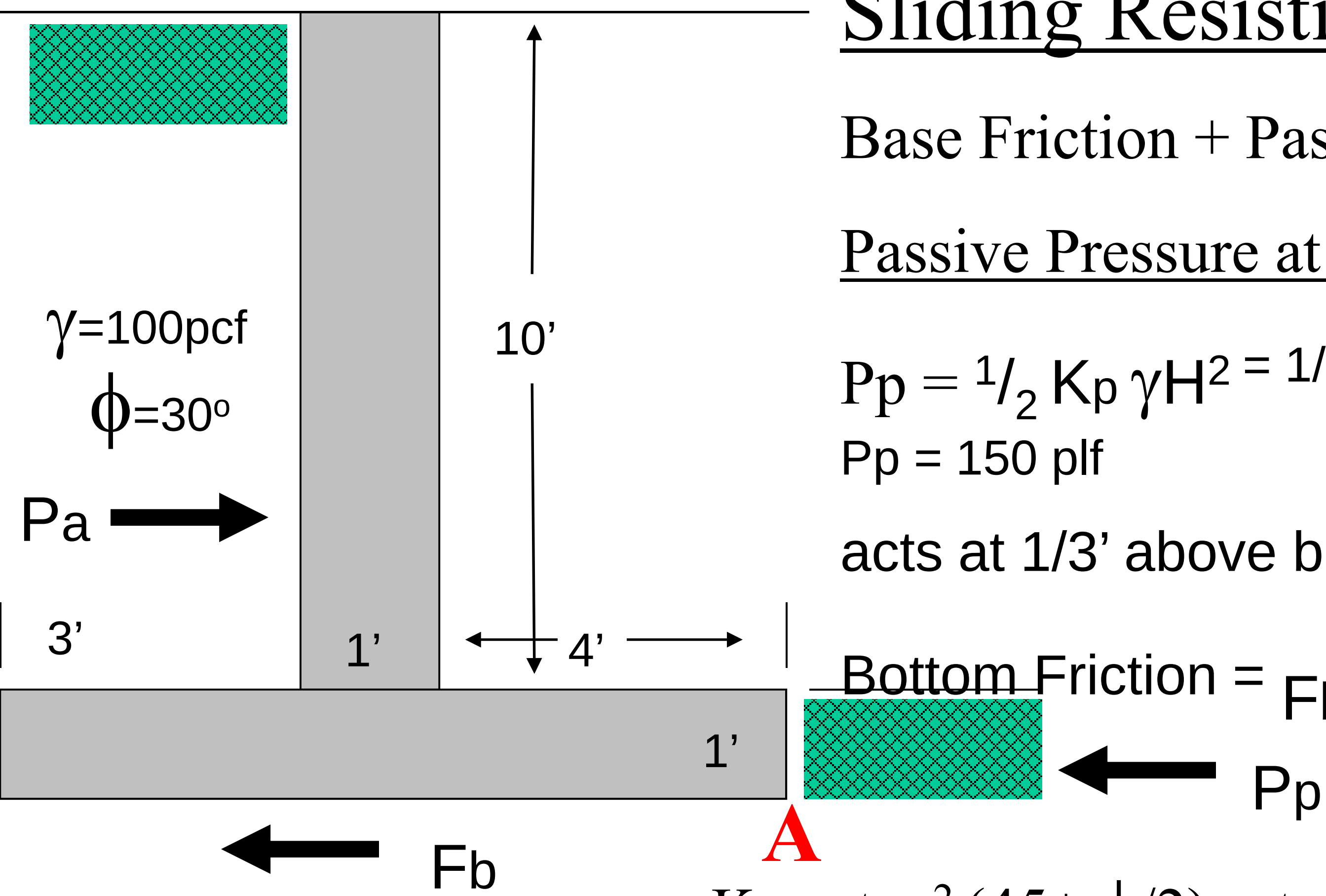
<u>Passive Pressure at toe</u>

$Pp = {}^1/_2\, K_p\, \gamma H^2$ = 1/2x 3x100x1

Pp = 150 plf

acts at 1/3' above base

Bottom Friction = F_b = weight x tan 2/3ϕ

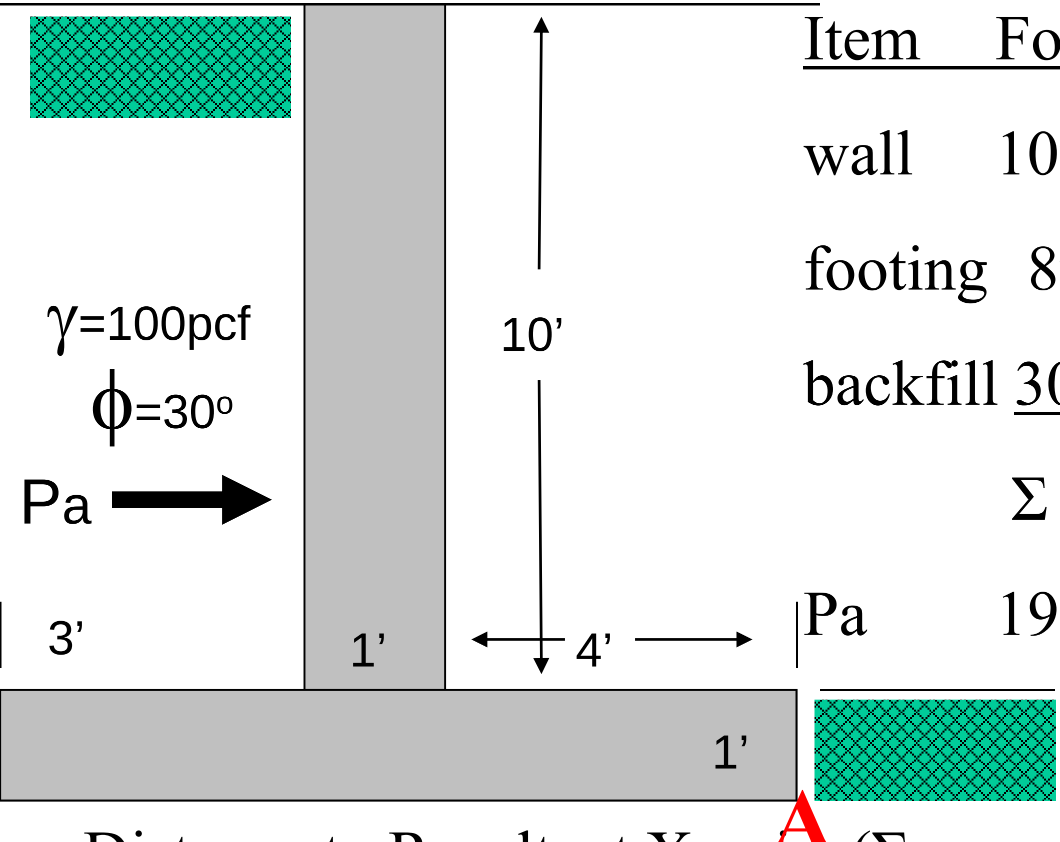


$Kp = \tan^2(45+\phi/2) = \tan^2(45+30/2) = 3$

Fb = 5700 tan (2/3)30 = 2075

Sliding FS = Resisting/Driving = (2075+150)/1997 = 1.1

Calculate Eccentricity



Item	Force	Arm	Moment
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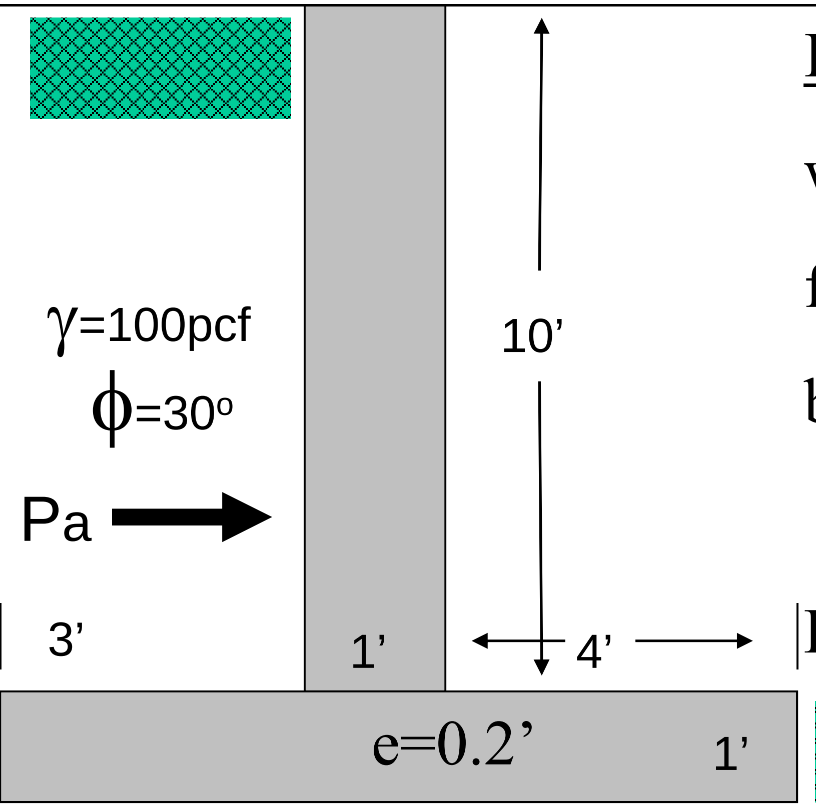
footing	8x150	-4	-4800
backfill	30x100	-6.5	-19500
	Σ 5700		Σ -31050
Pa	1997	3.67	7329

Σ Moments = -23721

Distance to Resultant X-axis: (Σ moments/resisting force)

Σ M/ Σ V = 23721/5700 = 4.2’ from point A

Middle 1/3= 2.7 to 5.3, eccentricity= e = 4-4.2 = 0.2’ beyond middle



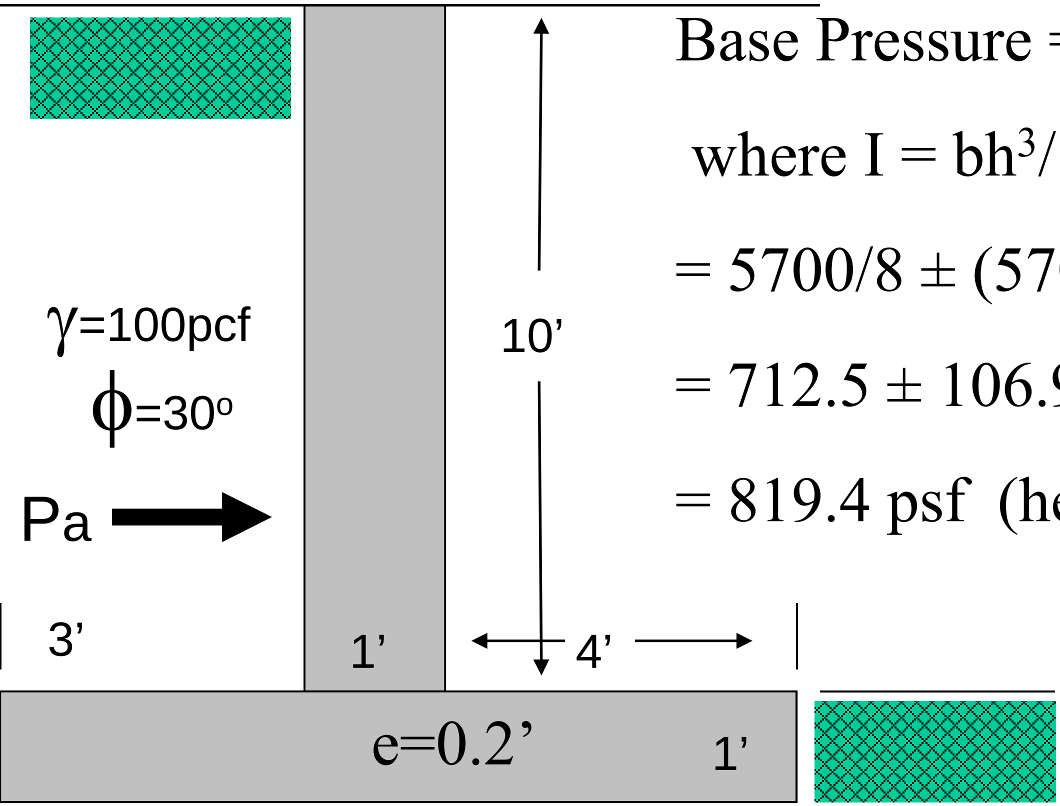
Item	Force	Arm	Moment
wall	10x150	-4.5	-6750↺
footing	8x150	-4	-4800↺
backfill	30x100	-6.5	-19500 ↺
	Σ 5700		Σ -31050↺
Pa	1997	3.67	7329↻

Σ Moments = -23721 ↺

Base Pressure = P/A ± Mc/I; or P/A ± Pec/I; where $I = bh^3/12$

$= 5700/8 \pm (5700 \times 0.2 \times 4)/(1/12 \times 1 \times 8^3) = 712.5 \pm 106.9$

