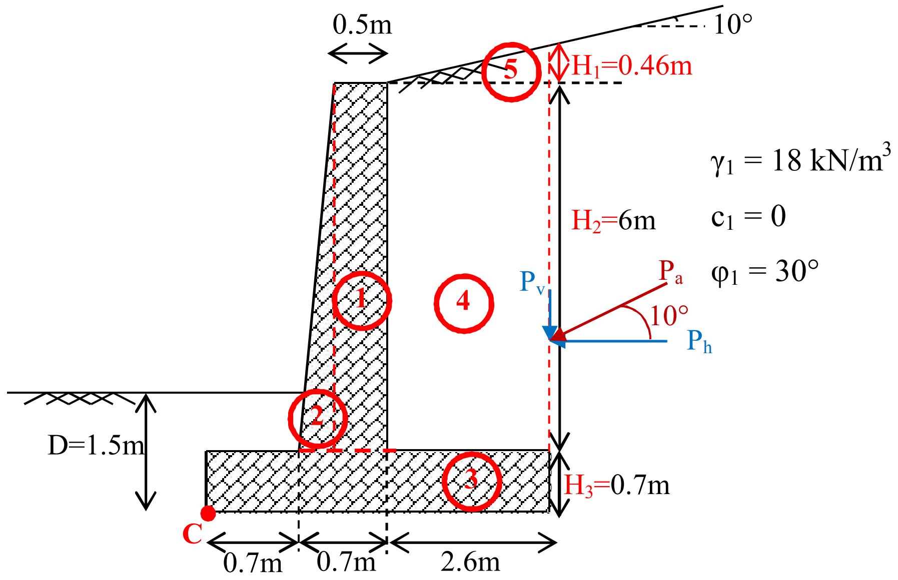
CE 366 FOUNDATION ENGINEERING I

2011 – 2012 SPRING SEMESTER

SOLUTIONS OF HOMEWORK 3

Solution 1:

Step 1: Draw freebody diagram and find forces



$\gamma_2 = 19$ kN/m^3

$c_2 = 40$ kPa

$\varphi_2 = 20°$

$$H' = H_1 + H_2 + H_3 = 2.6 \tan 10° + 6 + 0.7 = 0.46 + 6 + 0.7 = 7.16 \text{ m}$$

The Rankine active force per unit length of wall = $P_a = 0.5\gamma_1 H'^2 K_a$. For $\varphi_1 = 30°$, $\beta = 10°$, K_a is equal to 0.35 (from the given equation). Thus,

$$P_a = 0.5 \times 18 \times 7.16^2 \times 0.35 = 161.5 \text{ kN/m}$$

$$P_v = P_a \sin 10° = 161.5 \times \sin 10° = 28.0 \text{ kN/m}$$

$$P_h = P_a \cos 10° = 161.5 \times \cos 10° = 159.0 \text{ kN/m}$$

Factor of Safety Against Overturning

The following table can now be prepared for determination of the resisting moment:

Section No	Area (m^2)	Weight/unit length (kN/m)	Moment arm from point C (m)	Moment (kN.m/m)
1	6 x 0.5 = 3	3 x 24 = 72	1.15	82.8
2	0.5 x 0.2 x 6 = 0.6	0.6 x 24 = 14.4	0.83	12.0
3	4 x 0.7 = 2.8	2.8 x 24 = 67. 2	2.00	134.4
4	6 x 2.6 = 15.6	15.6 x 18 = 280.8	2.70	758.2
5	0.5 x 2.6 x 0.46 = 0.58	0.58 x 18 = 10.4	3.13	32.7
		P_v = 28.0	4.00	112.0
		ΣV = 472.8		ΣM_R = 1132.1

The overturning moment, M_o;

$$M_o = P_h (H'/3) = 159.0 \times (7.16/3) = 379.5 \text{ kN.m}$$

$$FS_{(overturning)} = \Sigma M_R / M_o = 1132.1 / 379.5 = \mathbf{2.98 > 2.0 \rightarrow OK}$$

Factor of Safety Against Sliding

$$FS_{(sliding)} = \frac{(\sum V)\tan\left(\frac{2}{3}\varphi_2\right) + B\frac{2}{3}c_2 + P_p}{P_a \cos\alpha}$$

