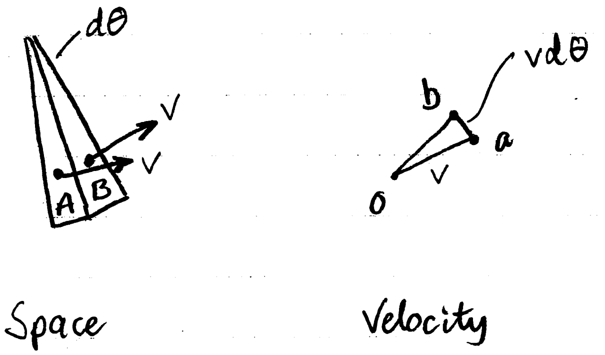


4D5 Foundation Engineering

2005 Exam crib.

1) a) Consider 2 infinitesimal elements (rigid)

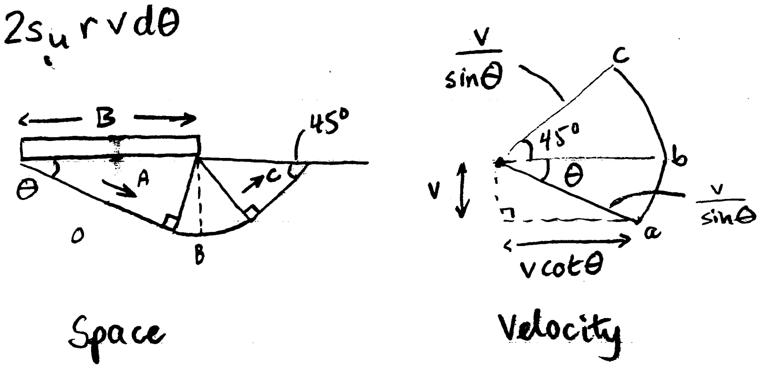


$\dot{D} = \Sigma\,(s_u \times \text{length} \times \text{relative velocity})$ (summed over all slip planes

$= \underbrace{s_u \underbrace{r\,d\theta}_{\text{length}} v}_{\text{on circumference}} + \underbrace{s_u \underbrace{r\,d\theta}_{\text{length}} v}_{\text{on radial plane}}$

$= 2 s_u r v\,d\theta$

b)



Call v, vertical component of foundation velocity.

Work dissipated $= \underbrace{s_u\, B\cos\theta \frac{v}{\sin\theta}}_{OA} + \underbrace{2 s_u\, B\sin\theta \frac{v}{\sin\theta}\left(\frac{\pi}{4}+\theta\right)}_{\text{fan}} + \underbrace{s_u B\sin\theta \frac{v}{\sin\theta}}_{OB}$

$= s_u B v \left(\cot\theta + \frac{\pi}{2} + 2\theta + 1\right)$

Work input $= Hv\cot\theta + Vv$

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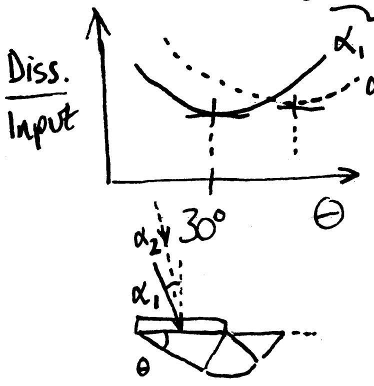
1) b) $\Rightarrow$ $$\frac{\text{Dissipated}}{\text{Input}} = \frac{s_u B\left(\cot\theta + \pi/2 + 2\theta + 1\right)}{H\cot\theta + V}$$

c) For a given load inclination, optimal mechanism will occur. (i.e. value of θ for which Dissipated/Input is minimum).

To find minimum:

Numerator of $\frac{d(\text{Dissipated}/\text{Input})}{d\theta}$ is (by quotient differentiation): (ignoring constant $s_u B$).



Higher θ optimal for smaller α. Given a value of θ, corresponding α is found from $d(\text{Diss}/\text{Input})/d\theta = 0$.

$$= \left(-\frac{1}{\sin^2\theta} + 2\right)\left(H\cot\theta + V\right) - \left(\cot\theta + \frac{\pi}{2} + 2\theta + 1\right)\left(\frac{-H}{\sin^2\theta}\right)$$

$$= 2H\cot\theta - \frac{V}{\sin^2\theta} + 2V + \left(\frac{\pi}{2} + 2\theta + 1\right)\frac{H}{\sin^2\theta}$$

$$= V\left(2\sin^2\theta - 1\right) + H\left(2\sin\theta\cos\theta + \frac{\pi}{2} + 2\theta + 1\right)$$

$$= V\left(2\sin^2\theta - 1\right) + H\left(\sin 2\theta + \frac{\pi}{2} + 2\theta + 1\right)$$