= 819.4 (heel) & 605.6 (toe) heel increases due to ↺ moment sum



Base Pressure = P/A ± Mc/I

where $I = bh^3/12$

$= 5700/8 \pm (5700 \times 0.2 \times 4)/(1/12 \times 1 \times 8^3)$

$= 712.5 \pm 106.9$

= 819.4 psf (heel) & 605.6 psf (toe)

or

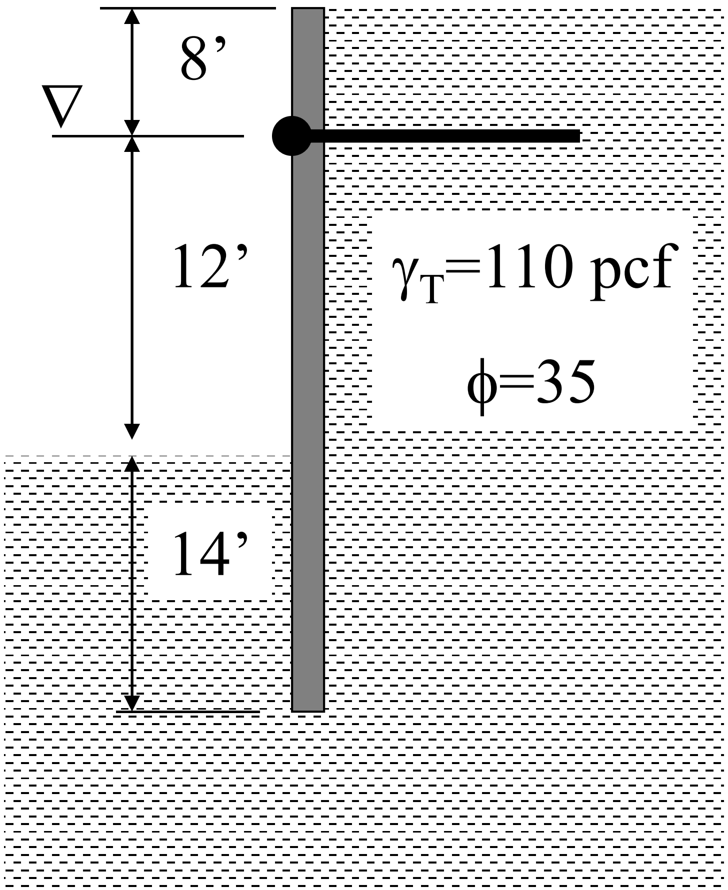
X-axis reaction = P/B(1 ±6e/B)

$= 5700 / 8 (1 + 6 \times 0.2 / 8)$

= 819.4 psf (heel)

Anchored Bulkhead

free earth method



$Ka = \tan^2 (45 - \phi /2) = 0.27$

$Kp = \tan^2 (45 + \phi /2) = 3.7$

Active Pressure = γ Ka H

at anchor

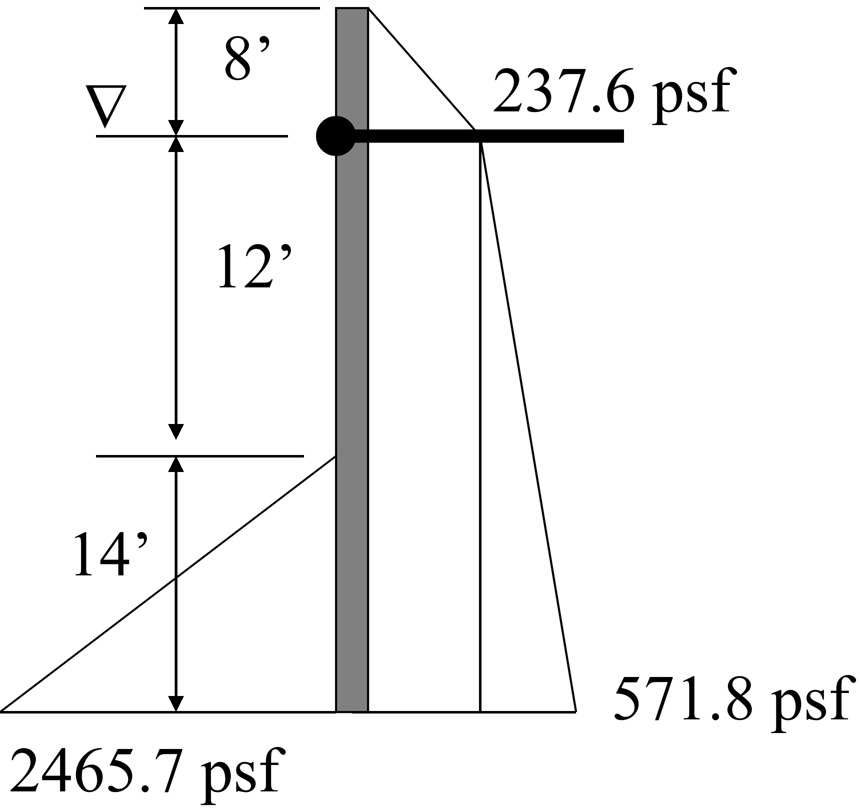
= 110 (0.27) 8= 237.6 psf per linear foot

at base of bulkhead

= 237.6 + (110-62.4) 0.27 (26)= 571.8 psf

per linear foot of wall

Anchored Bulkhead



$Kp = \tan^2 (45 + 35/2) = 3.7$

Passive Pressure = γ Kp H

below dredge line

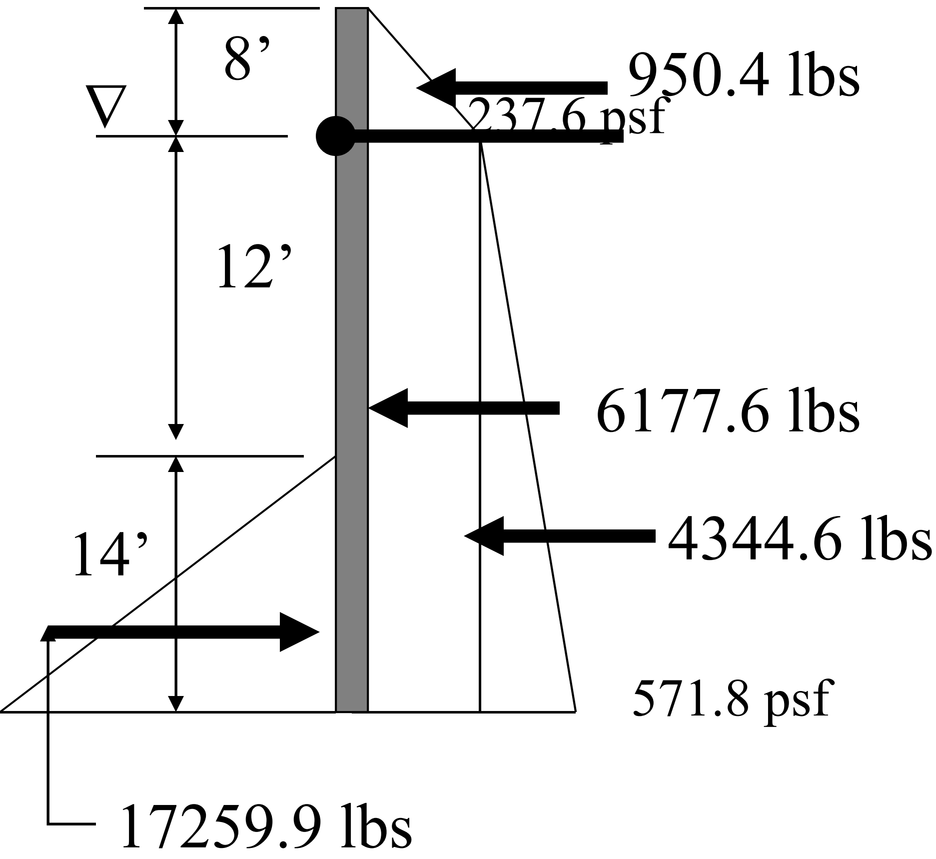
= (110-62.4) 3.7 (14) = 2465.7 psf

Lateral Passive Force = 1/2 γ Kp H^2

= 1/2 (2465.7) 14 = 17259.9 lbs/ft

Toe Failure Factor of Safety?

At toe failure, bulkhead will rotate about the anchor



Lateral Active Forces:

(Linearly Increasing)

$= 1/2\ \gamma Ka\ H^2$

= 1/2 (237.6)8 = 950.4 lbs/ft

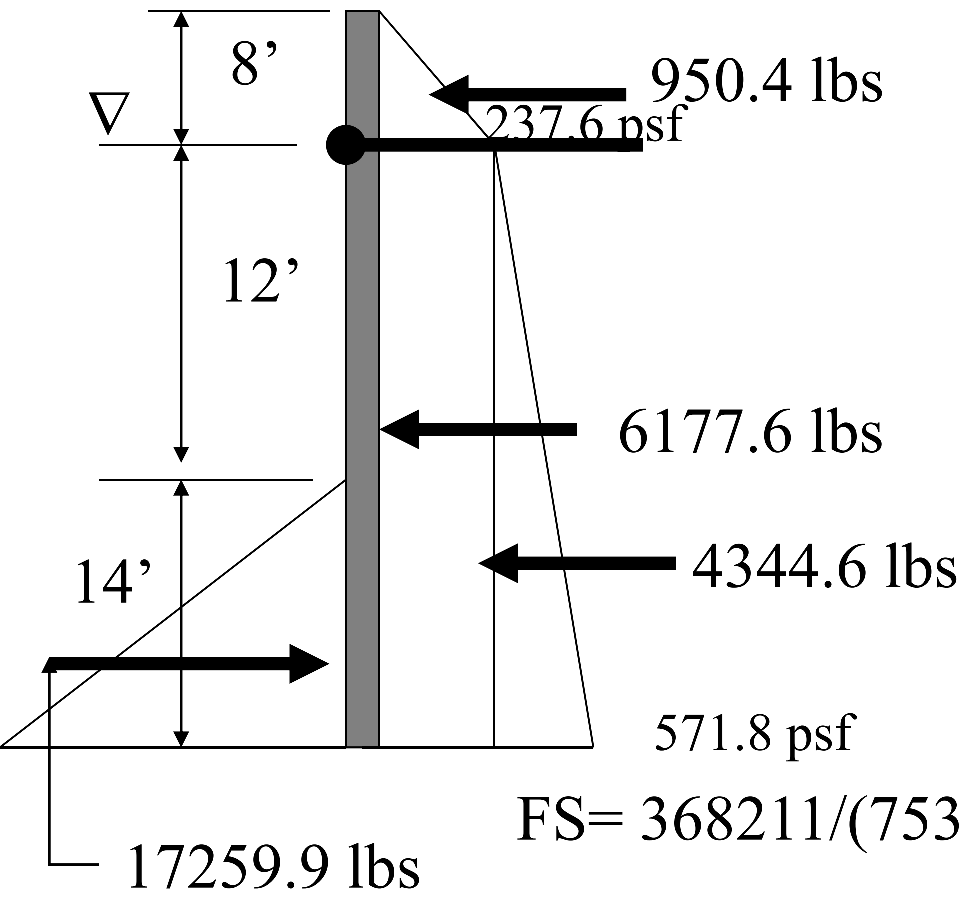
= 1/2 (571.8-237.6)26= 4344.6 lbs

(Uniform Forces)

237.6(26)= 6177.6 lbs

Toe Failure Factor of Safety?

At toe failure, bulkhead will rotate about the anchor ?



$$FS = \frac{\text{Resisting Moments}}{\text{Driving Moments}}$$

Driving Forces:

950.4 x-8/3= -2534.4 ft-lbs

6177.6 x 26/2= 80308.8 ft-lbs

4344.6 x (2/3)26= 75306.4 ft-lbs

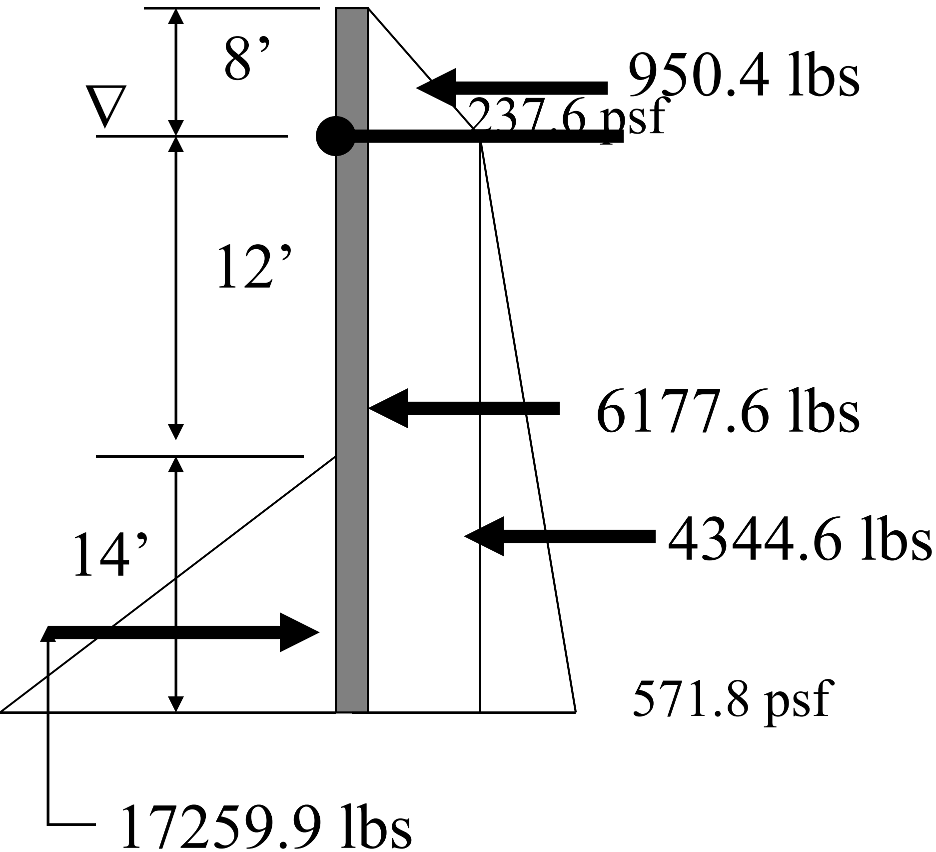
Resisting Forces:

17259.9[12+14(2/3)]=368211 ft-lbs

FS= 368211/(75306.4+80308.8-2534.4)= 2.4

What Force in Anchor for FS = 3?