Also,

$$P_p = 0.5\gamma_2 D^2 K_p + 2c_2\sqrt{(K_p D)}$$

$$K_p = \tan^2 [45+(\varphi_2/2)] = \tan^2 [45+(20/2)] = 2.04$$

$$D = 1.5 \text{ m}$$

So,

$$P_p = 0.5(19)(1.5)^2(2.04) + 2(40)\sqrt{(2.04)} \times (1.5) = 43.61 + 171.39 = 215 \text{ kN/m}$$

Hence,

$$FS_{(sliding)} = \frac{(472.8)\tan\left(\frac{2\times 20}{3}\right) + (4)\left(\frac{2}{3}\right)(40) + 215}{159} = \frac{111.5 + 106.67 + 215}{159} = \mathbf{2.73} > \mathbf{1.5} \rightarrow \boldsymbol{OK}$$

Factor of Safety Against Bearing Capacity Failure

$$e = \frac{B}{2} - \frac{\sum M_R - \sum M_o}{\sum V} = \frac{4}{2} - \frac{1132.1 - 379.5}{472.8} = 0.408 < \frac{B}{6} = \frac{4}{6} = 0.666\ m$$

$$q_{\substack{toe \\ heel}} = \frac{\sum V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{472.8}{4}\left(1 \pm \frac{6 \times 0.408}{4}\right) = \begin{matrix} 190.5\ kN/m^2 (toe) \\ 45.9\ kN/m^2 (heel) \end{matrix}$$

The ultimate bearing capacity of the soil can be determined from equation:

$$q_u = c_2 N_c d_c i_c + \gamma D N_q d_q i_q + \frac{1}{2}\gamma_2 B N_\gamma d_\gamma i_\gamma$$

For $\varphi_2 = 20° \rightarrow N_c = 14.8$, $N_q = 6.4$ and $N_\gamma = 3.6$.

Also,

$$q = \gamma_2 D = 19 \times 1.5 = 28.5\ kN/m^2$$

$$d_c = 1 + 0.4\left(\frac{D}{B}\right) = 1 + 0.4\left(\frac{1.5}{4}\right) = 1.15$$

$$d_q = 1 + 2\tan\varphi_2 (1 - \sin\varphi_2)^2 \left(\frac{D}{B}\right) = 1.118$$

$$d_\gamma = 1$$

$$i_c = i_q = \left(1 - \frac{\psi}{90}\right)^2 = \left(1 - \frac{18.6°}{90°}\right)^2 = 0.63$$

$$i_\gamma = \left(1 - \frac{\psi}{\varphi}\right)^2 = \left(1 - \frac{18.6°}{20°}\right)^2 = 0.14$$

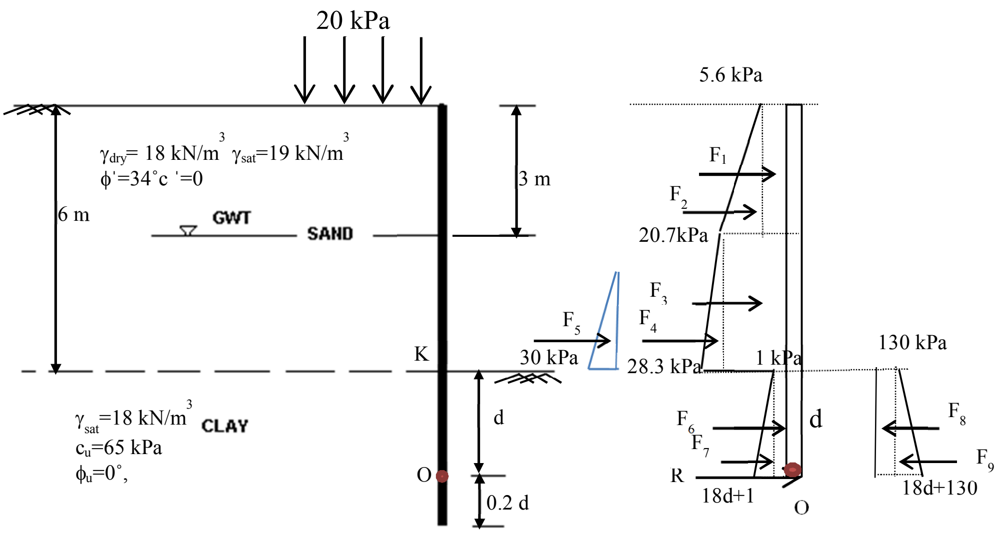
Hence,

$$q_u = (40)(14.8)(0.63)(1.15) + (28.5)(6.4)(1.118)(0.63) + \frac{1}{2}(19)(4)(3.6)(1)(0.14)$$
$$= 428.9 + 128.5 + 19.2 = 576.6\ kN/m^2$$

$$FS_{(bearing\ capacity)} = \frac{q_u}{q_{toe}} = \frac{576.6}{190.5} = \mathbf{3.03 \approx 3 \rightarrow \mathit{OK}}$$