Put equal to zero and substitute for $\theta = 30°$

$$\frac{H}{V} = \frac{\left(1 - 2\sin^2\theta\right)}{\left(\sin 2\theta + \pi/2 + 2\theta + 1\right)} = \frac{1/2}{\sqrt{3}/2 + \pi/2 + \pi/4 + 1}$$

$$\frac{H}{V} = 0.118 \quad \Rightarrow \quad \alpha = \tan^{-1}\left(\frac{H}{V}\right) = 6.75°$$

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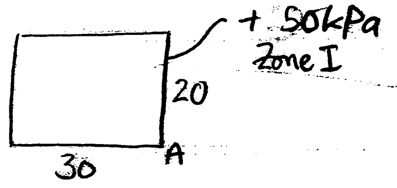
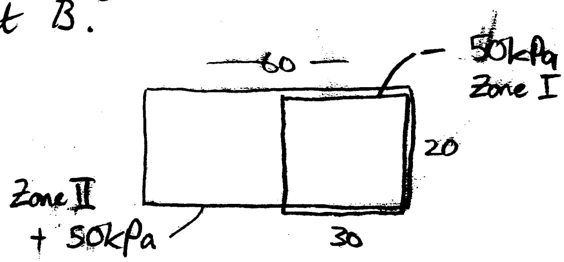
2) a) Divide subsoil into 3 layers, thinner close to ground surface, where $d(\Delta\sigma_v')/dz$ is highest.

Consider two loaded regions, using superposition to calculate settlement at point B.



Load case for settlement at A



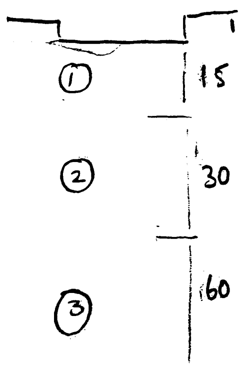
Load cases for settlement at B (using superposition of Zone I and II

Settlement of each layer, ΔH, estimated as: ($\kappa = 0.02$)

$$V_0 = 1.5 - 0.02 \ln \sigma_{v0}'$$
$$\Delta v = 0.02 \ln\left(\frac{\sigma_{v0}' + \Delta\sigma_v}{\sigma_{v0}'}\right)$$
$$\Delta H = H \frac{\Delta v}{V_0}$$

Using fadum
1) Divide soil into layers, thinner at top.
2) Calculate position of mid-depth normalised by L and B, hence m, n
3) Use fadum chart to find influence factor, and hence stress increment at each layer
4) Use expression on left to determine settlement of each layer

Using fadum chart to calculate influence factor for each layer:



Layer	Mid-depth (below fndn base)	Zone I $n \times m$	Zone II $n \times m$	Zone I I (Fadum)	Zone II I (Fadum)	$\Delta\sigma_v$ (kPa) Zone I	$\Delta\sigma_v$ (kPa) Zone II
1	7.5	4×2.67	8×2.67	0.24	0.24	12	12
2	30	1×0.67	2×0.67	0.145	0.16	7.25	8
3	75	0.4×0.27	0.8×0.27	0.045	0.07	2.25	3.5

Layer	Thickness, H	σ_{v0}' (inc. 1m surcharge)	Δv Zone I	Δv Zone II	V_0	ΔH (mm) zone I	ΔH (mm) zone II
1	15	85	2.64×10^{-3}	2.64×10^{-3}	1.41	28	28
2	30	310	4.62×10^{-4}	5.09×10^{-4}	1.39	9.97	10.99
3	60	760	5.91×10^{-5}	9.19×10^{-5}	1.37	2.59	4.02

[consider ~ 2-3B below foundation]
– see elastic stress distribution in databook.

Sum for settlement at A

Sum difference for settlement at B

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2) a) continued.

Settlement @ A = 41 mm
Settlement @ B = 2.5mm

Note: different layer thicknesses will lead to different answers: method is sensitive to chosen thicknesses. due to non-linear $\Delta\sigma_v'$ and v_0 with depth.
[In practice, spreadsheet is used, so number of layers >> 3.]

b) Settlement at B may be lower due to:

- higher initial soil stiffness close to existing building since $\sigma_{v0}' > \gamma' z$ due to consolidation under load due to existing building
- volume reduction during consolidation may be counteracted by dilation during shear deformation. Hence, a correction factor (e.g. Skempton - Bjerrum) may be necessary. Typical values for OC clay ~ 0.5-0.7.
- other building may be founded on piles, so less affected by movements close to ground surface.