Sum horizontal forces



T = ΣPa- Σ Pp/3

=950.4+6177.6+4344.6-17259.9/3

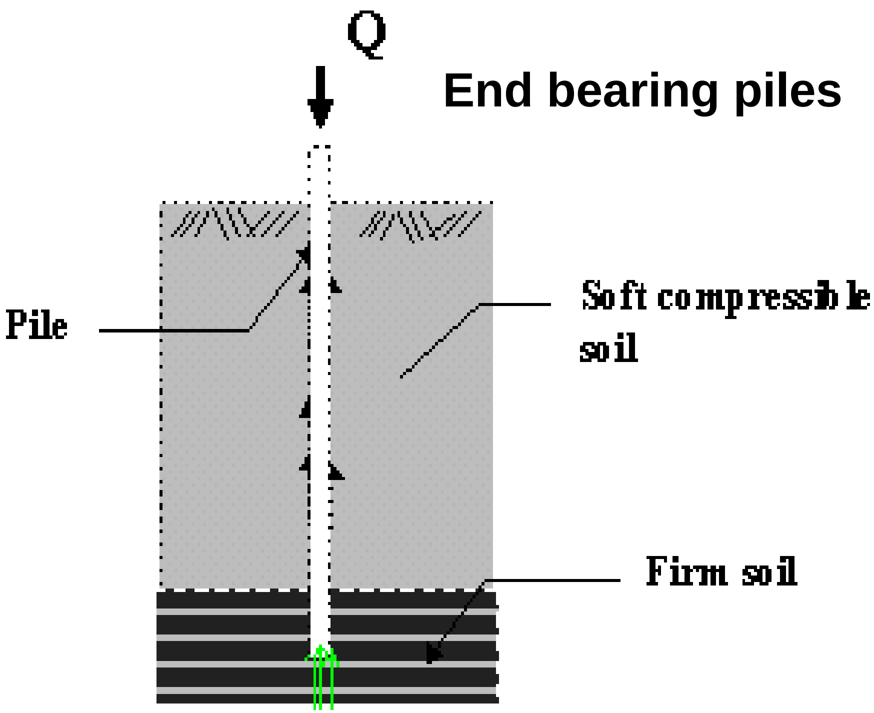
= 5719.3 lbs/ft of wall

if anchors at 5’ OC

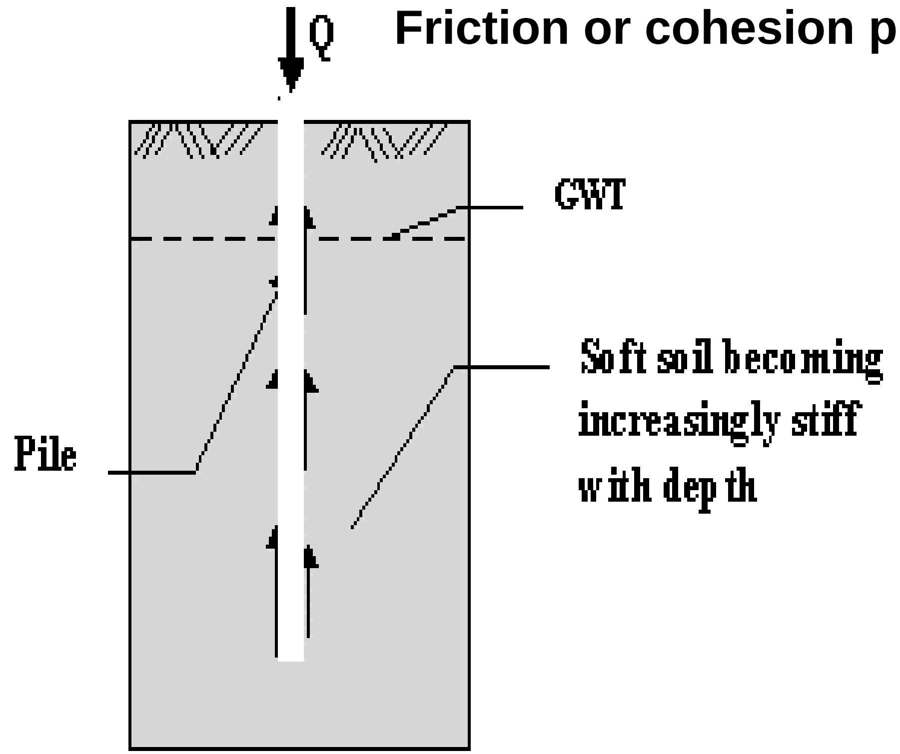
Force per anchor = 28,597 lbs

Pile Capacity

End bearing piles

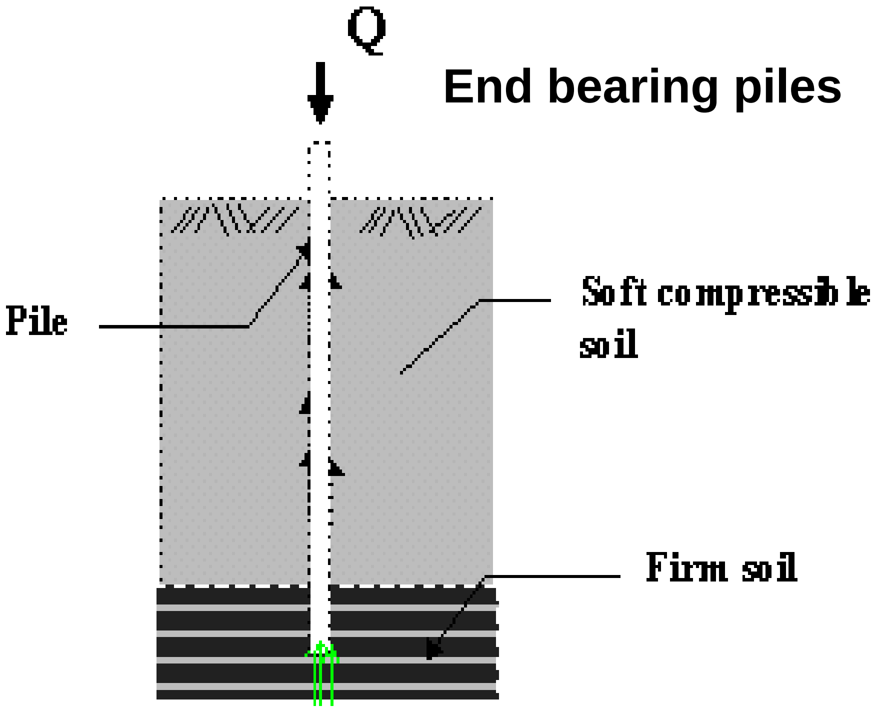


Friction or cohesion pile



Friction (Side Shear) + End Bearing

Pile Capacity



End bearing piles

A_T = Tip Area (B^2 or πR^2)

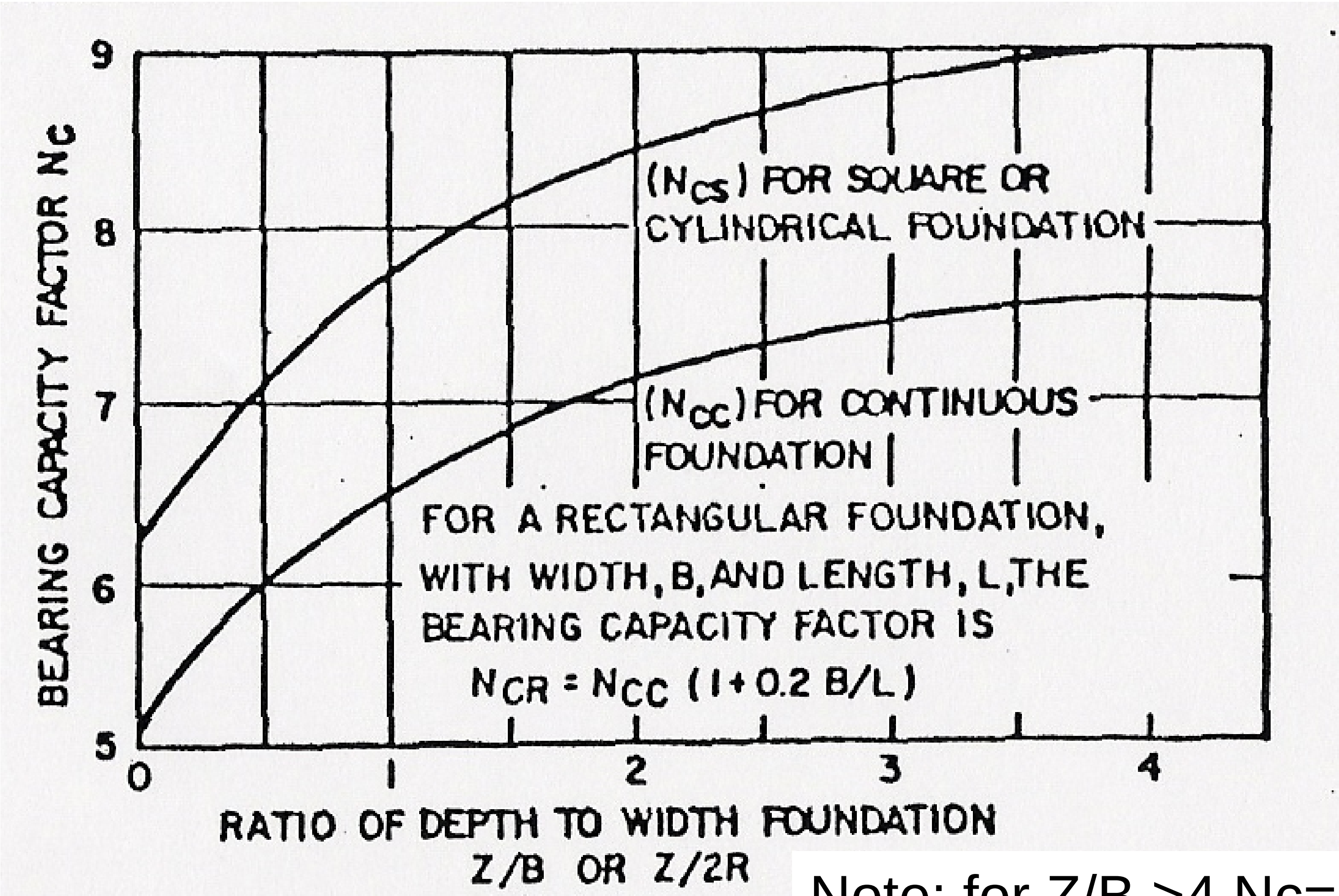
C = cohesion

q = overburden pressure

Nc & Nq Deep Bearing Factors

$$\text{Tip capacity} = A_T\,(c\,N_c + q\,Nq)$$

Pile End Bearing Factors Cohesive Soil



Note: for Z/B >4 Nc=9

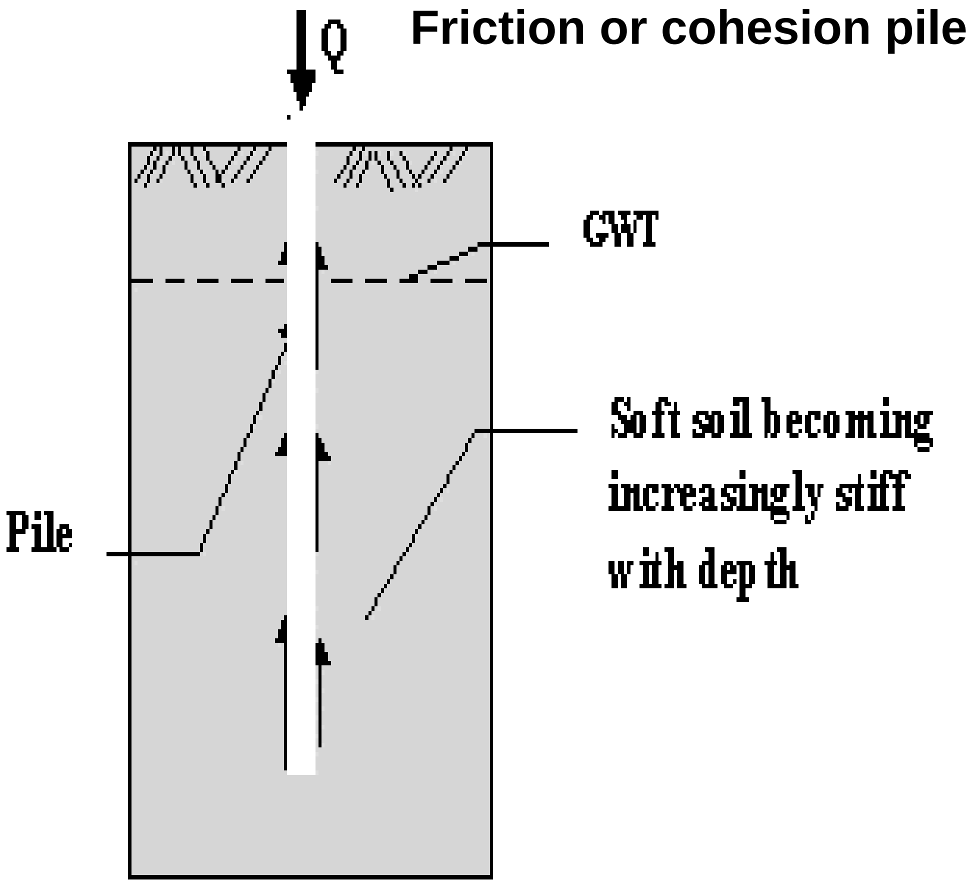
Pile End Bearing Factors Cohesionless Soil

BEARING CAPACITY FACTORS – N_q

ϕ* (DEGREES)	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q (DRIVEN PILE DISPLACE-MENT)	10	15	21	24	29	35	42	50	62	77	86	120	145
N_q** (DRILLED PIERS)	5	8	10	12	14	17	21	25	30	38	43	60	72