Solution 2:

a) draw horizontal stress distribution along the sheet pile height;



Active Pressure;

For sand layer; $K_A = \frac{1-sin\phi}{1+sin\phi} = \frac{1-sin34}{1+sin34} = 0.28$

$$P_A'(@z = 0m) = (0 + 20)x0.28 = 5.6\ kPa$$

$$P_A'(@z = 3m) = (18x3 + 20)x0.288 = 20.7\ kPa$$

$$P_A(@z = 6m) = (18x3 + (19 - 10)x3 + 20)x0.28 = 28.3\ kPa$$

For clay layer; $K_A = \frac{1-sin\phi}{1+sin\phi} = 1$

$$P_A(@z = 3m) = (18x3 + 19x3 + 20)x1 - 2x65 = 1kPa \quad \text{clay}$$

$$P_A(@z = 6 + d) = (18x3 + 19x3 + 20 + 18xd)x1 - 2x65 = 1 + 18d$$

Passive Pressure;

For clay layer; $K_P = \frac{1+sin\phi}{1-sin\phi} = 1$

$$P_p'^{(@z=0m)} = 2x65 = 130\ kPa$$

$$P_p'^{(@z=dm)} = (18xd) + 2x65 = 18d + 130$$

b) *estimate the minimum depth of penetration of cantilever sheet pile wall;*

Force (kN/m)	Moment Arm about "point O"(m)	Moment (kN.m/m)
F_1 =5.6*3=16.8	4.5+d	75.6+16.8d
F_2 =(20.7-5.6)x3/2=22.7	4+d	90.8+22.7d
F_3 =20.7x3=62.1	1.5+d	93.2+62.1d
F_4 =(28.3-20.7)x3/2=11.4	1+d	11.4+11.4d
F_5 =30x3/2=45	1+d	45+45d
F_6 =1xd=d	d/2	$d^2/2$
F_7 =18dxd/2=$9d^2$	d/3	$3d^3$
$-F_8/1.5$=(130xd)/1.5=-86.7d	d/2	$-43.3d^2$
$-F_9/1.5$ =(18dxd/2)/1.5=-$6d^2$	d/3	$-2d^3$
$\Sigma F = 3d^2-85.7d+158=0$		$\Sigma M_o = d^3-42.8d^2+158d+316$

$\Sigma M_o=0 \rightarrow \Sigma M_o = d^3-42.8d^2+158d+316=0 \rightarrow d=5.7$ m

<u>depth of penetration = d+0.2d=6.84 m</u>

c) *estimate the maximum shear in the sheet pile.*

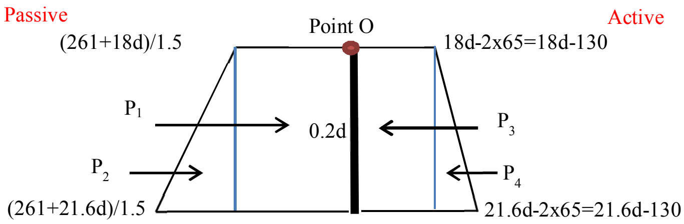
If M=0 then, $V=V_{max}$

$\Sigma F = 3d^2-85.7d+158+V_{max}=0 \rightarrow$ ***<u>V_{max}=233 kN/m</u>***

d) check if net passive resistance below point "O" is greater than "R"

$\Sigma F=0 \rightarrow \Sigma F = \Sigma F = 3d^2-85.7d+158+R=0$

$\Sigma F = \rightarrow 3x(5.7)^2-85.7x(5.7)+158+R=0 \rightarrow R=233$ kN/m $\longrightarrow$



