Geotechnical Engineering III

CE 4131

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Never give up hope of Allah’s Mercy (Quran: 12:87)

Special THANKS to-

My friend

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INDEX

CLICK ON THE TOPIC TO GO TO THAT PAGE

Sheet Pile

Cantilever Sheet Pile

Sheet Pile:

Sheet pile is a type of retaining wall that usually used to prevent movement of soil.

A sheet pile wall consists of sheet piles driven side by side into the ground thus forming a continuous vertical wall for the purpose of retaining an earth bank.

Types of sheet pile:

According to material used,

(i) Wooden sheet pile.

(ii) Pre cast concrete sheet pile.

(iii) Steel sheet pile.

(iv) Aluminium sheet pile.

(v) Vinyle sheet pile.

(i) Wooden sheet Pile:

* **Advantages:**

1. Low cost.
2. Easy to handle.
3. Easy to transport.

* **Disadvantages:**

1. Low resistance against driving force.
2. Easy to degrade.
3. Used only for temporary, light structures above water table.

(ii) Pre-cast Concrete sheet pile:

* Advantages:

1. cost effective.
2. corrosion free.
3. More durable than wooden sheet pile.

* Disadvantages:

1. Low resistance against driving.
2. Relatively heavy and bulky.

(iii) Steel sheet Pile:

* Advantages:

1. Light weight.
2. Easy to lift and handle.
3. Can be reused and recycled.
4. Long service life.
5. Easy to adapt the pile length.
6. Joints are less apt to deform during driving.
7. Convenient to use because of their resistance to the high driving stress

* Disadvantages:

1. May cause neighborhood disturbance.
2. Extremly difficult to install sheeting in rocky soil.

*** Why steel sheet pile is widely used? Explain.

Ans. (Advantages)

(iv) Aluminium Sheet Pile:

* Advantages:

1. Light in weight.
2. corrosion free.
3. Most effective strength to weight ratio.
4. Easy to handle.
5. Good Looking.

* Disadvantages:

1. special pile driving elements are needed.
2. Trained operation are needed.

(v) Vinyle sheet Pile:

* Advantages:

1. Lasts much longer than more traditional materials.
2. consistance appearence.
3. un affected by marine bores.

* Disadvantages:

1. Low strength.
2. Difficult to install in dense soils.
3. Leak in inter locking joints.

Disadvantages of sheet pile:

1. sections can be rarely used as part of the parmanent structure.
2. Difficult in installation in soils with boulders or cables.
3. Sheet pile driving may cause neighborhood disturbance.
4. settlements in adjacent properties due to intallation vibrations.

Use/Application of Sheet Pile:

1. sheet piles are often used to build continuous walls for large water front structure.
2. They are used to protect erosion of soil.
3. They are used to stabilize the ground slopes.
4. sheet pile walls are used to support excavations for below-grade parking structures, basements, pump-houses and foundations.
5. They are used to construct coffer dams.
6. They are also used to construct seawalls and bulk heads.

Depending on the way, the retaining structure is built and analyzed:

sheet pile walls may be divided into two basic categories:

1. Cantilever sheet Pile
2. Anchor sheet Pile

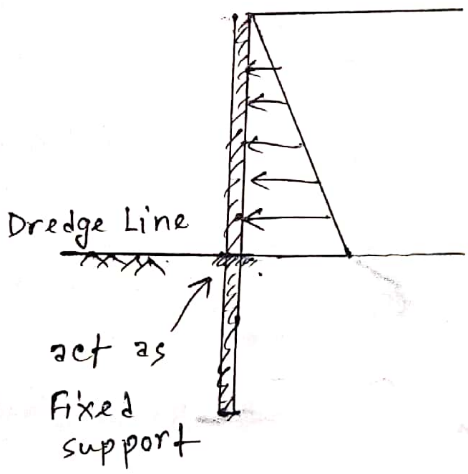
Dredge Line: The surface of soil on the water side is referred to as the mud line or dredge Line.

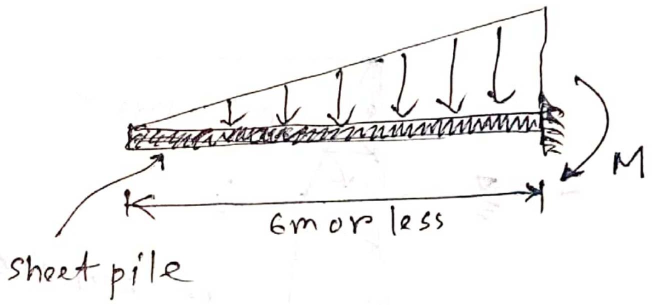
What is cantilever sheet pile?

Cantilever sheet piles are the piles which are usually recommended for walls of moderate height about 6m (≃ 20ft) or less, measured above the dredge line.

Why it is called cantilever?

In cantilever sheet pile walls, the sheet piles act as a wide cantilever beam above the dredge line.





As it acts as a cantilever beam, it is called cantilever sheet pile.

Cantilever sheet Pile penetrating sand / Granular soil:

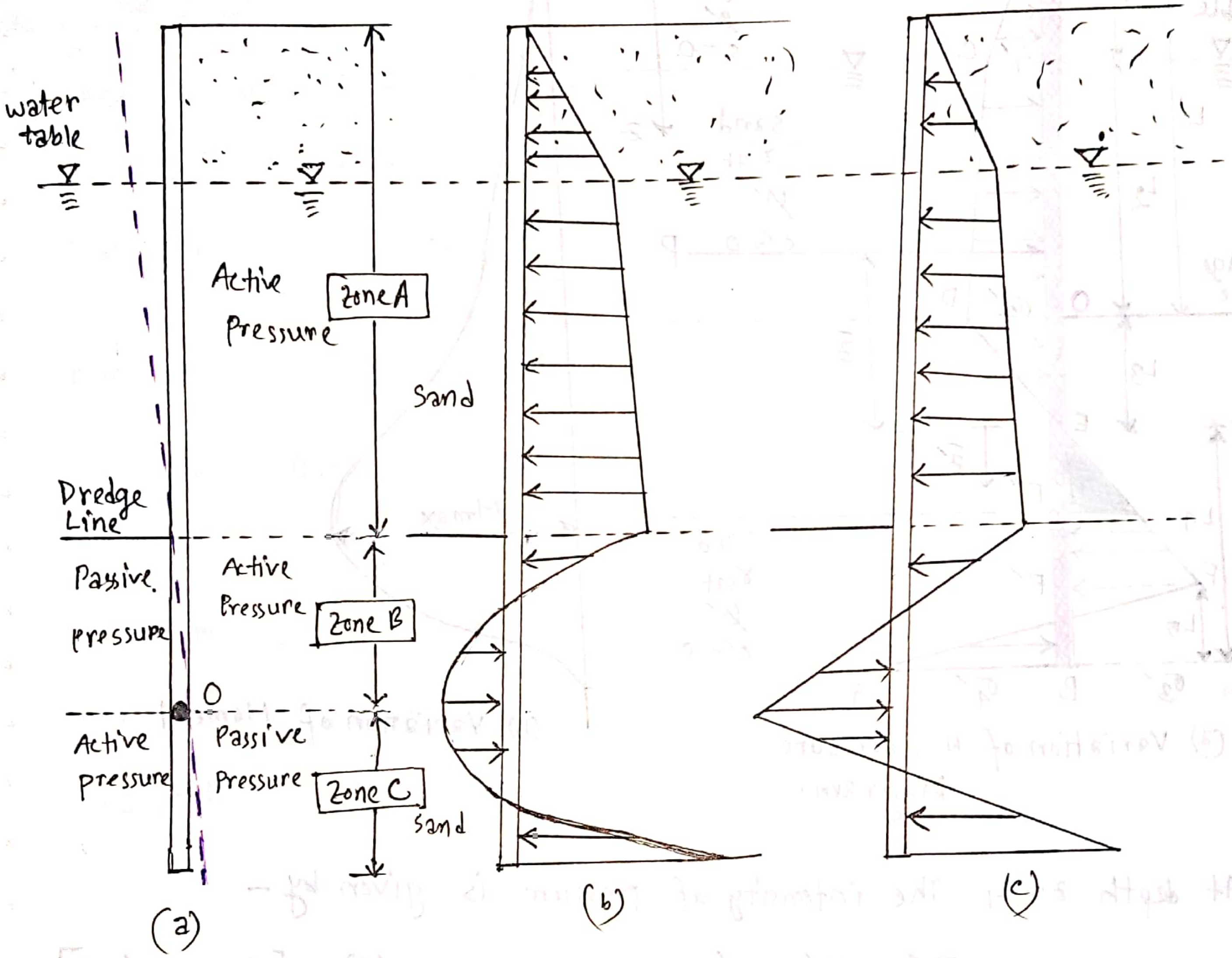
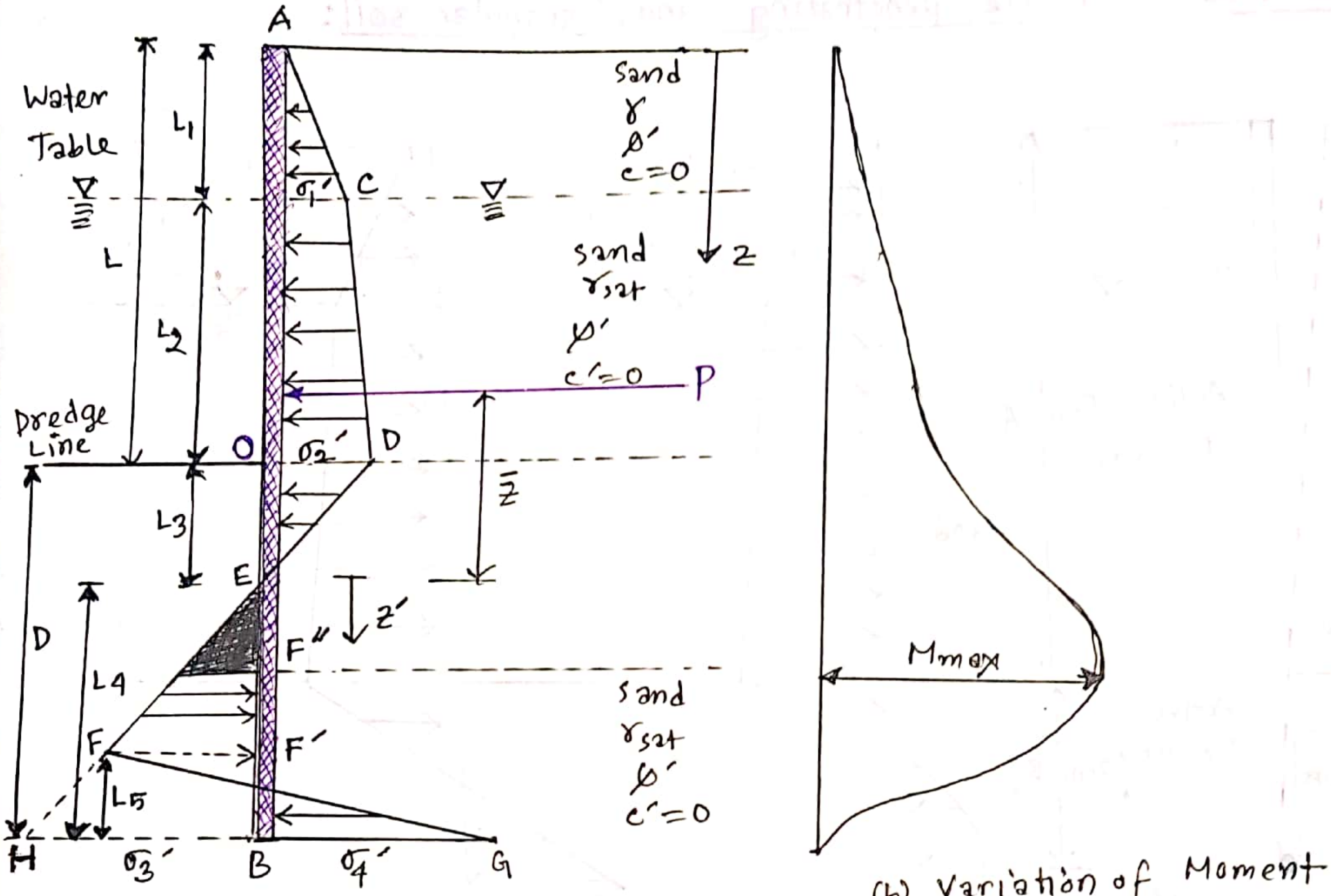


Fig. Cantilever pile penetrating Sand.

Formulation for depth of embedment: 2006

To develop the relationships for the proper depth of embedment of sheet piles driven into a granular soil. The soil retained by the sheet pile above the dredge line is also sand.



(a) Variation of Net pressure Diagram.

(b) Variation of Moment

At depth $z = L_1$ The intensity of pressure is given by -

$$\sigma_1' = \sigma_a' - \sigma_p' = K_a \gamma L_1 \quad \ldots\ldots (1) \quad [\text{Here, } \sigma_p' = 0]$$

where, $K_a = \tan^2(45 - \frac{\phi'}{2}) = \frac{1 - \sin\phi'}{1 + \sin\phi'}$

similarly, At depth $z = L_1 + L_2$

$$\sigma_2' = \sigma_1' + K_a \gamma' L_2 = K_a \gamma L_1 + K_a \gamma' L_2$$

$$\therefore \sigma_2' = K_a(\gamma L_1 + \gamma' L_2) \quad \ldots\ldots\ldots (2)$$

where, $\gamma' = \gamma_{sat} - \gamma_w$

At any depth z, The active pressure

$$\sigma_a' = [\gamma L_1 + \gamma' L_2 + \gamma'(z - L_1 - L_2)] K_a \quad \text{.......} (3)$$

Also, at any depth z, The passive pressure

$$\sigma_p' = \gamma'(z - L_1 - L_2) K_p \quad \text{........} (4)$$

where, $K_p = \tan^2(45 + \frac{\phi'}{2}) = \frac{1 + \sin\phi'}{1 - \sin\phi'}$

combining (3) & (4) yields the net pressure,

$$\sigma' = \sigma_a' - \sigma_p' = (\gamma L_1 + \gamma' L_2) K_a - \gamma'(z - L_1 - L_2)(K_p - K_a)$$

$$= \sigma_2' - \gamma'(z - L)(K_p - K_a) \quad \text{.......} (5)$$

where, $L = L_1 + L_2$

It is noted in Figure that, the net pressure is zero at a depth L_3 below the dredge line, so

$$\sigma' = \sigma_2' - \gamma'(z - L)(K_p - K_a) = 0$$

$$\Rightarrow (z - L) = \frac{\sigma_2'}{\gamma'(K_p - K_a)}$$

$$\Rightarrow L_3 = \frac{\sigma_2'}{\gamma'(K_p - K_a)} \quad \text{..........} (6)$$

Now from similar triangles,

ΔODE & ΔEHB, $\frac{\sigma_3'}{\sigma_2'} = \frac{L_4}{L_3}$

$$\Rightarrow \sigma_3' = \frac{L_4}{L_3} \times \sigma_2' = \frac{L_4}{\frac{\sigma_2'}{\gamma'(K_p - K_a)}} \times \sigma_2'$$

$$\therefore \sigma_3' = L_4 (K_p - K_a) \gamma' \quad \text{..............} (7)$$

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At the bottom of the sheet pile, passive pressure, σ_p' acts from the right toward the left side and active pressure, σ_2' acts from the left toward the right side of the sheet pile

So,

The intensity of passive pressure at depth $z = L + D$

$$\sigma_p' = (\gamma L_1 + \gamma' L_2 + \gamma' D) K_p \quad \ldots\ldots (8)$$

At the same depth, The active pressure,

$$\sigma_a' = \gamma' D K_a \quad \ldots\ldots (9)$$

Hence, the net lateral pressure at the bottom of the sheet pile is,

$$\sigma_4' = \sigma_p' - \sigma_a' = (\gamma L_1 + \gamma' L_2 + \gamma' D) K_p - \gamma' D K_a$$

$$= (\gamma L_1 + \gamma' L_2) K_p + \gamma' D (K_p - K_a)$$

$$= (\gamma L_1 + \gamma' L_2) K_p + \gamma' (L_3 + L_4)(K_p - K_a)$$

$$[\because D = L_3 + L_4]$$

$$= (\gamma L_1 + \gamma' L_2) K_p + \gamma' L_3 (K_p - K_a) + \gamma' L_4 (K_p - K_a)$$

$$\therefore \sigma_4' = \sigma_5' + \gamma' L_4 (K_p - K_a) \quad \ldots\ldots (10)$$

Where,

$$\sigma_5' = (\gamma L_1 + \gamma' L_2) K_p + \gamma' L_3 (K_p - K_a) \quad \ldots\ldots (11)$$

For the summation of horizontal forces, we have.

Area ACDE − Area EFHB + Area FHBG = 0

$$\Rightarrow P - \frac{1}{2}\sigma_3' L_4 + \frac{1}{2}(\sigma_3' + \sigma_4') \times L_5 = 0 \quad \ldots\ldots (12)$$

$$\Rightarrow L_5 = \frac{\sigma_3' L_4 - 2P}{\sigma_3' + \sigma_4'} \quad \ldots\ldots (13)$$

Now, summing the moment of all forces about point B yields

$$P\times(L_4+\bar{z}) - \left(\frac{1}{2}\sigma_3' L_4\right)\times\left(\frac{L_4}{3}\right) + \left[\frac{1}{2}(\sigma_3'+\sigma_4')\times L_5\right]\times\frac{L_5}{3} = 0 \quad \text{......(14)}$$

putting all values in Equation (14) we obtain,

$$L_4^4 + A_1 L_4^3 - A_2 L_4^2 - A_3 L_4 - A_4 = 0 \quad \text{..........(15)}$$

where,

$$A_1 = \frac{\sigma_5'}{\gamma'(K_p-K_a)} \quad \text{..........(16)}$$

$$A_2 = \frac{8P}{\gamma'(K_p-K_a)} \quad \text{..........(17)}$$

$$A_3 = \frac{6P\left[2\bar{z}\gamma'(K_p-K_a)+\sigma_5'\right]}{\gamma'^2(K_p-K_a)^2} \quad \text{..........(18)}$$

$$A_4 = \frac{P(6\bar{z}\sigma_5'+4P)}{\gamma'^2(K_p-K_a)^2} \quad \text{......(19)}$$

Calculation of Maximum Bending moment:

The maximum moment will occur between points E and F'.

Moment will be maximum where shear is zero.

Let us, consider the new axis z' (with origin at point E)

and Let the point of zero shear is at point F''

Now, $\Sigma F_x = 0$

Area ACDE − Black Shaded Area = 0

$$P - \frac{1}{2}\times z'\times(K_p-K_a)\times\gamma' z' = 0 \quad \text{..........(20)}$$

$$\Rightarrow z' = \sqrt{\frac{2P}{(K_p-K_a)\gamma'}} \quad \text{........(21)}$$

Maximum moment can be obtained by taking moment about point F"

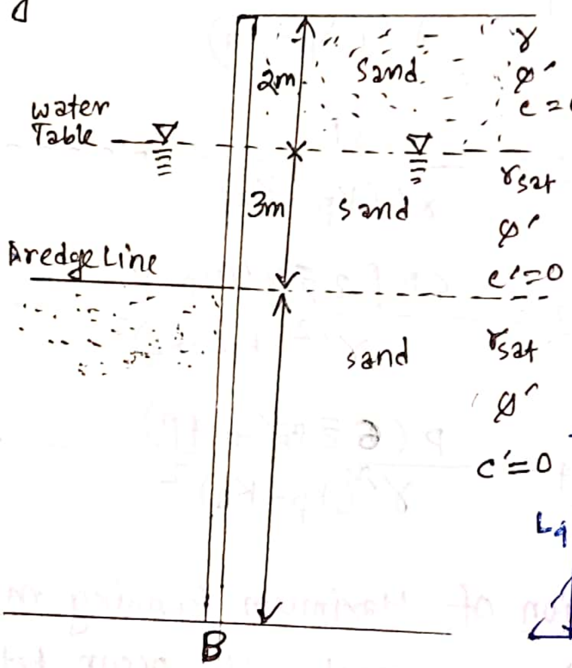
$$M_{max.} = P(\bar{z}+z') - \left[\tfrac{1}{2} \times \gamma' z'^2 \times (K_p - K_a)\right] \times \left(\tfrac{1}{3} \times z'\right) \quad \cdots\cdots (22)$$

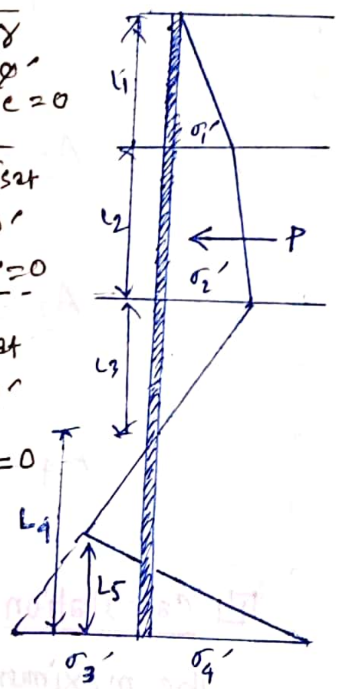
section modulus of sheet pile is calculated as follows:

$$S = \frac{M_{max.}}{\sigma_{all}} \quad \cdots\cdots (23)$$

Problem: 01

Cantilever sheet pile penetrating in to sand is shown in the adjacent figure.





Here, $\gamma = 16\ KN/m^3$

$\gamma_{sat} = 18\ KN/m^3$

$\phi' = 30°$

a. What is the theoretical depth of embedment D?

b. For 30% increase in D. What should be the total length of the sheet pile.

c. Determine the minimum sectional modulus of sheet piles if $\sigma_{all} = 170\ MN/m^2$

Solution:

(i) $K_a = \dfrac{1-\sin\phi}{1+\sin\phi} = \dfrac{1-\sin 30°}{1+\sin 30°} = \dfrac{1}{3}$

(ii) $K_p = \dfrac{1+\sin\phi}{1-\sin\phi} = 3$

(iii) $\sigma_1' = (\tfrac{1}{3} \times 16 \times 2) = 10.67\ KN/m^2$

(iv) $\sigma_2' = 10.67 + \tfrac{1}{3} \times (18 - 9.81) \times 3 = 18.86\ KN/m^2$

(v) $L_3 = \dfrac{\sigma_2'}{\gamma'(K_p - K_a)} = \dfrac{18.86}{(18-9.81)\times(3-\frac{1}{3})} = 0.864\ m$

(vi) $P = \frac{1}{2}\times 10.67\times 2 + \frac{1}{2}\times(18.86-10.67)\times 3 + 10.67\times 3 + \frac{1}{2}\times 0.864\times 18.86$

$= 63.11\ KN/m.$

(vii) $\bar{z} = \dfrac{\Sigma M_E}{P}$

$= \dfrac{\frac{1}{2}\times 10.67\times 2\times(3.864+\frac{2}{3}) + \frac{1}{2}\times(18.86-10.67)\times 3\times(0.864+\frac{3}{3}) + 10.67\times 3\times(0.864+\frac{3}{2}) + \frac{1}{2}\times 0.864\times 18.86\times \frac{0.864}{3}\times 2}{63.11}$

$= \dfrac{151.61}{63.11}$

$= 2.40\ m$

(viii) $\sigma_5' = (\gamma L_1 + \gamma' L_2)K_p + \gamma' L_3 (K_p - K_a)$

$= (16\times 2 + [18-9.81]\times 3)\times 3 + (18-9.81)\times 0.864\times(3-\frac{1}{3})$

$= 188.58\ KN/m^2$

(ix) $A_1 = \dfrac{\sigma_5'}{\gamma'(K_p-K_a)} = \dfrac{188.58}{(18-9.81)\times(3-\frac{1}{3})} = 8.63$

$A_2 = \dfrac{8P}{\gamma'(K_p-K_a)} = \dfrac{8\times 63.11}{(18-9.81)\times(3-\frac{1}{3})} = 23.11$

$A_3 = \dfrac{6P[2\bar{z}\gamma'(K_p-K_a)+\sigma_5']}{\gamma'^2(K_p-K_a)^2} = \dfrac{6\times 63.11\times[2\times 2.40^2\times(3-\frac{1}{3}) + 188.58]}{(18-9.81)^2\times(3-\frac{1}{3})^2}$

$\therefore A_3 = 174.094$

$A_4 = \dfrac{P(6\bar{z}\sigma_5' + 4P)}{\gamma'^2(K_p-K_a)^2} = \dfrac{63.11\times(6\times 2.4\times 188.58 + 4\times 63.11)}{(18-9.81)^2\times(3-\frac{1}{3})^2} = 392.695$

(x) $L_4^4 + A_1 L_4^3 - A_2 L_4^2 - A_3 L_4 - A_4 = 0$

$$\Rightarrow L_4^4 + 8.63\, L_4^3 - 23.12\, L_4^2 - 174.094\, L_4 - 392.695 = 0$$

$\therefore L_4 = 5.174$ m

(a) Thus, $D_{theory} = L_3 + L_4 = (0.864 + 5.174) = 6.038$ m

(b) For 30% Increase in D,

The total length of the sheet Pile $= L_1 + L_2 + 1.3\, D_{theory}$

$= (2 + 3 + 1.3 \times 6.038)$

$= 12.85$ m

(c)

$$z' = \sqrt{\frac{2P}{(K_p - K_a)\gamma'}} = \sqrt{\frac{2 \times 63.11}{(3 - \frac{1}{3}) \times (18 - 9.81)}} = 2.4 \text{ m.}$$

$$M_{max} = P(\bar{z} + z') - \left[\frac{1}{2} \times \gamma' z'^2 (K_p - K_a)\right] \times \left(\frac{1}{3} \times z'\right)$$

$$= 63.11 \times (2.4 + 2.4) - \left[\frac{1}{2} \times (18 - 9.81)^2 \times (2.40)^2 \times \left(3 - \frac{1}{3}\right)\right] \times \frac{2.4}{3}$$

$= 252.61$ KN·m/m

The sectional modulus of the sheet pile

$$S = \frac{M_{max}}{\sigma_{all}} = \frac{252.61 \text{ KN·m}}{170 \times 10^3 \text{ KN/m}^2} = 1.486 \times 10^{-3}$$

m^3/m of wall

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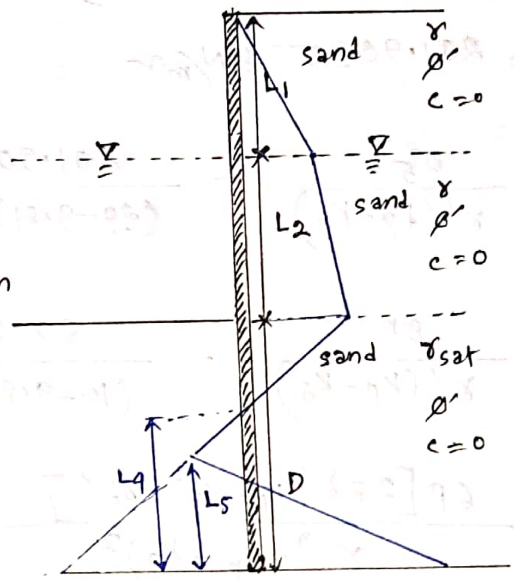
Problem: 02

A cantilever sheet pile penetrating into sand in shown in the adjacent Figure:

Here, $L_1 = 2m$ $\quad \gamma = 16.5\ KN/m^3$

$L_2 = 3m$ $\quad \gamma_{sat} = 19\ KN/m^3$

$\phi' = 30°$



(a) Draw quantative earth pressure diagram

(b) Find L_3, L_4, L_5, P and D

(c) Calculate D_{act}

(d) Find $\bar{z}'$, position of M_{max} from bottom point B, M_{max} and S if $\sigma_{all} = 165\ MN/m^2$

Solution: $K_a = \dfrac{1-\sin 30°}{1+\sin 30°} = \dfrac{1}{3}$; $K_p = \dfrac{1}{K_a} = 3$

$\sigma_1' = \gamma L_1 K_a = (16.5 \times 2 \times \frac{1}{3}) = 11\ KN/m^2$

$\sigma_2' = \sigma_1' + \gamma' L_2 K_a = 11 + (19-9.81) \times 3 \times \frac{1}{3} = 20.19\ KN/m^2$

$L_3 = \dfrac{\sigma_2'}{\gamma'(K_p - K_a)} = \dfrac{20.19}{(19-9.81)\times(3-\frac{1}{3})} = 0.824\ m.$

$P = \frac{1}{2}\sigma_1' L_1 + \sigma_1' L_2 + \frac{1}{2}(\sigma_2' - \sigma_1')L_2 + \frac{1}{2}\sigma_2' L_3$

$= \frac{1}{2} \times 11 \times 2 + 11 \times 3 + \frac{1}{2} \times (20.19 - 11) \times 2 + \frac{1}{2} \times 20.19 \times 0.824$

$= 61.51\ KN/m$

$\bar{z} = \dfrac{\Sigma M_E}{P} = \dfrac{1}{61.51} \times \left[\frac{1}{2} \times 11 \times 2 \times (3.824 + \frac{2}{3}) + 11 \times 3 \times (0.824 + \frac{3}{2}) + \frac{1}{2} \times (20.19 - 11) \times 2 \times (0.824 + \frac{3}{3}) + \frac{1}{2} \times 20.19 \times 0.824 \times (\frac{2}{3} \times 0.824)\right]$

$\therefore \bar{z} = \dfrac{147.42}{61.51} = 2.397\ m.$

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$$\sigma_5' = (\gamma L_1 + \gamma' L_2)K_p + \gamma' L_3 (K_p - K_a)$$

$$= [16.5\times 2 + (19-9.81)\times 3]\times 3 + (19-9.81)\times 0.824\times(3-\tfrac{1}{3})$$

$$= 201.9035 \text{ KN/m}^2$$

$$A_1 = \frac{\sigma_5'}{\gamma'(K_p - K_a)} = \frac{201.9035}{(19-9.81)\times(3-\frac{1}{3})} = 8.24$$

$$A_2 = \frac{8P}{\gamma'(K_p-K_a)} = \frac{8\times 61.51}{(19-9.81)\times(3-\frac{1}{3})} = 20.08$$

$$A_3 = \frac{6P[2\bar{z}\gamma'(K_p-K_a)+\sigma_5']}{\gamma'^2(K_p-K_a)^2} = \frac{6\times 61.51\times[2\times 2.397\times(19-9.81)+201.9035]}{(19-9.81)^2\times(3-\frac{1}{3})^2}$$

$$\Rightarrow A_3 = 151.145$$

$$A_4 = \frac{P(6\bar{z}\sigma_5'+4P)}{\gamma'^2(K_p-K_a)^2} = \frac{61.51\times(6\times 2.397\times 201.9035+4\times 61.51)}{(19-9.81)^2\times(3-\frac{1}{3})^2}$$

$$\Rightarrow A_4 = 322.6$$

Now,
$$L_4^4 + A_1 L_4^3 - A_2 L_4^2 - A_3 L_4 - A_4 = 0$$

$$\Rightarrow L_4^4 + 8.24 L_4^3 - 20.08 L_4^2 - 151.145 L_4 - 322.6 = 0$$

$$\therefore L_4 = 4.899 \text{ m} \simeq 4.9 \text{ m}$$

$$\therefore D_{theory} = (L_3 + L_4) = (0.824 + 4.9) = 5.724 \text{ m}$$

Now, $\sigma_3' = \gamma'(K_p - K_a)L_4 = (19-9.81)\times(3-\frac{1}{3})\times 4.9 = 120.08$ KN/m^2

$$\sigma_4' = \sigma_5' + \gamma'(K_p-K_a)L_4 = (201.9035 + 120.08) = 321.98 \text{ KN/m}^2$$

$$L_5 = \frac{\sigma_3' L_4 - 2P}{\sigma_3' + \sigma_4'} = \frac{120.08\times 4.9 - 2\times 61.51}{120.08+} = 1.05 \text{ m}$$

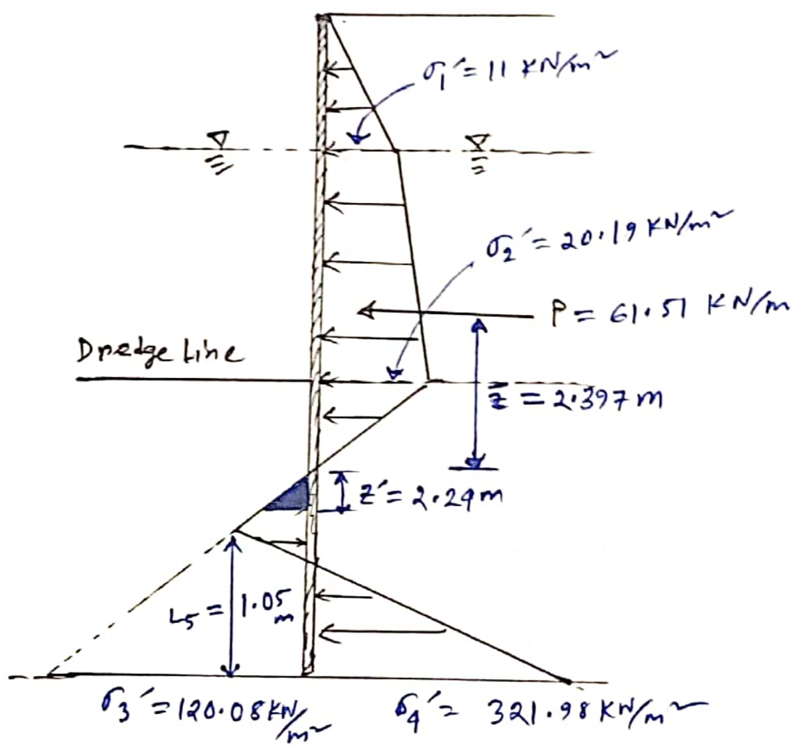
$$z' = \sqrt{\frac{2P}{\gamma'(K_p - K_a)}} = \sqrt{\frac{2 \times 61.51}{(19-9.81)\times(3-\frac{1}{3})}} = 2.24$$

$$M_{max} = P(\bar{z} + z') - \left[\frac{1}{2} \times \gamma' z'^2 \times (K_p - K_a)\right] \times \frac{z'}{3}$$

$$= 61.51 \times (2.397 + 2.24) - \left[\frac{1}{2} \times (19-9.81) \times 2.24^2 \times (3 - \frac{1}{3})\right] \times \frac{2.24}{3}$$

$$= 239.315 \text{ KN.m/m}$$

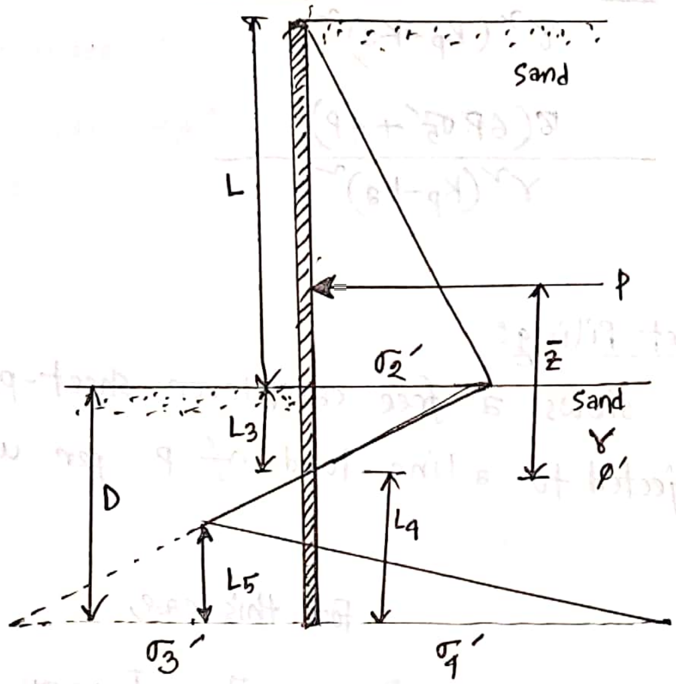
$$S = \frac{M_{max}}{\sigma_{all}} = \frac{239.315}{165 \times 10^3} = 1.45 \times 10^{-3} \text{ m}^3\text{/m of wall}$$



Special cases for cantilever sheet pile penetrating a sandy soil:

* Sheet-pile wall with the absence of water table:

In the absence of the water table, The net pressure diagram on the cantilever wall will be as shown in Figure:



In this case,

$$\sigma_2' = \gamma L K_a$$

$$\sigma_3' = L_4 (K_p - K_a)\gamma$$

$$\sigma_4' = \sigma_5' + \gamma L_4 (K_p - K_a)$$

$$\sigma_5' = \gamma L K_p + \gamma L_3 (K_p - K_a)$$

$$L_3 = \frac{\sigma_2'}{\gamma (K_p - K_a)} = \frac{L K_a}{(K_p - K_a)}$$

$$P = \frac{1}{2}\sigma_2' L + \frac{1}{2}\sigma_2' L_3$$

$$\bar{z} = L_3 + \frac{L}{3} = \frac{L K_a}{(K_p - K_a)} + \frac{L}{3} = \frac{L(2K_a + K_p)}{3(K_p - K_a)}$$

And, $L_4^4 + A_1' L_4^3 - A_2' L_4^2 - A_3' L_4 - A_4' = 0$

where, $A_1' = \dfrac{\sigma_5'}{\gamma (K_p - K_a)}$

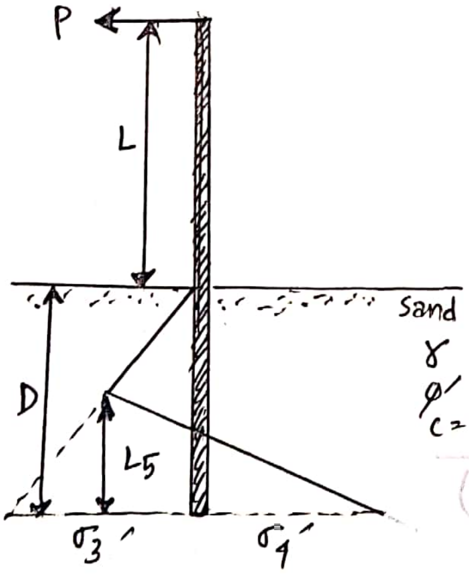
$$A_2' = \frac{8P}{\gamma (K_p - K_a)}$$

$$A_3' = \frac{6P\left[2\bar{z}\gamma (K_p - K_a) + \sigma_5'\right]}{\gamma^2 (K_p - K_a)^2}$$

$$A_4' = \frac{P(6\bar{z}\sigma_5' + 4P)}{\gamma^2 (K_p - K_a)^2}$$

* Free cantilever Sheet Piling:

The following figure shows a free cantilever sheet-pile wall penetrating a sandy soil and subjected to a line load of P per unit length of the wall.