* & ** limit ϕ to 28 if jetting or bailer used

Pile Capacity



C= Cohesion

α = adhesion/cohesion ratio

d= pile diameter

L= pile length

K= earth pressure coefficients

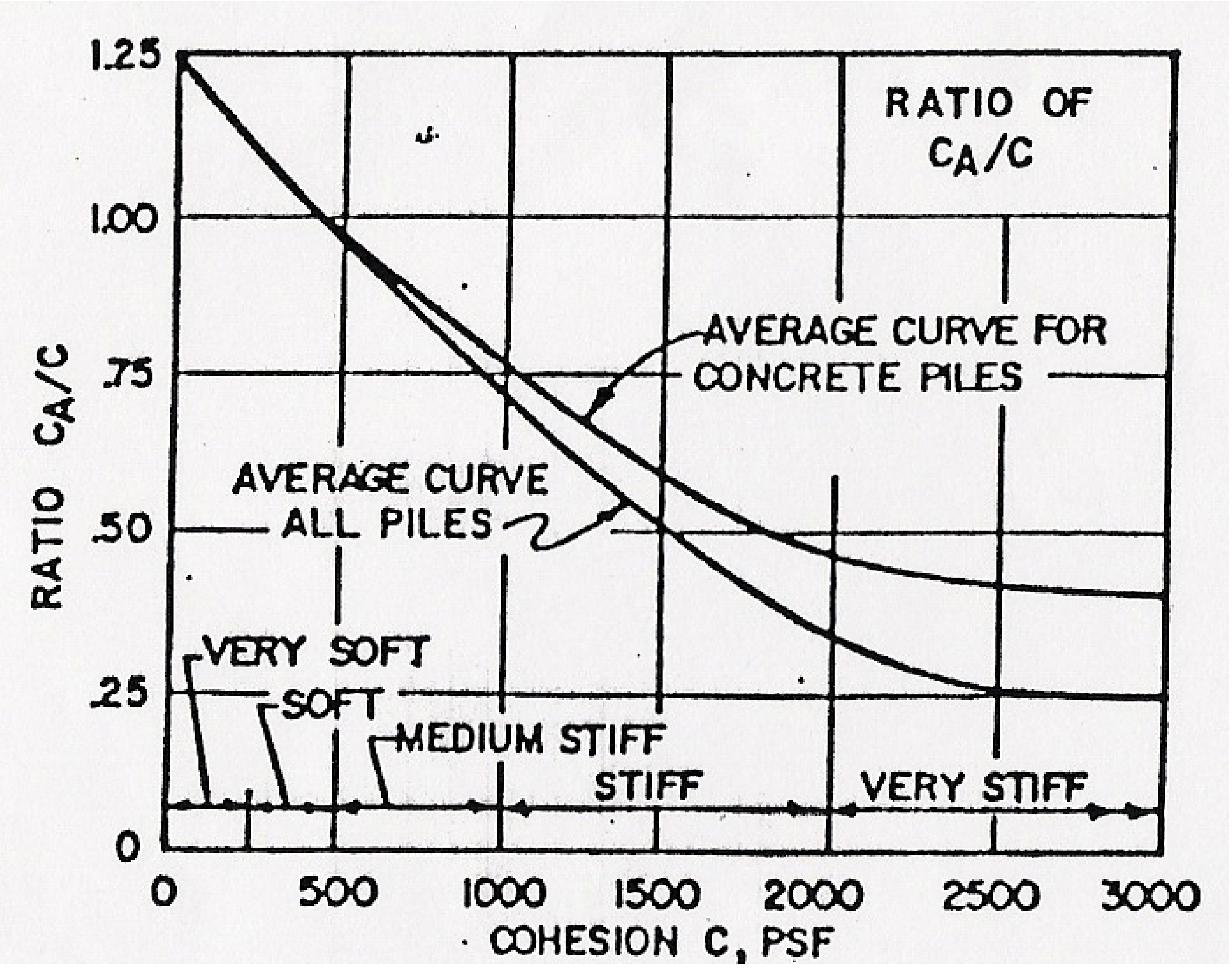
P= overburden pressure

δ= pile material friction angle

Side Shear = adhesion + friction

= C α π dL + KP tan δ πdL

Adhesion to Cohesion Ratio



Pile Earth Pressure Coefficients

EARTH PRESSURE COEFFICIENTS K_{HC} AND K_{HT}

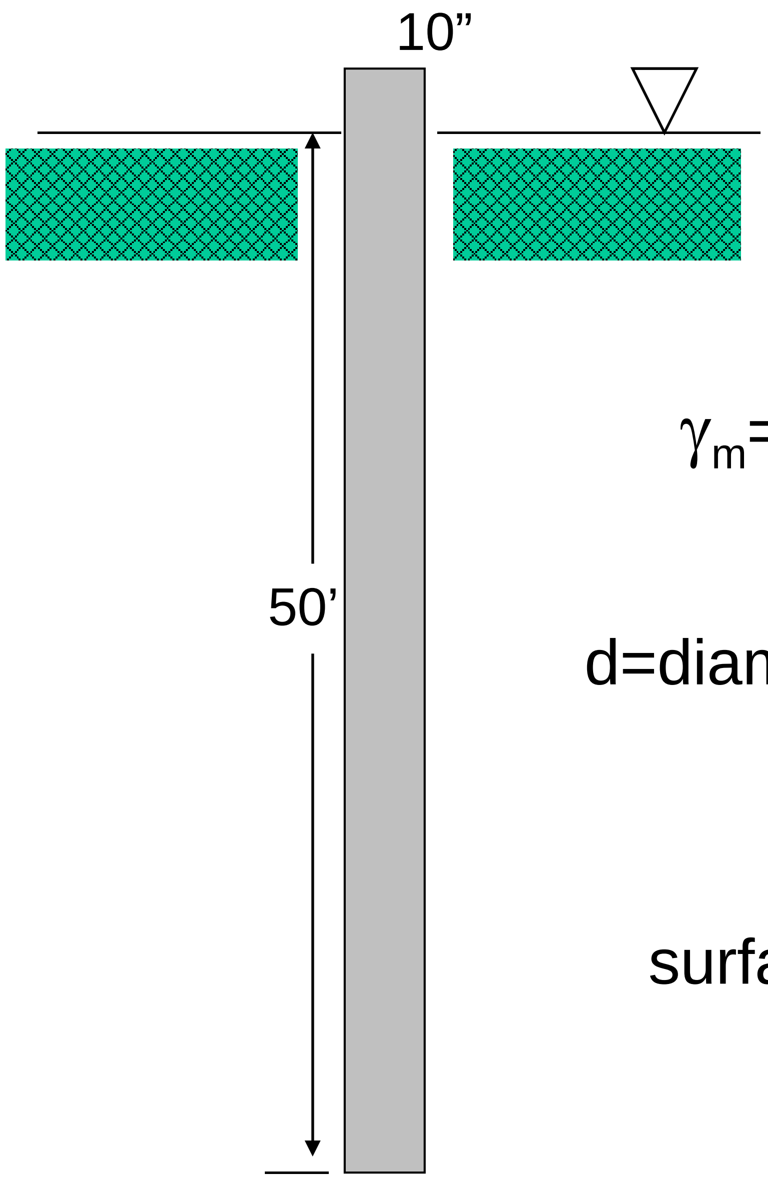
PILE TYPE	K_{HC}	K_{HT}
DRIVEN SINGLE H-PILE	0.5 – 1.0	0.3 – 0.5
DRIVEN SINGLE DISPLACEMENT PILE	1.0 – 1.5	0.6 – 1.0
DRIVEN SINGLE DISPLACEMENT TAPERED PILE	1.5 – 2.0	1.0 – 1.3
DRIVEN JETTED PILE	0.4 – 0.9	0.3 – 0.6
DRILLED PILE (LESS THAN 24" DIAMETER)	0.7	0.4

Pile Material Friction Angle

FRICTION ANGLE - δ

PILE TYPE	δ
STEEL	20°
CONCRETE	$3/4\ \phi$
TIMBER	$3/4\ \phi$

Pile Capacity - Clay



10” diameter concrete pile

qu=2800 psf ; c=1400 psf

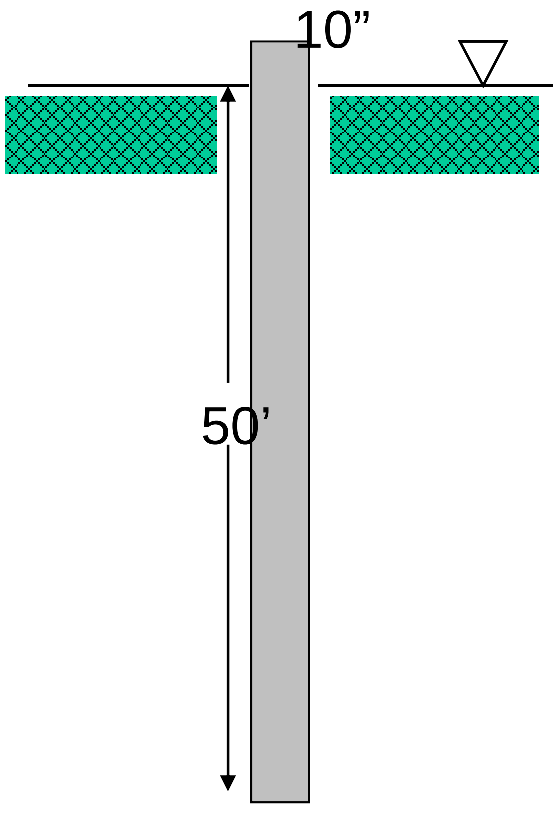
γ_m=120 pcf Ø=0

$$d = \text{diameter} = \frac{10''}{12} = 0.83' \quad \text{End Area} = \frac{\pi 0.83^2}{4}$$

$A = 0.54\ ft^2$

surface area=πdL=π0.83x50=130.4 ft^2

Pile Capacity - Clay



concrete driven pile;

C=1400 psf $\alpha \cong 0.6$

Tip capacity=A(CN_c+qN_q)

=ACN_c=0.54(1400)9=6804 lbs

Side Shear = C α π d L

=1400(0.6)130.4=109,536 lbs

Pile Capacity - Clay

Total Capacity

= 6804 + 109536
= 116,340

or

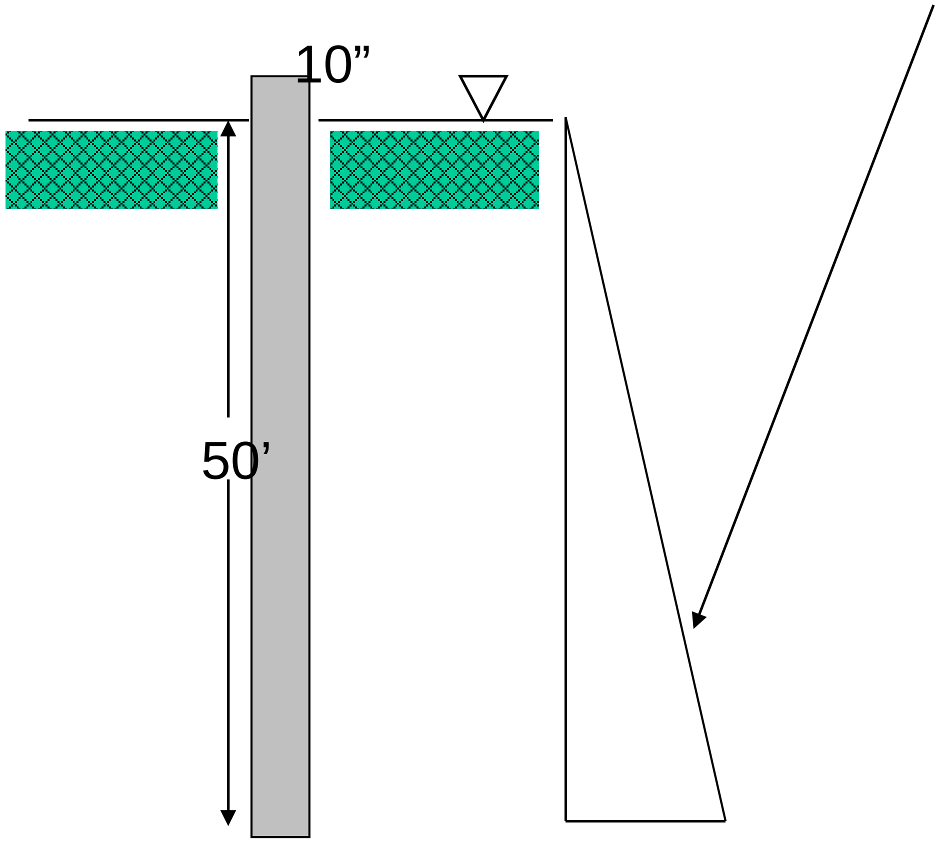
= 58 tons ultimate

for FS = 2

Working Capacity
= **29 tons**

Pile Capacity - Sand

Load Capacity in Sand Depends on Confining Pressure



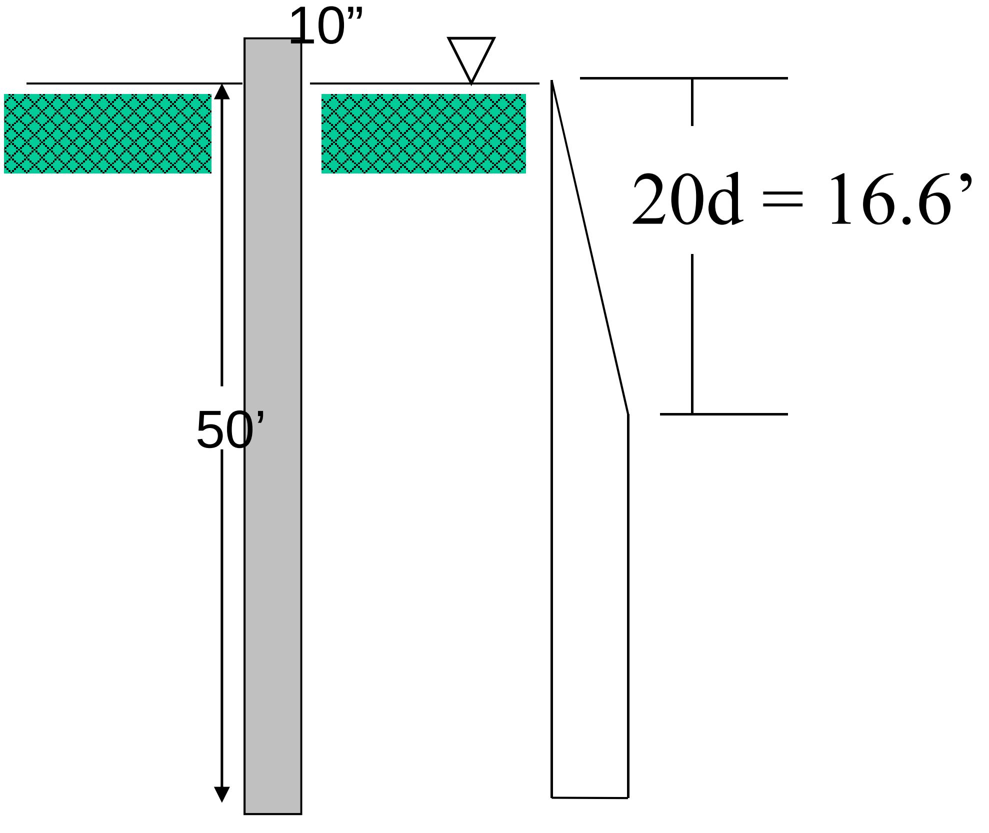
Do end bearing and side shear therefore increase infinitely with depth?