For d=5.7 m $\rightarrow$ P_1= (261+18x5.7)/1.5=242.4 kN/m

P_2= 256-242=14 kN/m

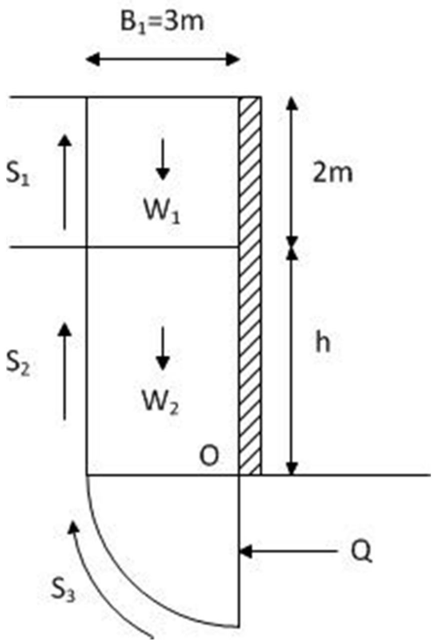
P_3=18x5.7-130=-27.4 kN/m $\rightarrow 0$

P_4=21.6x5.7-130=-6.9 kN/m $\rightarrow 0$

ΣP= 242.4+14-(-27.4)-(-6.9)=256.4 > R=233 kN/m **OK!**

Then passive resistance below 0.2d is ***enough.***

Solution 3:



$$F.S. = \frac{(S_1+S_2+S_3)xB_1 + Qx\frac{B_1}{2}}{(W_1+W_2)x\frac{B_1}{2}} = 1.0$$

$B_1 = 0.707xB = 0.707x4.25 = 3$ m. $K_a = \tan^2(45-30/2) = 0.33$

$S_1 = 0.5x\gamma xH^2xK_a x\tan\Phi = 0.5x20x2^2x0.33x\tan 30 = 7.6$ kN/m

$S_2 = 15h$ kN/m

$S_3 = \pi xB_1x15/2 = \pi x3x15/2 = 70.7$ kN/m

$Q = 30x3 = 90$ kN/m

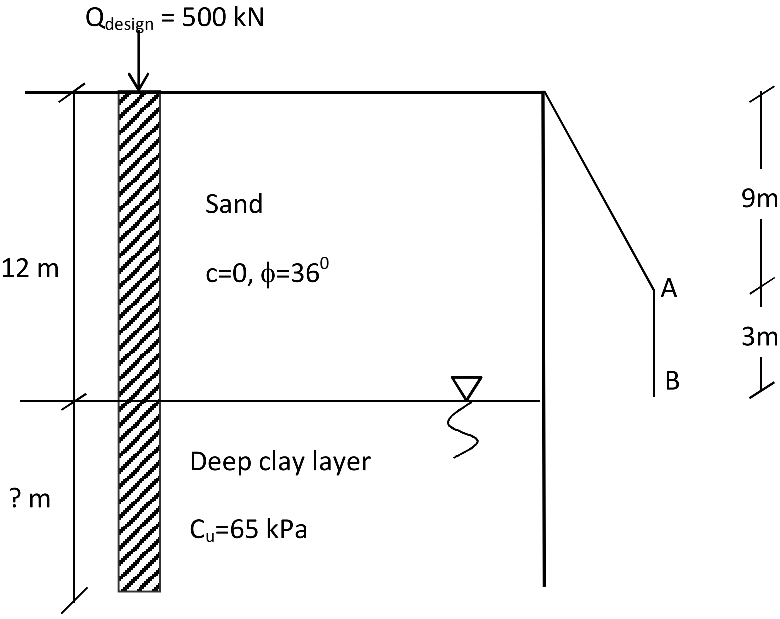
$W_1 + W_2 = (20x2 + 18xh)x3$ kN/m

$(7.6 + 15h + 70.7)x3 + 90x1.5 = (40 + 18h)x3x1.5$

$36h = 190$

$h = 5.28$ m; Thus; $H = h + 2 = 5.28 + 2 = 7.28$ m

Solution 4:



D_{cr} = 15*0.6 = 9m

$\sigma_A = \sigma_B = 9 * 17 = 153$ kPa

$$Q_{ult} = Q_p + Q_s \quad \text{where } Q_s = Q_{s,sand} + Q_{s,clay}$$

Sand:

$Q_{s,sand} = K_s\, \sigma_v'\, \tan(\delta)\, A_s$

where $K_s = 0.5$ for bored pile, $\delta = 3/4\Phi = 3/4\ 36 = 27°$

Above D_{cr} : $Q_{s,sand} = 0.5 * (0+153) / 2 \tan 27\ (\pi * 0.6 * 9) = 330$ kN

Below D_{cr} : $Q_{s,sand} = 0.5 * 153 \tan 27\ (\pi * 0.6 * 3) = 220$ kN

Clay

$Q_{s,clay} = \alpha\, c_u\, A_s$ where $\alpha = 0.8$; $Q_{s,clay} = 0.8 * 65 * (0.6 * \pi * L) = 98L$

$Q_p = N_c\, c_u\, A_p = 9 * 60 * (0.3^2 * \pi) = 153$ kN

$$Q_{design} = Q_{ult} / FS$$

$$500 = (330 + 220 + 153 + 98L) / 2$$

$$L = 5.6 \text{ m}$$

$$\text{Total pile length} = 12 + 5.6 = 17.6 \text{ m}$$

Solution 5:

$Q_{ult} = Q_s + Q_p$

$= \alpha\ c_u\ (A_{shaft}) + 9\ c_u\ A_p$

$= 1 * 30 * (\pi * 0.4 * 7) + 9 * 30 * \pi * 0.2^2$

$= 264 + 34$

$= 298$ kN/pile

$Q_{ult,group} = pD_f c_u + Aq_f - A\gamma d_f$

$= pD_f c_u + A(1.2cNc + \gamma d_f - \gamma d_f)$

$= pD_f c_u + A1.2cNc$

p= 2*[(3.0 + 0.4) + (5.0 + 0.4)] = 17.6 m and A= (3.4 x 5.4) = 18.36 m^2

D/B = 2 By using Skempton's chart $N_{c,square} = 8.4$

$N_{c,rectangle} = N_{c,square} * (0.84 + 0.16 * (B{=}3.4) / (L{=}5.4)) = 7.9$

$Q_{ult,group} = 17.6 * 7 * 30 + 18.36 * 1.2 * 30 * 7.9 = 8918$ kN

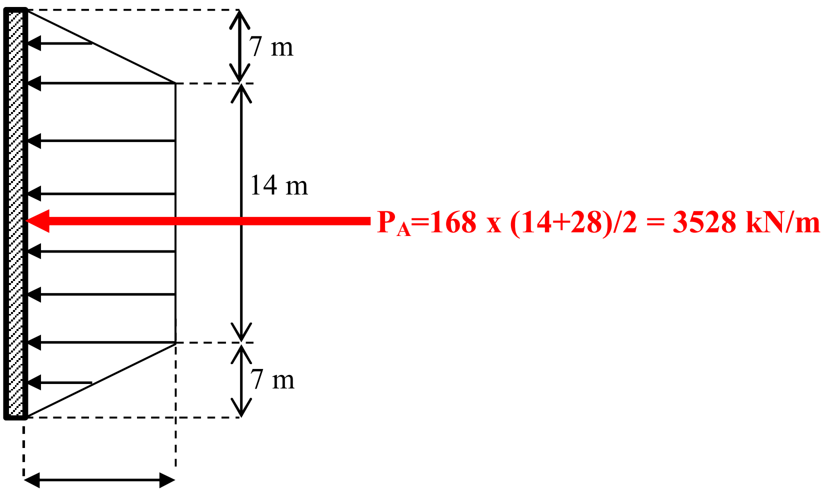
$Q_{ult,single} = Q_{ult,group} / 24 = 371$ kN / pile

Since 371 > 298 kN / pile, there is no need to consider group effect.

Solution 6:

- What is the final depth of excavation? **28** m
- Anchors are inclined at . . .**15 to 20** . . . . degree from horizontal.
- The fixed length (grouted length) of anchors are . . **8**. . . m.
- What type of soil exists at the site? Assuming that there is no surcharge at the ground surface, calculate the lateral earth pressure distribution and the active thrust, you expect at the back of this pile wall system for the final depth of excavation (assume any missing data).