For this case,

$$D^4 - \left[\frac{8P}{\gamma (K_p - K_a)}\right] D^2 - \left[\frac{12PL}{\gamma (K_p - K_a)}\right] D - \left[\frac{2P}{\gamma (K_p - K_a)}\right]^2 = 0$$

$$\sigma_3' = \gamma D (K_p - K_a)$$

$$\sigma_4' = \gamma D (K_p - K_a)$$

$$L_5 = \frac{\gamma (K_p - K_p) D^2 - 2P}{2D (K_p - K_a)\gamma}$$

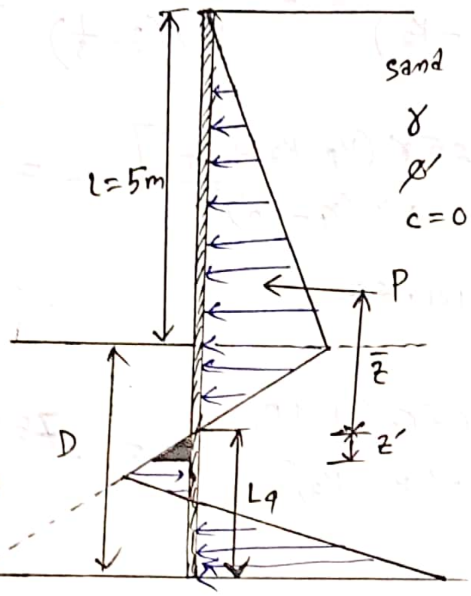
$$M_{max} = P(L + z') - \frac{\gamma z'^3 (K_p - K_a)}{6}$$

and, $z' = \sqrt{\dfrac{2P}{\gamma (K_p - K_a)}}$

Problem: 03

An cantilever sheet pile penetrating in to sand is shown in the adjacent Figure:

Here, $\gamma = 16\ KN/m^3$; $\phi' = 30°$



(a) What is the theoretical depth of embedment, D?

(b) For 30% increase in D, what should be the total length of the sheet pile.

(c) Determine the theoretical maximum moment and section modulus of sheet pile if $\sigma_{all} = 170\ MN/m^2$

Solution: $K_a = \dfrac{1-\sin 30°}{1+\sin 30°} = \dfrac{1}{3}$; $K_p = \dfrac{1}{K_a} = 3$

$$\sigma_2' = \gamma L K_a = (16 \times 5 \times \tfrac{1}{3}) = 26.67\ KN/m^2$$

$$L_3 = \frac{\sigma_2'}{\gamma(K_p - K_a)} = \frac{26.67}{16 \times (3-\frac{1}{3})} = 0.625\ m$$

$$P = \tfrac{1}{2} \times \sigma_2' L + \tfrac{1}{2} \times \sigma_2' L_3 = \tfrac{1}{2} \times 26.67 \times 5 + \tfrac{1}{2} \times 26.67 \times 0.625$$

$$\therefore P = 75\ KN$$

$$\bar{z} = L_3 + \frac{L}{3} = (0.625 + \frac{5}{3}) = 2.29$$

$$\sigma_5' = \gamma L K_p + \gamma L_3 (K_p - K_a) = 16 \times 5 \times 3 + 16 \times 0.625 \times (3 - \tfrac{1}{3}) = 266.67\ KN/m^2$$

$$A_1' = \frac{\sigma_5'}{\gamma(K_p - K_a)} = \frac{266.67}{16\times(3-\frac{1}{3})} = 6.25$$

$$A_2' = \frac{8P}{\gamma(K_p - K_a)} = \frac{8\times 75}{16\times(3-\frac{1}{3})} = 14.0625$$

$$A_3' = \frac{6P[2\bar{z}\gamma(K_p-K_a)+\sigma_5']}{\gamma^2(K_p-K_a)^2} = \frac{6\times 75\times[2\times 2.29\times 16\times(3-\frac{1}{3})+266.67]}{16^2\times(3-\frac{1}{3})^2}$$

$$\therefore A_3' = 114.22$$

$$A_4' = \frac{P(6\bar{z}\sigma_5' + 4P)}{\gamma^2(K_p-K_a)^2} = \frac{75\times(6\times 2.29\times 266.67 + 4\times 75)}{16^2\times(3-\frac{1}{3})^2} = 163.37$$

Now, $L_4^4 + A_1' L_4^3 - A_2' L_4^2 - A_3' L_4 - A_4' = 0$

$\Rightarrow L_4^4 + 6.25 L_4^3 - 14.0625 L_4^2 - 114.22 L_4 - 163.31 = 0$

$\Rightarrow L_4 = 4.465$ m

(a) $\therefore D_{act} = [L_3 + L_4] = (0.625 + 4.465) = 5.09$ m

For 30% increase in D,

(b) $\therefore$ The total length of the sheet pile $= L + 1.3 D_{act}$

$= (5 + 1.3\times 5.09) = 11.617$ m

$$z' = \sqrt{\frac{2P}{\gamma(K_p-K_a)}} = \sqrt{\frac{2\times 75}{16\times(3-\frac{1}{3})}} = 1.875 \text{ m}$$

(c) $M_{max} = P(\bar{z}+z') - [\frac{1}{2}\times\gamma z'^2\times(K_p-K_a)]\times\frac{z'}{3}$

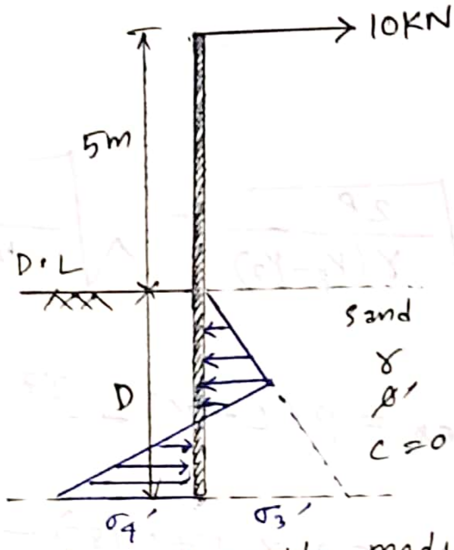
$= 75\times(2.29+1.875) - [\frac{1}{2}\times 16\times 1.875^2\times(3-\frac{1}{3})]\times\frac{1.875}{3}] = 265.5$ KN-m/m

$\therefore$ section modulus, $S = \frac{M_{max}}{\sigma_{all}} = \frac{265.5}{170\times 10^3} = 1.56\times 10^{-3}$ m^3/m of the wall

Problem: 04

A cantilever sheet pile penetrating into sand as shown in the adjacent figure:

Here, $\gamma = 16\ KN/m^3$, $\phi' = 30^\circ$



(a) What is the theoretical depth of embedment, D?

(b) For 30%, increase in D, what should be the total length of the sheet pile.

(c) Determine the theoretical maximum moment and section modulus of sheet pile if $\sigma_{all} = 170\ MN/m^2$

Solution: $K_a = \dfrac{1-\sin 30^\circ}{1+\sin 30^\circ} = \dfrac{1}{3}$; $K_p = \dfrac{1}{K_a} = 3$

$K_p - K_a = \left(3 - \frac{1}{3}\right) = \frac{8}{3} = 2.67$; $\gamma(K_p - K_a) = 2.67 \times 16 = 42.7$

Now, $$D^4 - \left[\frac{8P}{\gamma(K_p-K_a)}\right]D^2 - \left[\frac{12PL}{\gamma(K_p-K_a)}\right]D - \left[\frac{2P}{\gamma(K_p-K_a)}\right]^2 = 0$$

$$\Rightarrow D^4 - \left[\frac{8 \times 10}{42.7}\right]D^2 - \left[\frac{12 \times 10 \times 5}{42.7}\right]D - \left[\frac{2 \times 10}{42.7}\right]^2 = 0$$

$$\Rightarrow D^4 - 1.874 D^2 - 14.052 D - 0.22 = 0$$

$$\Rightarrow D = 2.675\ m$$

$\therefore$ Depth of embedment, $D = 2.675\ m$

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For 30% increase in D,

(b) Total length of the sheet pile = $L + 1.3D = (5 + 1.3 \times 2.675)$

$= 8.48$ m

$$z' = \sqrt{\frac{2P}{\gamma(K_p - K_a)}} = \sqrt{\frac{2 \times 10}{16 \times (3 - \frac{1}{3})}} = 0.685 \text{ m}$$

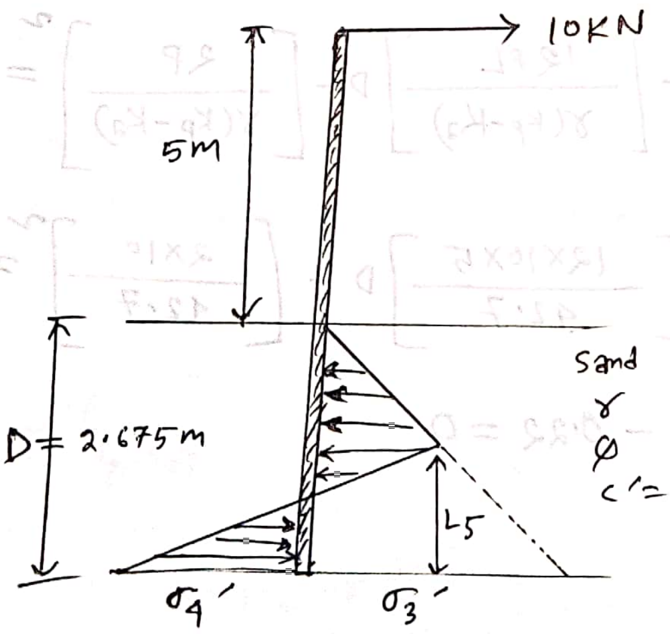
(c) $M_{max} = P(L + z') - \dfrac{\gamma z'^3 (K_p - K_a)}{6}$

$= 10 \times (5 + 0.685) - \dfrac{16 \times (0.685)^3 \times (3 - \frac{1}{3})}{6}$

$= 54.56$ KN-m/m

section modulus, $S = \dfrac{M_{max}}{\sigma_{all}} = \dfrac{54.56}{170 \times 10^3} = 3.21 \times 10^{-4}$ m³/m of the wall.

(Ans.)



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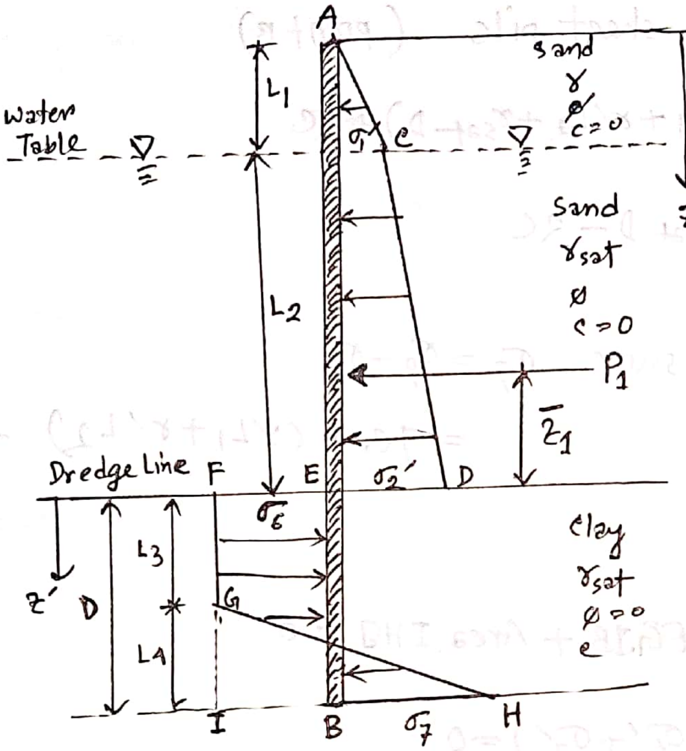
Cantilever Sheet piling Penetrating Clay:

The following figure shows a cantilever sheet pile wall driven into clay with a backfill of granular soil above the level of the dredge line:



The water table is at a depth L_1 below the top of the wall. The diagram for net pressure distribution below dredge line can be determined as:

At any depth greater than $L_1 + L_2$, for $\phi = 0$, $K_a = 1$

Similarly, for $\phi = 0$, $K_p = 1$

At depth $z = L_1$, $\sigma_1' = \gamma L_1 K_a$ ------ (i)

At depth $z = L_1 + L_2$, $\sigma_2' = \gamma L_1 K_a + \gamma' L_2 K_a$ ----- (ii)

where, $\gamma' = \gamma_{sat} - \gamma$

At any depth, $z > L_1 + L_2$, $K_a = K_p = 1$

$$\sigma_a' = \gamma L_1 + \gamma' L_2 + \gamma_{sat}(z - L_1 - L_2) - 2c$$

and, $\sigma_p' = \gamma_{sat}(z - L_1 - L_2) + 2c$

Net Pressure, $\sigma_6 = \sigma_p - \sigma_a = [\gamma_{sat}(z-L_1-L_2)+2c] - [\gamma L_1+\gamma' L_2+\gamma_{sat}(z-L_1-L_2)] + 2c$

$$= 4c - (\gamma L_1+\gamma' L_2) \quad \ldots\ldots (3)$$

At the bottom of the sheet pile (point B)

$$\sigma_p = (\gamma L_1+\gamma' L_2+\gamma_{sat} D)+2c$$

Similarly, $\sigma_a = \gamma_{sat} D - 2c$

Hence, the net pressure, $\sigma_7 = \sigma_p - \sigma_a$

$$= 4c + (\gamma L_1+\gamma' L_2) \quad \ldots\ldots (4)$$

Now,

$$\Sigma F_H = 0$$

$\Rightarrow$ Area ACEF $-$ Area EFIB $+$ Area IHG $= 0$

$$\Rightarrow P_1 - \sigma_6' D + \frac{1}{2} L_4 (\sigma_6 + \sigma_7) = 0$$

$$\Rightarrow L_4 = \frac{D[4c-(\gamma L_1+\gamma' L_2)] - P_1}{4c} \quad \ldots\ldots (5)$$

$$\Sigma M_B = 0$$

$$\Rightarrow P_1 \times (D+\bar{z}_1) - \sigma_6' D \times \frac{D}{2} + \frac{1}{2} L_4 (\sigma_6 + \sigma_7) \times \frac{L_4}{3} = 0$$

Putting the value of L_4 in this equation. we obtain,

$$D^2[4c-(\gamma L_1+\gamma' L_2)] - 2DP_1 - \frac{P_1(P_1+12c\bar{z}_1)}{(\gamma L_1+\gamma' L_2)+2c} = 0 \quad \ldots\ldots (6)$$

Maximum Moment Calculation:

According to the figure, The maximum moment (zero shear) will be between $L_1 + L_2 < z < L_1 + L_2 + L_3$.

Using a new co-ordinate system z' (with $z' = 0$ the dredge line) for zero shear gives,

$$P_1 - \sigma_6 z' = 0$$

$$\Rightarrow z' = \frac{P_1}{\sigma_6}$$

The magnitude of the maximum moment can be obtained,

$$M_{max} = P_1(z' + \bar{z}_1) - \frac{\sigma_6 z'^2}{2}$$

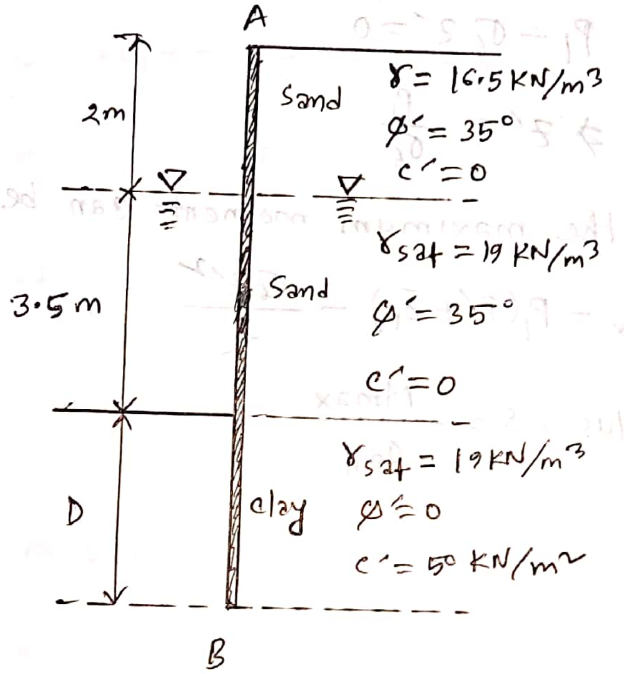
And, section Modulus, $S = \frac{M_{max}}{\sigma_{all}}$

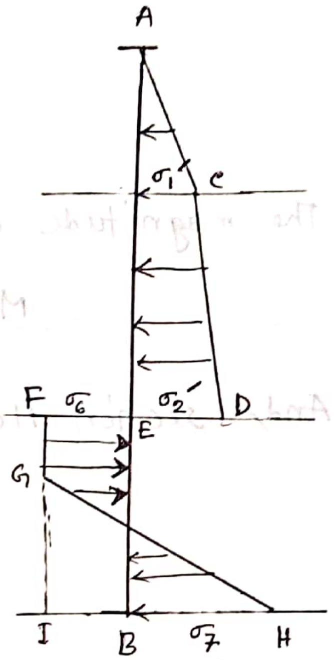
Problem 5: In the figure, for the sheet pile wall, determine

(a) Theoretical Depth of penetration

(b) Actual depth of penetration

(c) The minimum size of sheet pile section. use ($\sigma_{all} = 170.0 \ MN/m^2$)





Solution:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 35°}{1-\sin 35°} = 0.27 \qquad \therefore K_p = 3.69$$

$$\sigma_1' = K_a \gamma L_1 = 0.27 \times 16.5 \times 2 = 8.91 \ KN/m^2$$

$$\sigma_2' = K_a \gamma' L_2 + \sigma_1' = 0.27 \times (19-9.81) \times 3.5 + 8.91 = 17.6 \ KN/m^2$$

$$P_1 = \frac{1}{2}\sigma_1' L_1 + \sigma_1' L_2 + \frac{1}{2} \times (\sigma_2' - \sigma_1') \times L_2$$

$$\therefore P_1 = \frac{1}{2} \times 8.91 \times 2 + 8.91 \times 3.5 + \frac{1}{2} \times (19-9.81) \times 3.5$$

$$= 56.18 \ KN/m$$

$$\bar{z}_1 = \frac{\Sigma M_E}{P_1}$$

$$= \frac{1}{56.18} \times \left[\frac{1}{2} \times 8.91 \times 2 \times \left(3.5 + \frac{2}{3}\right) + 8.91 \times \frac{3.5^2}{2} + \frac{1}{2} \times (19 - 9.81) \times 3.5 \times \frac{3.5}{3}\right]$$

$$\bar{z}_1 = \frac{110.46}{56.18} = 1.97 \text{ m}$$

Now,

$$D^2\left[4c - (\gamma L_1 + \gamma' L_2)\right] - 2DP_1 - \frac{P_1(P_1 + 12c\bar{z}_1)}{(\gamma L_1 + \gamma' L_2) + 2c} = 0$$

$$\Rightarrow D^2\left[4 \times 50 - \{16.5 \times 2 + (19 - 9.81) \times 3.5\}\right] - 2D \times 56.18 - \frac{56.18 \times (56.18 + 12 \times 50 \times 1.97)}{[16.5 \times 2 + (19 - 9.81) \times 3.5] + 2 \times 50} = 0$$

$$\Rightarrow 134.835 D^2 - 112.36 D - 421.16 = 0$$

$$\Rightarrow D = 2.23 \text{ m}$$

(a) $\therefore D_{theory} = 2.23$ m

(b) Thus, $D_{act} = 1.3D = (1.3 \times 2.23) = 2.9$ m

(c) $\therefore$ The length of sheet pile ; $L = (2 + 3.5 + 2.9) = 8.4$ m.

(d) $z' = \dfrac{P_1}{\sigma_6'}$ Here, $\sigma_6' = 4c - (\gamma L_1 + \gamma' L_2)$

$$= 4 \times 50 - [16.5 \times 2 + (19 - 9.81) \times 3.5]$$

$$= 134.835 \text{ KN/m}^2$$

$$= \frac{56.18 \text{ KN/m}}{134.835 \text{ KN/m}^2}$$

$$= 0.417$$

$$M_{max} = P_1(z' + \bar{z}_1) - \frac{\sigma_6' z'^2}{2}$$

$$= 56.18 \times (0.417 + 1.97) - \frac{134.835}{2} \times 0.417^2 = 122.38 \text{ KN-m/m}$$

Hence, Section modulus, $S = \dfrac{M_{max}}{\sigma_{all}} = \dfrac{122.38}{170 \times 10^3} = 7.2 \times 10^{-4}$ m³/m of wall

(Ans.)

Problem-06: For the following sheet pile wall penetrating clay.

Given, sand: $L = 5.5$ m

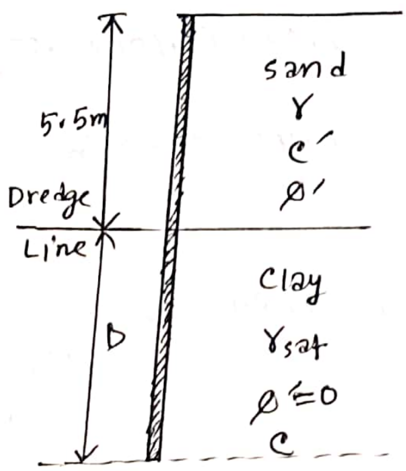
$\gamma = 16.5$ KN/m3

$c' = 0$; $\phi = 35°$

clay: $\gamma_{sat} = 16.5$ KN/m3

$\phi' = 0$

$c = 50$ KN/m²



Determine:

(a) Theoretical and actual Depth of penetration.

(b) The magnitude of maximum moment in the wall

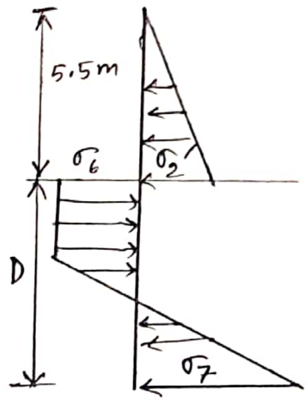
(c) And section modulus, S if $\sigma_{all} = 170$ MN/m²

Solution:

$\sigma_2' = \gamma L K_a$ Here, $K_a = \dfrac{1 - \sin 35}{1 + \sin 35} = 0.27$

$= 16.5 \times 5.5 \times 0.27$

$= 24.5$ KN/m²



$P_1 = \frac{1}{2} \sigma_2' L = \left(\frac{1}{2} \times 24.5 \times 5.5\right) = 67.38$ KN/m

Now, $\bar{z}_1 = \frac{L}{3} = \frac{5.5}{3} = 1.83$

(a) Then, $D^2 (\underbrace{4c - \gamma L}_{\sigma_6}) - 2 D P_1 - \dfrac{P_1 (P_1 + 12 c \bar{z}_1)}{\gamma L + 2c} = 0$

$\Rightarrow D^2 \times (4 \times 50 - 16.5 \times 5.5) - 2 \times D \times 67.38 - \dfrac{67.38 \times (67.38 + 12 \times 50 \times 1.83)}{16.5 \times 5.5 + 2 \times 50} = 0$

$\Rightarrow 109.25 D^2 - 134.76 D - 411.66 = 0$

$\therefore D = 2.654$ m $D_{theory} = 2.654$ m

$\therefore D_{act} = (1.3 \times 2.654) = 3.45$ m

(b) $M_{max} = P_1(z' + \bar{z}_1) - \frac{\sigma_6 z'^2}{2}$

$= 67.38 \times (1.696 + 1.83) - \frac{109.25 \times 1.696^2}{2}$

$= 80.46$ KN-m/m

Here, $z' = \frac{\frac{1}{2}\gamma L^2 K_a}{4c - \gamma L}$

$= \frac{0.5 \times 67.38 \times 5.5}{109.25}$

$= 1.696$

(c) The section modulus,

$S = \frac{M_{max}}{\sigma_{all}} = \frac{80.46}{170 \times 10^3} = 4.73 \times 10^{-4}$ m3/m of the wall

(Ans.)

Special Cases for Cantilever sheet pile Penetrating clay:

* Sheet pile wall in the absence of water Table:

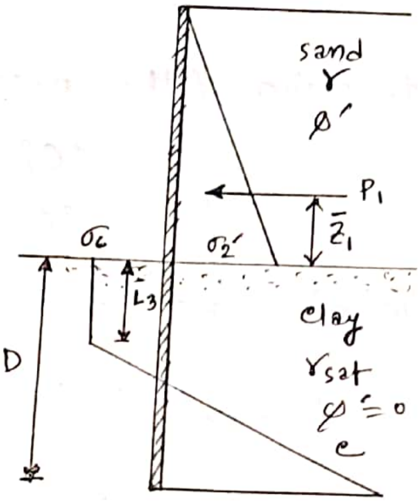
In this case,

$$\sigma_2' = \gamma L K_a$$

$$\sigma_6 = 4c - \gamma L$$

$$\sigma_7 = 4c + \gamma L$$

$$P_1 = \frac{1}{2} L \sigma_2' = \frac{1}{2} \gamma L^2 K_a$$



and,

$$L_4 = \frac{D(4c - \gamma L) - \frac{1}{2}\gamma L^2 K_a}{4c}$$

The theoretical Depth of penetration,

$$D^2(4c - \gamma L) - 2DP_1 - \frac{P_1(P_1 + 12c\bar{z}_1)}{\gamma L + 2c} = 0 \quad \text{where, } \bar{z}_1 = \frac{L}{3}$$

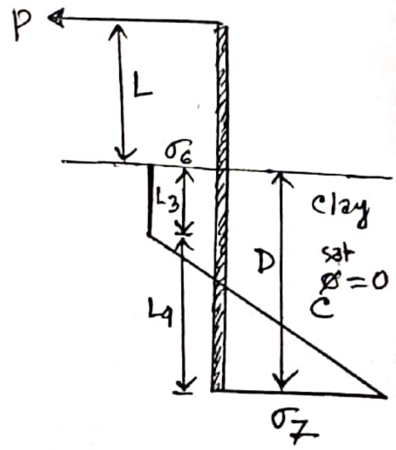
The magnitude of the maximum moment in the wall is,

$$M_{max} = P_1(z' + \bar{z}_1) - \frac{\sigma_6 z'^2}{2}$$

$$\text{where, } z' = \frac{P_1}{\sigma_6} = \frac{\frac{1}{2}\gamma L^2 K_a}{4c - \gamma L}$$

* Free cantilever sheet pile wall penetrating clay:

In the figure, A free cantilever sheet-pile is shown penetrating a clay layer. The wall is subjected a line load 'p' per unit length



For this case,

$$\sigma_6 = \sigma_7 = 4c$$

The depth of Penetration, D may be calculated as follows:

$$4D^2C - 2PD - \frac{P(P+12CL)}{2C} = 0$$

for construction of the pressure diagram,

$$L_4 = \frac{4CD - P}{4C}$$

The maximum moment in the wall is,

$$M_{max} = P(L+z') - \frac{4Cz'^2}{2}$$

where, $z' = \frac{P}{4C}$

Problem; 7:

For the sheet pile penetrating in to clay as shown in figure:

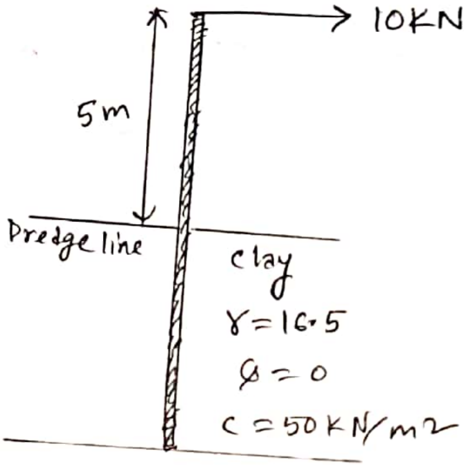
Determine;

(a) D

(b) Dact

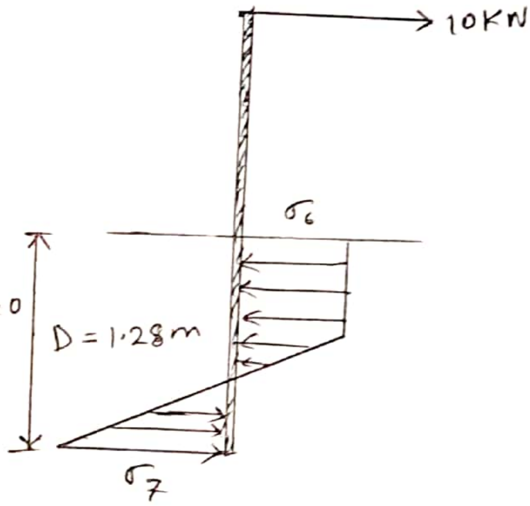
(c) Mmax

(d) S



Solution;

Depth of pentration:



$$4D^2c - 2PD - \frac{P(P+12cL)}{2c} = 0$$

$$\Rightarrow 4 \times D^2 \times 50 - 2 \times 10 \times D - \frac{10(10+12 \times 50 \times 5.0)}{2 \times 50} = 0$$

$$\Rightarrow 200D^2 - 20D - 301 = 0$$

$$\Rightarrow D = 1.28 \text{ m}$$

$$\therefore D_{theory} = 1.28 \text{ m}$$

$$D_{act} = (1.23 \times 1.28) = 1.664 \text{ m}$$

Now,

$$z' = \frac{P}{4c}$$

$$= \frac{10}{4 \times 50} = 0.05 \text{ m}$$

Maximum moment, $M_{max} = P(L+z') - \frac{4c\,z'^2}{2}$

$$= 10 \times (5.0 + 0.05) - \frac{4 \times 50 \times (0.05)^2}{2}$$

$$= 50.25 \text{ KN-m/m}$$

$\therefore$ section modulus, $S = \frac{M_{max}}{\sigma_{all}}$

$$= \frac{50.25}{170 \times 10^3} = 2.96 \times 10^{-4} \text{ m}^3\text{/m of the wall.}$$

(Ans.)

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Anchor Sheet Pile

Anchor sheet pile:

When the height of the backfill material behind a cantilever sheet-pile wall exceeds about 6m (≃ 20ft), tying the wall near the top to anchor plates, anchor walls or anchor piles becomes more economical. This type of construction is referred to as anchored sheet piles.

Methods of designing anchor sheet piles:

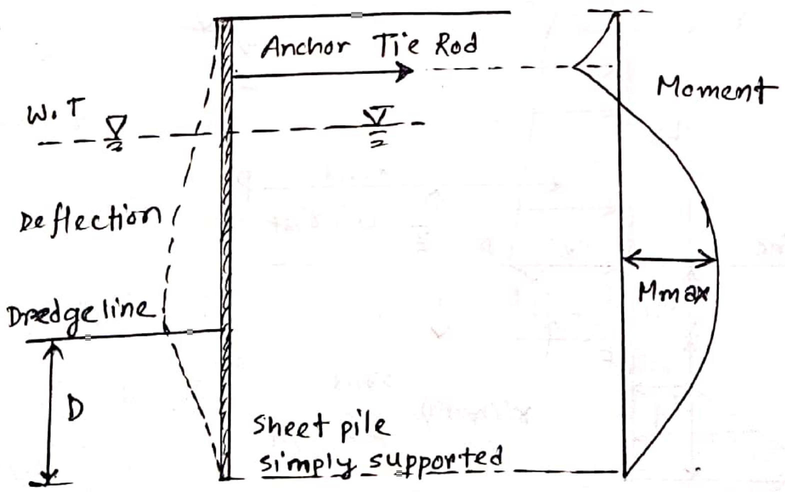
There are two basic methods:

(i) The free earth support method.

(ii) The fixed earth support method.

Free earth support method:

The free earth support method involves a minimum penetration depth. Below the dredge line, no pivot point exists for the static system. The nature of variation of the bending moment with depth for this method is shown in figure below.

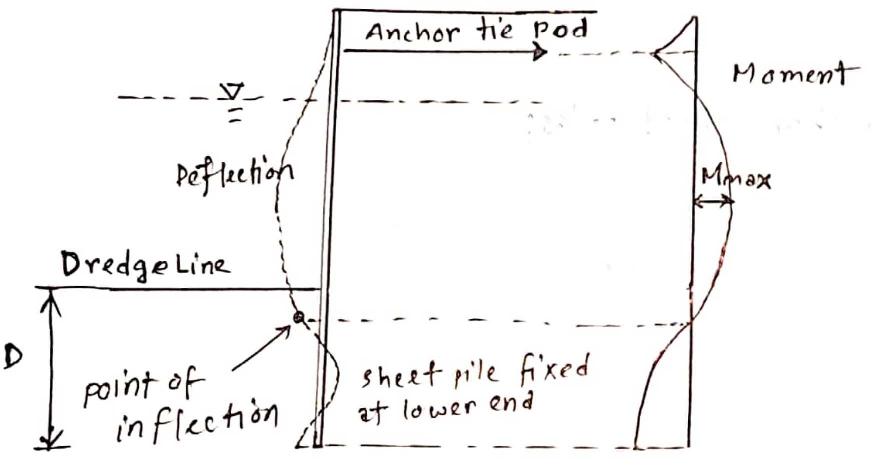


(a) Free earth support Method.