A) Yes

B) No

Pile Capacity - Sand

Answer is "B" No - Tests show confinement effects are constant below "critical depth"



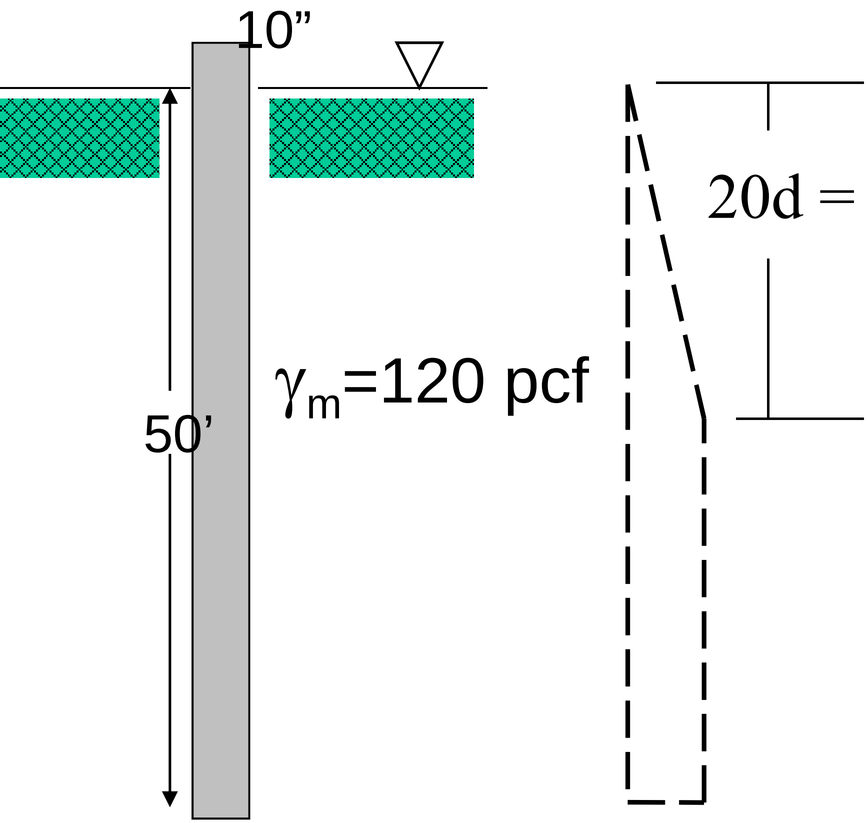
Critical Depth

20d for dense sand

15d for medium sand

10d for loose sand

Pile Capacity - Sand



<u>Confining Pressure</u>

$P_o = 0$

Max P = 16.6’(120-62.4)=956.2 psf

P= 956.2 psf

Pile Capacity - Sand

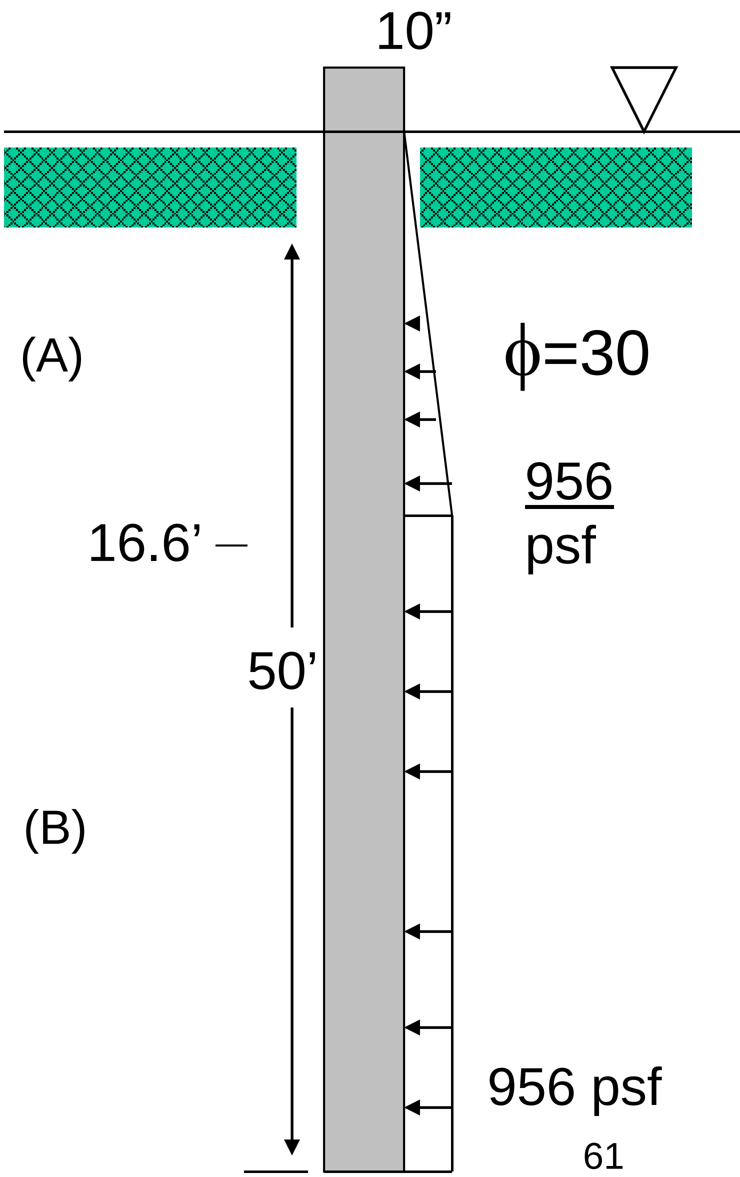
N_q=21; K=1.5

$\delta = {}^3/_4\phi = .75(30) = 23^o$

Tip Capacity=A_q Pmax N_q

=0.54(956)21

= 10,841 lbs

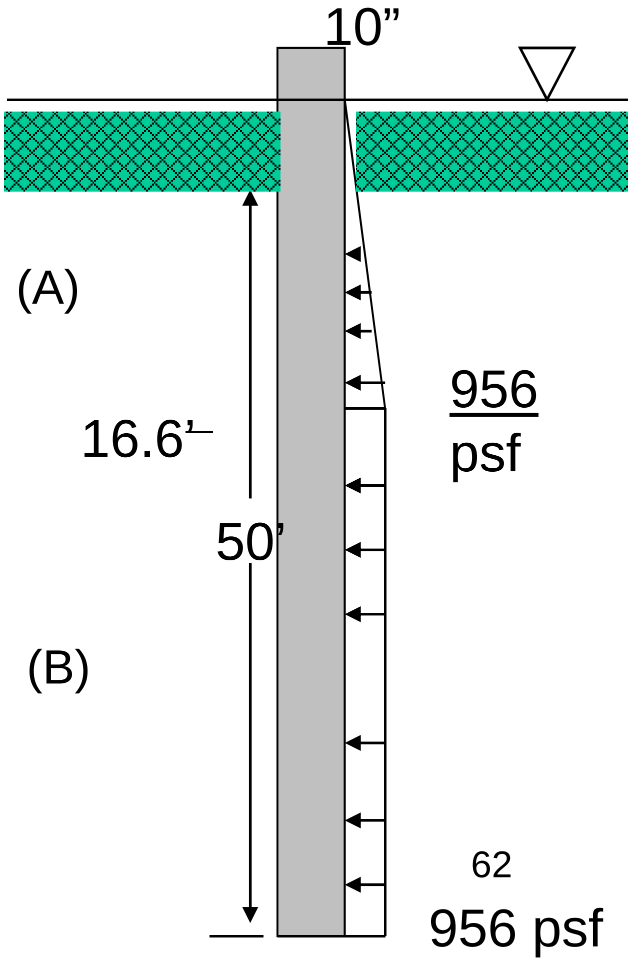


Side shear (friction)=KP tan δ πdL

Section A:

=1.5 (956 / 2) tan 23 π 0.83 (16.6)