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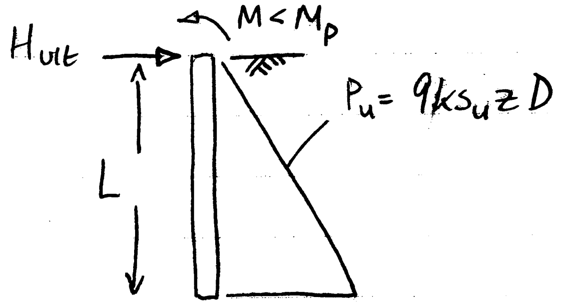
databook, can be quoted

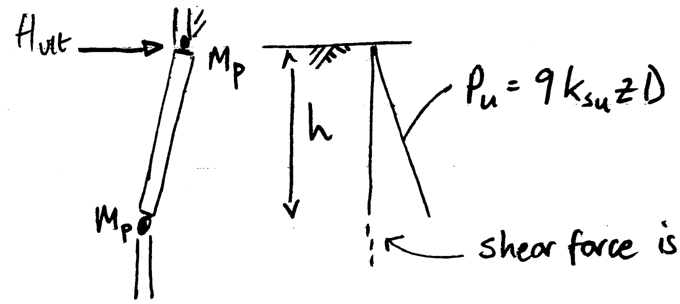
3) a) Horizontal resistance / unit pile length = $9 s_u D = 9 k_{su} z D$

in n.c. clay



Horizontal equilibrium: $H_{ult} = \bar{P}_u L = \frac{1}{2} 9 k_{su} L D . L = 5400 kN$

b)



First consider 2-hinge mechanism

$\frac{dM}{dx} = S$

shear force is zero where moment is a maximum.

Horizontal equilibrium: $H_{ult} = \bar{P}_u h = \frac{1}{2} 9 k_{su} h D . h$ — ①

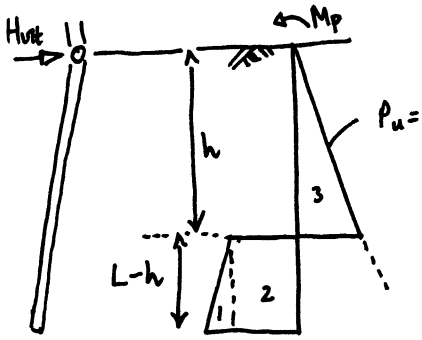
Moment eq^m about head: $2M_p = \frac{1}{2} 9 k_{su} h D . h . \underbrace{\frac{2}{3} h}_{\text{lever arm}}$ — ②

from ②, $h^3 = \frac{6 M_p}{9 k_{su} D} \Rightarrow h^2 = \left(\frac{2 M_p}{3 k_{su} D}\right)^{2/3}$

Sub into ① $H_{ult} = \frac{9}{2} k_{su} D \left(\frac{2}{3} \frac{M_p}{k_{su} D}\right)^{2/3}$

$H_{ult} = 3.43 (k_{su} D)^{1/3} M_p^{2/3} = 4.95 M_p^{2/3}$ (kN)

3 b) continued



Consider 1-hinge mechanism

Horizontal equilibrium:

$$H_{ult} = \overbrace{\underbrace{\tfrac{1}{2}(9k_{su}hD)}_{\bar{P}_u}\ \underbrace{h}_{\text{length}}}^{\text{region 3}} - \overbrace{\underbrace{\tfrac{1}{2}(9k_{su}(h+L)D)}_{\bar{P}_u}\ \underbrace{(L-h)}_{\text{length}}}^{\text{regions 1+2}}$$

$$H_{ult} = \tfrac{1}{2}9k_{su}D(2h^2-L^2) \qquad = 27h^2 - 5400 \text{ kN.}$$

Moment equilibrium:

$$M_p = \overbrace{\underbrace{\tfrac{1}{2}(9k_{su}hD)}_{\bar{P}_u}\ \underbrace{h}_{\text{length}}\ \underbrace{\tfrac{2}{3}h}_{\text{lever}}}^{\text{region 3}} - \overbrace{9k_{su}Dh\,(L-h)\left(\tfrac{h}{2}+\tfrac{L}{2}\right)}^{\text{region 2}}$$

$$- \overbrace{9k_{su}D\frac{L-h}{2}\,(L-h)\left(\tfrac{2L}{3}+\tfrac{h}{3}\right)}^{\text{region 1}}$$

simplifies to:

$$M_p = \tfrac{1}{2}9k_{su}D\left(\tfrac{4}{3}h^3 - \tfrac{2}{3}L^3\right) \qquad = 18h^3 - 72000 \text{ kN}$$

Combining equations to link H_{ult} and M_p: $H_{ult} = 27\left(\frac{M_p + 72000}{18}\right)^{2/3} - 5400$