Fixed earth support method:

Note that, $D_{free\ earth} < D_{fixed\ earth}$ —(2018) - Explain

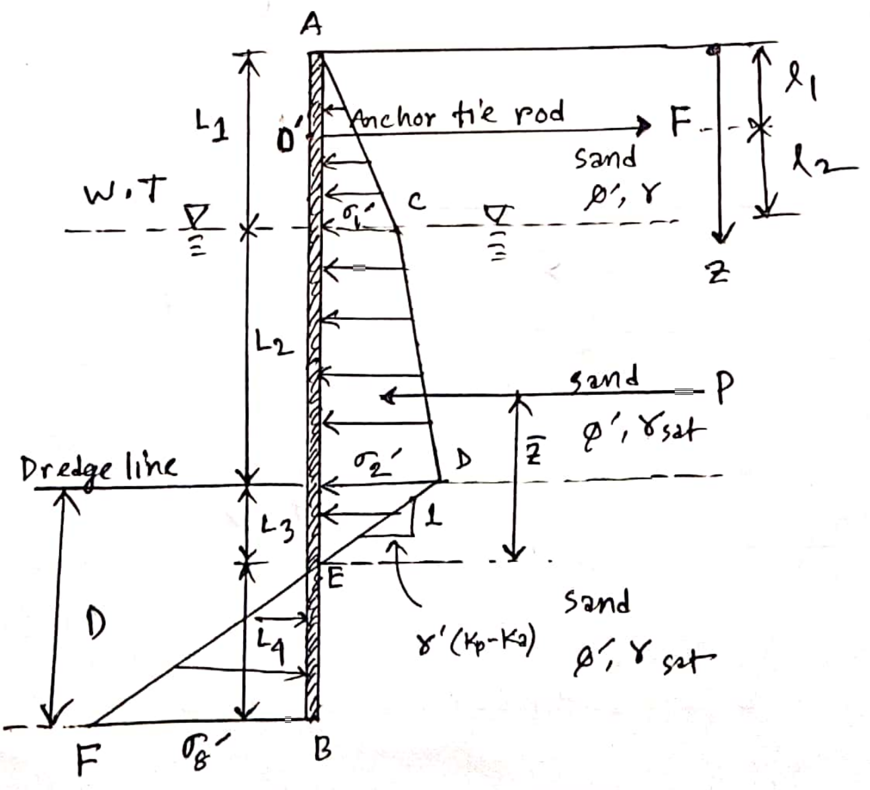
The nature of bending moment with depth for fixed earth support method is shown in figure below:



(b) Fixed earth support method

2018, 2015

Anchor sheet pile penetrating in to sand: (Free earth support Method)



Net Pressure Diagram

At depth $z = L_1$, $\sigma_1' = \gamma L_1 K_a$

At depth $z = L_1 + L_2$, $\sigma_2' = (\gamma L_1 + \gamma' L_2) K_a$

Below the dredge line the net pressure will be zero at $z = L_1 + L_2 + L_3$

At any depth z; $\sigma_a' = [\gamma L_1 + \gamma' L_2 + \gamma'(z - L_1 - L_2)] K_a$

and, $\sigma_p' = \gamma'(z - L_1 - L_2) K_p$

$\therefore$ Net pressure , $\sigma' = \sigma_p' - \sigma_a'$

$$= (\gamma L_1 + \gamma' L_2) K_a - \gamma'(z - L_1 - L_2)(K_p - K_a)$$

$$= \sigma_2' - \gamma(z - L)(K_p - K_a)$$

Now, Net pressure at point E is zero.

$$\sigma' = \sigma_2' - \gamma'(z - L)(K_p - K_a)$$

$$\Rightarrow z - L = \frac{\sigma_2'}{\gamma'(K_p - K_a)}$$

$$\therefore L_3 = \frac{\sigma_2'}{\gamma'(K_p - K_a)}$$

At depth $z = L_1 + L_2 + L_3 + L_4$,

$$\frac{\sigma_8'}{\sigma_2'} = \frac{L_4}{L_3}$$

$$\Rightarrow \sigma_8' = \frac{L_4}{L_3} \times \sigma_2'$$

$$\Rightarrow \sigma_8' = \frac{L_4 \sigma_2'}{\frac{\sigma_2'}{\gamma'(K_p - K_a)}} = L_4(K_p - K_a)\gamma'$$

$\Sigma F_H = 0$

Area ACDE − Area EBF − F = 0

$$\Rightarrow P - \frac{1}{2}\sigma_8' L_4 - F = 0$$

$$\Rightarrow F = P - \frac{1}{2}[\gamma'(K_p - K_a)]L_4^2$$

Taking moment about point O',

$$-P[(L_1+L_2+L_3) - (\bar{z}+l_1)] + \frac{1}{2}[\gamma'(K_p-K_a)]L_4^2\left(l_2+L_2+L_3+\frac{2}{3}L_4\right) = 0$$

$$\Rightarrow L_4^3 + 1.5L_4^2(l_2+L_2+L_3) - \frac{3P[(L_1+L_2+L_3)-(\bar{z}+l_1)]}{\gamma'(K_p-K_a)} = 0$$

Then,

$$D_{theoretical} = L_3 + L_4$$

$$D_{actual} = 1.3 \text{ to } 1.4\ D_{theoretical}$$

The maximum moment occurs at a depth between $z = L_1$ and $z = L_1 + L_2$,

$$\frac{1}{2}\sigma_1' L_1 - F + \sigma_1'(z-L_1) + \frac{1}{2}K_a\gamma'(z-L_1)^2 = 0$$

From this equation, 'z' is determined. And the magnitude of maximum moment is easily obtained.

$$M_{max} = F\times(z-L_1) - \frac{1}{2}\sigma_1' L_1 \times \left(z - \frac{2L_1}{3}\right) - \sigma_1'\frac{(z-L_1)^2}{2}$$
$$- \frac{1}{2}K_a\gamma'(z-L_1)\times(z-L_1)\times\frac{(z-L_1)}{3}$$

Then, The section modulus, $S = \dfrac{M_{max}}{\sigma_{all}}$

Moment reduction for Anchored Sheet Pile walls Penetrating in to sand:

* Sheet piles are flexible

* As a result sheet pile walls yield, which redistributes the lateral earth pressure.

* This change tends to reduce the maximum bending moment calculated by Free Earth support method.

* This is why, Rowe (1952, 1957) suggested a procedure to reduce the maximum design moment on sheet pile calculated by FESM.

* This reduction procedure is sometimes called Rowe's moment reduction theory/ technique.

Symbol used:

1. H' = total height of pile driven (i.e, $L_1 + L_2 + D_{act}$)

2. Relative flexibility of pile $= \rho = 10.91 \times 10^{-7} \times \left(\frac{H'^4}{EI}\right)$

where, H' is in meters

E = modulus of elasticity of the pile material (MN/m^2)

I = moment of inertia of the pile section per meter of the wall. (m^4/m of the wall)

* in English unit, $\rho = \frac{H'^4}{EI}$ where H is in (ft)

E is in (lb/in^2)

I is in (in^4/ft) of the wall

procedure for the use of moment reduction Diagram: (sand)

Step 1: choose a sheet pile section

step 2: Find the section modulus, S of the selected section per unit length of the wall.

Step 3: Determine the moment of inertia of the section per unit length of the wall.

step 4: Obtain H´ and calculate ρ

Step 5: Find log ρ

step 6: Find the moment capacity of the pile section, $M_d = \sigma_{all} S$

step 7: Determine M_d/M_{max}'

step 8: Plot log ρ and M_d/M_{max}

Step 9: Repeat step 1 through 8 for several sections.

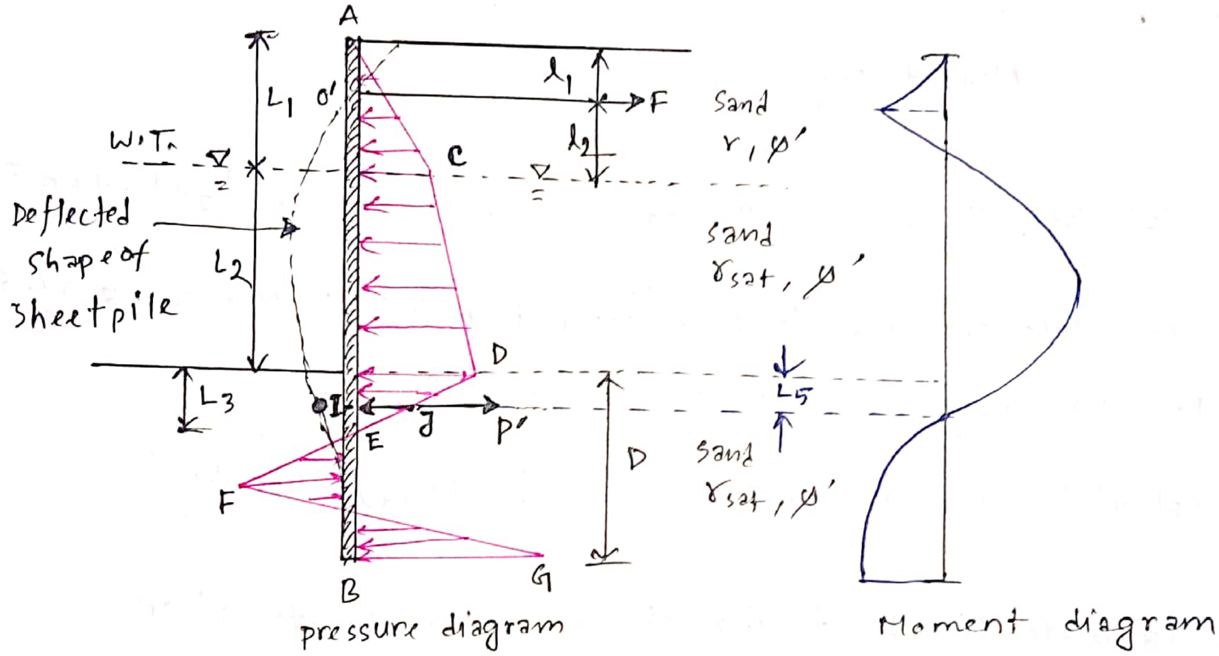
* The points that fall above the curve are safe sections and the points that fall below the curve are un safe section.

The cheapest section may now be choosen from those point which fall above the proper curve.

Note that, the section choosen will have an $M_d < M_{max}$.

Fixed Earth support Method for Penetration into Sandy soil.

When using the fixed earth support method, we assume that the toe of the pile is restrained from rotating as shown in figure below.



In the fixed earth support solution, a simplified method called the equivalent beam solution is generally used to calculate L_3 and thus, D.

Design procedure: (Cornfield, 1975)

Step 1: Determine L_5, which is a function of the soil friction angle ϕ' below the Dredgeline, from the following:

ϕ'(deg)	$\frac{L_5}{L_1+L_2}$
30	0.08
35	0.03
40	0

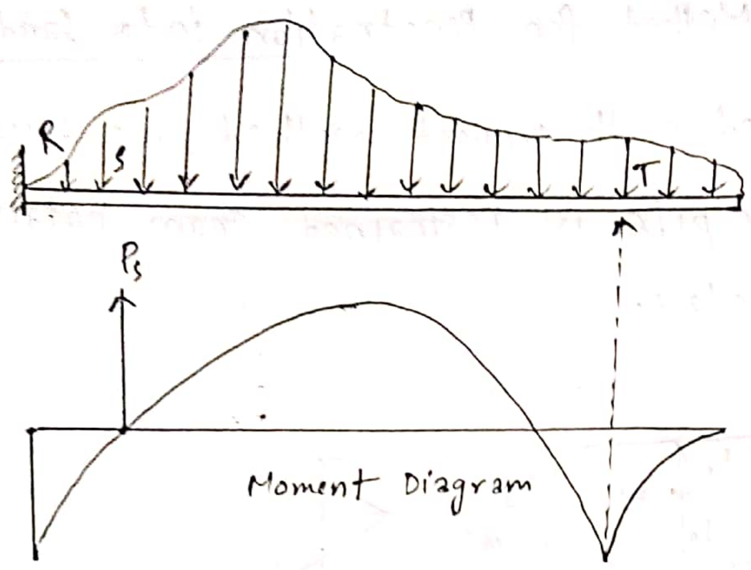


Fig. Equivalent Cantilever Beam Concept

<u>Step 2</u>: Calculate the span of the equivalent beam as $l_2 + L_2 + L_5 = L'$

<u>Step 3</u>: Calculate the total load of the span, W. This is the area of the pressure diagram between O' and I

<u>Step 4</u>: Calculate the maximum moment, M_{max} as $\frac{WL'}{8}$

<u>Step 5</u>: Calculate P' by taking the moment about O' or

$$P' = \frac{1}{L'} \text{(moment of area ACDJI)}$$

<u>Step 6</u>: Calculate D as,

$$D = L_5 + 1.2\sqrt{\frac{6P'}{(K_p - K_a)\gamma'}}$$

<u>Step 7</u>: Calculate the anchor force per unit length F, by taking the moment about I, or $F = \frac{1}{L'}$(Moment of area ACDJI about I)

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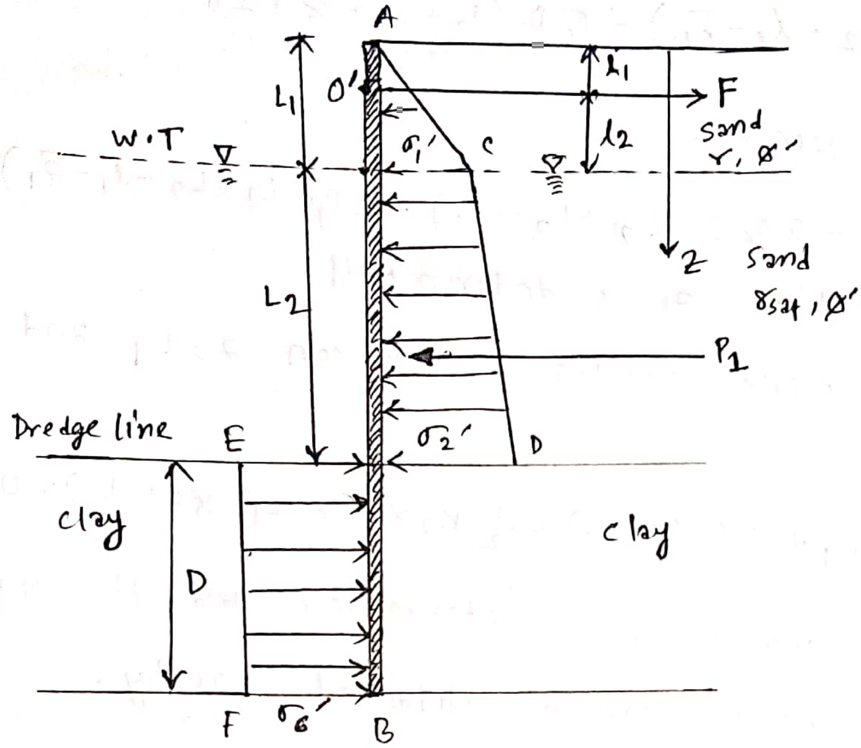
Free Earth Support Method for penetration of clay: (Free earth support method)



Net pressure diagram

At the depth z_1, $\sigma_1' = \gamma L_1 K_a$

At depth z_2, $\sigma_2' = \gamma L_1 K_a + \gamma' L_2 K_a$

At depth $z = L_1 + L_2 + D$, $[\phi = 0$; Thus $K_a = K_p = 1]$

$$\sigma_a' = \gamma L_1 + \gamma' L_2 + \gamma_{sat} D - 2c$$

$$\sigma_p' = \gamma_{sat} D + 2c$$

$$\sigma_6' = \sigma_p' - \sigma_a' = 4c - (\gamma L_1 + \gamma' L_2)$$

$$\Sigma F_H = 0$$

$$P_1 - F - \sigma_6' D = 0$$

$$\therefore F = P_1 - \sigma_6' D$$

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Taking moment about O:

$$P_1 (L_1 + L_2 - l_1 - \bar{z}_1) - \sigma_6 D (l_2 + L_2 + D/2) = 0$$

Simplification yields,

$$\sigma_6 D^2 + 2\sigma_6 D (L_1 + L_2 - l_1) - 2P_1 (L_1 + L_2 - l_1 - \bar{z}_1) = 0$$

From this equation, D is determined.

Maximum moment occurs at between $z = L_1$ and $z = L_1 + L_2$

$$\Sigma F_x = 0$$

$$-F + \frac{1}{2}\sigma_1' L_1 + \sigma_1' (z - L_1) + \frac{1}{2} K_a \gamma' (z - L_1) \times (z - L_1) = 0$$

From this equation, z is determined and the magnitude of Maximum moment can be obtained easily.

$$M_{max} = F \times (z - L_1) - \frac{1}{2}\sigma_1 L_1 \left(z - \frac{2L_1}{3}\right) - \sigma_1' (z - L_1) \times (z - L_1) \cdot - \frac{1}{2} K_a \gamma' (z - L_1) \times (z - L_1) \times \frac{(z-1)}{3}$$

Then,

The section modulus, $S = \dfrac{M_{max}}{\sigma_{all}}$

Moment Reduction technique for Anchor sheet pile pentrating clay:

Symbol used:

1. stability number, $S_n = 1.25 \times \dfrac{c}{(\gamma L_1 + \gamma' L_2)}$ where, c = undrained cohesion ($\phi = 0$)

2. The non dimensional wall height,

$$\alpha = \frac{L_1 + L_2}{L_1 + L_2 + D_{act}}$$

3. The flexibility number is e

4. M_d = design moment

M_{max} = Maximum theoretical moment

Procedure for moment reduction: (clay)

Step 1. Obtain $H' = L_1 + L_2 + D_{act}$

Step 2. Determine $\alpha = (L_1 + L_2)/H'$

Step 3. Determine S_n

Step 4. For the magnitude of α and S_n, determine M_d/M_{max} for various values of $\log e$ and plot M_d/M_{max} against $\log e$

Step 5. Follows step 1 through 9 as outlined for the case of moment reduction of sheet pile penetrating granular soil.

Problem: 8

An anchor sheet pile penetrating into sand is shown in adjacent figure:

Here, $L_1 = 2m$, $L_2 = 4m$, $l_1 = 1m$, $l_2 = 1m$, $\gamma = 16\ KN/m^3$, $\gamma_{sat} = 19\ KN/m^3$ and $\phi' = 31°$ Given, $\sigma_{all} = 170\ MN/m^3$

(i) what is the theoretical depth of embedment, D

(ii) D actual = ?

(iii) Anchor force, F = ?

(iv) Section of modulus, S = ?

(v) Draw net pressure diagram.

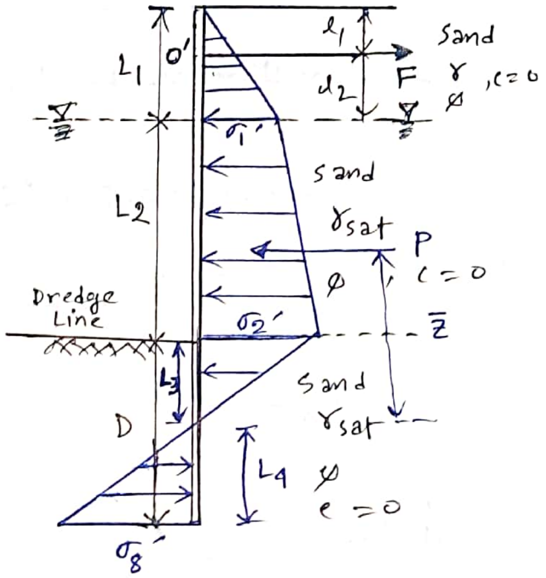

Fig. Net Pressure Diagram

Solution; $K_a = \dfrac{1-\sin 31}{1+\sin 31} = 0.32$; $K_p = \dfrac{1}{K_a} = 3.12$; $K_p - K_a = 2.8$

and, $\gamma' = (19 - 9.81)$
$= 9.19\ KN/m^3$

$\sigma_1' = K_a \gamma L_1 = (0.32 \times 16 \times 2) = 10.24\ KN/m^2$

$\sigma_2' = \sigma_1' + K_a \gamma' L_2 = (10.24 + \underbrace{0.32 \times 9.19 \times 4}_{11.76}) = 22\ KN/m^2$

$L_3 = \dfrac{\sigma_2'}{\gamma'(K_p - K_a)} = \dfrac{22}{9.19 \times 2.8} = 0.855\ m$

$P = \frac{1}{2} \times 10.24 \times 2 + 10.24 \times 4 + \frac{1}{2} \times 11.76 \times 4 + \frac{1}{2} \times 22 \times 0.855$

$= 84.125\ KN/m$

$\bar{z} = \dfrac{\Sigma M_E}{P} = \Big[\frac{1}{2} \times 22 \times 0.855 \times \frac{2}{3} \times 0.855 + \frac{1}{2} \times 11.76 \times 4 \times (0.855 + \frac{1}{3} \times 4) +$
$10.24 \times 4 \times (0.855 + \frac{4}{2}) + \frac{1}{2} \times 10.24 \times 2 \times (0.855 + 4 + \frac{1}{3} \times 2)\Big] \div 84.125$

$= \dfrac{230.31}{84.125} = 2.74\ m$

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$\Sigma M_{O'} = 0$

$84.125 \times (5.855 - 2.74) = \frac{1}{2} \times \sigma_8' \times L_4 \times \left(6.855 + \frac{2}{3} \times L_4\right)$

$\Rightarrow \frac{1}{2} \times \gamma' \times (K_p - K_a) L_4 \times L_4 \times \left(.855 + \frac{2}{3} L_4\right) = 262.05$

$\Rightarrow \frac{1}{2} \times 9.19 \times 2.8 \times \left(5.855 L_4^2 + \frac{2}{3} L_4^3\right) = 262.05$

$\Rightarrow 8.58 L_4^3 + 75.33 L_4^2 - 262.05 = 0$

$\therefore L_4 = 1.707$ m

(i) $D_{theoretical} = L_3 + L_4 = (0.855 + 1.707) = 2.562$ m

(ii) $D_{actual} = 1.3 \, D_{theoretical} = (1.3 \times 2.562) = 3.33$ m

(iii) anchor force:

$\Sigma F_H = 0$

$F - 84.125 + \frac{1}{2} \times 9.19 \times 2.8 \times 1.707^2 = 0$

$\Rightarrow F = 46.64$ KN/m

(iv) section modulus:

Maximum moment occurs between $z = L_1$ & $z = L_2$ and let, it is 'x' from top.

Thus,

$\frac{1}{2} \times 10.24 \times 2 + 10.24 \times (x-2) + \frac{1}{2} \times 9.19 \times 0.32 \times (x-2)^2 = \underset{F}{\underline{46.64}}$

$\Rightarrow 10.24(x-2) + 1.4704 (x-2)^2 = 46.64$

$\Rightarrow (x-2) = 3.19$

$\therefore x = 5.14$ m

Now, $M_{max} = -\frac{1}{2} \times 10.24 \times 2 \times (5.14 - \frac{2}{3} \times 2) - 10.24 \times \frac{(5.14-2)^2}{2} + 46.64 \times 4.14$

$- \frac{1}{2} \times 9.19 \times 1.32 \times 3.14^2 \times \frac{3.14}{3}$

$= 113.69 \text{ KN-m/m}$

$\therefore$ sectional modulus of sheet pile $= \frac{113.69}{170 \times 10^3} = 6.7 \times 10^{-4} \text{ m}^3/\text{m}$ of the wall

(Ans.)

Problem-9:

Refer to problem 8. Use moment reduction diagram to find an appropriate sheet pile section. For the sheet pile, use $E = 200 \times 10^3 \text{ MN/m}^2$ and $\sigma_{all} = 170 \text{ MN/m}^2$

Solution:

Problem: 10:

Given, $L_1 = 3m$, $L_2 = 6m$

$l_1 = 1.5m$, $l_2 = 1.5m$

$\gamma = 17 KN/m^3$, $\phi' = 35°$

$\gamma_{sat} = 20 KN/m^3$

$c = 41 KN/m^2$

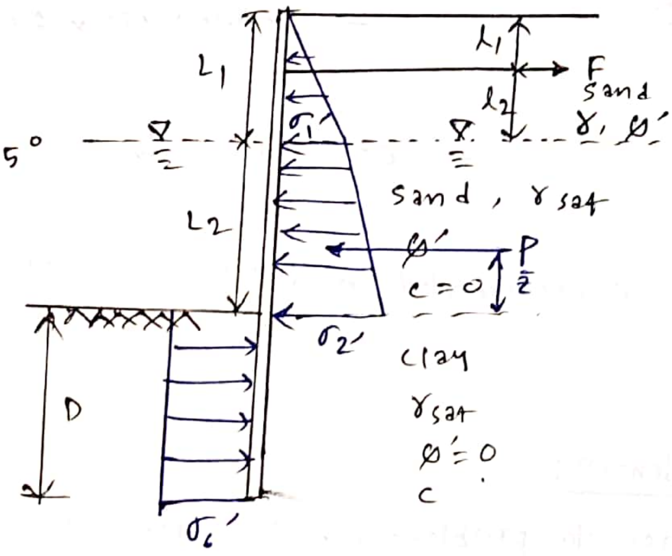
Calculate:

(a) D_{act}

(b) F

(c) M_{max}

(d) S if $\sigma_{all} = 170 MN/m^3$



Solution:

$$K_a = \frac{1-\sin 35}{1+\sin 35} = 0.271, \quad K_p = \frac{1}{K_a} = 3.69$$

$$\sigma_1' = \gamma K_a L = (17 \times 0.271 \times 3) = 13.82 \ KN/m^2$$

$$\sigma_2' = 13.82 + \gamma' K_a L_2 = 13.82 + (20-9.81) \times 0.271 \times 6 = 30.39 \ KN/m^2$$

$$P = \frac{1}{2} \times 13.82 \times 3 + 13.82 \times 6 + \frac{1}{2} \times (30.39 - 13.82) \times 6 = 153.36 \ KN/m$$

$$\bar{z} = \frac{\Sigma M_E}{P} = \left[\frac{1}{2} \times 13.82 \times 3 \times \left(6 + \frac{3}{3}\right) + 1.82 \times 6 \times \frac{6}{2} + \frac{1}{2} \times 16.57 \times 6 \times \frac{6}{3}\right] \div P$$

$$= \frac{493.29}{153.36} = 3.2$$

Now,

$$\sigma_6' = 4c - (\gamma L_1 + \gamma' L_2) = 4 \times 41 - [17 \times 3 + (20-9.81) \times 6]$$

$$\sigma_6' = 51.86 \ KN/m^2$$

$\Sigma M_{O'} = 0$

$153.36 \times (9-1.5-3.2) = 51.86 \times D \times (\frac{D}{2} + 7.5)$

$\Rightarrow 659.448 = \frac{51.86}{2} D^2 + 388.95 D$

$\Rightarrow \frac{51.86}{2} D^2 + 388.95 D - 659.448 = 0$

$\Rightarrow D = 1.54 m$ $\quad \therefore D \simeq 1.6 m$

$\therefore$ The actual depth of penetration, $D_{act} = (1.3 \times 1.6) = 2.08 m$

$\Sigma F_H = 0$

$F = 153.36 - 51.86 \times 1.6 = 70.384 KN/m$

Maximum moment occurs between $z = L_1$ & $z = L_2$,

Thus,

$70.384 - \frac{1}{2} \times 13.82 \times 3 - \frac{1}{2} \times (20-9.81) \times 0.271 \times (z-3) \times (z-3)$

$- 13.82 \times (z-3) = 0$

$\Rightarrow 1.38 (z-3)^2 + 13.82 (z-3) - 49.654 = 0$

$\Rightarrow z - 3 = 2.81$

$\therefore z = 5.81 m$

$\therefore M_{max} = 70.384 \times (5.81 - 1.5) - \frac{1}{2} \times 13.82 \times 3 \times (5.81 - \frac{2}{3} \times 3)$

$- 13.82 \times \frac{2.81^2}{2} - \frac{1}{2} \times (20-9.81) \times 0.271 \times 2.81^2 \times \frac{2.81}{3}$

$= 159.60 KN\text{-}m/m$

$\therefore$ sectional modulus, $S = \frac{M_{max}}{\sigma_{all}} = \frac{159.60}{170 \times 10^3} = 9.4 \times 10^{-4}$ m^3/m of the wall.

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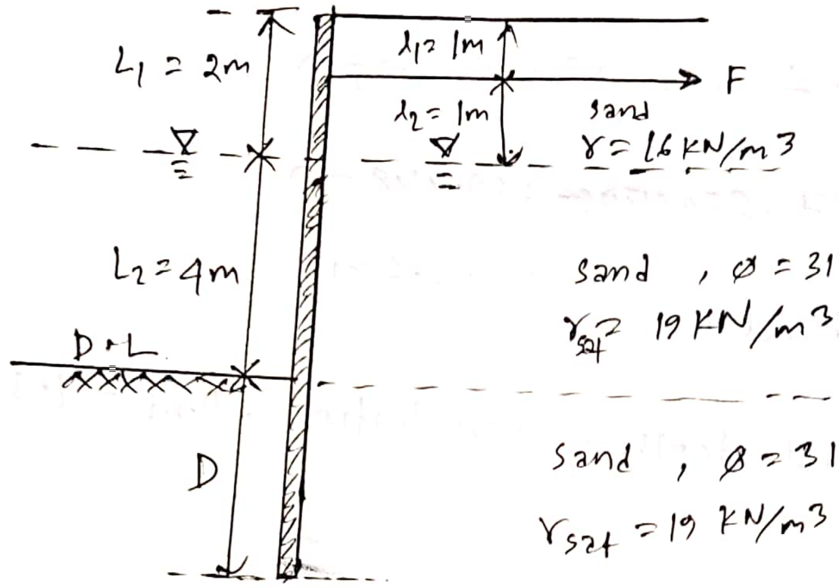
Problem 11: (Fixed Earth support Method)



Determine:

(a) Maximum moment.

(b) Theoretical depth of penetration.

(c) Anchor force per unit length of the structure.

Solution: (a)

Step 1: For $\phi' = 31°$,

$$\frac{L_5}{L_1 + L_2} \simeq 0.07$$

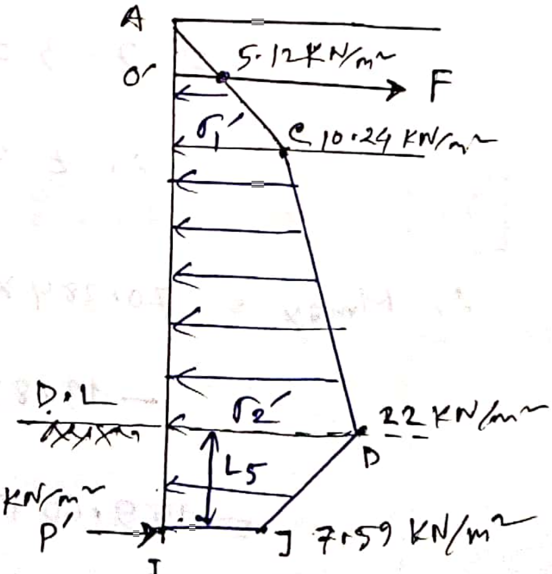
$$\Rightarrow L_5 = 0.07 \times (2+4) = 0.56\ m$$

Step 3: $K_a = \dfrac{1-\sin 31}{1+\sin 31} = 0.32$

$K_p = \dfrac{1}{0.32} = 3.12$

$K_p - K_a = 2.8$

$\gamma' = (19 - 9.81) = 9.19\ KN/m^3$



$\sigma_1' = K_a \gamma L_1 = (0.32 \times 16 \times 2) = 10.24\ KN/m^2$

$\sigma_2' = 10.24 + K_a \gamma' L_2 = (10.24 + 0.32 \times 9.19 \times 4) = 22\ KN/m^2$

The net pressure at a depth L_5 below dredge line,

$$\sigma_2' - \gamma'(K_p - K_a) \times L_5 = 22 - 9.19 \times (2.8) \times 0.56 = 7.59 \; KN/m^2$$

Now,

$$W = \frac{1}{2} \times (10.24 + 5.12) \times 1 + \frac{1}{2} \times 4 \times (10.24 + 22) + \frac{1}{2} \times (22 + 7.59) \times 0.56$$

$$= 80.45 \; KN/m$$

Step 2:

$$L' = L_2 + L_2 + L_5 = (1 + 4 + 0.56) = 5.56 \; m.$$

Step-4:

$$\therefore M_{max} = \frac{WL'}{8} = \frac{80.45 \times 5.56}{8} = 55.91 \; KN\text{-}m/m$$

(b) value of P': (Step-5:)

$$P' = \frac{1}{L'} \times (\text{moment of area ACDJI about } O')$$

$$= \frac{1}{5.56}\Big[\frac{1}{2} \times 10.24 \times 2 \times (\tfrac{2}{3} \times 2 - 1) + 10.24 \times 4 \times (1 + \tfrac{4}{2}) + \frac{1}{2} \times (22 - 10.24) \times 4 \times (1 + \tfrac{2}{3} \times 4) + \frac{1}{2} \times (22 + 7.59) \times 0.56 \times (1 + 4 + \tfrac{0.56}{2})\Big]$$

approximate value

$$= \left(\frac{256.28}{5.56}\right) KN/m = 46.09 \; KN/m$$

Step-6:

$$D = L_5 + 1.2\sqrt{\frac{6P'}{(K_p - K_a)\gamma'}}$$

$$= 0.56 + 1.2 \times \sqrt{\frac{6 \times 46.09}{2.8 \times 9.19}} = 4.494 \; m$$

(Step - 7):

Taking moment about J

$$F = \frac{1}{L'} \times (\text{moment of area ACDJI about I})$$

$$= \frac{1}{5.56} \times \left[\frac{1}{2} \times 10.24 \times 2 \times \left(4.56 + \frac{2}{3}\right) + 10.24 \times 4 \times \left(0.56 + \frac{4}{2}\right) \right.$$

$$+ \frac{1}{2} \times (22 - 10.24) \times 4 \times \left(0.56 + \frac{4}{3}\right) +$$

$$\left. \underbrace{\frac{1}{2} \times (22 + 7.59) \times 0.56 \times \left(\frac{0.56}{2}\right)}_{\text{approximate value}} \right]$$

$$F = \left(\frac{205.23}{5.56}\right) = 36.91 \text{ kN/m}$$

(Ans.)

Note:

In free earth support method, $D_{free} = 2.681$ m (see Problem-8)

But In fixed earth support method, $D_{fixed} = 4.494$ m

$$\boxed{D_{fixed} > D_{free}}$$

* যে method-এ D-এর মান বেশি সেই method comparatively better.

Anchors

The general types of anchor used in sheet pile walls are as follows:

1. Anchor plates and Beams (dead man)
2. Tie Back
3. Vertical anchor piles
4. Anchor beams supported by batter (compression and tension) piles.

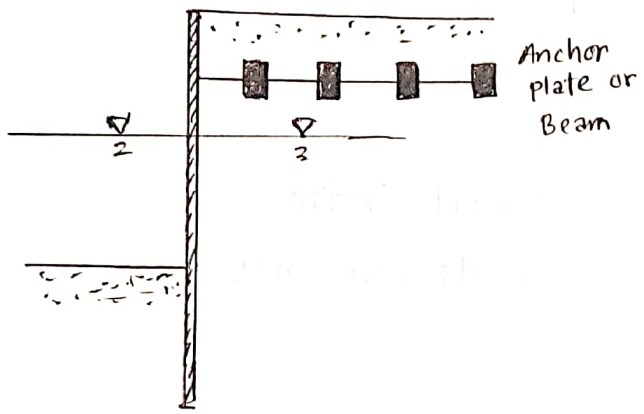


Fig. Anchor plates or Beam

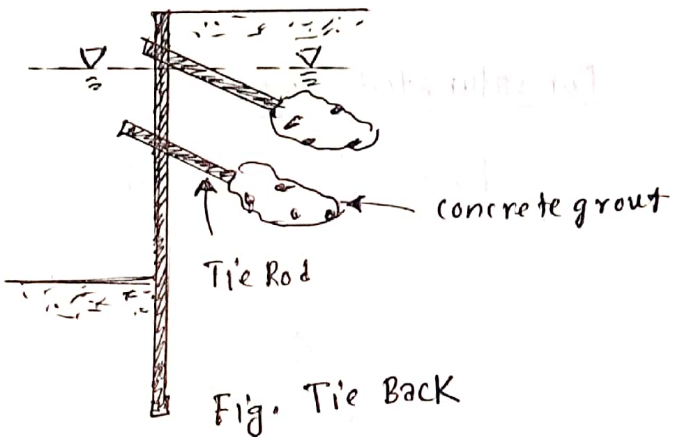


Fig. Tie Back

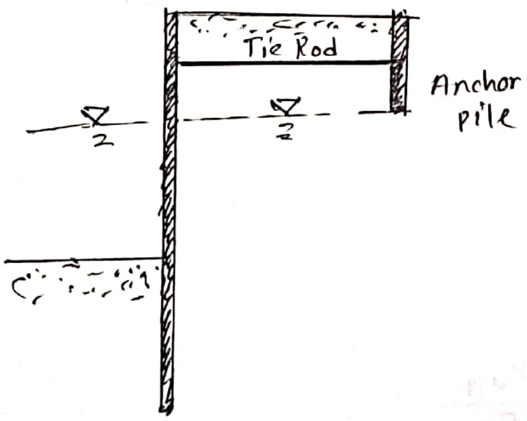


Fig. vertical anchor pile

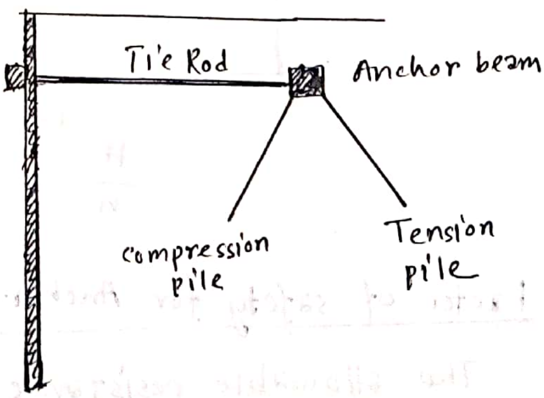


Fig. Anchor beams with batter piles.