= 13,174 lbs

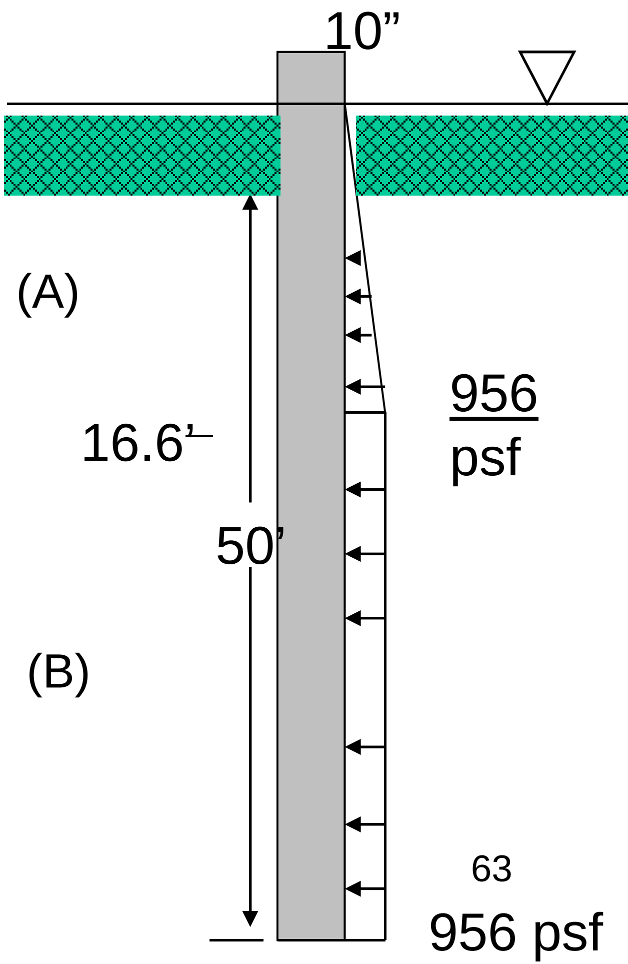


Side shear (friction)=KP tan δ πdL

Section B

$=1.5(956) \tan 23\ \pi\ 0.83(33.4)$

= <u>53,012 lbs</u>



Side shear (friction)=KP tan δ πdL

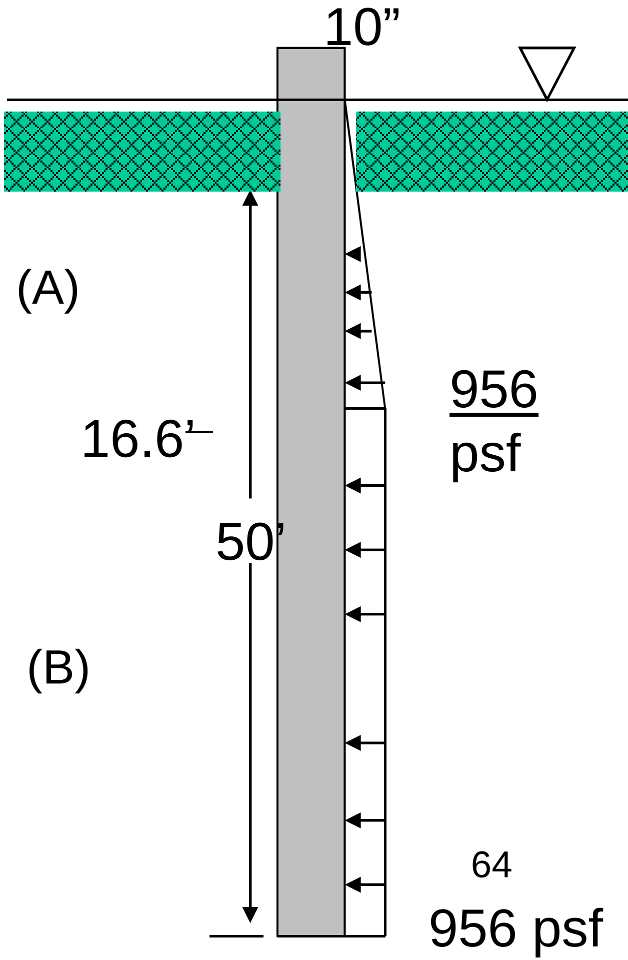
Total Capacity = tip + side shear

= 10841+13174+53012

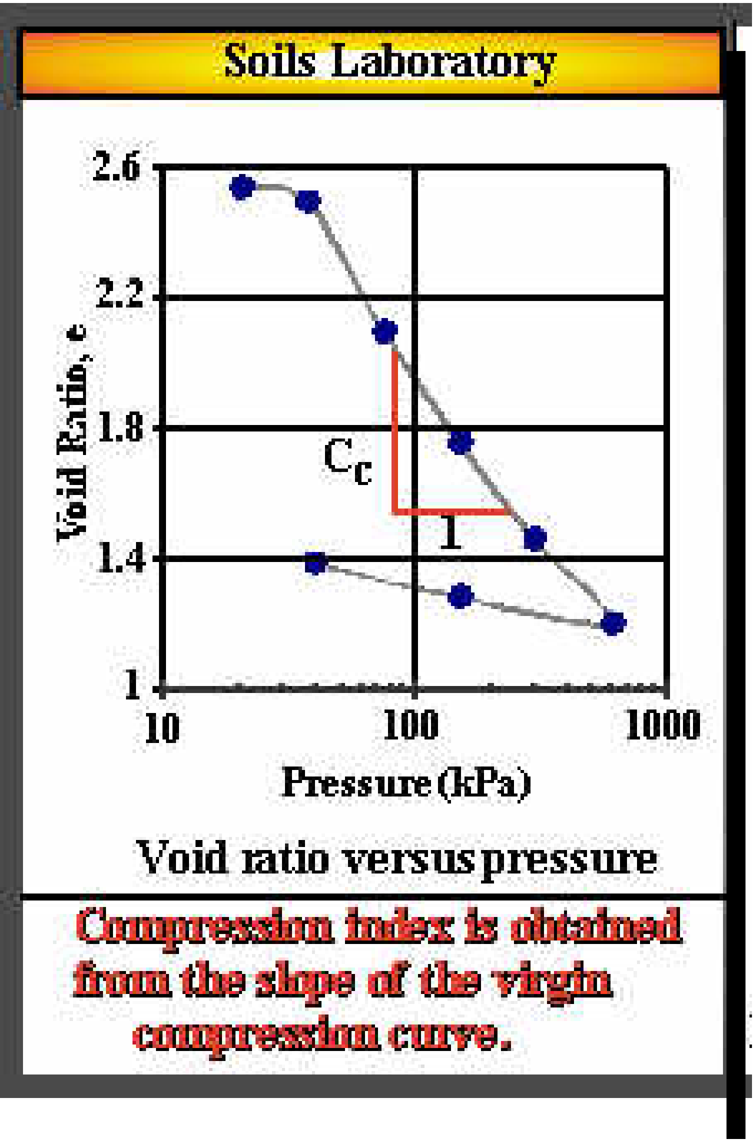
= 77027 lbs

= 38.5 tons Ultimate

for FS = 2, = 19 tons



Settlement Calculation



$S = \Delta H = (H \times \Delta e)/(1+e_o)$

or

$S = HC_c/(1+e_o)\log{(p_0 + \Delta p)/p_0}$

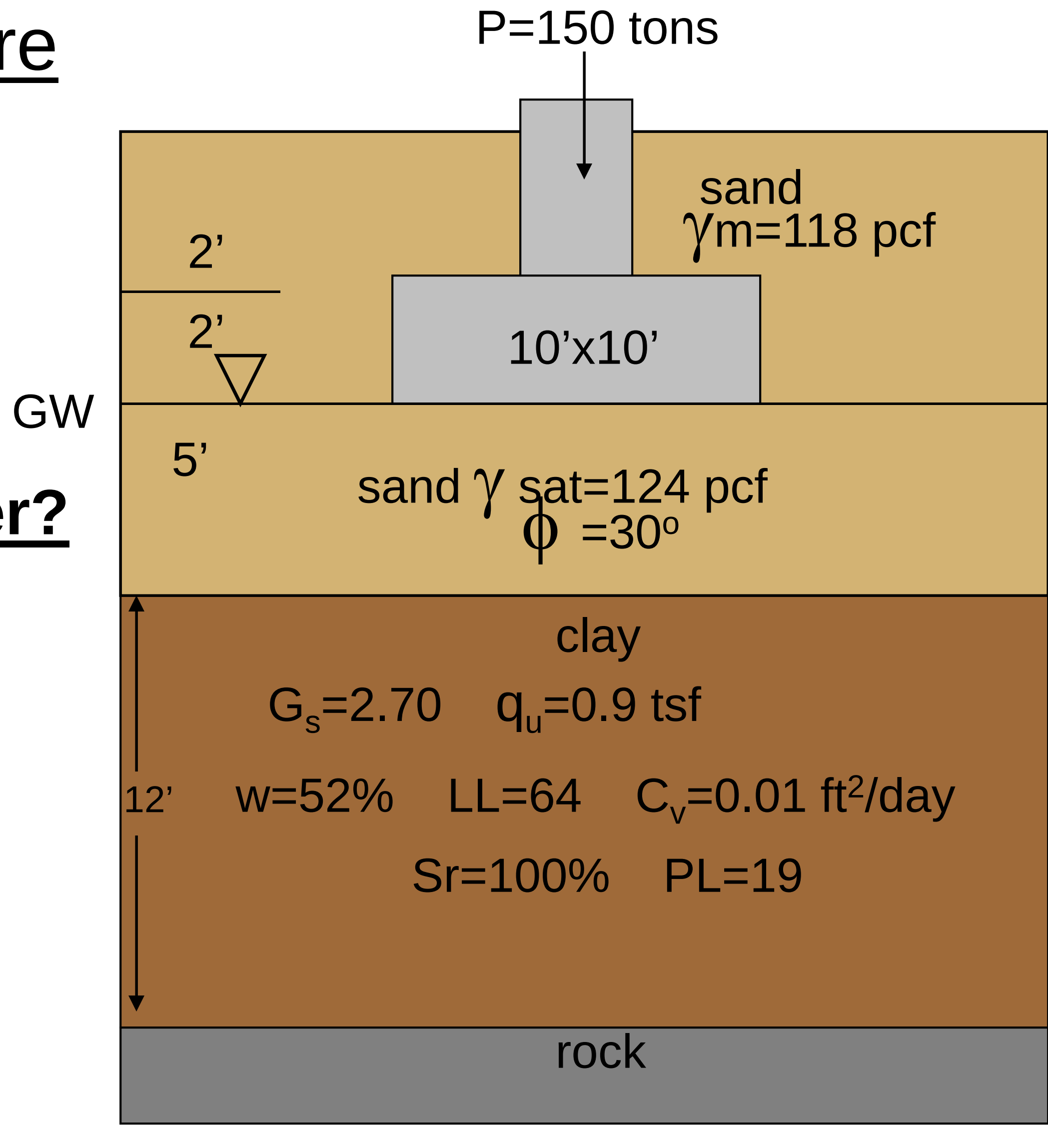
Footing Settlement

Calculate Bearing Pressure

$$BP = \frac{150 \times 2000}{10^2} = 3000 \text{ psf}$$

What is the stress change

Δ P at mid-height in clay layer?



A) 680 psf

B) 3000 psf

C) 2000 psf

Footing Settlement

Calculate Bearing Pressure

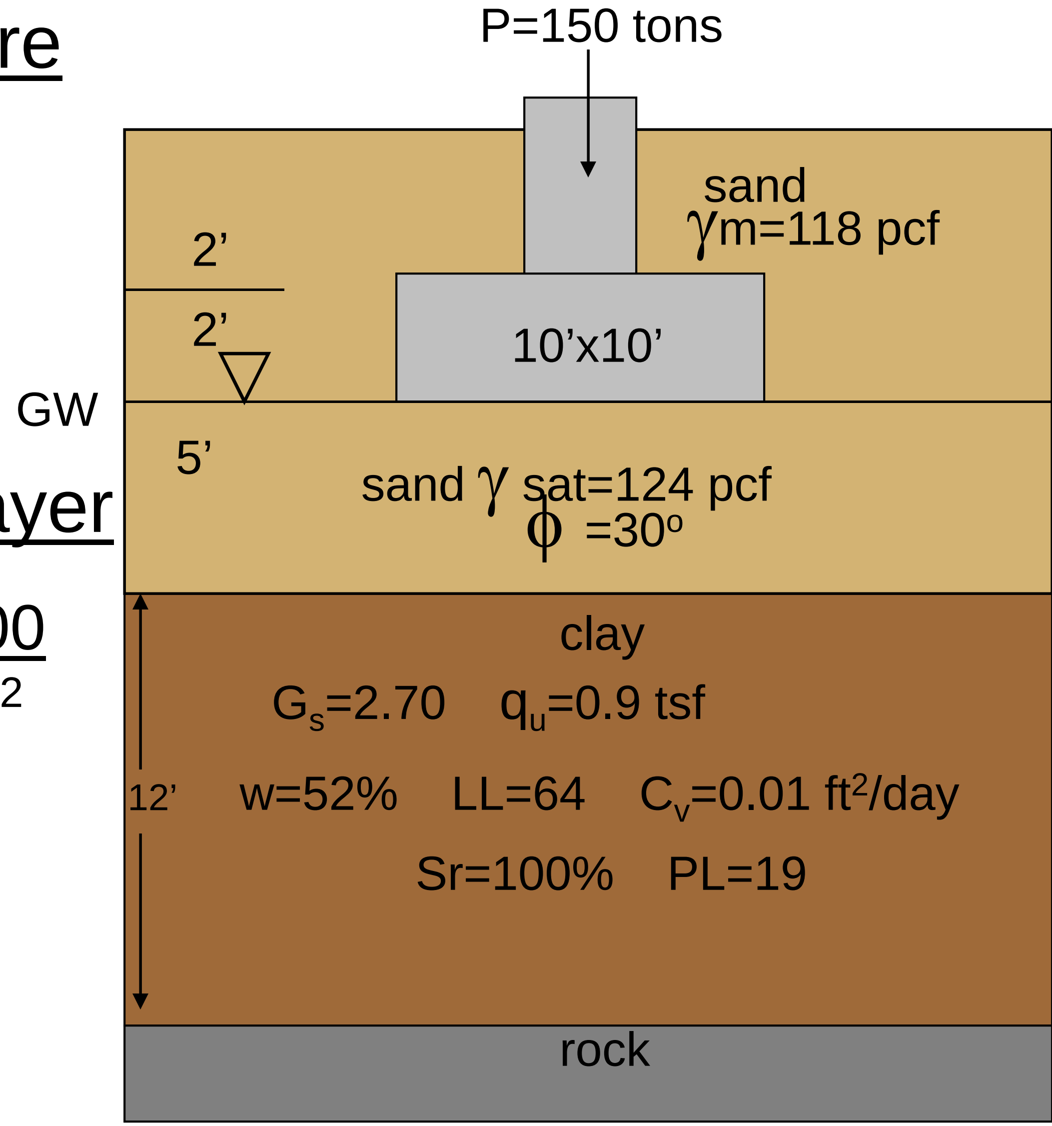
$$BP = \frac{150 \times 2000}{10^2} = 3000 \text{ psf}$$

Δ Pat mid-height in clay layer

$$\Delta P = \frac{P}{(B+Z)(L+Z)} = \frac{150 \times 2000}{(10+11)^2}$$

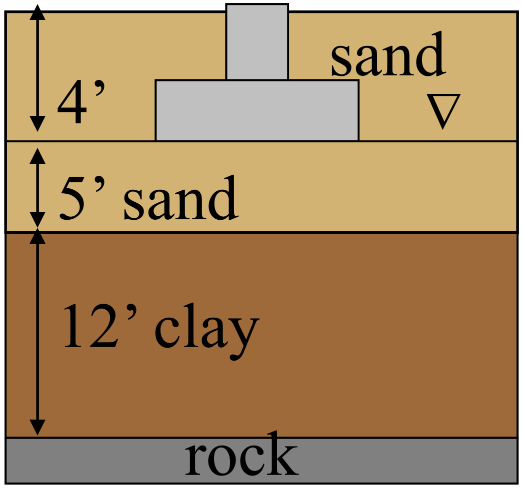
= 680 psf

Answer is "A"



Calculate e_0

$$e_o = \frac{wG_s}{S} = \frac{0.52\,(2.7)}{1} = 1.404$$



Calculate Unit Weight of Clay

$$\gamma_{sat} = \frac{(G_s + e)\gamma_\omega}{(1 + e)} = \frac{(2.7 + 1.404)\ 62.4}{(1 + 1.404)} = 106.5 \text{ pcf}$$

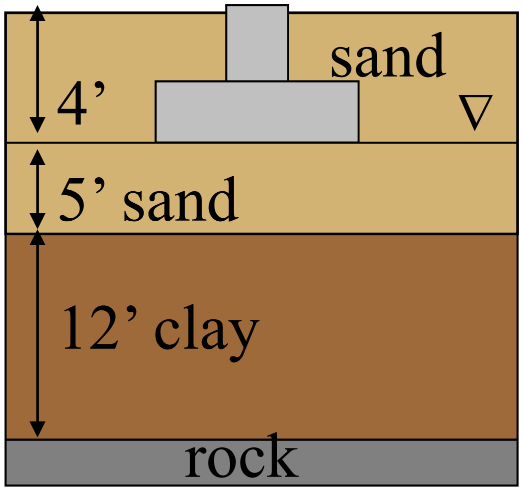
Calculate effective stress at mid height of clay layer

$$P_o = \sigma' = \sigma_T - \mu$$

$$= 4 \times 118 + 5 \times 124 + 6 \times 106.5 - 11 \times 62.4$$

$$= 1045 \text{ psf}$$

Calculate settlement of clay layer



Note: moisture is close to LL, void ratio is high & loading is large; expected settlement will be large.