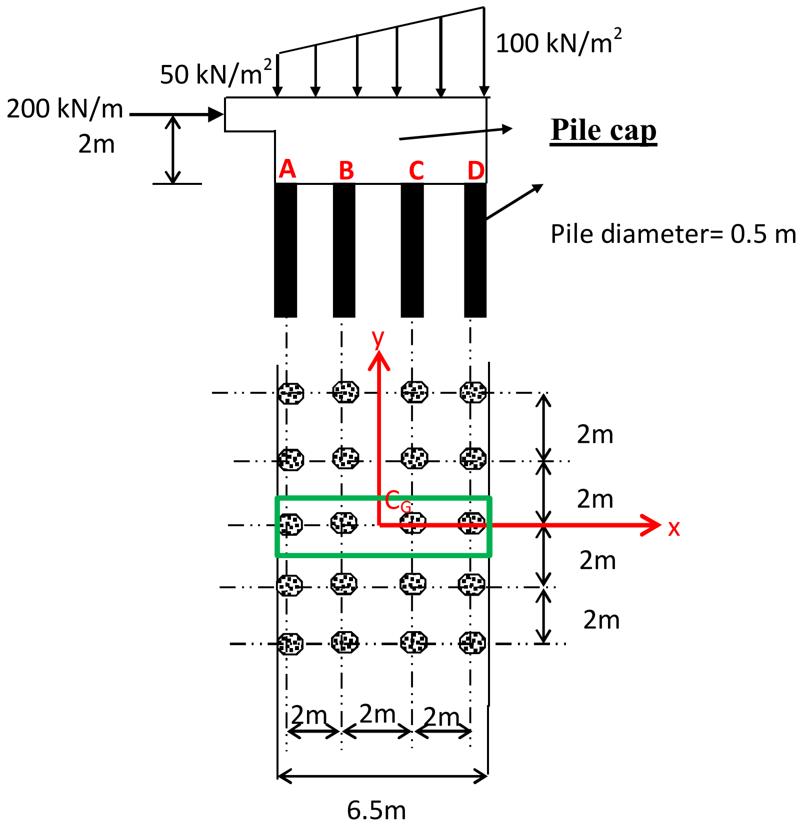
  Soil is stiff Ankara clay. Earth pressure distribution can be obtained from empirical earth pressure diagrams proposed for the multi-level anchored retaining walls (see page 135 of Lecture notes). Use 0.3γH, assume γ=20 kN/m³.



p_A=0.3γH=0.3x20x28 = 168 kPa

- Anchors are tensioned by a hydraulic jack to . . .**35** . . . tons.

Solution 7:



<u>If pile foundation is a strip foundation:</u>

For **2 m section** in y-y direction:

$$\sum F_V = \frac{(50+100)}{2}6.5 = 487.5\ kN/m$$

$$\sum M_{C_G} = \frac{(100-50)\times 6.5}{2}\left(\frac{6.5}{2}-\frac{6.5}{3}\right) + (200\times 2) = 576kNm/m$$

$$I_{y-y} = (3^2+1^2)\times 2 = 20\ pile - m^2$$

$$Q_A = \frac{487.5\times 2}{4} - \frac{(576\times 2)\times 3}{20} = 70.94\ kN$$

$$Q_B = \frac{487.5\times 2}{4} - \frac{(576\times 2)\times 1}{20} = 186.15\ kN$$

$$Q_C = \frac{487.5\times 2}{4} + \frac{(576\times 2)\times 1}{20} = 301.35\ kN$$

$$Q_D = \frac{487.5\times 2}{4} + \frac{(576\times 2)\times 3}{20} = 416.56\ kN$$

<u>If pile foundation is B_g = 6.5 m and L_g = 8.5 m:</u>

The piled foundation contains 20 piles within 6.5mx8.5m area.

$$\sum F_V = \frac{(50+100)}{2}x6.5x8.5 = 4143.75\ kN$$

$$\sum M_{C_G} = \frac{(100-50)x6.5x8.5}{2}\left(\frac{6.5}{2}-\frac{6.5}{3}\right) + (200x8.5x2) = 4896.35kNm$$

$$I_{y-y} = 5x2x(3^2+1^2) = 100\ pile - m^2$$

$$Q_A = \frac{4143.75}{20} - \frac{4896.35\times 3}{100} = 60.30\ kN$$

$$Q_B = \frac{4143.75}{20} - \frac{4896.35\times 1}{100} = 158.22\ kN$$

$$Q_C = \frac{4143.75}{20} + \frac{4896.35\times 1}{100} = 256.15\ kN$$

$$Q_D = \frac{4143.75}{20} + \frac{4896.35\times 3}{100} = 354.08\ kN$$