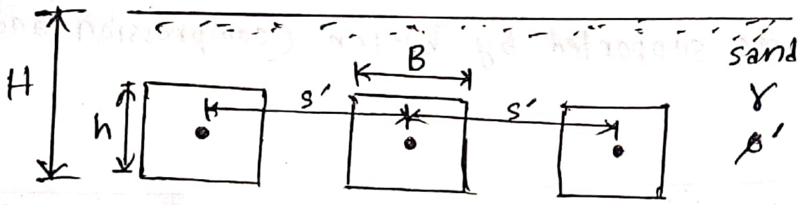
Empenical correlation Based on Model Tests:

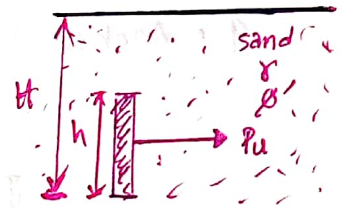
For Sand:

$$P_{ult} = \frac{5.4}{\tan\phi'}\left(\frac{H^2}{A}\right)^{0.28}\gamma A H \quad \text{where, } A = Bh$$

it is applicable for single anchors, For $\frac{S'}{B} = \infty$

Anchor behave as single plate, when, $\frac{S'}{B} \simeq 2$ (for all practical purposes)

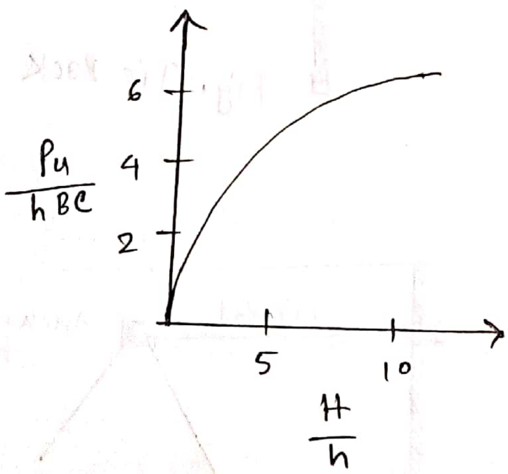




For saturated Clay:

$$F_c = \frac{P_u}{BhC} \quad \text{where, } F_c = \text{Breakout factor}$$

P_u = Ultimate resistance



[Based on Mackenzie (1955) and Tschebotarioff (1973)]

Factor of safety for Anchor plates:

The allowable resistance, $P_{all} = \frac{P_{ult}}{F_S}$

spacing of Anchor plates:

c/c spacing of anchor, $S' = \frac{P_{all}}{F}$

where F = force per unit length of sheet pile.

Ultimate resistance of Tie Back:

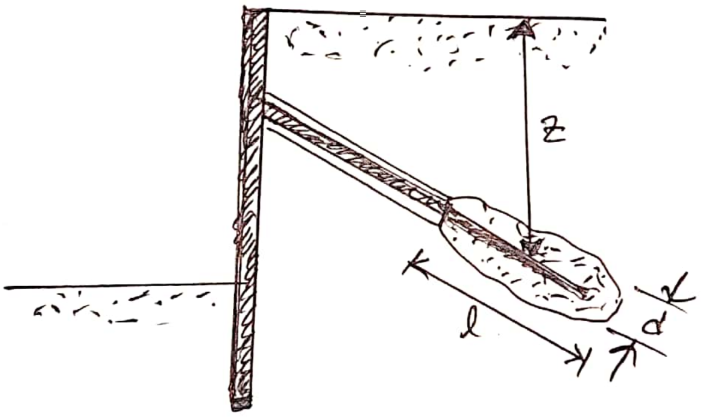
The ultimate resistance of tie Back is, (in sand)

$$P_{ult} = \pi d l \bar{\sigma_0}' K \tan \phi'$$

where, ϕ' = effective angle of friction of soil

$\bar{\sigma_0}'$ = Average effective vertical stress ($= \gamma z$ in dry sand)

K = earth pressure co-efficient



In clays, $P_{ult} = \pi d l C_a$ where,

$$C_a = \frac{2}{3} C_u$$

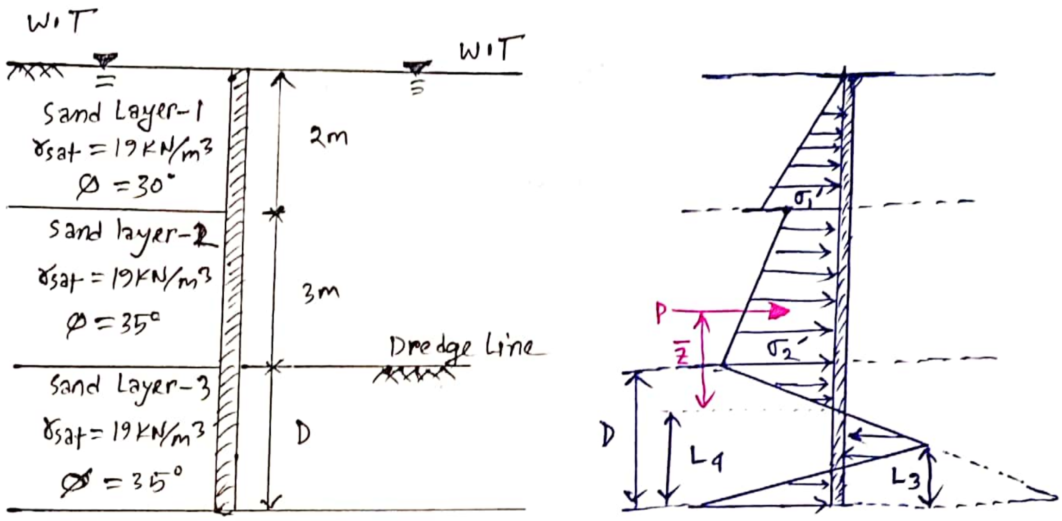
[C_u = undrained cohesion]

$$P_{all} = \frac{P_{ult}}{FS}$$ Here, $FS = (1.5 - 2.0)$

Sheet pile

2018

A sheet pile penetrating into sand is shown in figure below. compute the depth of embedment of sheet pile.



Solution: $K_{a_1} = \dfrac{1-\sin 30°}{1+\sin 30°} = \dfrac{1}{3}$ $\qquad \gamma' = (19-9.81) = 9.19\,KN/m^3$

$$K_{a_2} = \frac{1-\sin 35°}{1+\sin 35°} = 0.271$$

$$K_{p_1} = 3 \quad ; \quad K_{p_2} = \frac{1}{K_{a_2}} = \frac{1}{0.271} = 3.69$$

Now,

$$\sigma_1' = \gamma' L_1 K_{a_1} = 9.19 \times 2 \times \frac{1}{3} = 6.13\,KN/m^2$$

$$\sigma_2' = \gamma' L_1 K_{a_2} + \gamma' L_2 K_{a_2} = (\underbrace{9.19 \times 2 \times 0.271}_{4.98} + 9.19 \times 3 \times 0.271)\,KN/m^2$$

$$\therefore \sigma_2' = 12.45\,KN/m^2$$

$$L_3 = \frac{\sigma_2'}{\gamma'(K_{p_2} - K_{a_2})} = \frac{12.45}{9.19 \times (3.69 - 0.271)} = 0.396\,m$$

$$P = \frac{1}{2} \times 6.13 \times 2 + \frac{1}{2} \times (4.98 + 12.45) \times 3 + \frac{1}{2} \times 12.45 \times .396$$

$$= (6.13 + 26.145 + 2.4651) = 34.74\,KN/m$$

$$\bar{z} = \frac{6.13\left(0.396+3+\frac{2}{3}\right) + 4.98\times3\times\left(0.396+\frac{3}{3}\right) + \frac{1}{2}\times(12.45-4.98)\times3\times\left(0.396+\frac{3}{2}\right) + 2.4651\times\left(\frac{2}{3}\times0.396\right)}{34.74}$$

$= 1.9475\ m$

$\sigma_5' = (\gamma' L_1 + \gamma' L_2) K_{p_2} + \gamma' L_3 (K_{p_2} - K_{a_2})$

$= (9.19\times2 + 9.19\times3)\times3.69 + 9.19\times0.396\times(3.69-0.271)$

$= 181.998\ kN/m^2$

Now,

$$A_1 = \frac{\sigma_5'}{\gamma'(K_{p_2}-K_{a_2})} = \frac{181.998}{9.19\times(3.69-0.271)} = 5.7923$$

$$A_2 = \frac{8P}{\gamma'(K_{p_2}-K_{a_2})} = \frac{8\times34.74}{9.19\times(3.69-0.271)} = 8.845$$

$$A_3 = \frac{6P\left[2\gamma'\bar{z}(K_{p_2}-K_{a_2})+\sigma_5'\right]}{\gamma'^2(K_{p_2}-K_{a_2})^2} = \frac{6\times34.74\times\left[2\times9.19\times1.9475\times(3.69-0.271)+181.998\right]}{9.19^2\times(3.69-0.271)^2}$$

$\therefore A_3 = 64.2643$

$$A_4 = \frac{P(6\bar{z}\sigma_5' + 4P)}{\gamma'^2(K_{p_2}-K_{a_2})^2} = \frac{34.29\times(6\times1.9475\times181.998 + 4\times34.74)}{9.19^2\times(3.69-0.271)^2}$$

$\therefore A_4 = 78.69$

we know,

$L_4^4 + A_1 L_4^3 - A_2 L_4^2 - A_3 L_4 - A_4 = 0$

$\Rightarrow L_4^4 + 5.7923 L_4^3 - 8.845 L_4^2 - 64.2643 L_4 - 78.69 = 0$

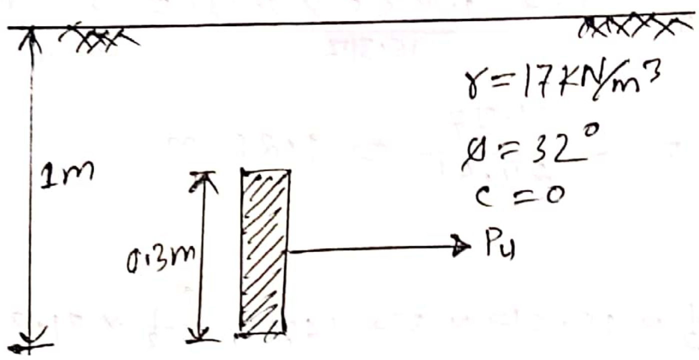
$\therefore L_4 = 3.55\ m$

$\therefore$ Depth of embedment $= (L_3 + L_4) = (0.396 + 3.55) = 3.946\ m$

(Ans.)

2018

A single anchor slab is shown in figure below. Calculate the ultimate holding capacity of the anchor slab if the width is (i) 0.3m (ii) 0.6m. center to center spacing, $S' = \alpha$.



Solution:

we Know,

$$P_u = \frac{5.4}{\tan^2\phi}\left(\frac{H^2}{A}\right)^{0.28} \gamma A H$$

Here, $A = Bh$ when, $B = 0.3m$, $A = 0.3 \times 0.3 = 0.09$

$B = 0.6m$, $A = 0.3 \times 0.6 = 0.18$

$B = 0.3m$, $$P_u = \frac{5.4}{\tan 32°} \times \left(\frac{1^2}{0.09}\right)^{0.28} \times 17 \times 0.09 \times 1$$

$$= 25.95 \text{ KN}$$

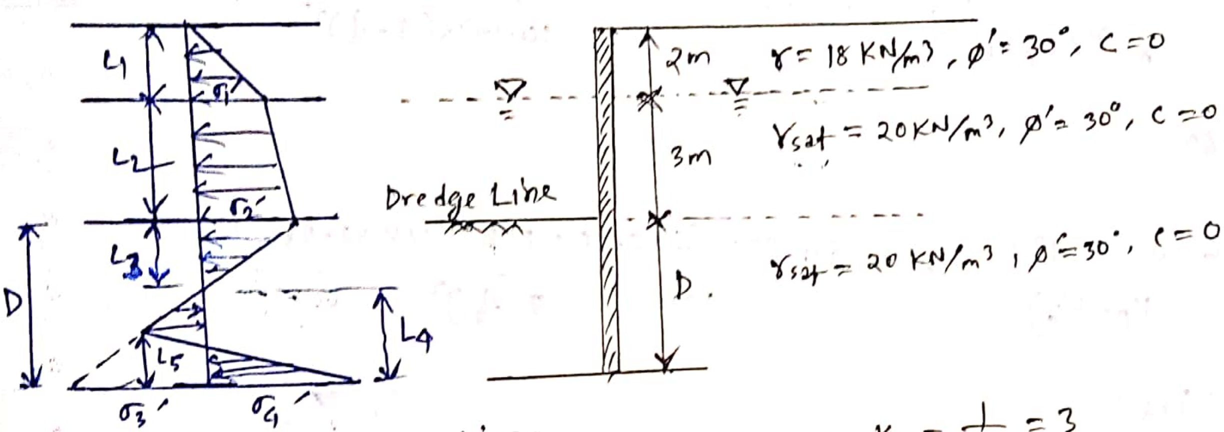
$B = 0.6m$, $$P_u = \frac{5.4}{\tan 32°} \times \left(\frac{1^2}{0.18}\right)^{0.28} \times 17 \times 0.18 \times 1$$

$$= 42.74 \text{ KN}$$

(Ans)

2017

An cantilever sheet pile is shown in figure. compute the depth of embedment of the sheet pile.



Solution: $K_a = \dfrac{1-\sin 30}{1+\sin 30} = 0.333$; $K_p = \dfrac{1}{K_a} = 3$

$\gamma' = (\gamma_{sat} - \gamma_w) = (20-9.81) = 10.19\ KN/m^3$

$\sigma_1' = \gamma L_1 K_a = 18 \times 2 \times 0.333 = 12\ KN/m^2$

$\sigma_2' = \sigma_1' + \gamma' L_2 K_a = 12 + 10.19 \times 3 \times 0.333 = 22.19\ KN/m^2$

$L_3 = \dfrac{\sigma_2'}{\gamma'(K_p - K_a)} = \dfrac{22.19}{10.19 \times (3-\frac{1}{3})} = 0.82\ m$

$P = \frac{1}{2} \times 12 \times 2 + \frac{1}{2} \times 10.19 \times 3 + 12 \times 3 + \frac{1}{2} \times 0.82 \times 22.19 = (12 + 15.285 + 36 + 9.098)$

$= 72.383\ KN/m$

$\bar{z} = \dfrac{12 \times (.82 + 3 + \frac{2}{3}) + 15.285\ (.82 + \frac{3}{3}) + 36 \times (0.82 + \frac{3}{2}) + 9.098 \times \frac{2}{3} \times 0.82}{72.383}$

$= \dfrac{170.152}{72.383} = 2.35\ m$

$\sigma_5' = (\gamma L_1 + \gamma' L_2) \times K_p + \gamma' L_3\ (K_p - K_a)$

$= (18 \times 2 + 10.19 \times 3) \times 3 + 10.19 \times 0.82 \times (3 - \frac{1}{3}) = 222\ KN/m^2$

Now,

$A_1 = \dfrac{\sigma_5'}{\gamma'(K_p - K_a)} = \dfrac{222}{10.19 \times (3-\frac{1}{3})} = 8.17$

$$A_2 = \frac{8P}{\gamma'(K_p - K_a)} = \frac{8 \times 72.383}{10.19 \times (3 - \frac{1}{3})} = 21.31$$

$$A_3 = \frac{6P \times [2\bar{z}\gamma'(K_p - K_a) + \sigma_5']}{\gamma'^2 (K_p - K_a)^2} = \frac{6 \times 72.383 \times [2 \times 2.35 \times 10.19 \times (3 - \frac{1}{3}) + 222]}{10.19^2 \times (3 - \frac{1}{3})^2}$$

$$\therefore A_3 = 205.69$$

$$A_4 = \frac{P(6\bar{z}\sigma_5' + 4P)}{\gamma'^2 (K_p - K_a)^2} = \frac{72.383 \times (6 \times 2.35 \times 222 + 4 \times 72.383)}{10.19^2 \times (3 - \frac{1}{3})^2}$$

$$\therefore A_4 = 335.23$$

Then,

$$L_4^4 + A_1 L_4^3 - A_2 L_4^2 - A_3 L_4 - A_4 = 0$$

$$\Rightarrow L_4^4 + 8.17 L_4^3 - 21.31 L_4^2 - 205.69 L_4 - 335.23 = 0$$

$$\therefore L_4 = 5.32 \text{ m}$$

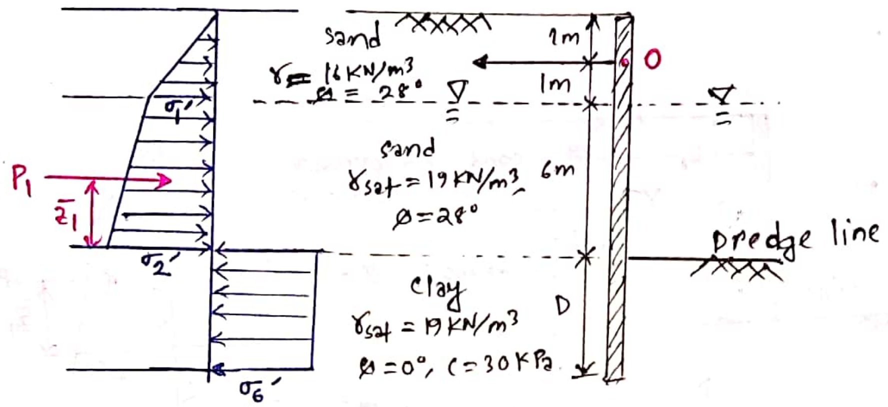
$\therefore$ Depth of embedment of the sheet pile

$$D = (L_3 + L_4) = (0.82 + 5.32) = 6.14 \text{ m}$$

(Ans.)

2016, 2009 (same type)

An anchor sheet pile bulkhead is shown in figure below. Use free earth method, Determine (i) Depth of embedment, D (ii) Anchor force per unit length of the sheet pile wall.



Solution:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 28}{1+\sin 28} = 0.36 \quad \text{and,} \quad K_p = \frac{1}{K_a} = 2.77$$

$$\gamma' = \gamma_{sat} - \gamma_w = (19 - 9.81) = 9.19\ KN/m^3$$

$$\sigma_1' = \gamma L_1 K_a = (16 \times 2 \times 0.36) = 11.52\ KN/m^2$$

$$\sigma_2' = (\gamma L_1 + \gamma' L_2) K_a = (16 \times 2 + 9.19 \times 6) \times 0.36 = 31.37\ KN/m^2$$

$$P_1 = \underbrace{\tfrac{1}{2} \times 11.52 \times 2}_{11.52} + \tfrac{1}{2} \times (11.52 + 31.37) \times 6 = 140.19\ KN/m$$

$$\bar{z}_1 = \frac{11.52 \times (6 + \frac{2}{3}) + \overset{(11.52\times6)}{69.12} \times (\frac{6}{2}) + \overset{0.5\times(31.37-11.52)\times6}{59.55} \times \frac{6}{3}}{140.19} = 2.88\ m$$

$$\sigma_6' = 4C - (\gamma L_1 + \gamma' L_2) = 4 \times 30 - (16 \times 2 + 9.19 \times 6) = 32.86\ KN/m^2$$

Now, we know, $\Sigma M_o = 0$ (সূত্র মুখস্থ না করে O point-এ Moment নিয়েও D বের করতে পারি)

$$\sigma_6 D^2 + 2\sigma_6 D (L_1 + L_2 - l_1) - 2P_1 (L_1 + L_2 - l_1 - \bar{z}_1) = 0$$

$$\Rightarrow 32.86 \times D^2 + 2 \times 32.86 \times D \times (2+6-1) - 2 \times 140.19 \times (2+6-1-2.88) = 0$$

$$\Rightarrow 32.86 D^2 + 460.04 D - 1155.1656 = 0$$

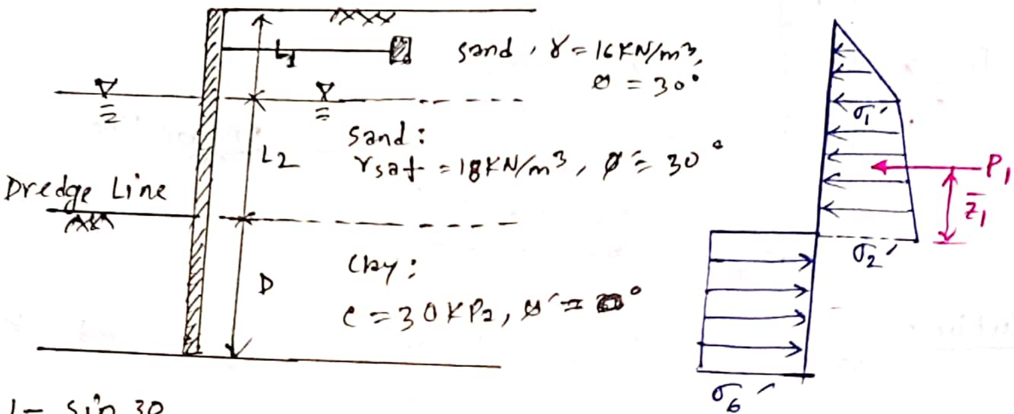
$\therefore D = 2.174\ m \simeq 2.2\ m$ $\quad \therefore$ Depth of embedment, $D = 2.2\ m$

Anchor force, $F = P_1 - \sigma_6 D = (140.19 - 32.86 \times 2.2) = 67.9\ KN/m$

(Ans.)

2015,

An anchor sheet pile wall is shown in figure below. Given $L_1 = 2m$, $L_2 = 6m$, $l_1 = 1m$, $\gamma = 16\ KN/m^2$, $\gamma_{sat} = 18\ KN/m^3$, $\phi' = 30°$ and $c = 30\ KPa$. Use free earth support method. (i) Determine the theoretical depth of embedment (ii) calculate the anchor force per unit length of the sheet pile.



Solution:

$$K_a = \frac{1 - \sin 30}{1 + \sin 30} = 0.333, \quad K_p = 3$$

$$\sigma_1' = \gamma L_1 K_a = (16 \times 2 \times 0.333) = 10.65\ KN/m^2$$

$$\sigma_2' = \sigma_1' + \gamma' L_2 K_a = 10.66 + (18 - 9.81) \times 6 \times 0.333 = 27.02\ KN/m^2$$

$$P_1 = \frac{1}{2} \times 10.66 \times 2 + \frac{1}{2} \times (27.02 - 10.66) \times 6 + 10.66 \times 6$$
$$= (10.66 + 49.02 + 63.96) = 123.64\ KN/m$$

$$\bar{z}_1 = \frac{10.66 \times (6 + \frac{2}{3}) + 49.02 \times \frac{6}{3} + 63.96 \times \frac{6}{2}}{123.64} = 2.92\ m$$

$$\sigma_6' = 4c - (\gamma L_1 + \gamma' L_2) = 4 \times 30 - [16 \times 2 + (18 - 9.81) \times 6]$$
$$= 38.86\ KN/m^2$$

We know, $\sigma_6' D^2 + 2\sigma_6 D (L_1 + L_2 - l_1) - 2P_1 (L_1 + L_2 - l_1 - \bar{z}_1) = 0$

$$\Rightarrow 38.86 D^2 + 2 \times 38.86 \times (2 + 6 - 1) \times D - 2 \times 123.64 (2 + 6 - 1 - 2.92) = 0$$

$$\Rightarrow 38.86 D^2 + 544.04 D - 1008.9024 = 0$$

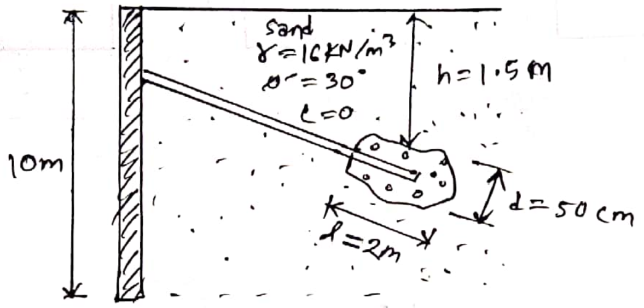
$\therefore D = 1.66m$ $\qquad$ $\therefore$ Depth of embedment $= 1.66m$

Anchor force, $F = P_1 - \sigma_6' D = (123.64 - 38.86 \times 1.66) = 59.13\ KN/m$

(Ans.)

2015

A Tie back is shown in figure below. Determine the ultimate resistance of the tiebacks.



Solution:

we know,

the ultimate resistance of tie back is,

$P_{ult} = \pi d l \bar{\sigma}_o' K \tan\phi'$

$= (3.1416 \times 0.5 \times 2 \times \frac{1}{2} \times 24 \times 0.58)$

$= 21.87$ KN

(Ans.)

Here,

$d = 50\,cm = 0.5\,m$

$l = 2m$

$\bar{\sigma}_o' = \gamma h = (16 \times 1.5) = 24\,KN/m^2$

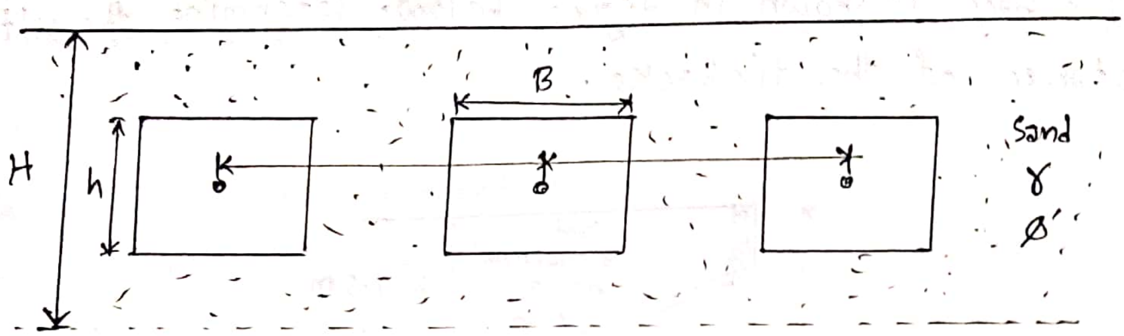
$\tan\phi = \tan 30 = 0.58$

$K = 1 - \sin 30 = 0.5$

2014 (Example-14.15)-(B.M Das- 8th ed)

The anchor plates are placed in a row with center to center spacing s' as shown in figure below,

Given, $B = h = 0.5\,m$, $s' = 1.5\,m$, $H = 1.0\,m$, $\gamma = 18\,KN/m^3$ and $\phi' = 30°$. Determine the ultimate resistance for each anchor plate. The anchor plates are made of concrete and have thickness of 0.15



Solution: we know,

The ultimate anchor resistance,

$$P_{ult} = \frac{5.4}{\tan\phi'} \times \left(\frac{H^2}{A}\right)^{0.28} \gamma A H$$

$$= \frac{5.4}{\tan 30^\circ} \times \left(\frac{1^2}{0.25}\right)^{0.28} \times 18 \times 0.25 \times 1.0$$

$$= 62.05 \text{ KN}$$

(Ans)

Here,

$A = Bh = (0.5 \times 0.5)$

$= 0.25 \ m^2$

$\gamma = 18 \ KN/m^3$

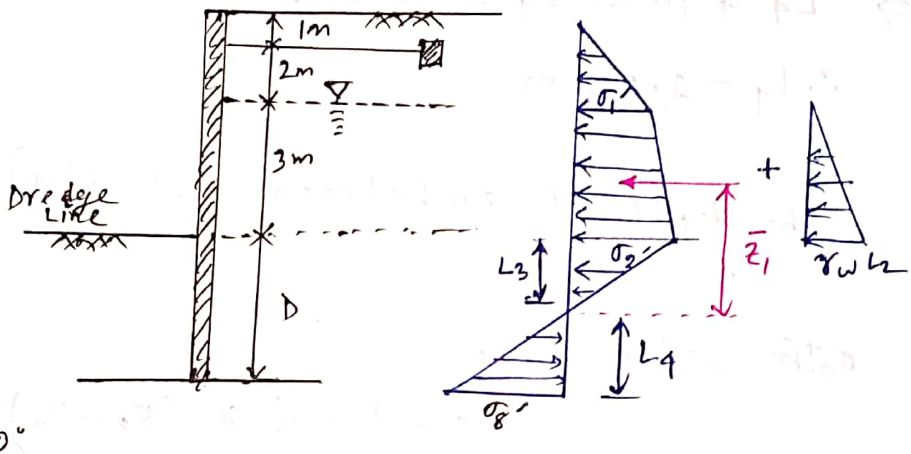
$H = 1.0 \ m$

$\phi' = 30^\circ$

2014

Compute the embedment length and the pull in the anchor rod for the sheet pile structure shown in properties:

$\phi = \phi' = 30°$, $c = 0$, $\gamma_{sat} = 20 KN/m^3$, $\gamma = 18 KN/m^3$. Use free earth support method.



Solution: $K_a = \dfrac{1-\sin 30°}{1+\sin 30} = 0.333$; $K_p = \dfrac{1}{K_a} = 3$

$\gamma' = (\gamma_{sat} - \gamma_w) = (20-9.81) = 10.19\ KN/m^3$

$\gamma'(K_p - K_a) = 10.19 \times (3-0.333) = 27.177\ KN/m^3$

$\sigma_1' = \gamma L_1 K_a = 18 \times 3 \times 0.333 = 18\ KN/m^2$

$\sigma_2' = (\gamma' L_2 K_a + \sigma_1' + \gamma_w L_2) = (10.19 \times 3 \times 0.333 + 18 + \underbrace{9.81 \times 3}_{29.43})\ KN/m^2$

$\therefore \sigma_2' = 57.62\ KN/m^2$

$L_3 = \dfrac{\sigma_2'}{\gamma'(K_p - K_a)} = \dfrac{57.62}{27.177} = 2.12$

$P = \frac{1}{2} \times 18 \times 3 + \frac{1}{2} \times 10.19 \times 3 + 18 \times 3 + \frac{1}{2} \times 29.43 \times 3 + \frac{1}{2} \times 57.62 \times 2.12$

$= (27 + 15.285 + 54 + 44.145 + 61.0772) = 201.5072\ KN/m$

$\bar{z} = \dfrac{27 \times (2.12 + 3 + \frac{3}{3}) + 15.285 \times (2.12 + \frac{3}{3}) + 54 \times (2.12 + \frac{3}{2}) + 44.145 \times (2.12 + \frac{3}{3}) + 61.0772 \times \frac{2 \times 2.12}{3}}{201.5072}$

$= \dfrac{632.464}{201.5072} = 3.14\ m$

Scanned by CamScanner

$$L_4^3 + 1.5 L_4^2 (l_2 + L_2 + L_3) - \frac{3P[(L_1+L_2+L_3) - (\bar{z}+l_1)]}{\gamma'(K_p - K_a)} = 0$$

$$\Rightarrow L_4^3 + 1.5 \times (2+3+2.12) L_4^2 - \frac{3 \times 201.5072 \times (8.12 - 4.14)}{27.177} = 0$$

$$\Rightarrow L_4^3 + 10.68 L_4^2 - 88.5306 = 0$$

$\therefore L_4 = 2.584$ m

$\therefore$ The depth of embedment $= (L_3 + L_4) = (2.12 + 2.584) = 4.704$m

The achor force,

$$F = P - \frac{1}{2}\gamma'(K_p - K_a) L_4^2$$

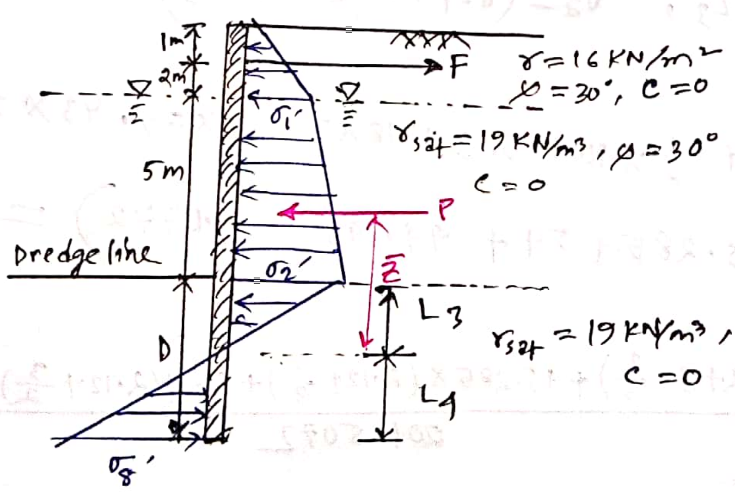
$$= 201.5072 - 0.5 \times 27.177 \times (2.584)^2$$

$$= 110.776 \text{ KN/m.}$$

(Ans.)

2013 $\boxed{2011 \text{ (same type)}}$ $\boxed{2012}$

An anchor sheet pile wall with a granular soil backfill and penetrating sand is shown. The net pressure distribution is shown as well. Determine the following using free earth support method: (i) P (ii) L_3 (iii) D (iv) F, (v) $\bar{z}$



Scanned by CamScanner

Solution: $K_a = \frac{1-\sin 30}{1+\sin 30} = 0.333$; $K_p = \frac{1}{K_a} = 3$

$$\gamma'(K_p - K_a) = (19 - 9.81) \times (3 - 0.333) = (9.19 \times 2.667) = 24.51 \ KN/m^3$$

$$\sigma_1' = \gamma L_1 K_a = 16 \times 3 \times 0.333 = 16$$

$$\sigma_2' = \sigma_1' + \gamma' L_2 K_a = 16 + \underbrace{9.19 \times 5 \times 0.333}_{15.317} = 31.317$$

$$L_3 = \frac{\sigma_2'}{\gamma'(K_p - K_a)} = \frac{31.317}{24.51} = 1.28 \ m$$

$$P = \tfrac{1}{2} \times 16 \times 3 + \tfrac{1}{2} \times 15.317 \times 5 + 16 \times 5 + \tfrac{1}{2} \times 31.317 \times 1.28$$
$$= (12 + 38.2925 + 80 + 20.043) = 150.34 \ KN/m$$

$$\bar{z} = \frac{12 \times (1.28 + 5 + \frac{3}{3}) + 38.2925 \times (1.28 + \frac{5}{3}) + 80 \times (1.28 + \frac{5}{2}) + 20.043 \times \frac{2}{3} \times 1.28}{150.34}$$

$$= \frac{519.7}{150.34} = 3.46 \ m$$

Now,

$$L_4^3 + 1.5 L_4^2 (l_2 + L_2 + L_3) - \frac{3P[(l_1 + L_2 + L_3) - (\bar{z} + l_1)]}{\gamma'(K_p - K_a)} = 0$$

$$\Rightarrow L_4^3 + 1.5 \times (2 + 5 + 1.28) L_4^2 - \frac{3 \times 150.34 \times (9.28 - 4.46)}{24.51} = 0$$

$$\Rightarrow L_4^3 + 12.42 L_4^2 - 88.695 = 0$$

$$\therefore L_4 = 2.443 \ m$$

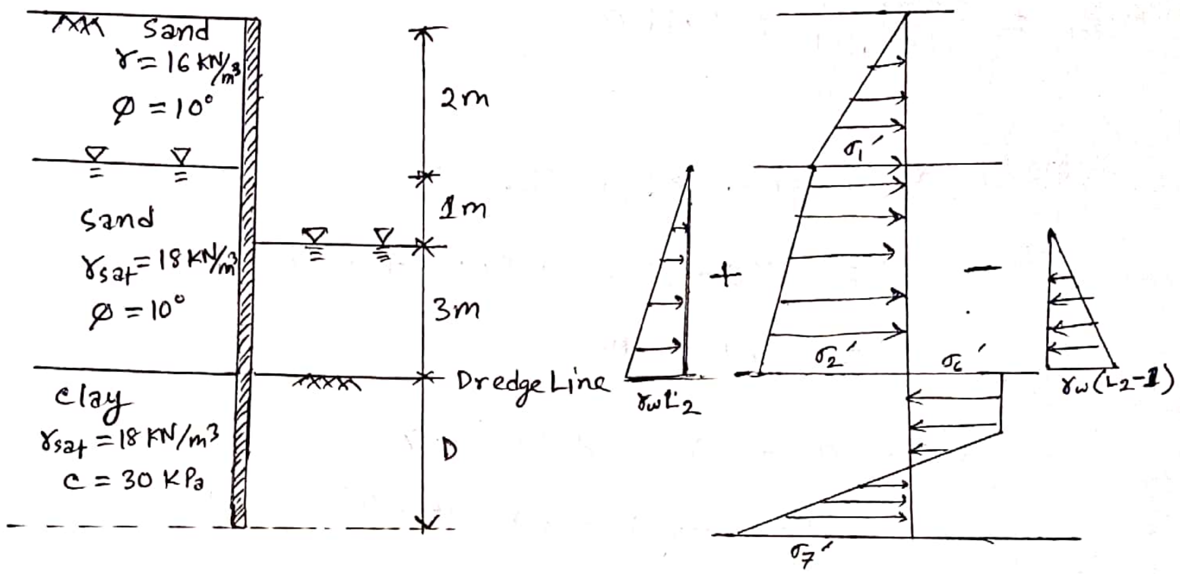
$\therefore$ The depth of Embedment, $D = (L_3 + L_4) = (1.28 + 2.443)$
$= 3.723 \ m$

The anchor force, $F = P - \tfrac{1}{2} \gamma'(K_p - K_a) L_4^2$
$= 150.34 - 0.5 \times 24.51 \times (2.443)^2 = 77.2 \ KN/m$

(Ans.)