Cc=.009(LL-10) after Skempton

Cc=.009(64-10)= 0.49

$$S=\Delta H = [C_c/ (1+e_o)]\ H \log [(Po+ \Delta P)/ Po]$$

$$= [0.49 / (1+1.404)]12 \log [(1045+680)/ 1045]$$

= 0.53’ or 6.4”

Calculate settlement using e-log p curve

$P_1 = 1045$; $e_1 = 1.395$

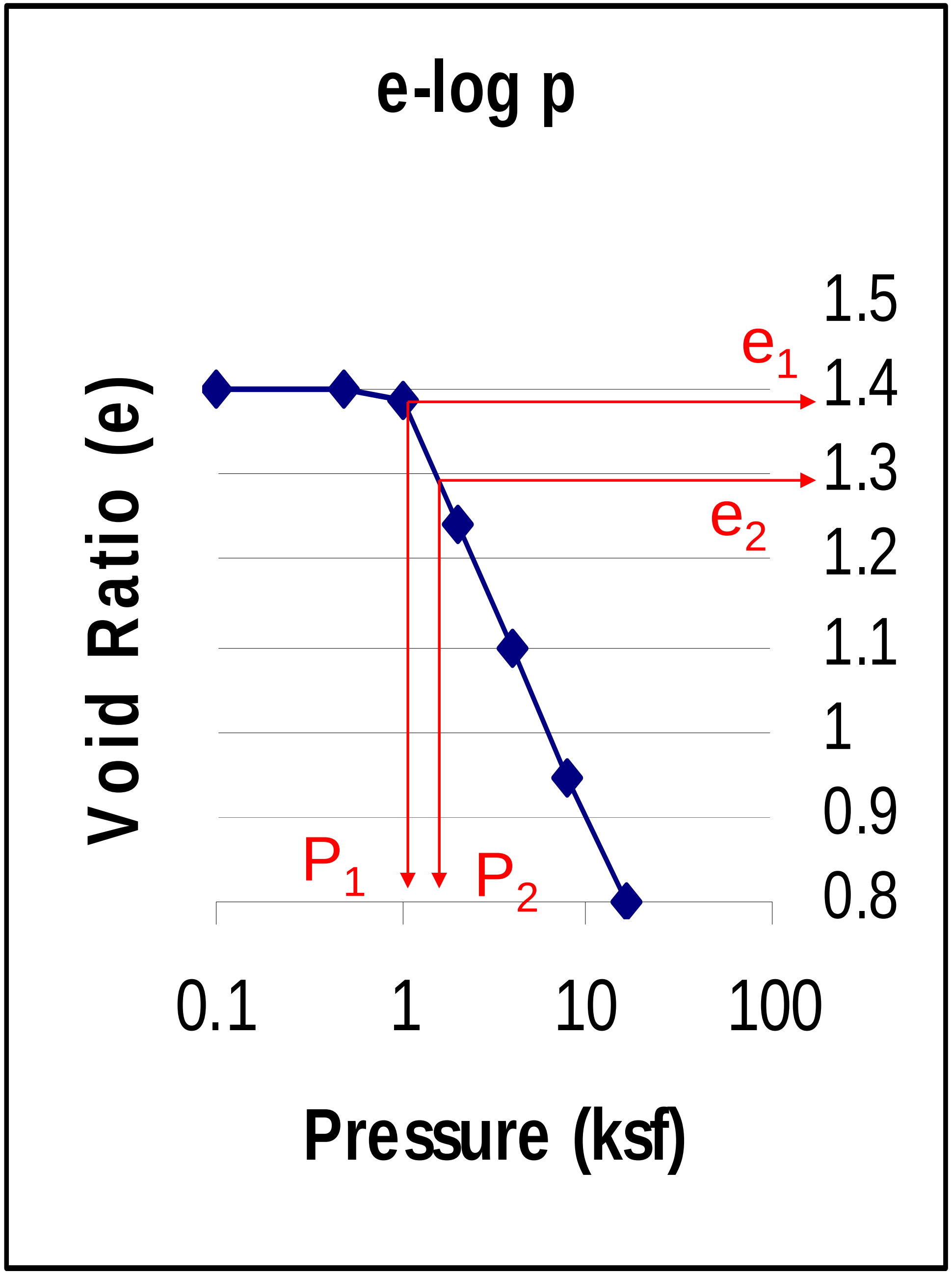
$\Delta P = 680$ psf

$P_2 = 1725$; $e_2 = 1.288$

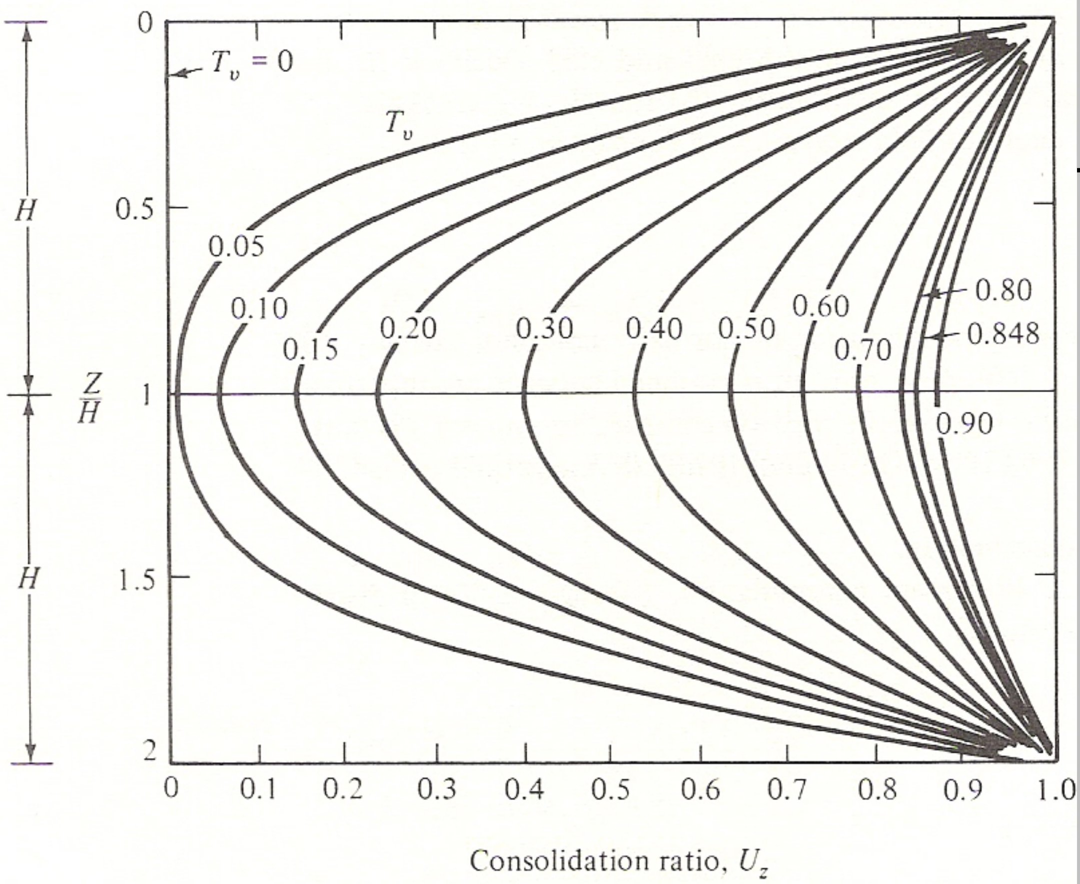
$S = \Delta H = H \Delta e / (1 + e_o)$

$= 12(1.395 - 1.288)/(1+1.404)$

= 0.53' or 6.4"



Rate of Consolidation



Avg. Degree of Consolidation U%	Time Factor Tv
10	0.008
20	0.031
30	0.071
40	0.126
50	0.197
60	0.287
70	0.403
80	0.567
90	0.848
100	

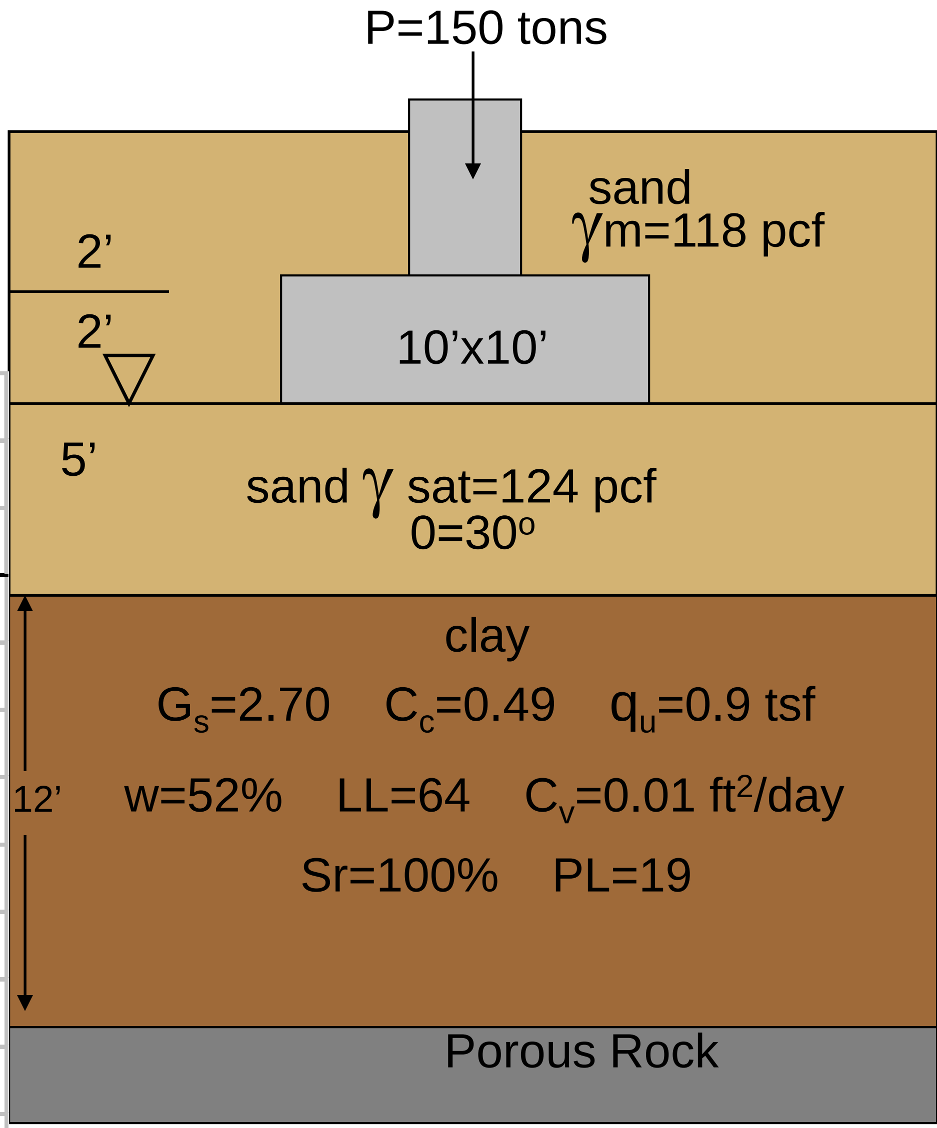
$$T_v = C_v\, t / H^2$$

How Long Will it Take to Achieve 70 Percent Consolidation?

A) 16 years

B) 4 years

Degree of Consolidation U%	Time Factor Tv
10	0.008
20	0.031
30	0.071
40	0.126
50	0.197
60	0.287
70	0.403
80	0.567
90	0.848
100	∞