Hence, capacity is minimum of offered by 3 mechanisms:

$$H_{ult} = \min\Big(\underbrace{5400}_{\text{no hinge}},\ \underbrace{4.95M_p^{2/3}}_{\text{1 hinge}},\ \underbrace{27\left(\frac{M_p+72000}{18}\right)^{2/3} - 5400}_{\text{2 hinges.}}\Big)$$

c) To prevent bending failure mechanisms, M_p must be sufficiently large that no-hinge mechanism is critical. To find this value, equate capacities of mechanisms to find required M_p.

no hinge vs. 1-hinge : $5400 \leq 27\left(\frac{M_p+72000}{18}\right)^{2/3} - 5400 \Rightarrow M_p \geq 72\text{ MNm}$ critical mechanism

no hinge vs. 2-hinge : $5400 \leq 4.95\,M_p^{2/3} \Rightarrow M_p \geq 36\text{ MNm}$

Plastic section modulus, $Z_p = D^2t$

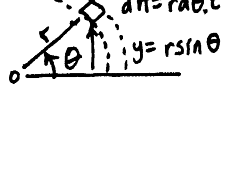
$$\Rightarrow \quad t = \frac{M_p}{D^2\sigma_y} = \frac{72}{2^2 \cdot 200} = \underline{\underline{90\text{mm}}}$$

bookwork, integration of elements:

$dZ_p = y\,dA = r\sin\theta\,(r\,d\theta)t = r^2t\sin\theta\,d\theta$

Integrate $0 \to \pi/2$ then $\times 4$ quadrants

$Z_p = 4\,[-\cos\theta]_0^{\pi/2}\,r^2t.$



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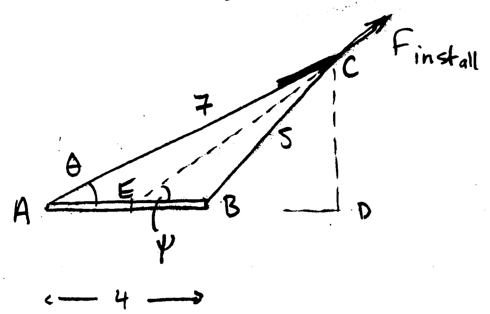
4) a) <u>Installation force:</u>

- Calculate angle of installation line to plate, ψ



Cosine rule

$$\cos\theta = \frac{5^2 - 7^2 - 4^2}{2\times7\times4}$$

$$\Rightarrow \theta = 44.4^\circ$$

$CD = 7\sin 44.4 = 4.90$
$BD = \sqrt{5^2 - 4.9^2} = 1$

$$\Rightarrow \psi = \tan^{-1}\frac{4.90}{3} = 58.5^\circ$$

- Calculate resistance in direction parallel to plate:

$$F_{parallel} = \underbrace{2\alpha S_u (A_{plan} + A_{edge})}_{\text{'shaft friction' on top, bottom and 2 edges}} + \underbrace{2N_c S_u A_{edge}}_{\text{+ve and -ve 'bearing capacity' on front and back}}$$

$S_u/\sigma'_{v0} = 50/5\times20 = 0.5 \Rightarrow \alpha = 1/\sqrt{2}$ (API / Randolph / Murphy)

$N_c =$ say 7 : deeply embedded strip
Skempton depth factor → 1.33
$1.33\times(2+\pi) \sim 7$

databook.

$$F_{parallel} = 2\frac{1}{\sqrt{2}}50\,(4^2 + 4\times0.05) + 2.7.50\,(4\times0.05)$$

$$= 1145 + 140 = 1285\text{ kN}$$

For equilibrium, $F_{install} = \dfrac{F_{parallel}}{\cos\psi} = 1799\text{ kN}.$

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4) <u>Mooring capacity</u>

flow-round mechanism: $s_c N_c = 2(2+\pi)\,1.18$ (2 square surface footings back to back)

$F_{mooring} = s_c N_c s_u A_{plan}$

(ignore shear on thickness)

$F_{mooring} = 9707\ kN.$

Hence, anchor efficiency $= \dfrac{\text{mooring}}{\text{installation}} = \dfrac{9707}{1799} = 5.4$

b) Crack – free water at anchor
- infinitesimal drainage distance to edge
- 'zero' nominal consolidation time
- hence, tension via $-ve\ \Delta u$ unsustainable
- so, no 'negative' bearing capacity.

$F_{install} = 1701\ kN$ ↓5% (ignoring $N_c s_u$ on rear edge.)

$F_{mooring} = 4853.5\ kN$ ↓50% (ignoring tension on back face)

efficiency = 2.85.

Also, crack could lead to softening of soil around anchor in long term.

Note: vertical shear mechanism in part a) is not optimal.

$F = A s_u = (4 \times 4 \times 20)\,50 = 16000\ kN$