2008

A cantilever sheet pile is shown in figure below. Determine the depth of penetration and size of the sheet pile section, if $\sigma_{all} = 170\ KN/m^2$.



Solution: $K_a = \dfrac{1 - \sin 10}{1 + \sin 10} = 0.704$

$$K_p = \frac{1}{K_a} = 1.42$$

$$\gamma' = \gamma_{sat} - \gamma_w = (18 - 9.81) = 8.19\ KN/m^3$$

$$\sigma_1' = K_a \gamma L_1 = 0.704 \times 16 \times 2 = 22.528\ KN/m^2$$

$$\sigma_2' = \sigma_1' + K_a \gamma' L_2 + L_2 \gamma_w - (L_2 - 1)\gamma_w$$

$$= 22.528 + 0.704 \times 8.19 \times 4 + 4 \times 9.81 - 3 \times 9.81$$

$$= 22.528 + 23.063 + 39.24 - 29.43$$

$$= 55.4\ KN/m^2$$

Now,

$$P_1 = \frac{1}{2} \times 22.528 \times 2 + 22.528 \times 4 + \frac{1}{2} \times 23.063 \times 4 + \frac{1}{2} \times 39.24 \times 4 - \frac{1}{2} \times 29.43 \times 3$$

$$= 22.528 + 90.112 + 46.126 + 78.48 - 44.145$$

$$= 193.101\ KN/m$$

$$\bar{z}_1 = \frac{\Sigma M_E}{P_1}$$

$$= \frac{1}{193.101} \times \left[22.528 \times \left(4 + \frac{2}{3}\right) + 90.112 \times \frac{4}{2} + 46.126 \times \frac{4}{3} + 78.48 \times \frac{4}{3} - 44.145 \times \frac{3}{3} \right]$$

$$\bar{z}_1 = \frac{407.351}{193.101} = 2.11 \text{ m}$$

Now, $\sigma_6' = 4C - (\gamma L_1 + \gamma' L_2 + \gamma_w)$ $(4-3)\gamma_w$

$= 4 \times 30 - (16 \times 2 + 8.19 \times 4 + 9.81)$

$= 45.43 \text{ KN/m}^2$

$(4-3)\gamma_w$

$\sigma_7' = 4C + (\gamma L_1 + \gamma' L_2 + \gamma_w)$

$= 4 \times 30 + (16 \times 2 + 8.19 \times 4 + 9.81)$

$= 194.57$

$\Sigma F_H = 0$

$P_1 - \sigma_6' \times D + \frac{1}{2} L_4 (\sigma_6 + \sigma_7) = 0$

$\Rightarrow 193.101 - 45.43 D + \frac{1}{2} L_4 \times (45.43 + 194.57) = 0$

$\Rightarrow L_4 = \frac{45.43 D - 193.101}{120}$

Now,

$\Sigma M_B = 0$

$P_1 \times (D + \bar{z}_1) - \sigma_6 \times D \times \frac{D}{2} + \frac{1}{2} L_4 \times (\sigma_6 + \sigma_7) \times \frac{L_4}{3} = 0$

$\Rightarrow 193.101 D + 193.101 \times 2.11 - 45.43 \times \frac{D^2}{2} + \frac{1}{6} \times (45.43 + 194.57) \times L_4^2 = 0$

$\Rightarrow 193.101 D + 407.44 - 22.715 D^2 + 40 L_4^2 = 0$

$\Rightarrow 193.101 D + 407.44 - 22.715 D^2 + 40.0 \times \frac{(45.43 D - 193.101)^2}{(120)^2} = 0$

$\Rightarrow D = 11.19 \text{ m}$ (Ans.)

Braced Cut

Braced Cut:

An excavation supported by bracing system is Known as Braced Cut.

Why is Braced Cut:

i. Keep the sides of excavation stable.
ii. Ensure that deep excavation will not hamper the neighbour structures.
iii. Minimize Excavation Area.

Types of Braced Cut:

i. Sheet Pile Braced Cut.
ii. Wooden Board Braced cut.

Uses of Braced Cut:

i. Laying underground pipeline.
ii. construction of Bridge abutment.
iii. construction of Basement.
iv. Metro railway construction.
v. construction of subway tunnel.

2017, 2011, 2009

Components of Braced Cut:

i. Struts
ii. Wale
iii. Lagging
iv. Soldier Beam

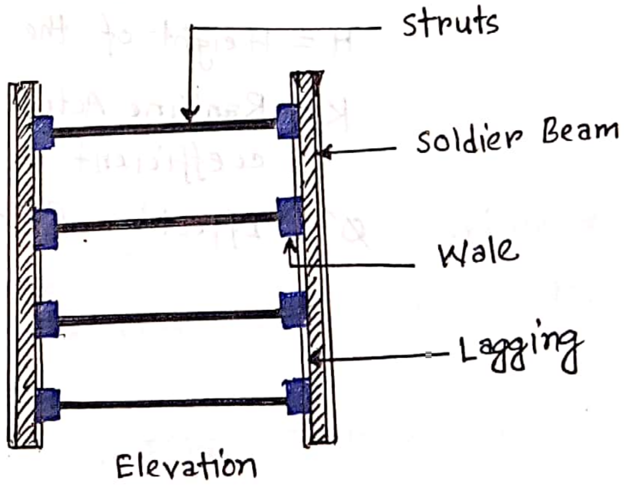


Figure: Components of Braced Cut

(i) Strut:

(a) A horizontal compression member.
(b) It is subjected to bending.
(c) The load bearing capacity of strut depends on their slenderness ratio.

(ii) **Wale:**

(a) continuous horizontal members.

(b) They are pinned at the struts.

(c) The maximum moments for the wale are,

$$M_{max} = \frac{Rs^2}{8}$$

where, R = strut Reaction per Length of wall
s = spacing of struts

(iii) **Lagging:**

(a) It is placed between soilder beam as the excavation proceed.

(b) It is a horizontal timber planks.

(iv) **Soilder Beam:**

(a) It is driven into ground before excavation.

(b) It is a vertical steel or timber beam.

Pressure Envelope For Braced Cut Design:

1. **Sandy Soil:**

For Sandy soil, Pressure σ_a is given by,

$$\sigma_a = 0.65 \gamma H K_a$$

Where, γ = unit weight of soil
H = Height of the cut
K_a = Rankine Active earth pressure coefficient $= \frac{1-\sin\phi'}{1+\sin\phi'}$
ϕ' = Effective friction angle

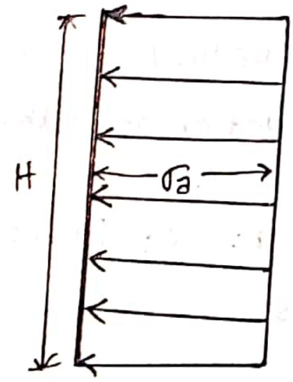


Fig. Peek's (1969) Apparent-Pressure envelope for cuts in sand

2. Clayey Soil:

(i) If $\frac{\gamma H}{c} > 4$, Soft to Medium clay is applied.

(ii) If $\frac{\gamma H}{c} \leqslant 4$, Stiff Clay is applied.

Here, γ = Unit weight of clay.

c = undrained cohesion. ($\phi = 0$)

(i) Soft to Medium Clay: Pressure is the largest of the followings:

(a) $\sigma_a = \gamma H\left[1 - \left(\frac{4c}{\gamma H}\right)\right]$ and,

(b) $\sigma_a = 0.3\gamma H$

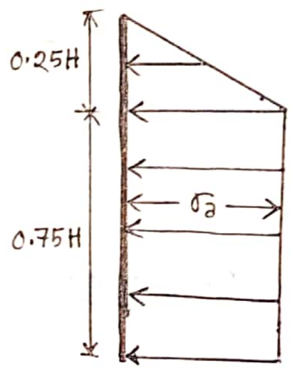


Figure. Peck's (1969)
Apparent pressure envelope for cuts in soft to medium clay

(ii) Stiff clay:

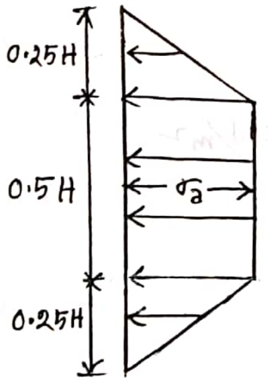


Figure. Peck's (1969)
Apparent pressure envelope for cuts in stiff clay

The pressure, σ_a is as follows:

$$\sigma_a = 0.2\gamma H \text{ to } 0.4\gamma H$$

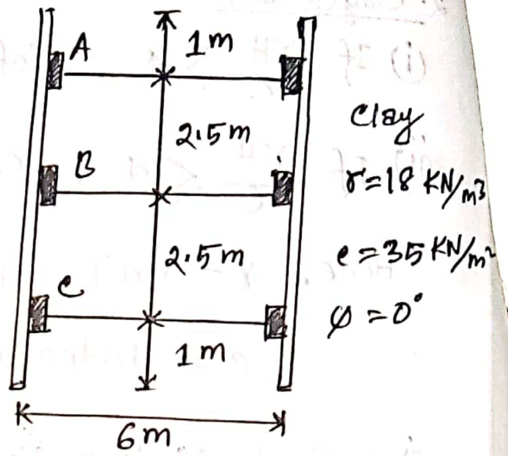
Average value of σ_a is used, $\sigma_a = 0.3\gamma H$

Precautions in using the Pressure Envelope:

1. They apply to excavations having depths greater than about 6 m ($\simeq$ 20 ft)
2. They are based on the assumption that the water table is below the bottom of the cut.
3. Sand is assumed to be drained with zero pore water pressure.
4. Clay is assumed to be undrained and pore water pressure is not considered.

Scanned by CamScanner

Problem: The cross section of a braced cut is shown in Figure:



(i) Draw the earth pressure envelope.

(ii) Determine the strut loads at level A, B and C

(iii) Determine the section modulus of the sheet pile section required.

(iv) Determine a design section modulus for the wales at B.

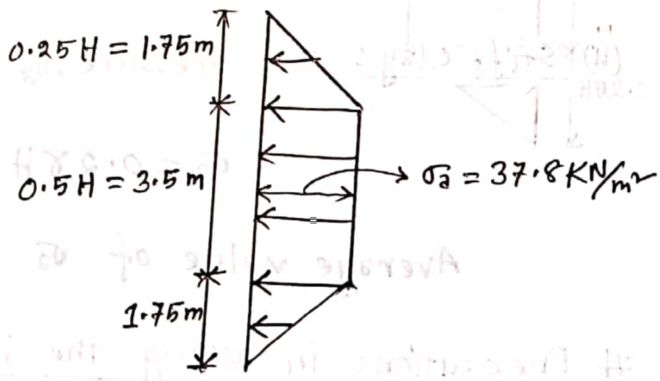
The struts are placed at 3 m, center to center, in the plan. Use $\sigma_{all} = 170\ MN/m^2$

Solution:

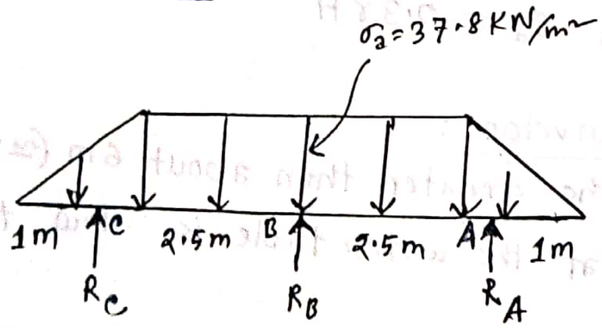
(i) Earth Pressure Envelope: $\frac{\gamma H}{c} = \frac{18 \times 7}{35} = 3.6 < 4$

so, stiff clay is applied.

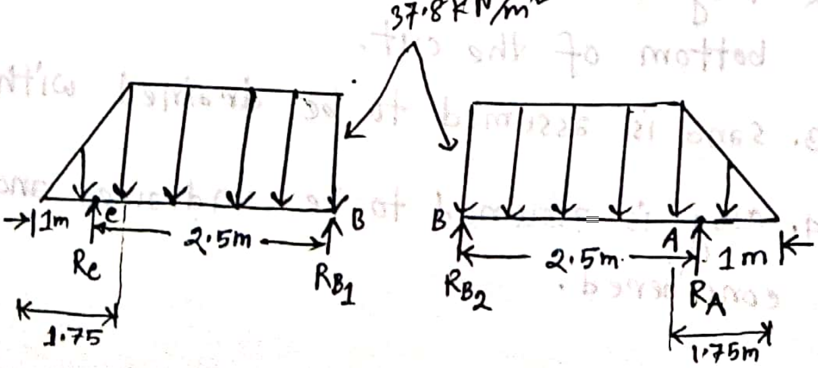
The pressure, $\sigma_a = 0.3 \gamma H = 0.3 \times 18 \times 7 = 37.8\ KN/m^2$



(ii) Strut Loads:



Assume Hinge at Point B.



$\Sigma M_B = 0$

$37.8 \times (2.5-0.75) \times \frac{1}{2} \times (2.5-0.75) + \frac{1}{2} \times 37.8 \times 1.75 \times \left[(2.5-0.75) + \frac{1}{3} \times 1.75\right] = R_c \times 2.5$

$\Rightarrow R_c = 54.0225 \ KN/m$

$\Sigma F_y = 0 \quad \therefore R_{B_1} = 45.2025 \ KN/m$

For symmetrical section, $R_{B_1} = R_{B_2} = 45.2025 \ KN/m$

and $R_c = R_A = 54.0225 \ KN/m$

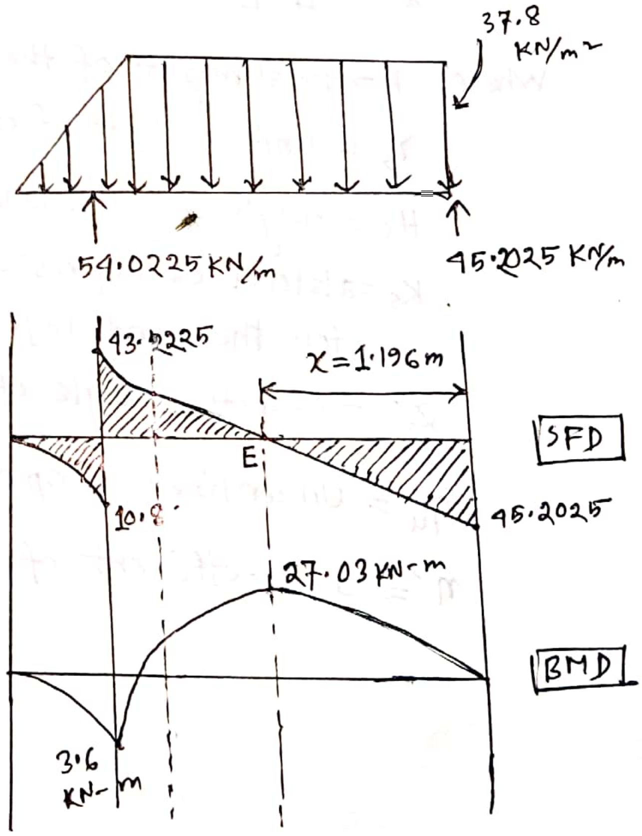
$\therefore R_B = R_{B_1} + R_{B_2} = (2 \times 45.2025) = 90.405 \ KN/m$

Struts Load at A, $P_A = R_A \times S = (54.0225 \times 3) = 162.0675 \ KN$

at B, $P_B = R_B \times S = (90.405 \times 3) = 271.215 \ KN$

at C, $P_c = R_c \times S = (54.0225 \times 3) = 162.0675 \ KN$

(iii) <u>section Modulus of sheet Pile</u>:



The location of Maximum moment,

$$x = \frac{45.2025}{37.8} = 1.196 \ m$$

Magnitude of the maximum Moment,

$M_{max} = \left(\frac{1}{2} \times 45.2025 \times 1.196\right) KN\text{-}m/m$

$= 27.03 \ KN\text{-}m$/meter of wall

$\therefore$ The section modulus of sheet pile,

$$S = \frac{M_{max}}{\sigma_{all}} = \frac{27.03}{170 \times 10^3} = 1.59 \times 10^{-4} \ m^3/m$$

(iv) Section modulus of wale at B:

Maximum moment for wale at B, $M_{max} = \frac{R_B s^2}{8}$

$$= \frac{90.405 \times 3^2}{8} \text{ KN-m}$$

$$= 101.71 \text{ KN-m}$$

$\therefore$ Secton modulus, $S = \frac{M_{max}}{\sigma_{all}}$

$$= \frac{101.71}{170 \times 10^3} = 5.983 \times 10^{-4} \text{ m}^3$$

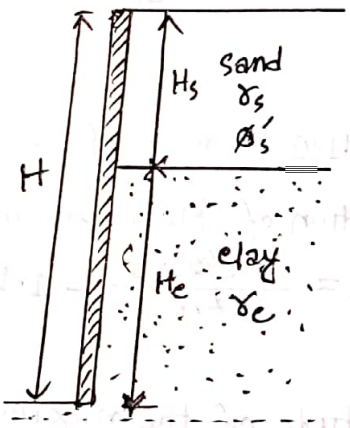
(Ans.)

Pressure Envelope for cuts in Layered Soil:

(a) Equivalent value of cohesion and unit weight (Peck, 1943)

$$c_{av} = \frac{1}{2H}\left[\gamma_s K_s H_s^2 \tan\phi_s' + (H-H_s)n' q_u\right]$$

$$\gamma_a = \frac{1}{H}\left[\gamma_s H_s + (H-H_s)\gamma_c\right]$$



Where, H = total height of the cut

γ_s = unit weight of sand

H_s = Height of the sand layer

K_s = a lateral earth pressure co-efficient for the sand layer ($\simeq 1$)

ϕ_s' = effective angle of friction of sand

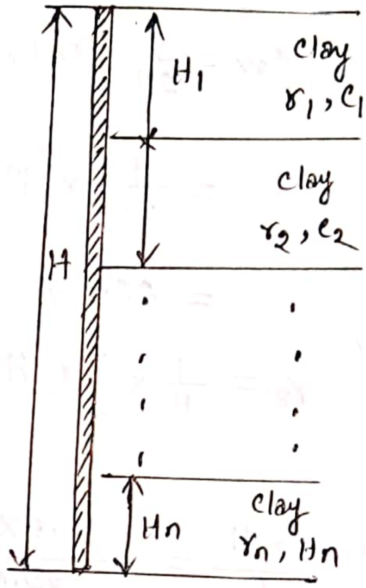
q_u = Unconfined compression strength of clay $= 2c_u$

η' = a co-efficient of progressive failure (ranging 0.5 to 1.0; average value 0.75)

(b) several clay layers in cut,

The average undrained cohesion becomes,

$$c_{av} = \frac{1}{H}\left(c_1 H_1 + c_2 H_2 + \cdots + c_n H_n\right)$$



Where,

$c_1, c_2, \ldots, c_n$ = Undrained cohesion in layers $1, 2, \ldots, n$

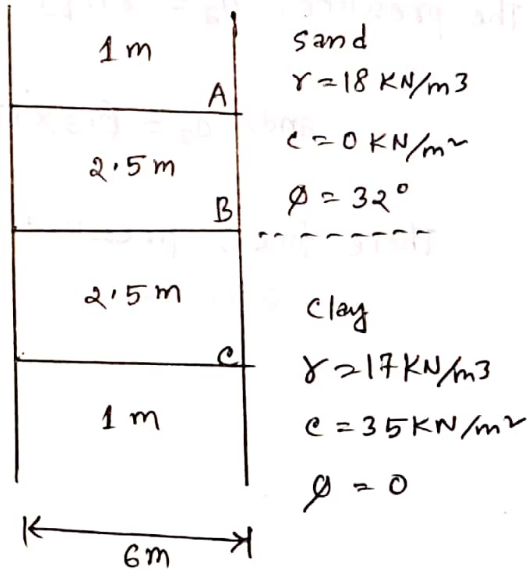
$H_1, H_2, \ldots, H_n$ = Thickness of layers $1, 2, \ldots, n$

The average unit weight,

$$\gamma_a = \frac{1}{H}\left(\gamma_1 H_1 + \gamma_2 H_2 + \cdots + \gamma_n H_n\right)$$

Problem:

The cross section of a braced cut is shown in Figure:



(i) Draw the earth pressure envelope

(ii) Determine the strut loads at level A, B and C

(iii) Determine the section modulus of the sheet pile section required.

(iv) Determine a design section modulus for the wales at B

The struts are placed at 3m, center to center, in the plan. Use $\sigma_{all} = 170$ MN/m²

Solution:

(i) Earth Pressure envelope:

$$c_{av} = \frac{1}{2H}\left[\gamma_s K_s H_s^2 \tan\phi_s' + (H-H_s)n' q_u\right]$$

$$= \frac{1}{2\times 7}\times\left[18\times \boxed{1} \times 3.5^2 \times \tan 32^\circ + (7-3.5)\times \boxed{0.75}\times(2\times 35)\right]$$

K_s

$$= 23.54 \text{ KN/m}^2$$

and,

$$\gamma_a = \frac{1}{H}\times\left[\gamma_s H_s + (H-H_s)\gamma_c\right] = \frac{1}{7}\times\left[18\times 3.5 + 3.5\times 17\right] = 17.5 \text{ KN/m}^3$$

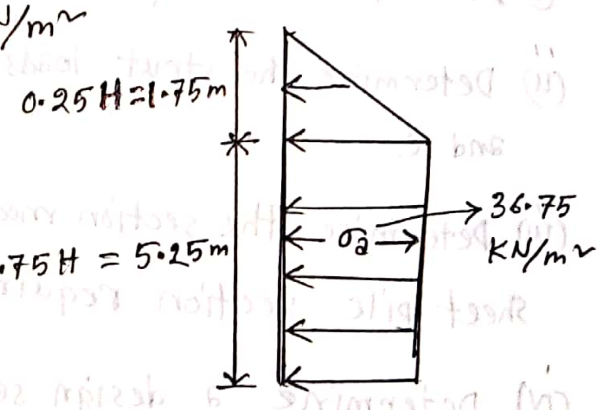
Now, $\frac{\gamma H}{c} = \frac{17.5\times 7}{23.54} = 5.204 > 4$

So, soft to Medium clay is applied

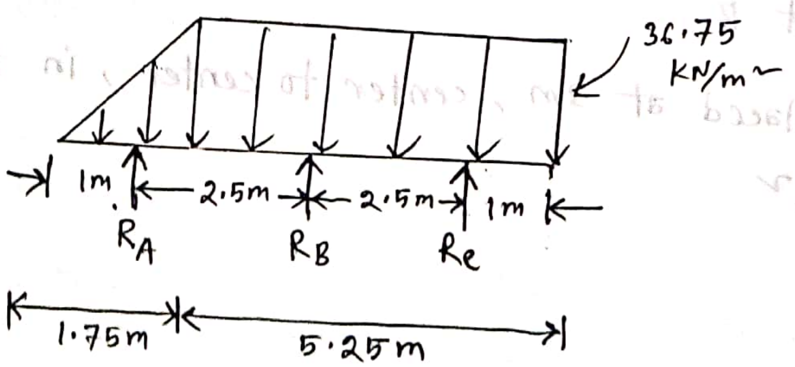
The pressure, $\sigma_a = \gamma H\left[1-\left(\frac{4c}{\gamma H}\right)\right] = 17.5\times 7\times\left[1-\frac{4\times 23.54}{17.5\times 7}\right] = 28.34 \text{ KN/m}^2$

and, $\sigma_a = (0.3\times 17.5\times 7) = 36.75 \text{ KN/m}^2$ (Larger value)

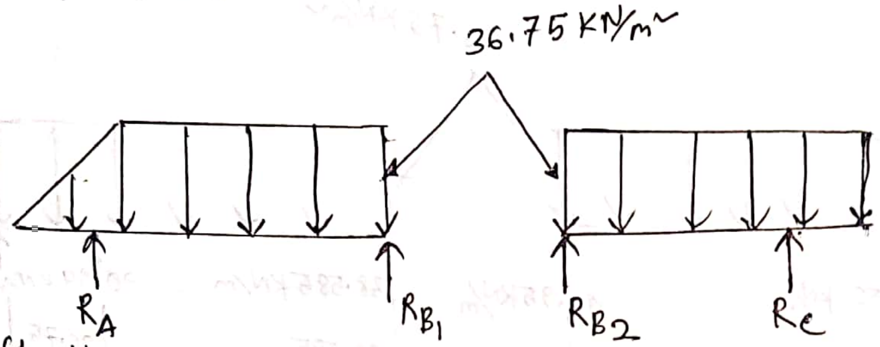
There fore, pressure will be 36.75 KN/m^2



(ii) Strut Loads:



Assume Hinge at B:



Taking Left side,

$\Sigma M_B = 0$

$$36.75 \times \frac{(2.5-.75)^2}{2} + \frac{1}{2} \times 36.75 \times 1.75 \times \left[(2.5-0.75) + \frac{1.75}{3}\right] = R_A \times 2.5$$

$\therefore R_A = 52.52$ KN/m

$\therefore R_{B_1} = 43.95$ KN/m

Taking Right side,

$\Sigma M_B = 0$

$$36.75 \times (2.5+1) \times \frac{1}{2} \times (2.5+1) = R_C \times 2.5$$

$\therefore R_C = 90.04$ KN/m

$\therefore R_{B_2} = 38.585$ KN/m

$\therefore R_B = R_{B_1} + R_{B_2} = (43.95 + 38.585) = 82.535$ KN/m

strut loads at A, $P_A = R_A \times S = (52.52 \times 3) = 157.56$ KN

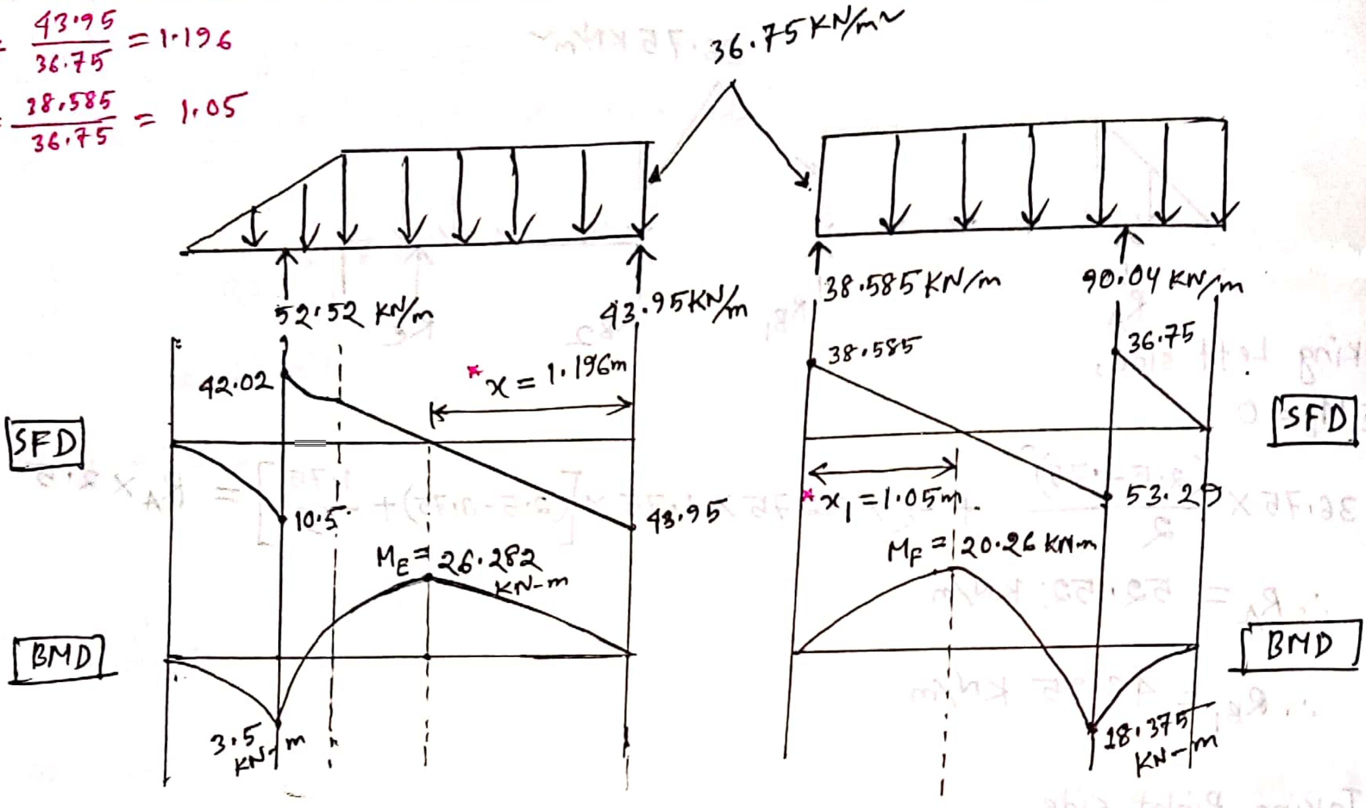
at B, $P_B = R_B \times S = (82.535 \times 3) = 247.605$ KN

at C, $P_C = R_C \times S = (90.04 \times 3) = 270.12$ KN

(iii) section modulus of sheet pile:

$*x = \frac{43.95}{36.75} = 1.196$

$*x_1 = \frac{38.585}{36.75} = 1.05$



Moment at E, $M_E = \left(\frac{1}{2} \times 43.95 \times 1.196\right) = 26.282$ KN-m/m

Moment at F, $M_F = \left(\frac{1}{2} \times 38.585 \times 1.05\right) = 20.26$ KN-m/m

$\therefore M_{max} = 26.282$ KN-m/m

$\therefore$ section modulus of sheet pile, $S = \frac{M_{max}}{\sigma_{all}} = \frac{26.282}{170 \times 10^3}$ m³/m

$= 1.546 \times 10^{-4}$ m³/m of wall

(iv) section modulus of wale at B:

Max. Moment for wale, $M_{max} = \frac{R_B s^2}{8} = \frac{82.535 \times 3^2}{8}$ KN-m

$= 92.852$ KN-m

$\therefore$ Section modulus, $S = \frac{M_{max}}{\sigma_{all}} = \frac{92.852}{170 \times 10^3} = 5.46 \times 10^{-4}$ m³/m

(Ans)

2015, 2012, 2010

Bottom Heaving of a cut in Clay: (Terzaghi, 1943)

Bottom cuts in clay may become unstable as a result of heaving of the bottom of the excavation.

The failure surface for such a case in a homogeneous soil is shown in Figure below:

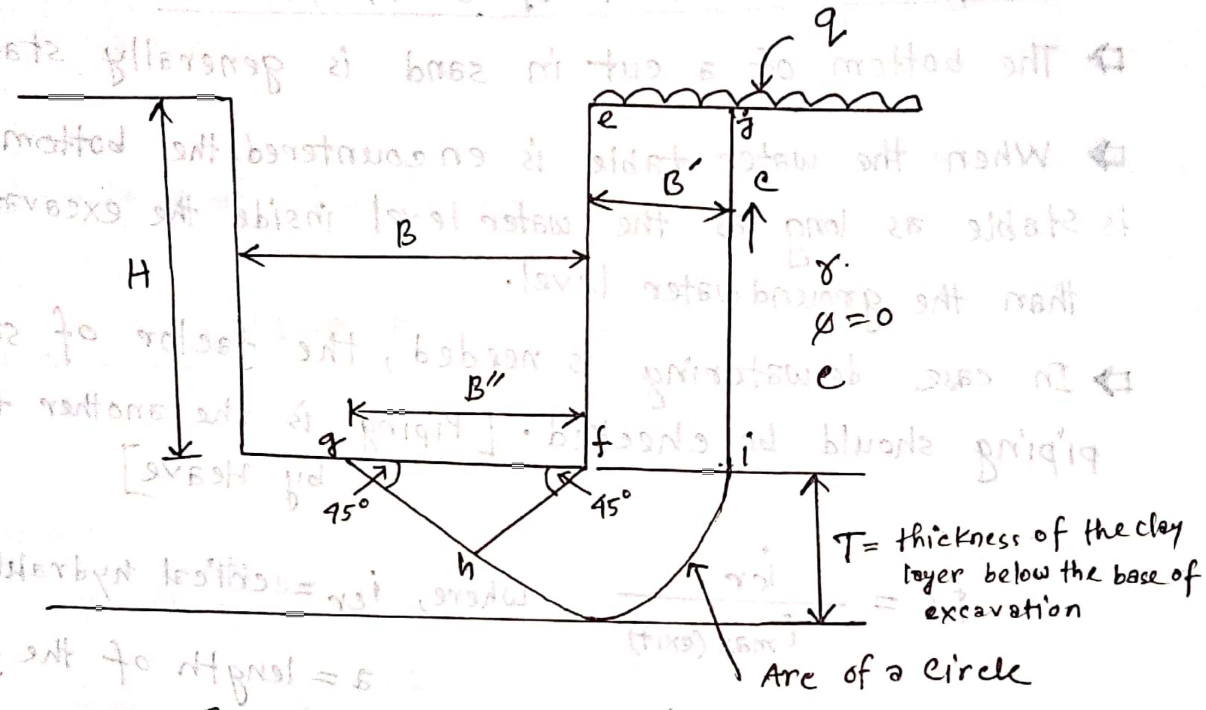


Fig. Heaving in Braced Cut in Clay

The Ultimate bearing capacity at the base of a soil column,

$q_{ult} = cN_c$ where, $c = 5.7$ (for a perfectly rough foundation)

The vertical load per unit area along fi is,

$$q_v = \gamma H + q - \frac{cH}{B'}$$

Hence, The factor of safety against bottom heave is,

$$FS = \frac{q_{ult}}{q_v} = \frac{cN_c}{\gamma H + q - \frac{cH}{B'}} = \frac{cN_c}{\left(\gamma + \frac{q}{H} - \frac{c}{B'}\right)H}$$

For excavations of limited length L, The factor of safety can be modified to,

$$FS = \frac{cN_c\left(1+0.2\times\frac{B'}{L}\right)}{\left(\gamma+\frac{q}{H}-\frac{c}{B'}\right)H}$$ where, $\left[B' = T \text{ or } \frac{B}{\sqrt{2}}\right]$ (which is smaller)

Modified Formula: Chang (2000) — see Example Problem

Stability of the bottom of a cut in Sand:

- The bottom of a cut in sand is generally stable.
- When the water table is encountered, the bottom of the cut is stable as long as the water level inside the excavation is higher than the ground water level.
- In case dewatering is needed, the factor of safety against piping should be checked. [Piping is the another term for failure by Heave]

$$FS = \frac{i_{cr}}{i_{max\,(exit)}}$$ where, i_{cr} = critical hydraulic gradient

a = length of the flow element

N_d = Number of drops

Here, $i_{cr} = \frac{G_s - 1}{e+1}$

and, $i_{max(exit)} = \frac{h/N_d}{a} = \frac{h}{aN_d}$

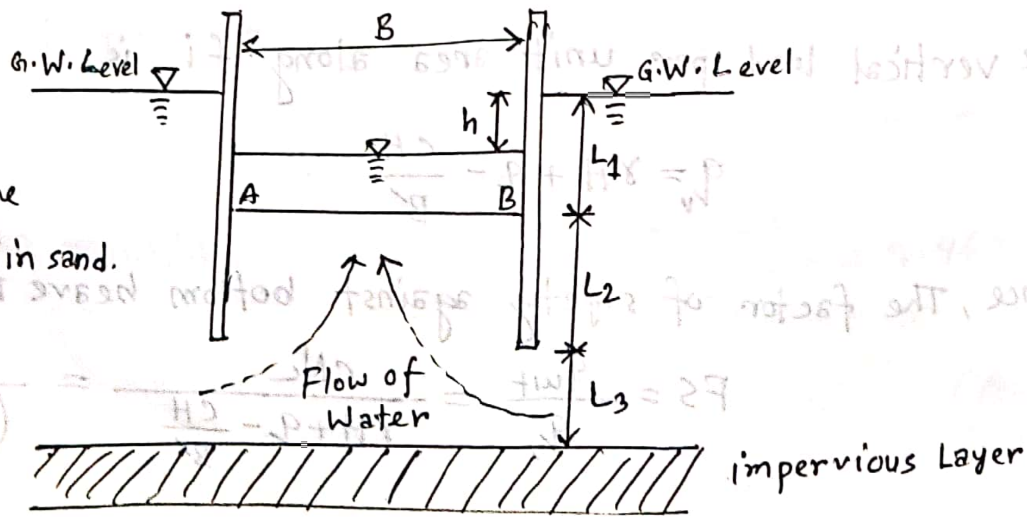


Fig. Stability of the bottom of a cut in sand.