Time to 70 Percent Consolidation

$$t = \frac{T_v\,(H/2)^2}{C_v} = \frac{0.403\,(12/2)^2}{0.01}$$

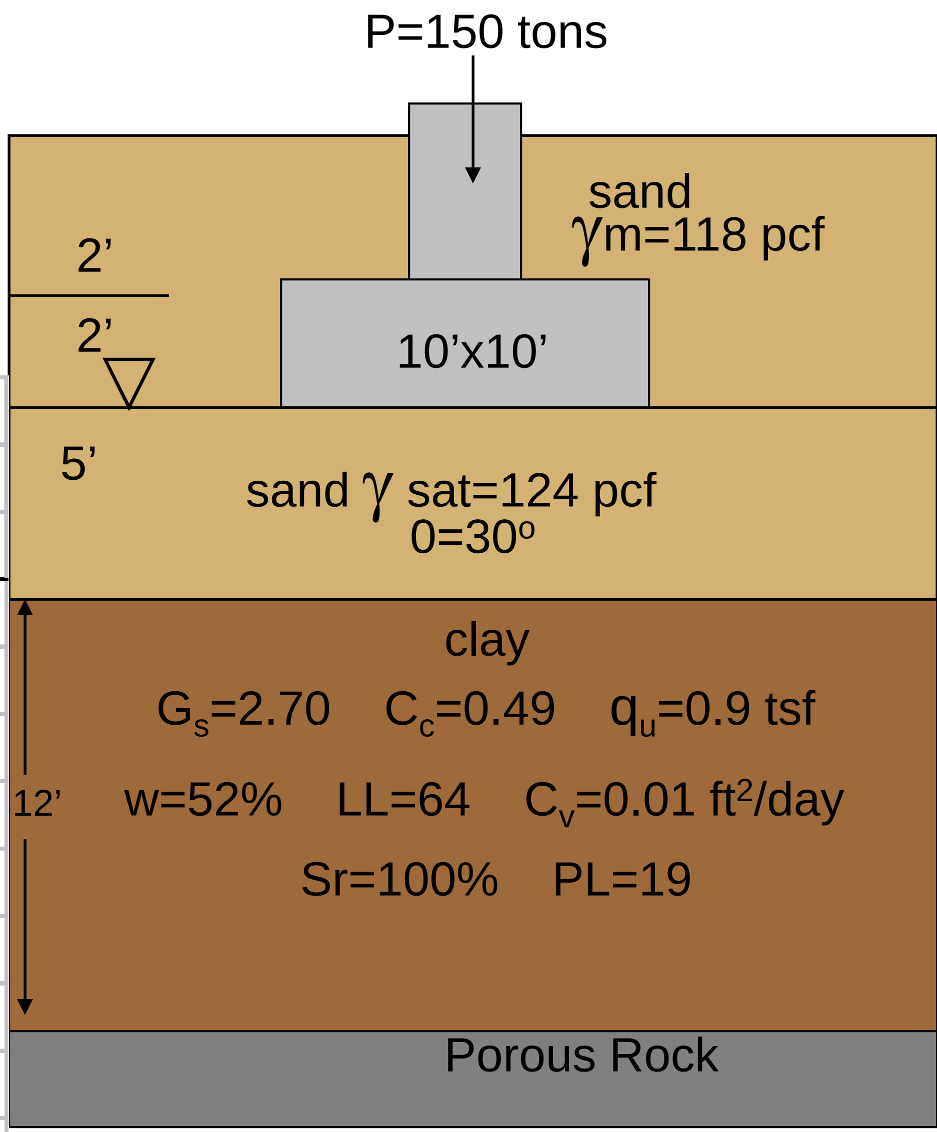
= 1450.8 days

or

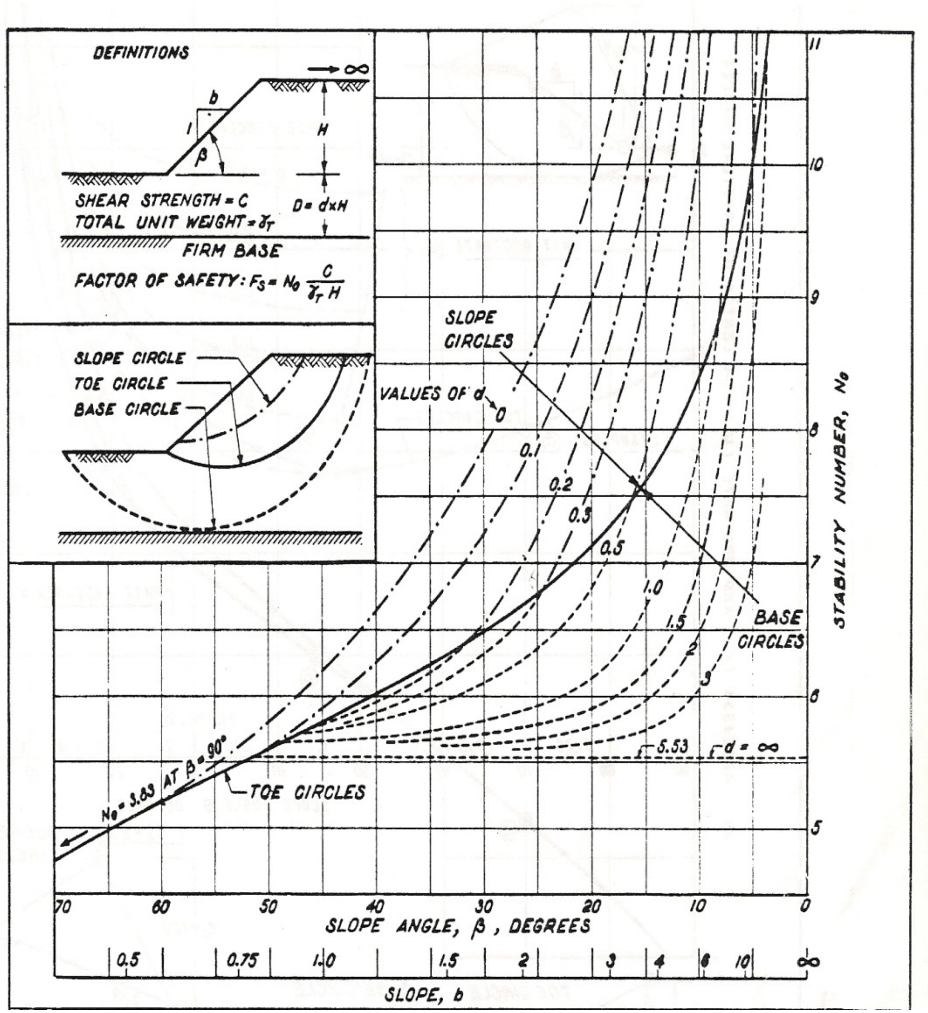
±4 years

Answer is "B"

Degree of Consolidation U%	Time Factor Tv
10	0.008
20	0.031
30	0.071
40	0.126
50	0.197
60	0.287
70	0.403
80	0.567
90	0.848
100	∞

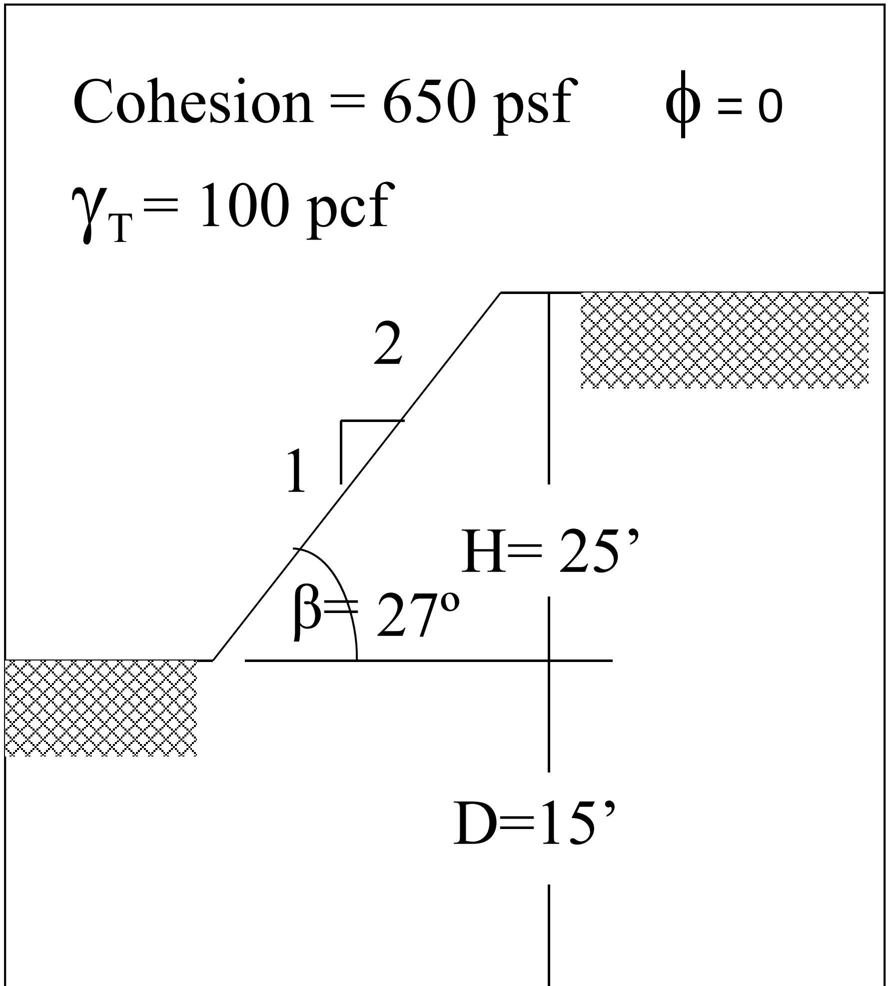


Circular Failure Stability



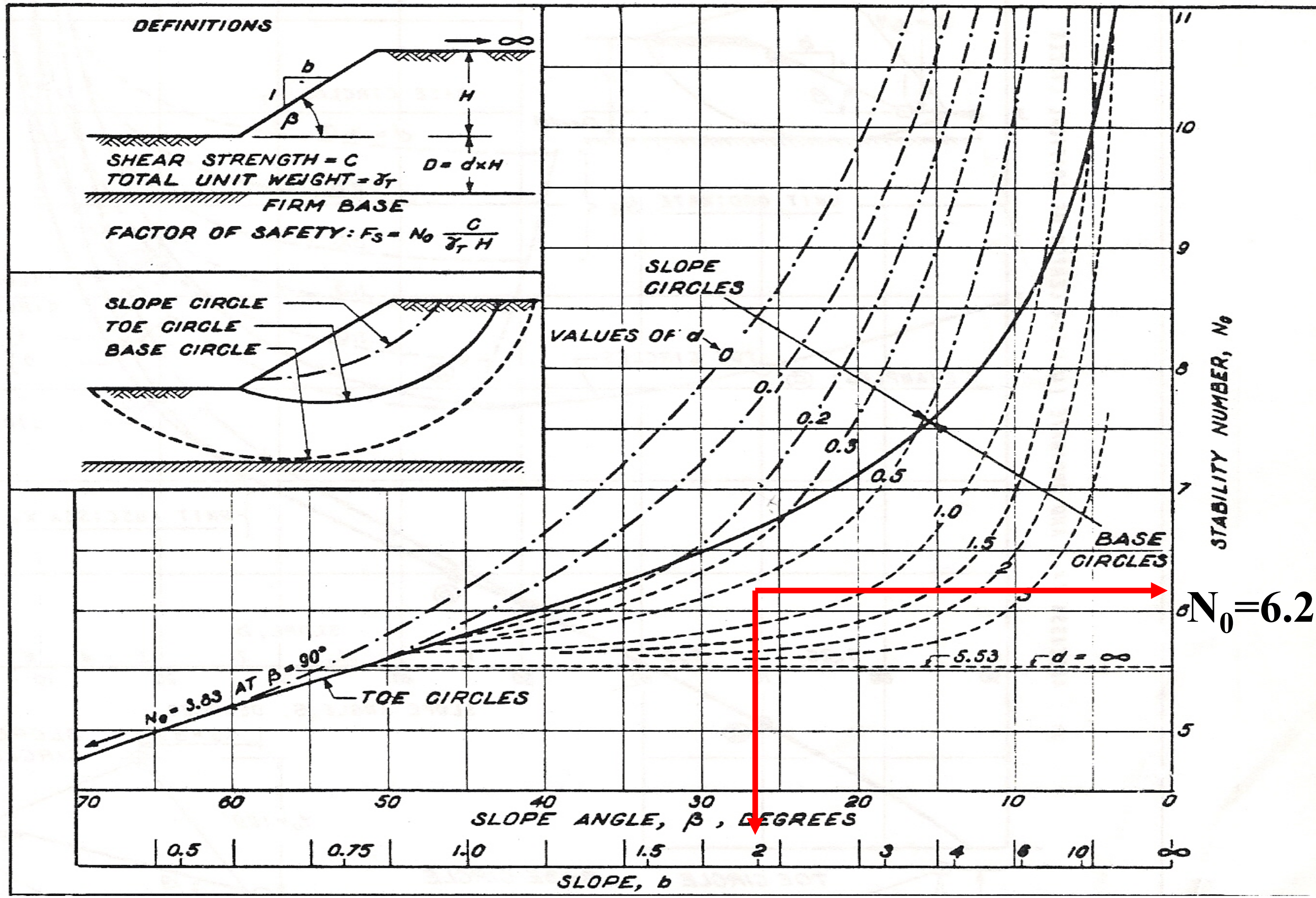
- Homogeneous soils
- No surcharge
- No tension cracks
- ϕ = zero
- Circular arc failure
- No water on slope

Slope Stability by Charts

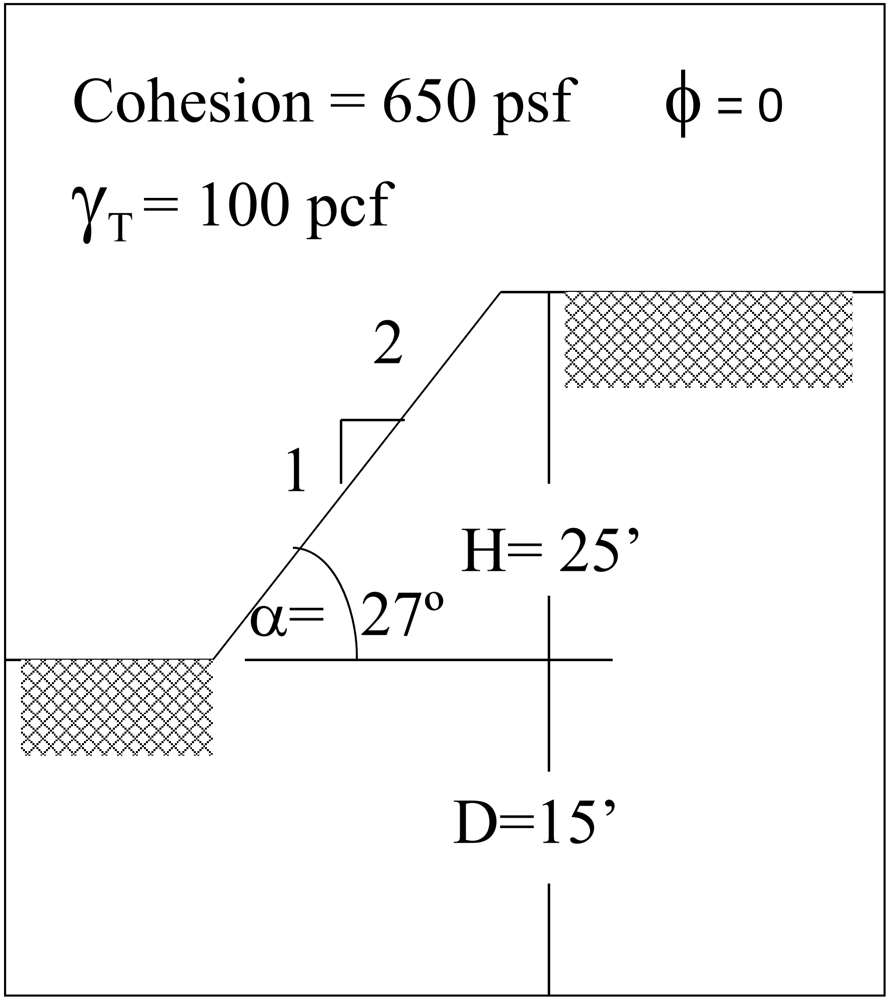


$d = D/H = 15/25 = 0.6$

$FS = N_0 C/(\gamma_T H)$


DEFINITIONS
b
1
β
H
D = d×H
SHEAR STRENGTH = C
TOTAL UNIT WEIGHT = γ_T
FIRM BASE
FACTOR OF SAFETY: $F_S = N_0 \frac{C}{\gamma_T H}$
SLOPE CIRCLE
TOE CIRCLE
BASE CIRCLE
SLOPE CIRCLES
VALUES OF d → 0
0.1
0.2
0.3
0.5
1.0
1.5
2
3
BASE CIRCLES
5.53
d = ∞
N_0 = 3.83 AT β = 90°
TOE CIRCLES
STABILITY NUMBER, N_0
SLOPE ANGLE, β, DEGREES
SLOPE, b
N_0=6.2

Slope Stability by Charts



$d = 15/25 = 0.6$

$FS = N_0 C/(\gamma_T H)$

$FS = 6.2(650)/(100 \times 25)$

$FS = 1.6$

Thanks for participating in the PE review course on Foundation Engineering!

More questions or comments?

You can email me at:

gtv@gemeng.com