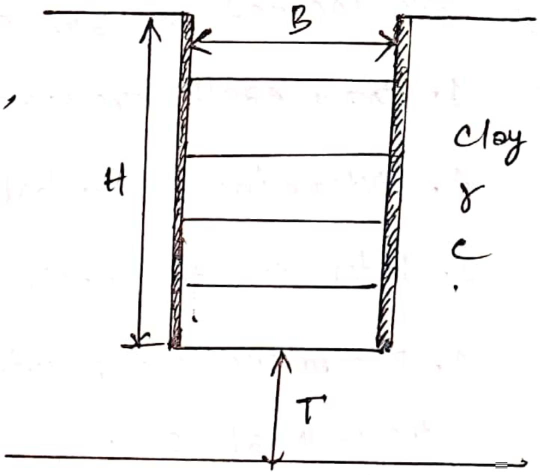
Problem:

For a braced cut in clay as shown in figure:

$L = 15\ m$, $q = 0$, $H = 6m$, $T = 2m$, $B = 4m$,

$\gamma = 17\ KN/m^3$, $c = 40\ KN/m^2$

Calculate the factor of safety against heave.



Solution:

Modified formula from chang (2000) we obtain,

$$FS = \frac{5.14c\left(1 + 0.2 \times \frac{B''}{L}\right) + \frac{cH}{B'}}{\gamma H + q}$$

Here,

$B = 4\ m$

$B' = T$ or $\frac{B}{\sqrt{2}}$ (smaller one)

$= 2m$ or $\frac{4}{\sqrt{2}} = 2.83\ m$

Hence $B' = 2\ m$

$\therefore B'' = \sqrt{2}B' = (\sqrt{2} \times 2) = 2.83\ m$

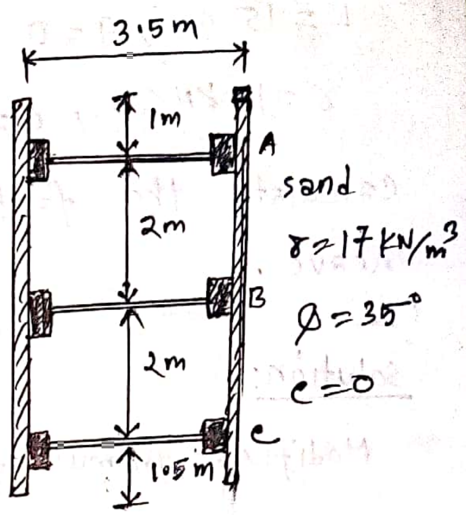
$$\therefore FS = \frac{40 \times 5.14 \times \left(1 + 0.2 \times \frac{2.83}{15}\right) + \frac{40 \times 6}{2}}{17 \times 6 + 0} = 2.55$$

(Ans.)

Problem: A braced cut is shown in figure below. The struts are located at 3m center to center in plan.

1. Draw earth pressure envelope
2. Determine struts load at levels A, B and C.
3. Determine the section modulus of sheet pile.
4. Determine the required section modulus of the wale at C.

Given, $\sigma_{all} = 170\ MN/m^2$

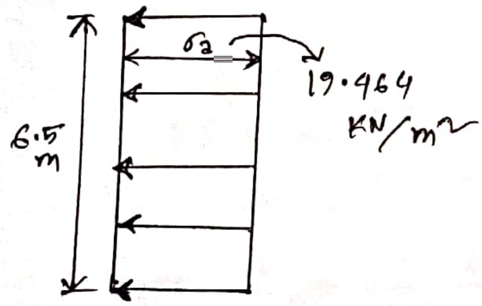


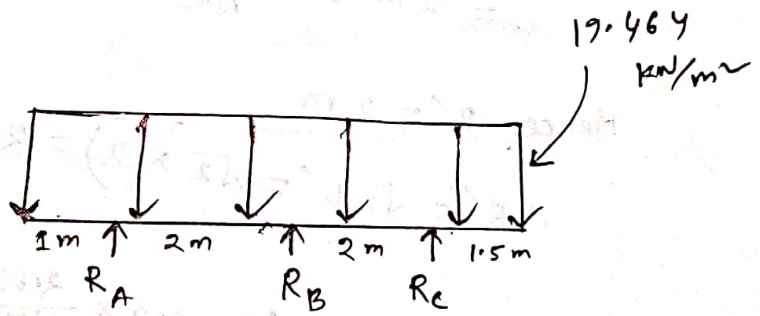
solution: (1) Earth pressure Envelope:

We know, The pressure, $\sigma_a = 0.65\ K_a \gamma H$

$$= 0.65 \times \frac{1 - \sin 35^\circ}{1 + \sin 35^\circ} \times 17 \times 6.5$$

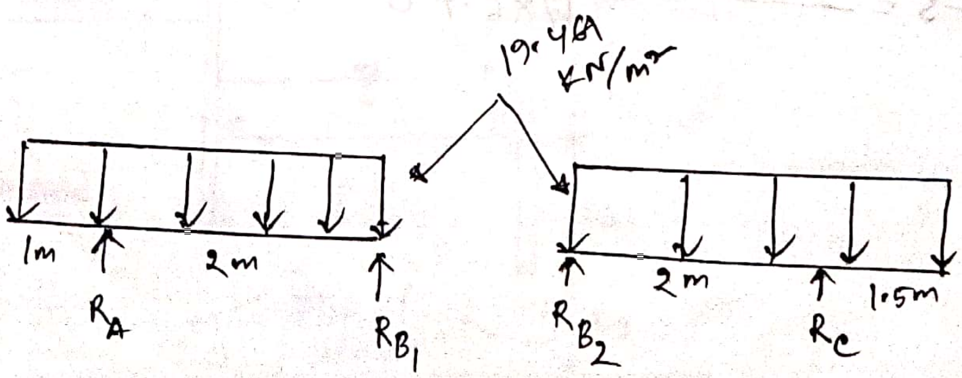
$$= 19.464\ KN/m^2$$





(2) Strut Loads:

Assume, Hinge at B



Taking left side,

$\Sigma M_B = 0$

$19.464 \times 3 \times 1.5 = R_A \times 2.0$

$\Rightarrow R_A = 43.794$ KN/m

$\therefore R_{B_1} = 14.598$ KN/m

Taking Right side,

$\Sigma M_B = 0$

$19.464 \times 3.5 \times \frac{3.5}{2} = R_C \times 2.0$

$\Rightarrow R_C = 59.6085$ KN/m

$\therefore R_{B_2} = 8.5155$ KN/m

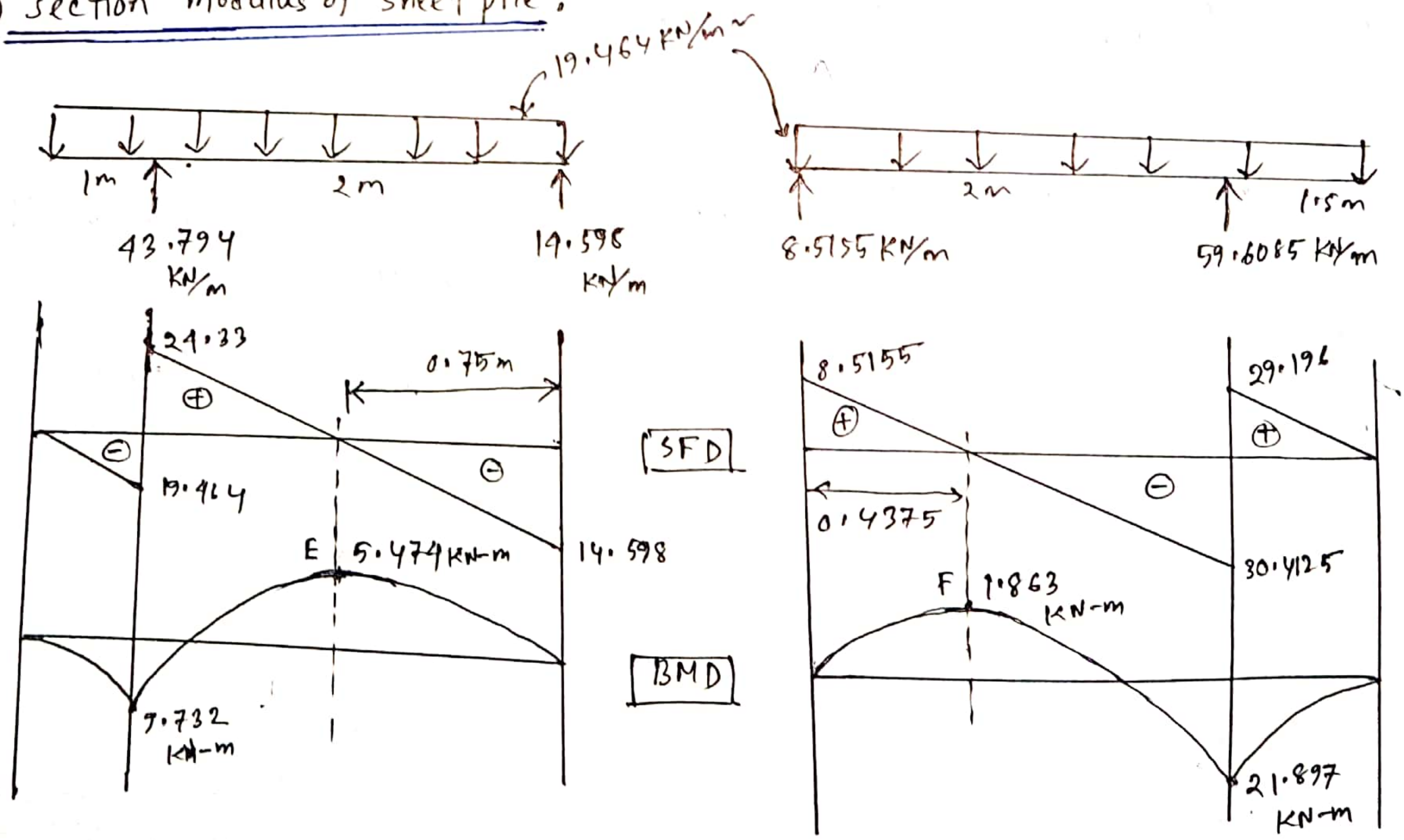
$\therefore R_B = (R_{B_1} + R_{B_2}) = 23.1135$ KN/m

strut load at A, $P_A = R_A \times S = (43.794 \times 3) = 131.382$ KN

at B, $P_B = R_B \times S = (23.1135 \times 3) = 69.3405$ KN

at C, $P_C = R_C \times S = (59.6085 \times 3) = 178.8255$ KN

(3) section modulus of sheet pile :



Moment at A = 9·732 KN-m/m

Moment at E = 5·479 KN-m/m

Moment at F = 1·863 KN-m/m

Moment at C = 21·897 KN-m/m

∴ Maximum moment, $M_{max} = 21·897$ KN-m/m

∴ Section modulus of sheet pile, $S = \dfrac{M_{max}}{\sigma_{all}}$

$= \dfrac{21·897}{170 \times 10^3} = 1·29 \times 10^{-4}$ m³/m of the wall

(4) **Section modulus of wale at c :**

At wale c, $M_{max} = \dfrac{R_c s^2}{8} = \dfrac{59·6085 \times 3^2}{8} = 67·06$ KN-m/m

∴ Section modulus,

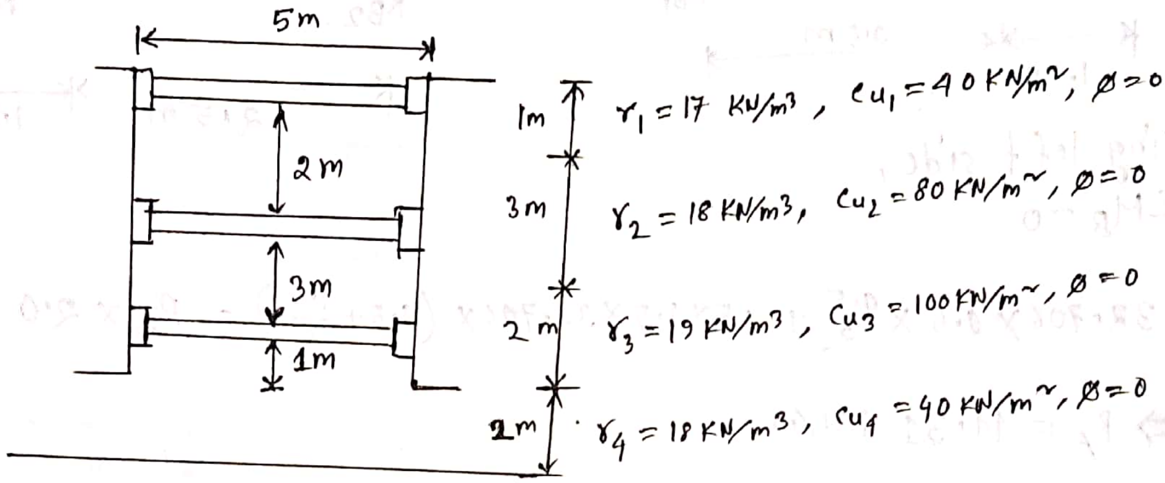
$S = \dfrac{M_{max}}{\sigma_{all}} = \dfrac{67·06}{170 \times 10^3} = 3·94 \times 10^{-4}$ m³/m

(Ans.)

Braced Cut

2018

\# For the braced cut shown in figure below, determine (i) strut loads (ii) section modulus of wale (iii) section modulus of sheet pile. The struts are placed 4m c/c in the plan. Use $\sigma_{all} = 180\ MN/m^2$.



Solution:

(i) Strut Loads:

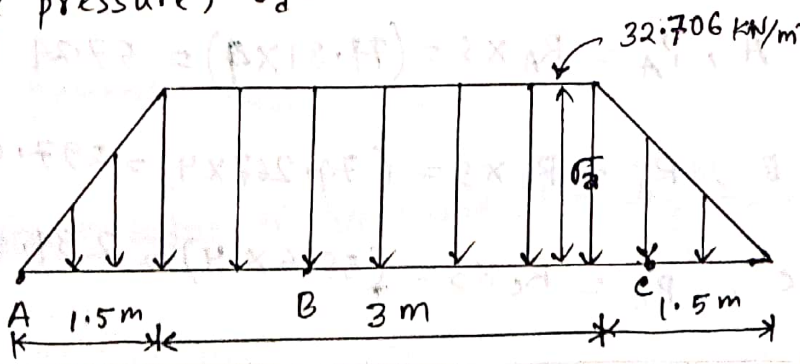
For layered clays, $c_{av} = \frac{1}{6} \times (40\times1 + 80\times3 + 100\times2) = 80\ KN/m^2$

$$\gamma_a = \frac{1}{6} \times (17\times1 + 18\times3 + 19\times2) = 18.17\ KN/m^3$$

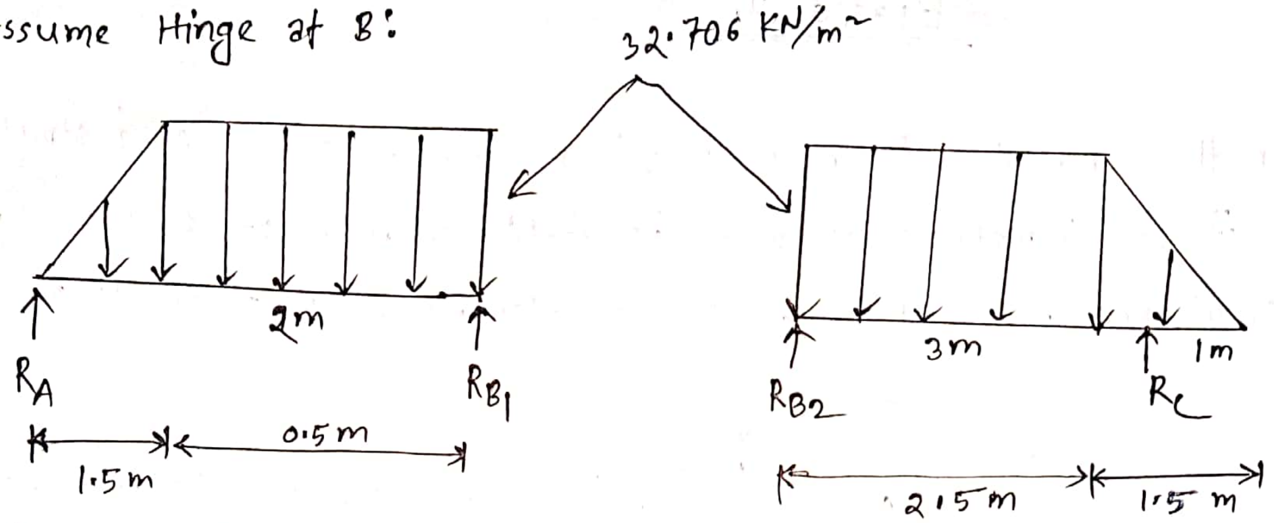
Now, $\frac{\gamma H}{c} = \frac{18.17\times6}{80} = 1.363\ m < 4m.$

So, stiff clay is applied.

The pressure, $\sigma_a = 0.3\gamma H = (0.3\times18.17\times6) = 32.706\ KN/m^2$



Assume Hinge at B:



Taking left side,

$\Sigma M_B = 0$

$32.706 \times 0.5 \times \frac{0.5}{2} + .5 \times 1.5 \times 32.706 \times \left(0.5 + \frac{1.5}{3}\right) = R_A \times 2.0$

$\Rightarrow R_A = 14.31$ KN/m

$\therefore R_{B_1} = 26.5725$ KN/m

Taking Right side,

$\Sigma M_B = 0$

$32.706 \times 2.5 \times \frac{2.5}{2} + .5 \times 1.5 \times 32.706 \times \left(2.5 + \frac{1.5}{3}\right) = R_C \times 3.0$

$\Rightarrow R_C = 58.6$ KN/m

$\therefore R_{B_2} = 47.6945$ KN/m

$\therefore R_B = R_{B_1} + R_{B_2} = (26.5725 + 47.6945) = 74.267$ KN/m

$\therefore$ strut load at A, $P_A = R_A \times S = (14.31 \times 4) = 57.24$ KN

at B, $P_B = R_B \times S = (74.267 \times 4) = 297.068$ KN

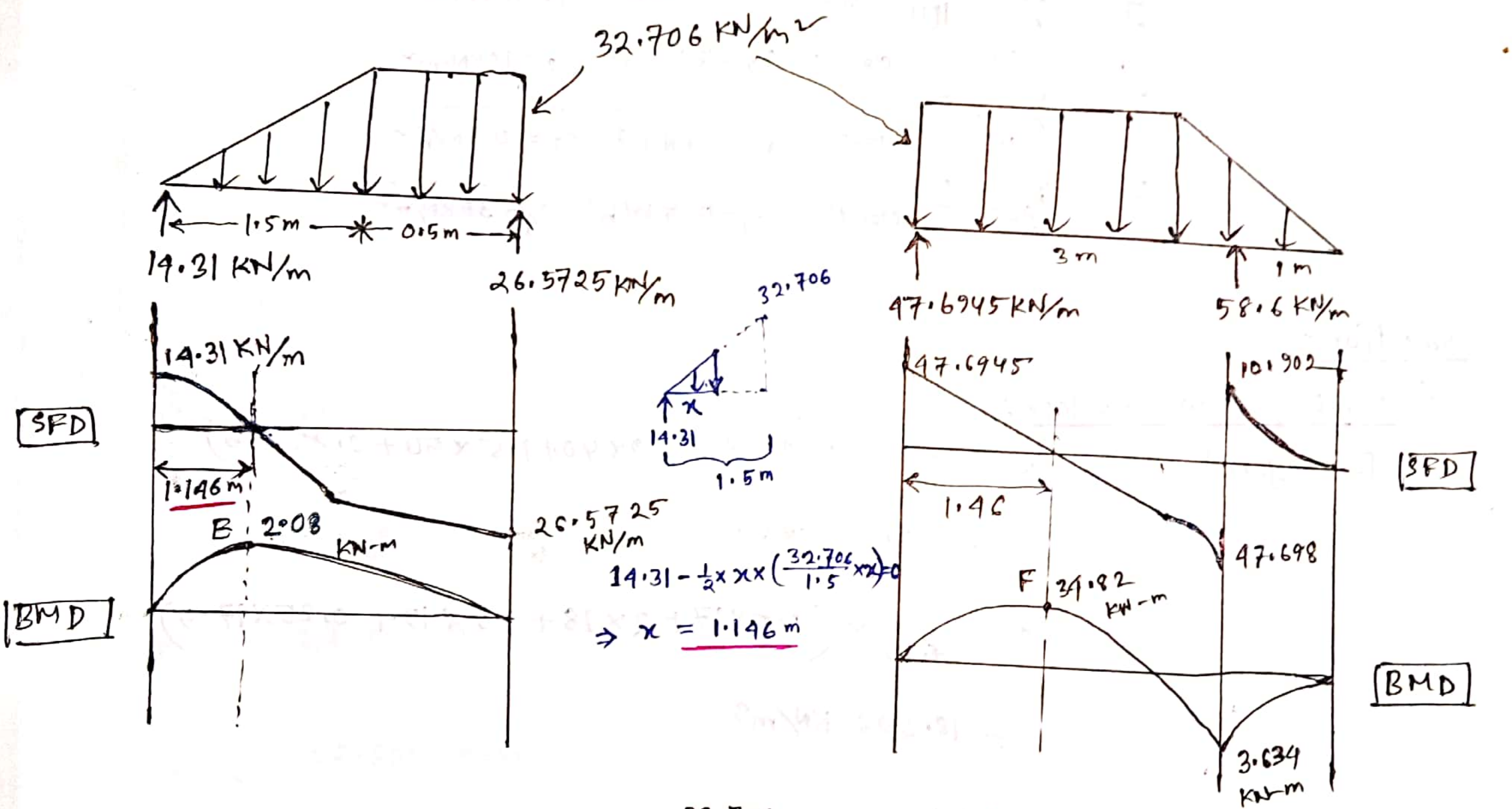
at C, $P_C = R_C \times S = (58.6 \times 4) = 234.4$ KN

(ii) section modulus of wale:

Max. moment for wale, $M_{max} = \frac{74.267 \times 4^2}{8} = 148.534$ KN-m/m

∴ section modulus, $S = \frac{M_{max}}{\sigma_{all}} = \frac{148.534}{180 \times 10^3} = 8.25 \times 10^{-4}$ m³/m

(iii) section modulus of sheet pile:



Moment at E $= -\frac{1}{2} \times 1.146 \times \left(\frac{32.706}{1.5} \times 1.146\right) \times \frac{1.146}{3} + 14.31 \times 1.146 = 2.08$ KN-m

Moment at F = 34.82 KN-m

Moment at e $= .5 \times 1.0 \times \left(\frac{32.706}{1.5} \times 1\right) \times \frac{1.0}{3} = 3.634$ KN-m

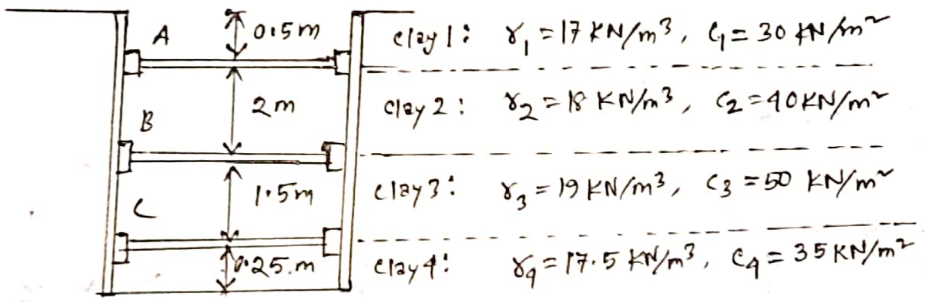
∴ Maximum Moment, $M_{max} = 34.82$ KN-m

∴ section modulus of sheet pile, $S = \frac{M_{max}}{S} = \frac{34.82}{180 \times 10^3} = 1.934 \times 10^{-4}$ m³/m of the wall.

(Ans.)

2017

A braced cut is shown in figure below. (i) Draw the earth pressure envelope, (ii) Determine the struts loads at A, B, C, (iii) Determine the section modulus of the sheet pile section required, and (iv) Determine the design section modulus for the wales at level B. Use $\sigma_{all} = 170$ MN/m^2. The struts are placed at 3 m c/c in the plan.



Solution:

(i) Earth pressure envelope:

For layered clays, $C_{av} = \frac{1}{4.25} \times (0.5 \times 30 + 2 \times 40 + 1.5 \times 50 + 0.25 \times 35)$

$= 42.06$ KN/m^2

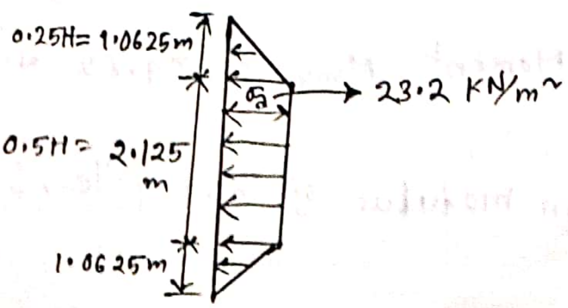
$\gamma_a = \frac{1}{4.25} \times (0.5 \times 17 + 2 \times 18 + 1.5 \times 19 + 0.25 \times 17.5)$

$= 18.206$ KN/m^3

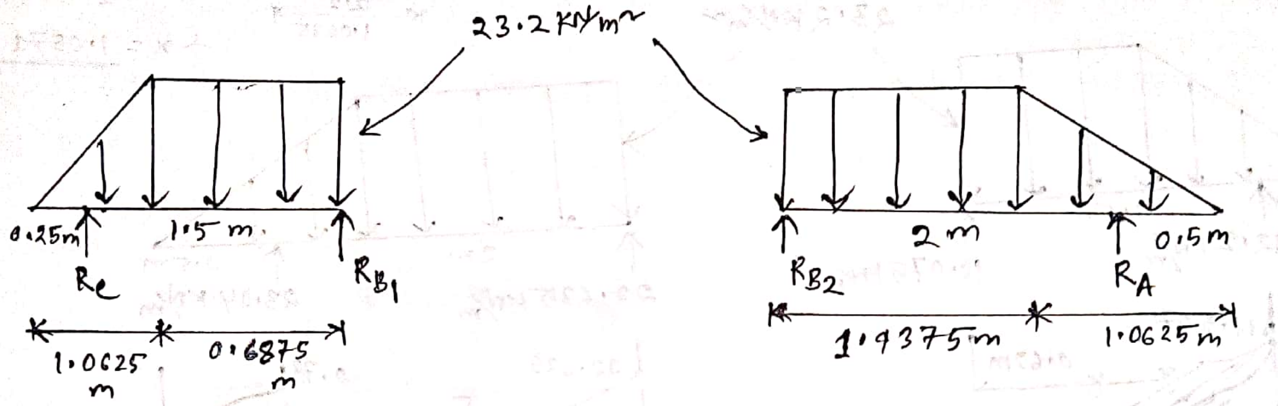
Now,

$\frac{\gamma H}{c} = \frac{18.206 \times 4.25}{42.06} = 1.84 < 4$; So, stiff clay is applied.

$\therefore$ The pressure, $P_a = 0.3 \gamma H = (0.3 \times 18.206 \times 4.25) = 23.2$ KN/m^2



Assume Hinge at B:



Taking left side,

$\Sigma M_B = 0$

$23.2 \times 0.6875 \times \frac{0.6875}{2} + \frac{1}{2} \times 1.0625 \times 23.2 \times \left(0.6875 + \frac{1.0625}{3}\right) = R_c \times 1.5$

$\Rightarrow R_c = 12.2$ KN/m

$\therefore R_{B_1} = 16.075$ KN/m

Taking Right side,

$\Sigma M_B = 0$

$23.2 \times \frac{1.4375^2}{2} + 0.5 \times 1.0625 \times 23.2 \times \left(1.4375 + \frac{1.0625}{3}\right) = R_A \times 2$

$\Rightarrow R_A = 23.04$ KN/m

$\therefore R_{B_2} = 22.635$ KN/m

$\therefore R_B = (R_{B_1} + R_{B_2}) = (16.075 + 22.635) = 38.71$ KN/m

(ii) Strut Loads:

$\therefore$ Strut loads at A, $P_A = R_A \times S = (23.04 \times 3) = 69.12$ KN

at B, $P_B = R_B \times S = (38.71 \times 3) = 116.13$ KN

at C, $P_c = R_c \times S = (12.2 \times 3) = 36.6$ KN

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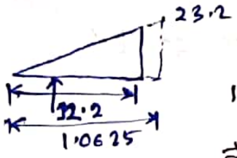
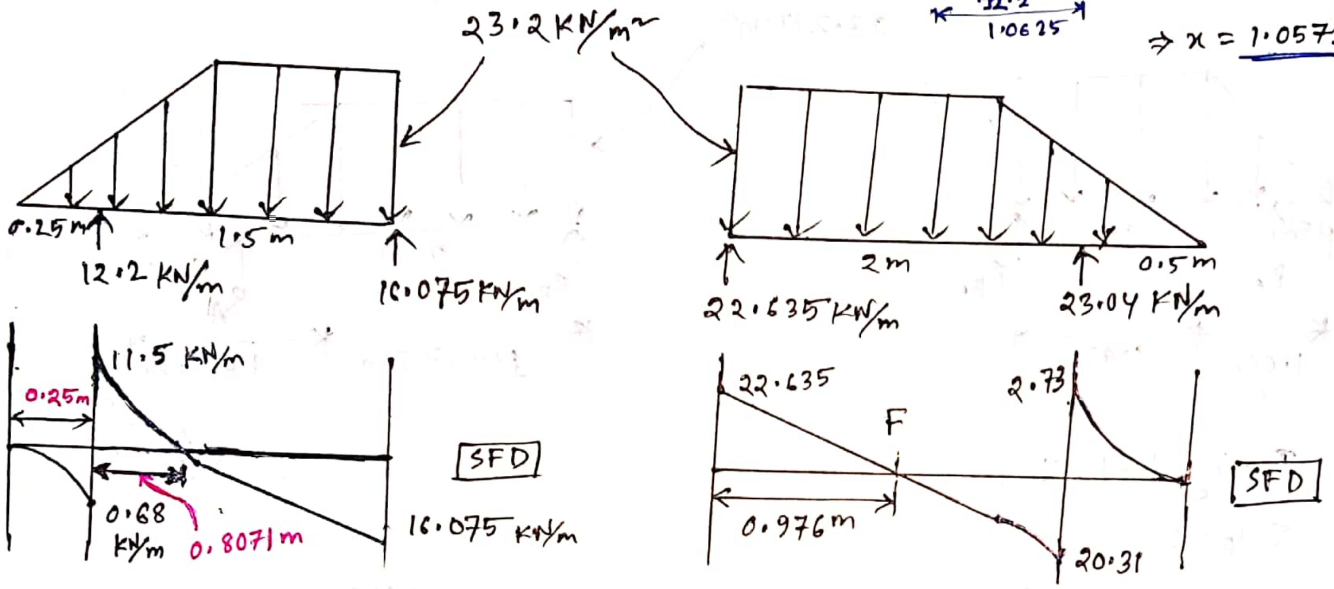
(iii) section modulus of sheet pile:



$12.2 - \frac{1}{2} \times x \times \left(\frac{23.2x}{1.0625}\right)$

$\Rightarrow x = 1.0571$ m



$$\text{Moment At } C = \frac{1}{2} \times 0.25 \times \left(\frac{23.2}{1.0625} \times 0.25\right) \times \frac{0.25}{3} = 0.057 \text{ KN-m/m}$$

$$\text{Moment at } E = -\frac{1}{2} \times 1.0571 \times \left(\frac{23.2}{1.0625} \times 1.0571\right) \times \frac{1.0571}{3} + 12.2 \times 0.8071 = 5.55 \text{ KN-m/m}$$

$$\text{Moment at } F = \frac{1}{2} \times 0.976 \times 22.635 = 11.046 \text{ KN-m/m}$$

$$\text{Moment at } A = \frac{1}{2} \times 0.5 \times \left(\frac{23.2}{1.0625} \times 0.5\right) \times \frac{0.5}{3} = 0.455 \text{ KN-m/m}$$

$\therefore M_{max} = 11.046$ KN-m/m

$\therefore$ section modulus of the sheet pile, $S = \frac{M_{max}}{\sigma_{all}} = \frac{11.046}{170 \times 10^3} = 6.5 \times 10^{-5}$ m³/m of the wall.

(iv) section modulus for wales at B:

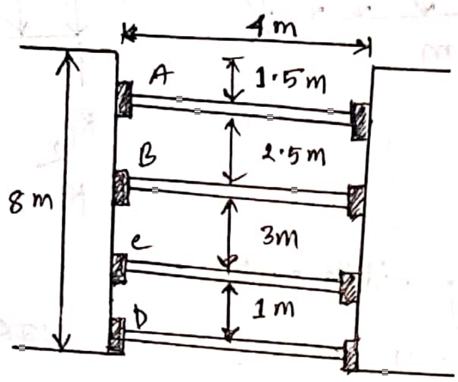
Maximum moment at B for wales $= \frac{R_B \times s^2}{8} = \frac{38.71 \times 3^2}{8} = 43.55$ KN-m/m

$\therefore$ Section modulus, $S = \frac{M_{max}}{\sigma_{all}} = \frac{43.55}{170 \times 10^3} = 2.56 \times 10^{-4}$ m³/m

(Ans)

2016

\# For the braced cut hown in figure below. Draw the earth pressure envelope. Determine the loads in struts A, B, C, D. The spacing of struts is 2.5 m. Take $\gamma = 18\ KN/m^3$ and $c = 30\ KN/m^3$



Solution:

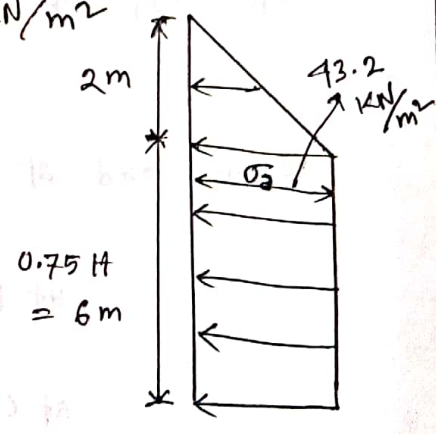
(i) Earth pressure envelope:

$$\frac{\gamma H}{c} = \frac{18 \times 8}{30} = 4.8 > 4$$; So, soft to medium clay is applied.

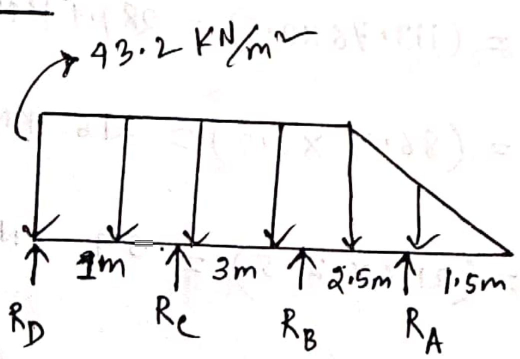
The pressure, $P_a = \gamma H \left(1 - \frac{4c}{\gamma H}\right) = 18 \times 8 \left(1 - \frac{4 \times 30}{18 \times 8}\right) = 24\ KN/m^2$

again $P_a = 0.3\,\gamma H = 0.3 \times 18 \times 8 = 43.2\ KN/m^2$

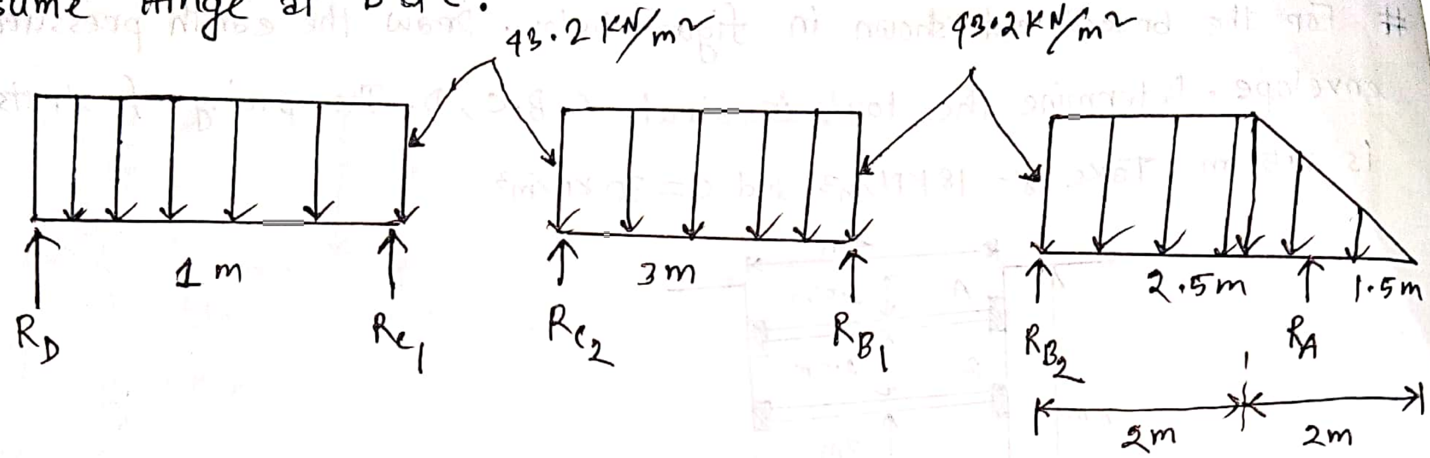
Therefore, the pressure will be $= 43.2\ KN/m^2$



(ii) strut loads:



Assume Hinge at B & C:



Taking left side,

$\Sigma M_D = 0$,

$43.2 \times 1 \times \frac{1}{2} = R_{C_1} \times 1$

$\Rightarrow R_{C_1} = 21.6$ KN/m

$\therefore R_D = 21.6$ KN/m

Taking middle part,

$\Sigma M_C = 0$

$43.2 \times 3 \times \frac{3}{2} = R_{B_1} \times 3$

$\Rightarrow R_{B_1} = 64.8$ KN/m

$\therefore R_{C_2} = 64.8$ KN/m

Taking Right side,

$\Sigma M_B = 0$

$43.2 \times 2 \times \frac{2}{2} + \frac{1}{2} \times 2 \times$
$43.2 \times (2 + \frac{2}{3}) = R_A \times 2.5$

$\Rightarrow R_A = 80.64$ KN/m

$\therefore R_{B_2} = 48.96$ KN/m

$\therefore R_C = R_{C_1} + R_{C_2} = (21.6 + 64.8) = 86.4$ KN/m

$R_B = R_{B_1} + R_{B_2} = (64.8 + 48.96) = 113.76$ KN/m

Strut load at A, $P_A = R_A \times S = (80.64 \times 2.5) = 201.6$ KN.

at B, $P_B = R_B \times S = (113.76 \times 2.5) = 284.4$ KN

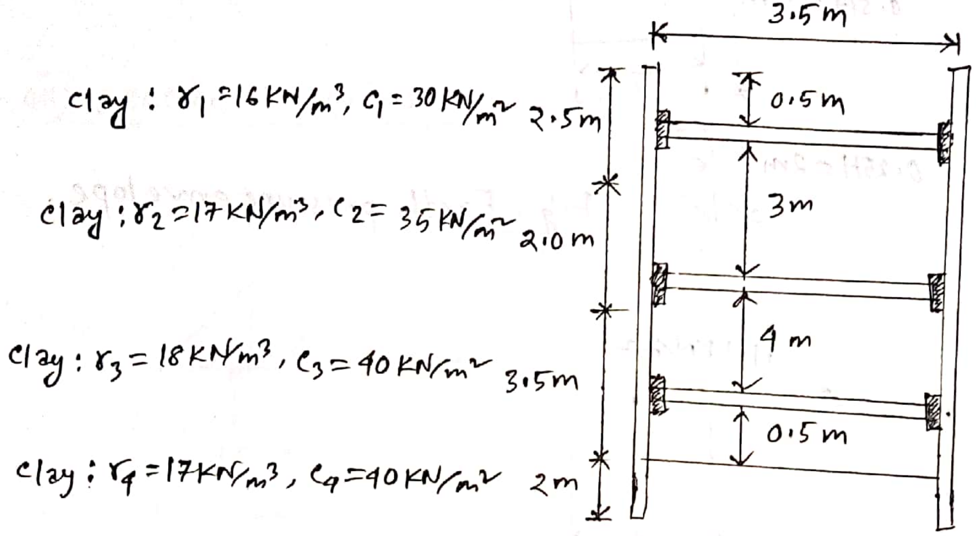
at C, $P_C = R_C \times S = (86.4 \times 2.5) = 216$ KN

at D, $P_D = R_D \times S = (21.6 \times 2.5) = 54$ KN

(Ans.)

2015

\# A braced cut is shown in figure below. Assuming, $\sigma_{all} = 170\ MN/m^2$.
(i) Draw the earth pressure envelop (ii) Determine the struts loads
(iii) Determine the section modulus of the sheet pile required. (iv) Determine the design section modulus for the wales at level B. The struts are placed at 3.5 m c/c in plan.



Solution:

For layered soil,

$$c_{av} = \frac{1}{H}(c_1 H_1 + c_2 H_2 + c_3 H_3)$$

$$= \frac{1}{8} \times (30 \times 2.5 + 35 \times 2 + 40 \times 3.5)$$

$$= 35.625\ KN/m^2$$

$$\gamma_{av} = \frac{1}{H} \times (\gamma_1 H_1 + \gamma_2 H_2 + \gamma_3 H)$$

$$= \frac{1}{8} \times (16 \times 2.5 + 17 \times 2 + 18 \times 3.5)$$

$$= 17.125\ KN/m^2$$

Now, $\frac{\gamma H}{c} = \frac{17.125 \times 8}{35.625} = 3.85 < 4$, Hence stiff clay is applied.

(i) Thus, $\sigma_a = 0.3\gamma H = 0.3 \times 17.125 \times 8 = 41.1\ KN/m^2$

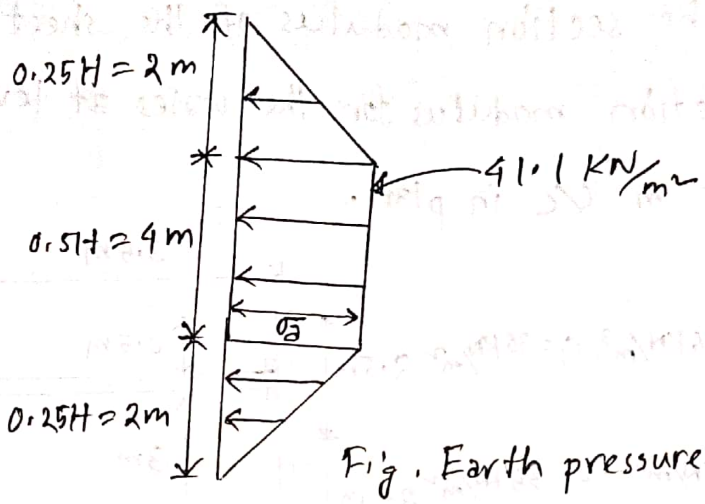
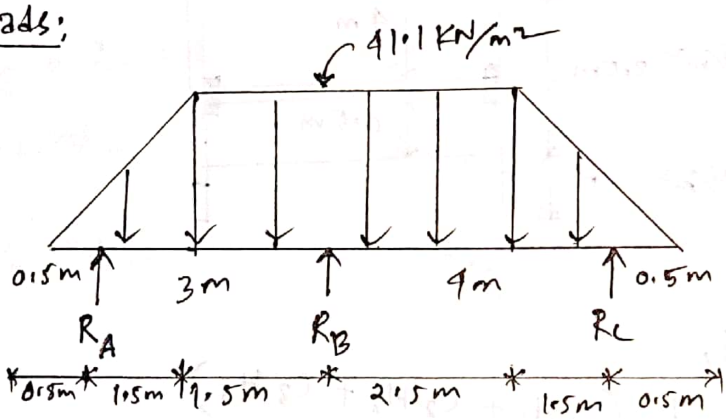
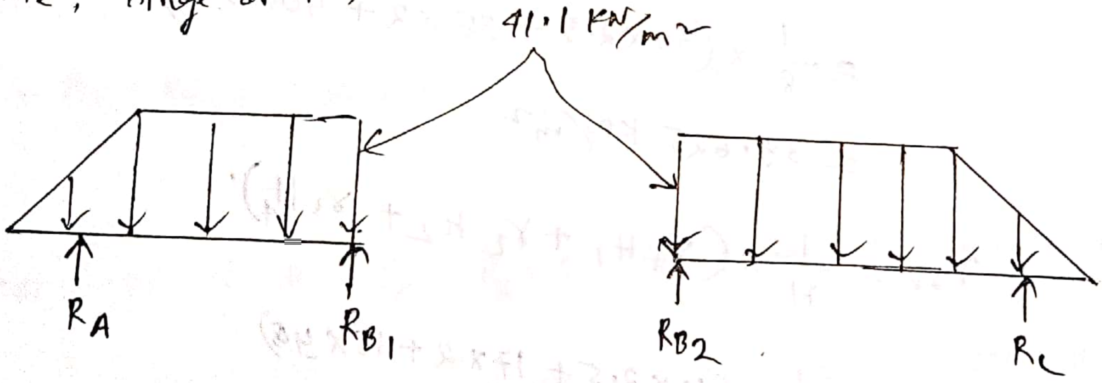


Fig. Earth pressure envelop.

(ii) <u>Struts loads:</u>



Assume, Hinge at B,



$\Sigma M_B = 0$

$3 \times R_A = 41.1 \times 1.5 \times 0.75 + \frac{1}{2} \times 2 \times 41.1 \times (1.5 + \frac{2}{3})$

$\therefore R_A = 45.096\ KN/m$

$\therefore R_{B1} = 57.654\ KN/m$

$\Sigma M_B = 0$

$R_C \times 4 = 41.1 \times 2.5 \times 1.25 + \frac{1}{2} \times 2 \times 41.1 \times (2.5 + \frac{2}{3})$

$\Rightarrow R_C = 64.65\ KN/m$

$\therefore R_{B2} = 79.2\ KN/m$

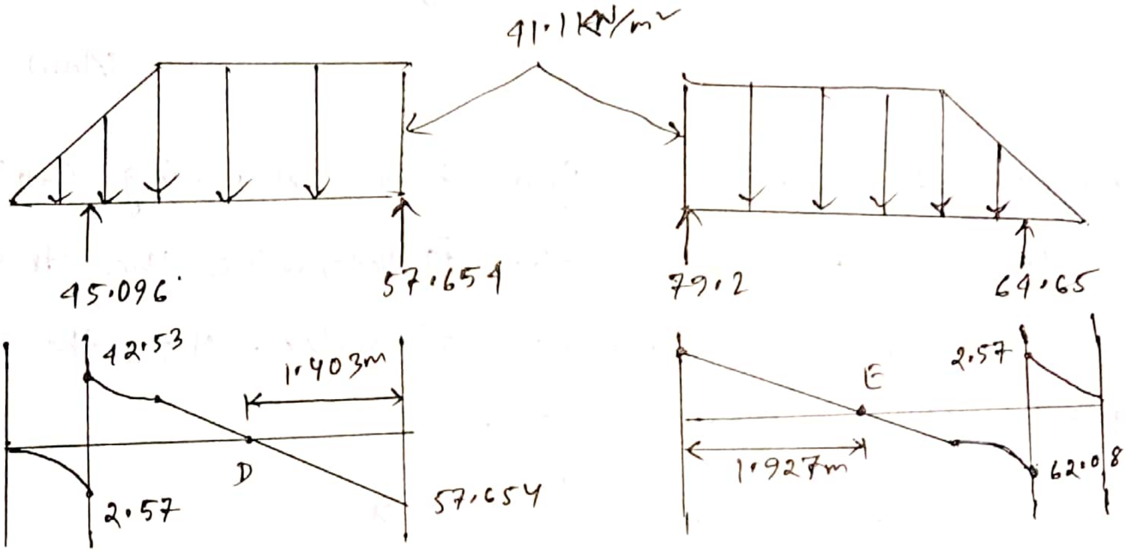
$\therefore R_B = (R_{B_1} + R_{B_2}) = (57.654 + 79.2) = 136.854 \text{ KN/m}$

$\therefore$ strut load at A, $P_A = R_A \times S = (45.096 \times 3.5) = 157.836 \text{ KN}$

at B, $P_B = (136.854 \times 3.5) = 478.989 \text{ KN}$

at C, $P_C = (64.65 \times 3.5) = 226.275 \text{ KN}$

(iii) <u>section modulus of sheet pile:</u>



Moment at D $= \left(\frac{1}{2} \times 57.654 \times 1.403\right) = 40.44 \text{ KN-m/m}$

Moment at E $= \left(\frac{1}{2} \times 79.2 \times 1.927\right) = 76.31 \text{ KN-m/m}$

$\therefore M_{max} = 76.31 \text{ KN-m/m}$

$\therefore$ section modulus of the steel pile, $S = \frac{M_{ax}}{\sigma_{all}}$

$= \frac{76.31}{170 \times 10^3} \text{ m}^3/\text{m}$ of the wall

$= 4.49 \times 10^{-4} \text{ m}^3/\text{m}$ of the wall

(iv) section modulus of wales at B:

Maximum moment at wale B, $M_{max} = \frac{R_B \times s^2}{8}$

$= \frac{136.854 \times 3.5^2}{8}$ KN-m/m

$= 59.87$ KN-m/m

$\therefore$ section modulus, $S = \frac{M_{max}}{\sigma_{all}}$

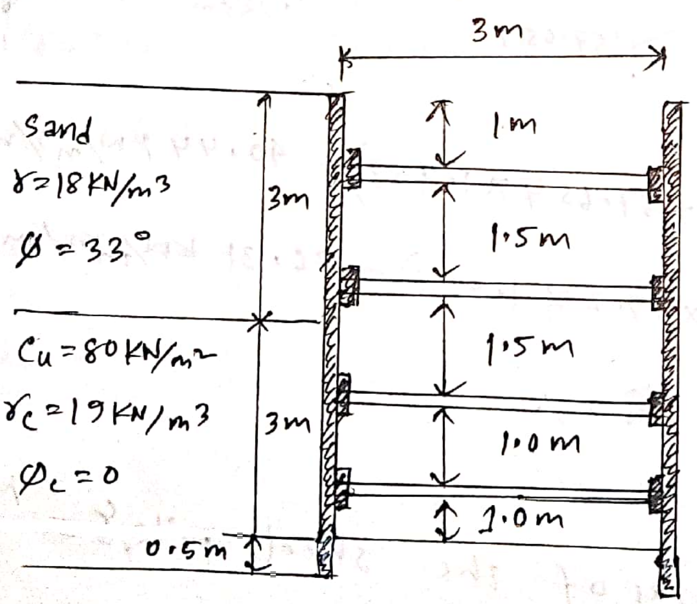
$= \frac{59.87}{170 \times 10^3} = 3.52 \times 10^{-4}$ m³/m of the wall.

(Ans.)

2014

A braced cut is shown in figure below. Assuming $\sigma_{all} = 160$ MN/m²

(i) Draw the earth pressure envelop (ii) Determine the strut loads (iii) Determine the section modulus of wales. Note the struts are placed at 3m c/c in plan.



Solution:

$c_{av} = \frac{1}{2H}\left[K_s \gamma_s H_s^2 \tan\phi_s' + (H - H_s) n' q\right]$

$= \frac{1}{2 \times 6} \times \left[1 \times 18 \times 3^2 \times \tan 33° + 3 \times 0.75 \times 2 \times 80\right]$

$= 38.77$ KN/m²

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$\gamma_{av} = \frac{1}{H}[\gamma_s H_s + (H-H_s)\gamma_c]$

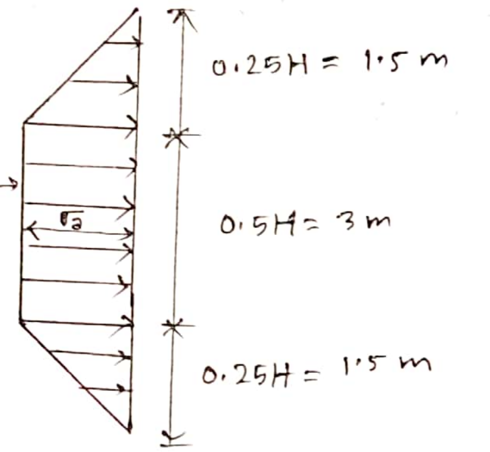
$= \frac{1}{6}\times[18\times3+3\times19]$

$= 18.50\ KN/m^3$

Now, $\frac{\gamma H}{c} = \frac{18.5\times6}{38.77} = 2.86 < 4$; so stiff clay is applied.

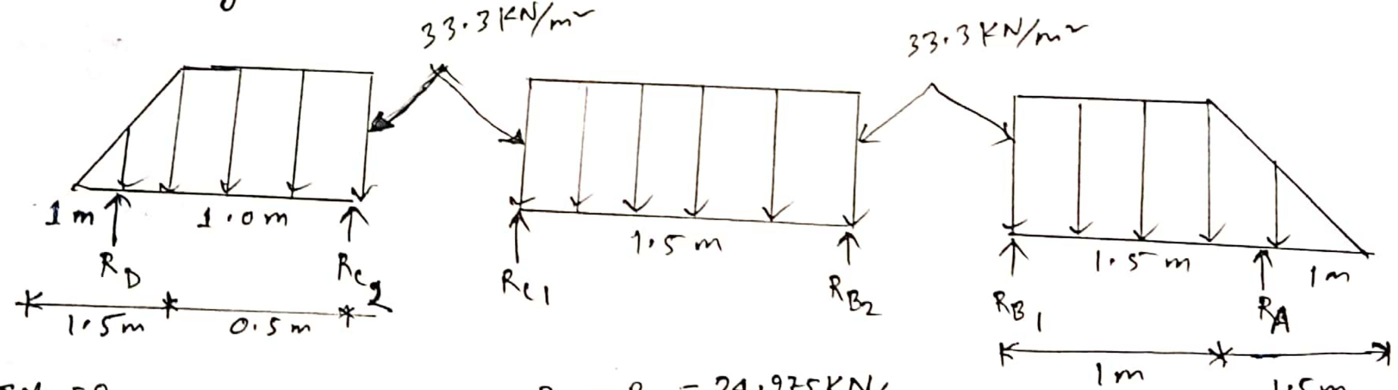
$\sigma_a = 0.3\gamma H = (0.3\times18.5\times6) = 33.3\ KN/m^2$

(i) Earth pressure envelop:



(ii) strut loads:

Assume Hinge at B & C.



$\Sigma M_c = 0$

$R_D = 29.1375\ KN/m$

$R_{c_2} = 12.4875\ KN/m$

$R_{c_1} = R_{c_2} = 24.975\ KN/m$

$\Sigma M_B = 0$

$R_A = 36.075\ KN/m$

$R_{B_1} = 22.2\ KN/m$

$\therefore R_c = (12.4875 + 24.975) = 37.4625\ KN/m$

$R_B = (24.975 + 22.2) = 47.175\ KN/m$

strut loads at A, $P_A = (36.075 \times 3) = 108.225$ KN

$P_B = (47.175 \times 3) = 141.525$ KN

$P_C = (37.4625 \times 3) = 112.3875$ KN

$P_D = (29.1375 \times 3) = 87.4125$ KN

(iii) section modulus of wales:

Maximum Reaction occurs at level B.

$\therefore$ Maximum moment at wale B, $M_{max} = \dfrac{R_B \times s^2}{8}$

$= \dfrac{47.175 \times 3^2}{8}$

$= 53.072$ KN-m/m

$\therefore$ section modulus of wales, $S = \dfrac{M_{max}}{\sigma_{all}} = \dfrac{53.072}{160 \times 10^3}$

$= 3.317 \times 10^{-4}\ m^3/m$ of the wall.

(Ans.)

Caisson

Caisson:

Caisson is a type of foundation of the shape of a hollow prismatic, which is built above the ground level and then sunk to the required depth as a single unit.

It is a watertight chamber used for laying foundation under water as in rivers, lakes, harbours etc.

Type of caisson:

The caisson are of three types:

(i) Open caisson.

(ii) Pneumatic caisson.

(iii) Box caisson, or Floating Caisson.

Open Caisson:

* Open caissons are hollow chambers.
* open both at the top and the bottom.
* walls are made of RCC.
* concreat seal is used.
* Any shape like oval, rectangular, circular etc.

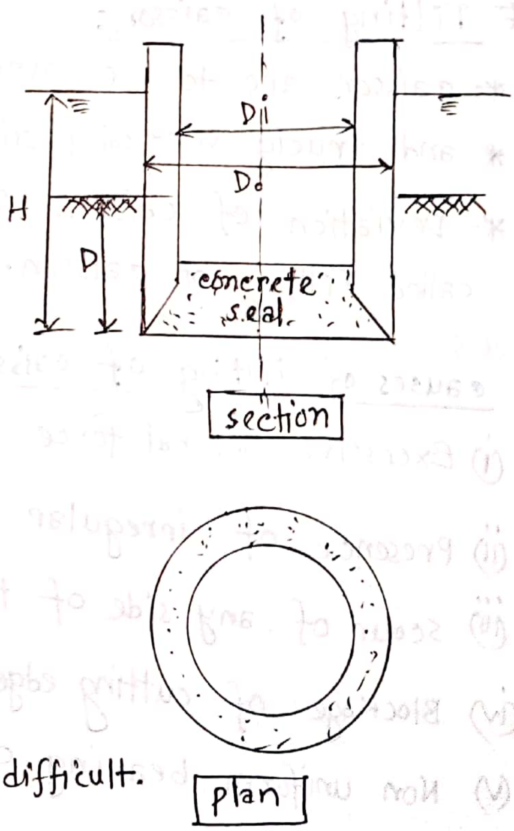


Fig. Open Circular caisson.

Advantages:

1. Feasibility to sunk to great depth
2. construction cost is low.

Disadvantages:

1. concrete seal may not be satisfactory.
2. Slower work process in case of boulders or logs.
3. cleaning and inspection at the bottom of caisson is difficult.

construction procedure:

1. The cutting edge of caisson is first fabricated at the site and the first segment of the shaft is built on it.

2. Soil inside the shaft is removed up to the depth of cutting edge.

3. The first segment is then sunk vertically.

4. Another segment of the shaft is added to the top and the process of sinking is continued.

5. After reaching the required depth concrete is placed on the open bottom as well as seal.

6. When the concrete seal is completely cured, the water in the caisson can be pumped out.

2018

Tilting of caisson:

* caisson are to be sunk perfectly straight, and truely vertical position during the entire process of sinking.
* Deviation of caisson from the true vertical position during sinking is called Tilting of caisson.

2018

causes of tilting of caissons:

(i) Excessive lateral force on the caisson.

(ii) Presence of irregular bed rock during sinking.

(iii) scour of any side of the caisson.

(iv) Blockage of cutting edge.

(v) Non uniform bearing capacity.

2018

Corrective measures or control of tilting of caisson:

(i) Excavation of high sidehead of the low side

(ii) Dredging on the outside of the high side

(iii) pulling the caisson attached with a cables to a dolphin and applying tension as the sinking process.

(iv) Blocking under the cutting edge on the low side.

(v) Jetting on the outside and inside of the high side.

Seal thickness of open caisson:

Type of failure to be considered: Two types of failure are to be considered:

(i) Perimeter shear failure

(ii) Buoyance failure.

(i) perimeter shear failure:

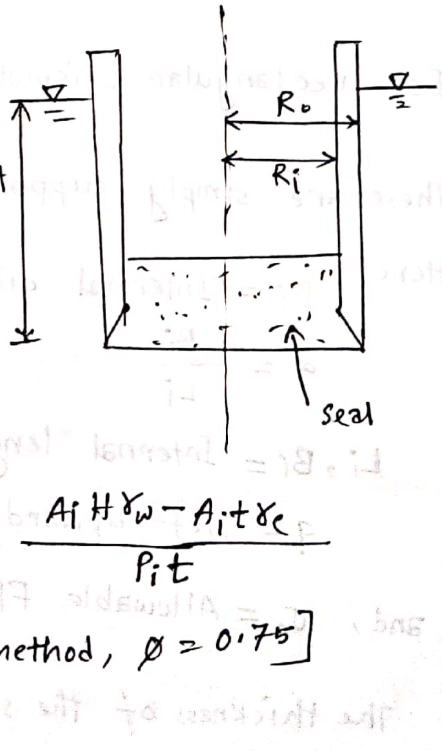
Net upward hydrostatic force from bottom of seal,

$$F_{up} = A_i H \gamma_w - A_i t \gamma_c$$

Here, t = thickness of seal

P_i = Inside perimeter of caisson $= 2\pi R_i$

A_i = Internal Area $= \pi R_i^2$



Resisting area along the perimeter of concrete seal, $A_r = P_i t$

Developed perimetric shear, $V_d = \frac{F_{up}}{A_r} = \frac{A_i H \gamma_w - A_i t \gamma_c}{P_i t}$

Allowable shear, $V_a = 2\sqrt{f_c'}$ [In USD method, $\phi = 0.75$]

For safety, $V_d \leq V_a$

(ii) Buoyance Failure:

If the shaft is totally dewatered, upward buoyant force,

$$F_b = A_o H \gamma_w = \pi R_o^2 H \gamma_w$$

The total downward force,

$$F_d = W_c + W_s + Q_s$$

where, W_c = weight of caisson
W_s = weight of seal
Q_s = Skin friction

To prevent buoyance failure of completely dewatered shaft

$$F_d > F_b$$

* Seal Thickness:

After a caisson is placed in its final position, a thick concrete layer is placed at the bottom to plug it. This is known as the concrete seal or bottom plug and it forms the permanent base of the caisson.

For cylindrical caissons, $t = 0.59 D_i \sqrt{\frac{q}{\sigma_c}}$

For rectangular prismatic caissons, $t = 0.866 B_i \sqrt{\frac{q}{\sigma_c (1+1.61\alpha)}}$

These are simply supported conditions.

Here, D_i = Internal dia. of caisson.

$$\alpha = \frac{B_i}{L_i}$$

L_i, B_i = Internal length and breadth of caisson

q = Net upward pressure on the seal $= \gamma_w H - \gamma_c t$

and, σ_c = Allowable Flexural stress for concrete ($\leq 3500\ KN/m^2$)

The thickness of the seal should be safe against perimeter shear.

2016

Pneumatic caisson:

* Pneumatic caissons are closed at the top, but open at the bottom.
* Water chamber at the bottom of caisson is kept dry.
* can be sunk with the aid of compressed air.
* Air pressure is used to force out the water.

Advantages: 2015

1. Good quality control as work is done in dry conditions.
2. concrete gain more strength in dry condition.
3. Fastest method.
4. suitable for nearly any soil.
5. Low construction related risk
6. can be sealed properly.
7. can be sunk vertically.

2016 * components with sketch

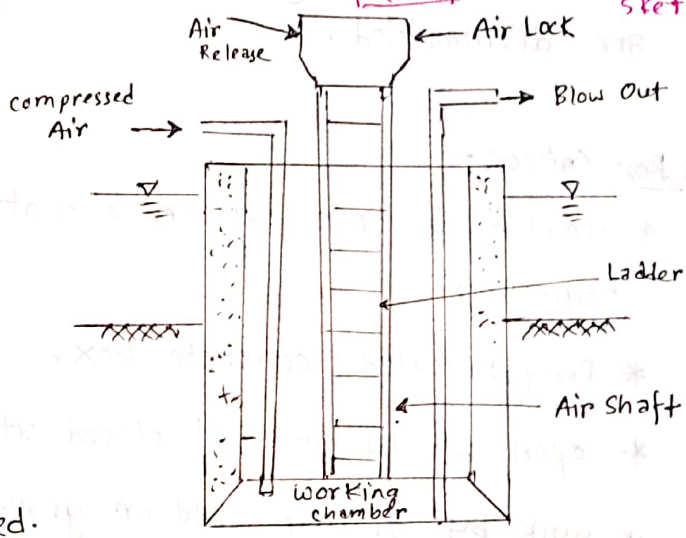


Fig. Pneumatic Caisson

Disadvantages: 2015

1. Expensive than open caisson
2. Highly skilled and fit labour is required.
3. Proper case is needed to avoid accident.
4. Lobour cost is high.
5. More chance of caisson deseases due to high air pressure.

construction procedure:

1. The cutting edge of the caisson is carefully positioned.
2. Compressed are is introduced in the working chamber to expel water.
3. After the working chamber has been dewatered, workmen descend through the air lock in to the working chember.
4. The material is excavated by hand tools in dry.
5. As the excavation progresses, the eassion gradually sinks.

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6. concreting of the caisson is then done.

7. The air pressure in the caisson is increased to equalise the increase in the head of water as the caisson goes down.

8. The excavated material is removed by buckets through the air shaft

9. After caisson has attained its design depth, the working chamber is filled with concrete.

10. After concreting of the working chamber is completed, the shaft tubes are dismantled.

Box Caisson:

* similar to open caisson except that the bottom is sealed from the beginning.
* Prefabricated concrete box.
* open at the top but closed at the bottom.
* sunk by filling sand or gravel or concrete in the empty place inside

Advantages:

1. Easy to construct.
2. Faster than open caisson.
3. Minimise the risk of tilting in construction.

Disadvantages:

1. Not suitable for sites where high water current can erode the foundation
2. can not be used when the construction location is in the land.
3. Foundation base shall be prepared in advance of sinking.
4. Not suitable for high depth.

construction procedure:

1. The caisson is first cast on land.
2. The foundation base is levelled.
3. Then the caissons are floated to the place where to be placed.
4. The caissons are then sunk by filling them with ballast such as sand, dry concrete, gravel.
5. It simply rests on a levelled surface and does not penetrate the soil.
6. After the caissons has been sunk to its final position, It is completely filled with sand or gravel.
7. A concrete cap is cast on its top to recieve structural loads.

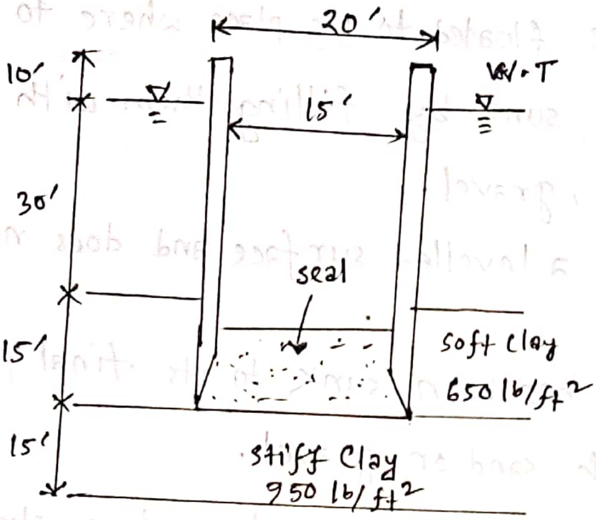
2013

Application of caisson:

1. Bridge pier.
2. Bridge Abutments.
3. Protection of sea-shore.
4. subway tunnel.
5. Pump house.
6. Quay docks.
7. Railway bridge.
8. Garbage pits.
9. water and sewage supply facilities.

2012, 2009

Problem: An open caisson (circular) is shown in figure below. Determine the thickness of the seal that will enable complete dewatering.



Solution: For circular open caisson,

$$t = 0.59\, D_i \sqrt{\frac{q}{\sigma_c}}$$

Height of the caisson below water level,

$$H = (30+15) = 45\text{ ft}$$

The net upward pressure on the seal,

$$q = \gamma_w H - \gamma_c t$$

$$= 62.4 \times 45 - 150 \times t$$

$$\therefore q = 2808 - 150t$$

Allowable flexural stress for concrete, $\sigma_c = 0.1\, f_c' = (0.1 \times 3000)$ psi

$= 300$ psi

$= 43200\ \text{lb/ft}^2$

Hence,

$$t = 0.59 \times 15 \times \sqrt{\frac{2808-150t}{43200}}$$

$$\Rightarrow t^2 = \frac{(0.59 \times 15)^2}{43200} \times (2808 - 150t)$$

$\Rightarrow 551.566\,t^2 + 150t - 2808 = 0$

$\therefore t = 2.124$ ft

considering the thickness, $t = 2.25$ ft

(i) perimeter shear check:

Inner Area, $A_i = \pi R_i^2 = \pi \times \left(\frac{15}{2}\right)^2 = 176.715\ ft^2$

Inner perimeter, $P_i = 2\pi R_i = 2 \times 3.1416 \times \left(\frac{15}{2}\right) = 47.124$ ft

Developed perimeter shear,

$$V_d = \frac{A_i H \gamma_w - A_i \gamma_c t}{P_i t} = \frac{176.715 \times (45 \times 62.4 - 150 \times 2.25)}{47.124 \times 2.25}$$

$\therefore V_d = 4117.5\ lb/ft^2 = 28.6$ psi

Allowable shear, $V_{all} = 2\sqrt{f_c'}$

$= 2 \times \sqrt{3000}$

$\therefore V_{all} = 109.54$ psi $> V_d$

(OK)

(ii) Buoyancy failure check:

If the shaft is totally dewatered,

upward buoyant force, $F_b = A_o H \gamma_w$

$= \frac{\pi}{4} \times 20^2 \times 45 \times 62.4$

$= 882161.3$ lb $= 882.2$ Kip

The total down ward force,

$$F_d = W_c + W_s + Q_s$$

weight of caisson,

$W_c = (A_o - A_i)\gamma_c H = \left(\frac{\pi}{4} \times 20^2 - \frac{\pi}{4} \times 15^2\right) \times 150 \times 55 = 1133921.25$ lb

$= 1133.9$ Kip.

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weight of seal,

$W_s = A_i \gamma_c t = \frac{\pi}{4} \times 15^2 \times 150 \times 2.25 = 59641.3125 \text{ lb} = 59.64 \text{ Kip}$

skin friction, $Q_s = f\,p\,L$

For $C_u = 650 \text{ lb/ft}^2$,

$\alpha = 0.80$

(from graph)

$= \alpha\, C_u\, \underbrace{P_o}_{\pi D_o} L$

$= 0.8 \times 650 \times \pi \times 20 \times 15$

$= 490089.6 \text{ lb}$

$= 490.1 \text{ Kip}$

Thus, $F_d = (1133.9 + 59.64 + 490.1) = 1683.64 \text{ Kip} > F_b$

(OK)

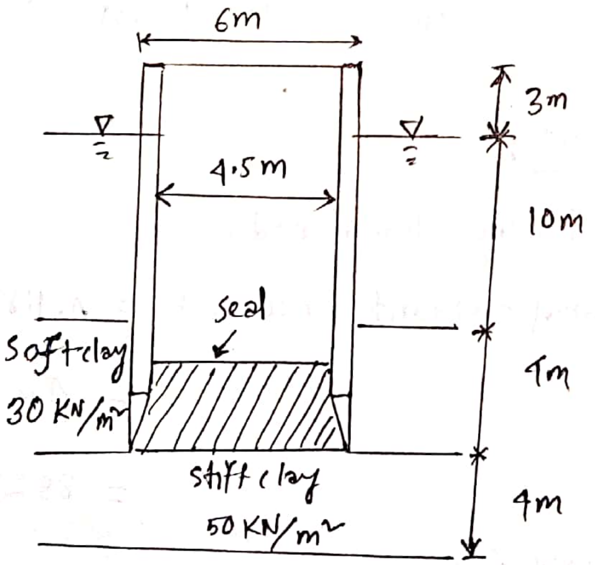
So,

Thickness of seal, $t = 2.25$ ft

(Ans.)

2018

An open caisson is shown in figure below. Determine the thickness of the seal that will enable complete dewatering.



Solution: (perimeter shear failure থেকে thickness বের করলেও হবে)

(i) perimeter shear check:

Inside Area, $A_i = \frac{\pi}{4} \times D_i^2 = \frac{\pi}{4} \times (4.5)^2 = 15.9 \ m^2$

Inside perimeter, $P_i = \pi D_i = (3.1416 \times 4.5) = 14.14 \ m$

developed shear,

$$V_d = \frac{A_i H \gamma_w - A_i \gamma_c t}{P_i t}$$

Here, $\gamma_w = 9.81 \ KN/m^3$
$\gamma_c = 24 \ KN/m^3$

$$\Rightarrow V_d = \frac{15.9 \times 14 \times 9.81 - 15.9 \times 24 \times t}{14.14t} = \frac{2183.706 - 381.6t}{14.14t}$$

And,

Allowable shear, $V_{all} = 2\sqrt{f_c'} = 2 \times \sqrt{3000} = 109.54 \ psi$

$= (109.54 \times 6.895) \ KN/m^2$

$\therefore V_{all} = 755.28 \ KN/m^2$

We know,

$V_{all} \geq V_d$

$1 \ psi = 6.895 \ KN/m^2$

Hence,

$$755.28 = \frac{2183.706 - 381.6t}{14.14t}$$

$$\Rightarrow 10679.66t + 381.6t = 2183.706$$

$$\Rightarrow t = \frac{2183.706}{11061.26} = 0.2 \ m \approx 0.25 \ m$$

$\therefore$ The thickness of seal $= 0.25 \ m$

(ii) Buoyancy failure check:

Upward buoyant force, $F_b = A_o H \gamma_w$

$= \frac{\pi}{4} \times 6^2 \times 14 \times 9.81$

$= 3883.21 \ KN$

Total downward force, $F_d = W_c + W_s + Q_s$

weight of caisson, $W_c = (A_o - A_i)\gamma_c H$

$= (\frac{\pi}{4} \times 6^2 - \frac{\pi}{4} \times 4.5^2) \times 24 \times 17$

$= 5046.98$ KN

weight of seal, $W_s = A_i \gamma_c t$

$= \frac{\pi}{4} \times (4.5)^2 \times 24 \times 0.25$

$= 95.43$ KN

skin friction, $Q_s = P_o L f$

$= \pi D_o L \times (\alpha c_u)$

$= 3.1416 \times 6 \times 4 \times 0.82 \times 30$

$= 1854.8$ KN

Here, For $c_u = 30$ KN/m²

$\alpha = 0.82$

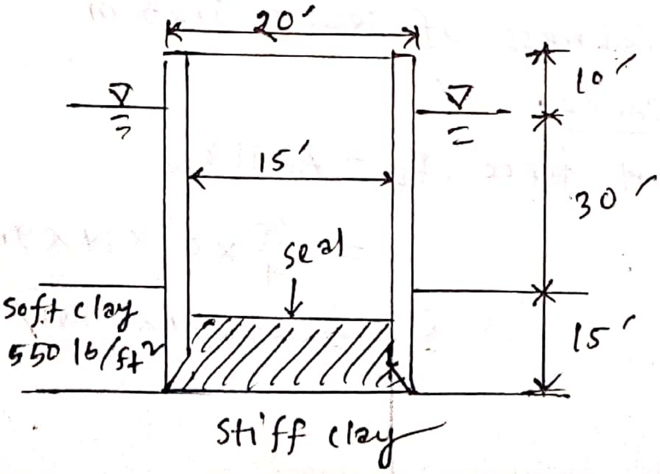
(from graph)

$\therefore F_d = (W_c + W_s + Q_s) = 6997.21$ KN $> F_b$

(OK)

2010

An Open caission (Rectangular) is shown in figure below. Determine the thickness of the seal that will enable to complete dewatering.



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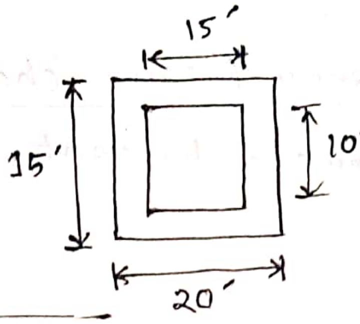
Solution: Given, $L_o = 20'$ & $L_i = 15'$
Assume, $B_o = 15'$



$\therefore B_i = 10'$

For Rectangular caission,

$\therefore$ The thickness, $t = 0.866\, B_i \sqrt{\dfrac{q}{\sigma_c (1 + 1.61\alpha)}}$

Here, $q = \gamma_w H - \gamma_c t = 62.4 \times 45 - 150t = 2808 - 150t$

$\sigma_c = 0.1 f_c' = (0.1 \times 3000) = 300 \text{ psi} = 43200 \text{ lb/ft}^2$

$\alpha = \dfrac{B_i}{L_i} = \dfrac{10}{15} = 0.67$

Thus, $t = 0.866 \times 10 \sqrt{\dfrac{2808 - 150t}{43200 \times (1 + 1.61 \times 0.67)}}$

$\Rightarrow t = 1.47 \text{ ft} \simeq 1.5 \text{ ft}$

$\therefore$ The thickness of seal $= 1.5$ ft

(i) punching shear check:

$A_i' = L_i \times B_i' = (15 \times 10) = 150 \text{ ft}^2$

$P_i' = 2 \times (L_i + B_i) = 2 \times (15 + 10) = 50 \text{ ft}$

developed shear, $V_d = \dfrac{A_i (\gamma_w H - \gamma_c t)}{P_i t}$

$\therefore V_d = \dfrac{150 \times (62.4 \times 45 - 150 \times 1.5)}{50 \times 1.5} = 5166 \text{ lb/ft}^2$
$= 35.875 \text{ psi}$

$V_{all} = 2\sqrt{f_c'}$
$= 2\sqrt{3000} = 109.54 \text{ psi} > V_d$
(ok)

(ii) Buoyancy Failure check;

Upward buoyant force, $F_b = A_o H \gamma_w$

$= L_o B_o H \gamma_w$

$= 20 \times 15 \times 45 \times 62.4$

$= 842400$ lb

$= 842.4$ Kip.

Total downward force, $F_d = W_c + W_s + Q_s$

Here,

$W_c = (A_o - A_i)\gamma_c H = (20\times15 - 15\times10)\times150\times55 = 1237500$ lb

$= 1237.5$ Kip.

$W_s = A_i \gamma_c t = (15\times10\times150\times1.5) = 33750$ lb $= 33.75$ Kip.

and,

$Q_s = p_o L f = 2\times(20+15)\times15\times\alpha C_u$ For, $C_u = 550$ lb/ft^2

$= 2\times35\times15\times0.85\times550$ $\alpha = 0.85$

$= 490875$ lb

$= 490.875$ Kip

$\therefore F_d = (W_c + W_s + Q_s) = 1762.125$ Kip $> F_b$

(OK)

if not OK.

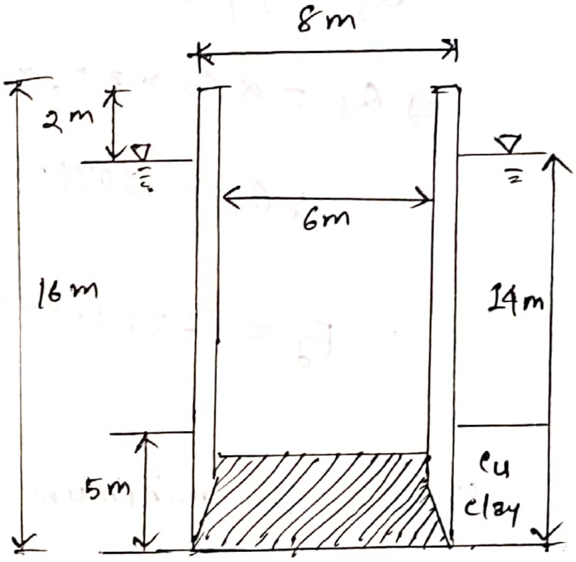
Then, $t' = t + \Delta t$

where, $\Delta t = \dfrac{F_b - F_d}{A_i \gamma_c}$

2017

An open caisson 16 m deep has external and internal diameters of 8 m and 6 m respectively. If the water level is 2 m below the top of the well and depth of base below the scour level is 5 m. Determine the minimum thickness of the seal that will enable complete dewetering of the caisson. Assume, $f_c = 2000$ KN/m², $\gamma_c = 24$ KN/m³ and allowable perimeter shear of 650 KN/m².

Solution:



developed shear,

$$v_d = \frac{A_i \gamma_w H - A_i \gamma_c t}{P_i t}$$

Here,

$$A_i = \frac{\pi}{4} \times (6)^2 = 28.2744 \text{ m}^2$$

$$P_i = \pi (6) = 18.85 \text{ m}$$

$$H = (16 - 2.0) = 14 \text{ m}$$

Now,

$$v_d = \frac{28.2744 (9.81 \times 14 - 24t)}{18.85t}$$

Given, $v_{all} = 650$ KN/m²

Thus,

$$\frac{3883.2 - 678.6t}{18.85t} = 650$$

$$\Rightarrow 650 \times 18.85t + 678.6t = 3883.2$$

$$\Rightarrow t = \frac{3883.2}{12931.1} = 0.30 \text{ m}$$

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Buoyancy failure check:

$F_b = A_o \gamma_w H$

$= \left(\frac{\pi}{4} \times 8^2 \times 9.81 \times 14\right) = 6903.46$ KN

$W_c = (A_o - A_i) \gamma_c H = \frac{\pi}{4}(8^2 - 6^2) \times 24 \times 16 = 8444.601$ KN

$W_s = A_i \gamma_c t = \left(\frac{\pi}{4} \times 6^2 \times 24 \times 0.3\right) = 203.6$ KN

$Q_s = f p L$ Assume, $C_u = 30$ KN/m^2 ∴ $\alpha = 0.82$

$\Rightarrow Q_s = \alpha C_u \times \pi D_o \times L = (0.82 \times 30 \times 3.1416 \times 8 \times 5)$

∴ $Q_s = 3091.33$ KN,

∴ $F_d = (8444.601 + 203.6 + 3091.33) = 11739.531 > F_b$

(OK)

Hence, the minimum thickness of seal, $t = 0.30$ m.

(Ans.)

Coffer Dam

What is Coffer Dam?

* The word 'Coffer' means a casket, chest or trunk
* A coffer dam is a temporary structure built to enclose an area for excavation of foundation.
* Coffer dams are designed and placed, when the size of excavations is very large and sheeting and bracing system becomes difficult, incovenient or uneconomical.
* coffer dams are generally required for foundations of structures such as bridge piers, docks, locks and dams, which are built in open water.

Requirements of coffer dam:

(i) coffer dam should be resoanably water high.

(ii) The design and layout of a coffer dam should be such that the total cost of construction, maintenance and pumping should be minimum.

(iii) It should be sufficiently stable against bursting, overturning and sliding under the floods, waves and anticipated loads.

(iv) It should be generally constructed at site of work.

(v) It should be so planned as to facilitate easy dismantling and reuse of materials.

Advantages of a coffer dam:

(i) Allow excavation and construction of structures in poor environment

(ii) Provides safe environment to work.

(iii) Sheet piles are easily installed and removed.

(iv) Materials can typically be reused on other projects.

Types of cofferdam: 2015

1. Earth Fill Coffer dam.
2. Rock Fill Coffer dam.
3. Single Wall Coffer dam.
4. Double Wall Coffer dam.
5. Braced Coffer dam.
6. Cellular Coffer dam.

1. Earth Fill Coffer dam:

* Simplest form of coffer dam
* Built of local soils, preferably fine sand
* Usually have a clay core or a vertically driven sheet piling in the middle.
* Upstream slope of the bank is covered with a rip tap.
* Use where impervious earth is avaiable and water depth is shallow with low velocity of flow.

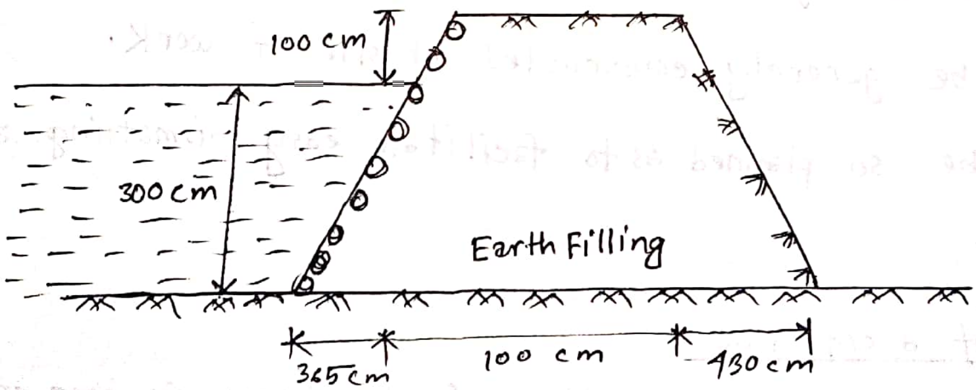

Figure. Section of Earth fill Coffer dam

2. Rockfill Coffer dam:

* Made of rock fill.
* Permeable in nature, An impervious membrane can be used.
* suitable even in case of swift water.
* with stand the over topping of water without any serious damage.

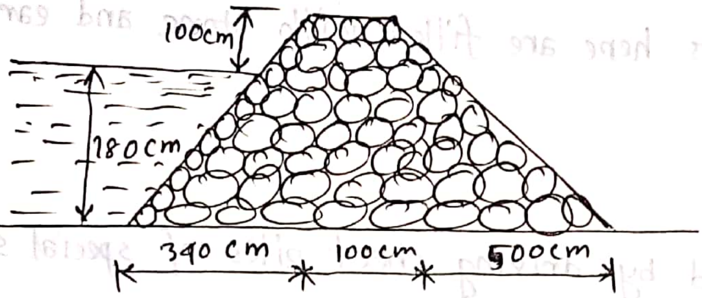

Fig. section of Rock fill coffer dam.

3. Single wall coffer dam:

* suitable for relatively small enclosed area.
* suitable for moderate flow velocities of water and for depth up to 1m.
* The walls of this coffer dam are generally made of steel sheet pile.
* Reinforced and pre-stressed concrete sheet piles are also used.

4. Double Coffer dam:

* These coffer dams are provided to enclose a large area.
* The double wall gives stability to the coffer dam.
* This type is useful where scour problems and space limitations are prevalent.
* These dam consists of two straight, parallel vertical walls of sheet pilling, tied to each other and the space between walls filled with soil.

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5. Braced Coffer dam:

* When it is difficult to drive piles inside the bed in the water, then this type of coffer dam is used.
* In braced coffer dam, two piles are driven into the bed and they are laterally supported with the help of wooden cribs installed in alternate courses to form pockets.

* The empty pockets here are filled with stone and earth

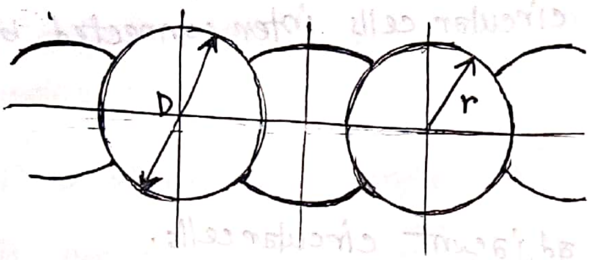
6. Cellular Cofferdam:

* This is constructed by driving sheet piles of special shape to form a series of cells.
* The cells are interconnected to form a water tight wall.
* These cells are filled with soil to provide stabilizing force against lateral pressure.
* They are suitable for dewatering large areas.
* It can withstand over topping of water.
* These types of cofferdams are quite expensive.

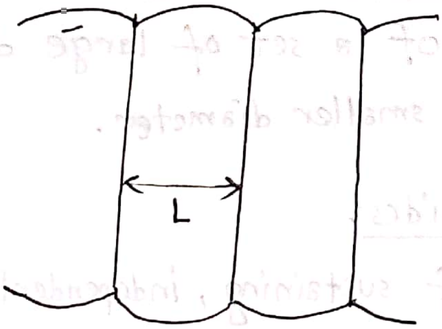
Types of cellular coffer dam:

Cellular coffer dams are of three basic types:

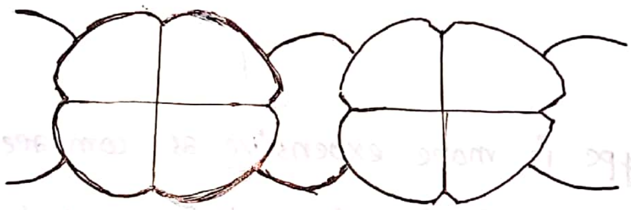
(i) Circular Coffer dam.
(ii) Diaphragm Coffer dam.
(iii) Clover leaf Coffer dam.



(i) circular Type



(ii) Diaphragm Type



(iii) Clover Leaf Type

Diaphragm Type Cofferdam:

consists of circular arcs at the inner and outer sides, which are connected by straight diaphragm walls.

*positive sides:

(i) The effective width of the coffer dam may be increased easily by lengthening the diaphragm

* Negative Sides:

(i) The collapse or failure of one cell will cause the failure of a number of adjoing cells.

(ii) During filling operation, the fill level in any one cell must not vary more than a few feet above or below the fill level in an adjoining cell.

(iii) More piles per linear foot of coffer dam are required for the diaphragm cell than the circular cell.

Circular Type Coffer dam:

consists of a set of large diameter circular cells inter connected by arcs of smaller diameter.

***Positive sides:**

(i) It is self sustaining, independent of the adjacent circular cells.

(ii) Each cell can be filled independently

(iii) The stability of such cells is much larger as compared with the diaphram type.

*** Negative sides:**

(i) The circular type is more expensive as compared to the diaphram type.

(ii) It requires more sheet piles for setting and driving the pile.

(iii) It requires skilled technology for setting and driving the pile.

(iv) The diameter of circular cell can not be enlarged as desired due to limitations in inter locking system.

Clover Leaf Type coffer dam:

This type of cell consists of four arc wall. The clover leaf is used where a large cell width is required for stability against a high head of water.

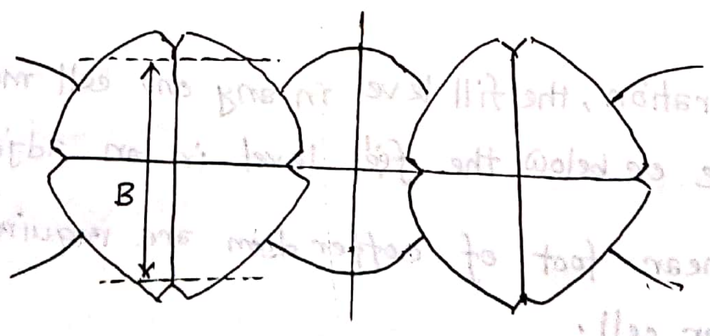


Fig. Clover Leaf Type Coffer Dam

Which one is preferable, circular or diaphragm? 2016, 2012

The circular cell is generally preferable to other cellular types for the following reasons:

(i) It is stable as a single unit.

(ii) It can be filled as soon as it is constructed

(iii) The collapse of a diaphragm cell may cause the entire coffer dam to fail, whereas the collapse of a circular cell is generally a local cell failure.

(iv) The circular cell usually requires less sheet piling.

2012, 2010, 2008

Requirements of cell Filling Material:

(i) It is free draining.

(ii) It has a high angle of internal friction

(iii) It contains small amounts of No. 200 sieve materials (less than 5%)

(iv) It is resistant to scour.

Stability and Design of Cellular Cofferdam:

Stability Check:

1. Cell sliding.
2. Cell Overturning.
3. Cell bursting.
4. Cell shear.
5. Bearing Capacity and Settlement.

Design Method:

Coffer Dam design is semi-empirical and there are at least three design approaches to the problem:

(1) Former Tennessee Valley Authority (TVA) Method. Also called Terzaghi's Method.

(2) Cummings Method.

(3) Hansen's (or Danish) Method.

TVA and cummings method are mostly used as they are simplier.

TVA Method of Cellular Coffer Dam Design: 2015, 2011

Sliding Stability: 2018

Here,

W = Self weight of filling material.

$= B H \gamma_f$ (For Rectangular)

$= \frac{\pi D^2}{4} \gamma_f$ (For circular)

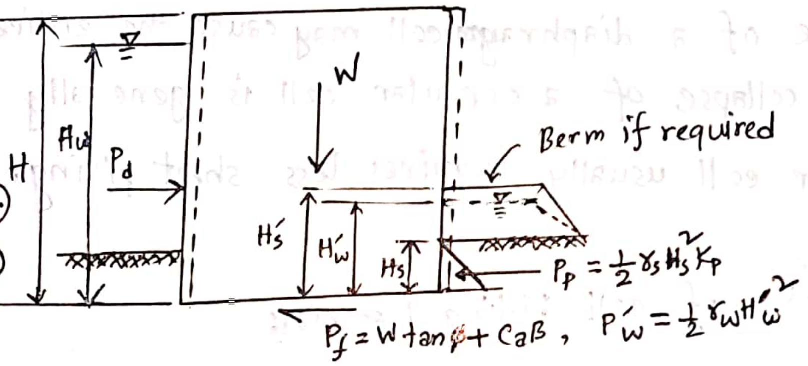


Fig. Sliding resistance.

Factor of safety for sliding stability,

$$FS_s = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{P_r}{P_d} = \frac{P_f + P_p + P_w'}{P_a + P_w}$$

Factor of safety FS_s should be at least 1.25.

A sliding number 'N_s' is defined as,

$$N_s = \frac{P_f + P_p + P_w'}{P_a + P_w}$$

where, P_f = friction on base

P_d = Driving force $(= P_w = \frac{1}{2}\gamma_w H_w^2)$

P_p = Passive resistance but may include a berm.

In this equation, The active force P_a on the water side (not shown) is usually neglected unless the embankment depth is more than about 1.75 m.

Overturning Stability: 2018

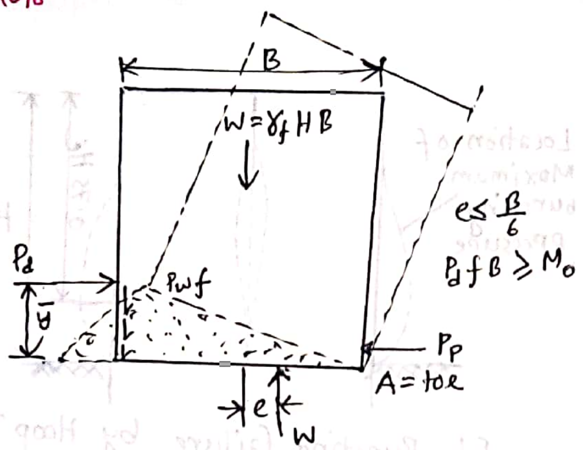


Fig. Overturning Resistance.

Factor of safety against overturning,

$$FS_{ot} = \frac{M_r}{M_o} = \frac{We}{P_d \bar{y}}$$

A value of FS should be 1 to 1.25.

e is calculated as follows: $M_o = M_r$

$$\Rightarrow P_d \bar{y} = We$$

$$\Rightarrow e = \frac{P_d \bar{y}}{W} = \frac{P_d \bar{y}}{\gamma_f HB} \leq \frac{B}{6}$$

Weight W should be with in the middle One-third of the base.

To occur the friction resistance between the cell fill and the water-side sheet piling must develop from the water force $P_d = P_w$

Summing moments about the toe of the cell,

$$BP_w \tan\delta = P_w \bar{y}$$

$$\Rightarrow B = \frac{\bar{y}}{\tan\delta}$$

The stability Number, $N_{ot} = \frac{B\tan\delta}{\bar{y}}$ where, δ = Angle of friction between cell fill and steel. (0.6 to 0.7ϕ)

Bursting Failure:

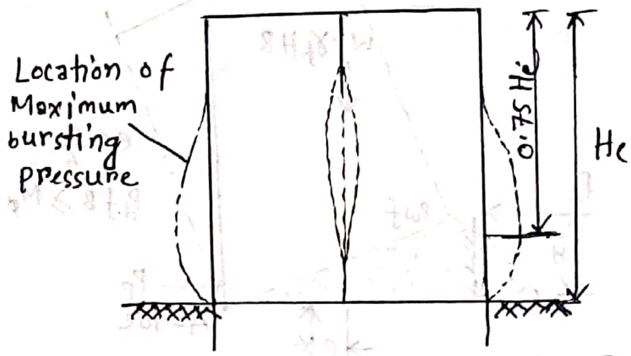


Fig. Bursting failure by Hoop Tension.

The bursting pressure at a depth z_i in the cell is,

$q_t = \bar{q}_h + q_w$ where, $\bar{q}_h$ = effective lateral pressure due to soil.

$= \gamma_e z_i K_a$

q_w = Hydrostatic Pressure $= \gamma_w z_i$

The bursting force per unit of height is,

$T = q_t r$

T should be smaller than allowable tensile stress of sheet pile material.

Cell Shear:

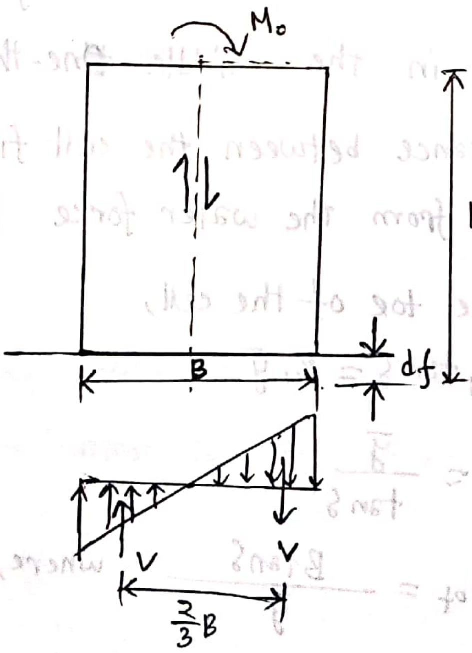


Fig. shear along centerline of cell.

Shear along a plane through the center of the cell is another possible mode of failure.

For stability, the shearing resistance along this plane must be equal to or greater than the shear due to overturning effect.

$$\text{Overturning Moment, } M_o = V \times \frac{2}{3} B$$

$$\Rightarrow V = 1.5 \frac{M_o}{B}$$

For stability, $V_r \geq V$

soil shear resistance V_r is computed as follows:

$$V_r = F_s + T = \frac{1}{2} \gamma H^2 K' \tan\phi + \frac{1}{2} \gamma H^2 K_a r$$

where, $K' = \dfrac{\cos^2\phi}{2 - \cos^2\phi}$ and,

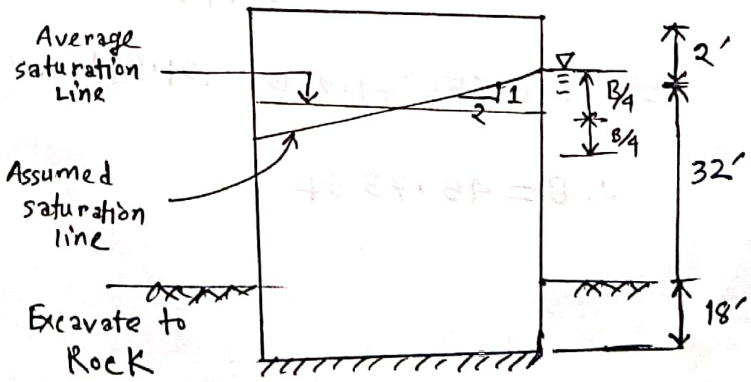
$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$$

Factor of Safety against cell shear,

$$FS_{cr} = \frac{V_r}{V} \quad \left[\text{It should be } 1.25 - 1.5\right]$$

Problem:

Design a diaphragm cell. Assume the cell saturation line to be as shown in figure:



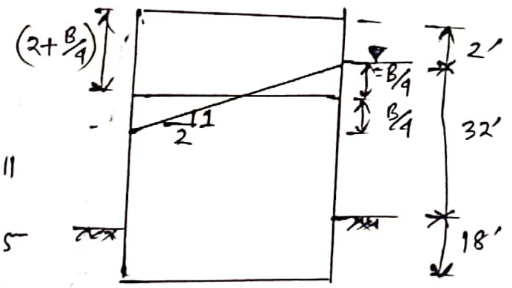
Use $\gamma_{fill} = 110$ pcf for clean sand and gravel. Given that $\gamma'_{fill} = 65$ pcf, $\phi_{fill} = 30°$ for both saturated and damp and interlock friction $= 0.3$.

Assume other values if necessary.

Solution: The width of the cell = B

1. Sliding Stability:

Total weight of the cell,



$$W = B \times (2 + \frac{B}{4}) \times \gamma_{fill} + B \times (52 - 2 - \frac{B}{4}) \times \gamma'_{fill}$$

$$= (2B + \frac{B^2}{4}) \times 0.110 + (50B - \frac{B^2}{4}) \times 0.065$$

$$= 0.22B + 0.0275B^2 + 3.25B - 0.01625B^2$$

$$\therefore W = 3.47B + 0.01125B^2$$

Friction co-efficient, $f = \tan\phi = \tan 30° = 0.5774$

Frictional force, $P_f = fW = 0.5774 \times (3.47B + 0.01125B^2)$

$$= 2.004B + 0.0065B^2$$

Driving force, $P_d = P_a + P_w$

$$= \frac{1}{2} K_a \gamma'_{fill} H_{fill}^2 + \frac{1}{2} \gamma_w H_w^2$$

$$= \frac{1}{2} \times 0.33 \times 0.065 \times 18^2 + \frac{1}{2} \times 0.0624 \times 50^2$$

$$= 3.475 + 78$$

$$= 81.475 \text{ Kip.}$$

$\therefore$ Factor of safety, $FS_s = \dfrac{P_f}{P_d}$

$$\Rightarrow 1.25 = \frac{2.004B + 0.0065B^2}{81.475}$$

$$\Rightarrow 0.0065B^2 + 2.004B - 101.844 = 0$$

$$\therefore B = 44.42 \text{ ft}$$

2. Overturning Stability:

$$M_o = \left(P_a \times \frac{H_{fill}}{3}\right) + \left(P_w \times \frac{H_w}{3}\right)$$

$$= \left(3.475 \times \frac{18}{3}\right) + \left(78 \times \frac{50}{3}\right)$$

$$= 1320.85 \text{ K-ft}$$

$$FS = \frac{W \cdot e}{M_o} = \frac{(3.47B + 0.01125B^2) \times \frac{B}{6}}{1320.85}$$

$$\Rightarrow 1.25 = \frac{0.5783 B^2 + 0.001875 B^3}{1320.85}$$

$$\Rightarrow 0.001875B^3 + 0.5783B^2 - 1651.0625 = 0$$

$$\therefore B = 49.6 \text{ ft}$$

Checking for overturning friction on heel.

$$fB(P_a + P_w) = M_o \times FS$$

$$\Rightarrow 0.5 \times B \times (3.475 + 78) = 1320.85 \times 1.25$$

$$\Rightarrow B = 40.53 \text{ ft} < 49.6 \text{ ft}$$

Hence provide width of the cell, $B \simeq 50$ ft

3. Cell Shear: Along center line, Average weight of soil in the cell of strip (1 ft × B ft)

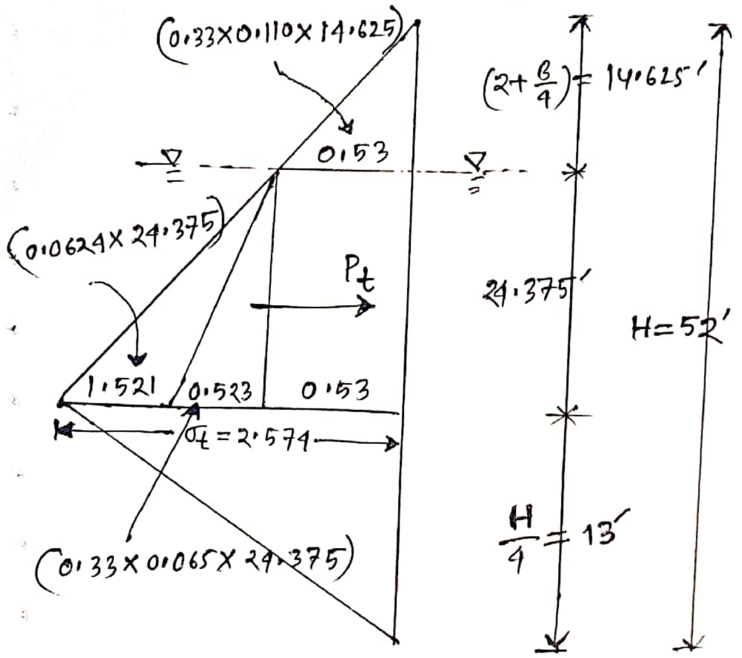
$$W = (3.47B + 0.01125B) = (3.47 \times 50.0 + 0.01125 \times 50)$$

$$\therefore W = 174.0625 \text{ Kip.}$$

Shear resistance, $F_s = \frac{1}{2} \gamma H^2 K' \tan \phi'$

Here, $K' = \frac{\cos^2 30^\circ}{2 - \cos^2 30^\circ} = 0.6$

$$\therefore F_s = \frac{1}{2} \times 0.110 \times 50^2 \times 0.6 \times \tan 30 = 47.63 \text{ Kip.}$$



$$P_t = 0.53 \times \frac{14.625}{2} + \frac{0.53 + 2.574}{2} \times 24.375 + 2.574 \times \frac{13}{2}$$

$$= 58.44 \text{ Kip}$$

$$R_{i1} = P_t f_i = (58.44 \times 0.3) = 17.53 \text{ Kip}$$

$$V_p = F_s + R_{i1}$$

$$= (47.63 + 17.53) = 65.16 \text{ Kip}$$

The developed shear,

$$V = 1.5 \times \frac{M_o}{B}$$

$$= 1.5 \times \frac{1320.85}{50} = 39.6255 \text{ Kip}$$

$$\therefore V < V_p$$

$$FS_{er} = \frac{V_r}{V} = \frac{65.16}{39.6255} = 1.64 > 1.25$$

(OK)

(iv) Interlocking Friction:

$$t_i = \sigma_t r \leq \frac{190 \text{ K/ft}}{F_s}$$

$$\Rightarrow 2.574\, r = \frac{190}{1.64}$$

$$\therefore r = 45 \text{ ft}$$

we now have design dimensions for the cell as follows,

$B = 50$ ft, $r = L = 45$ ft and cell Height $= 52$ ft

Machine Foundation

Type of Foundation:

1. Foundation subjected to static loads.
2. Foundation subjected to dynamic loads.

2007, 2006

Machine Foundation:

Foundations which are subjected to dynamic forces caused by machine is referred to as Machine Foundations.

Sources of Dynamic loads:

1. Vibratory Motion of machines.
2. Movements of vehicles.
3. Impacts of hammers.
4. Earth quake.
5. Winds.
6. Waves.
7. Nuclear Blast.
8. Mine Exploration.
9. Pile Driving etc.

2017

Types of Machines:

1. Machines which produce a periodic unbalanced force, such as: Reciprocating Engine (speed < 600 rpm)
2. Machines which produce on impact loads, such as: Hammer (speed in between 60 – 150 blows per minute)
3. High speed machines, such as: Turbines (speed is high even more than 3000 rpm)

Centrifugal Machine:

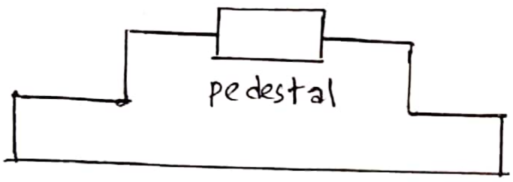
Based on Configuration:

1. **Block Type:** consists of a pedestal resting on a footing.

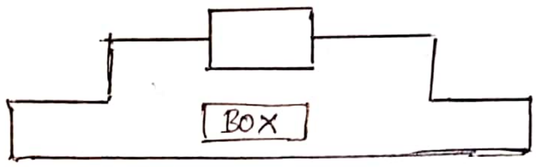
2. **Box Type:** consists of a hollow concrete block.

3. **Wall Type:** consists of a pair of walls having a top slab.

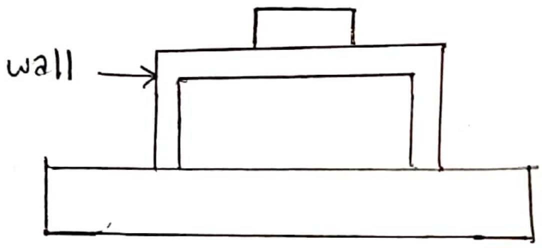
4. **Frame Type:** consists of vertical columns having a horizontal frame at their tops.



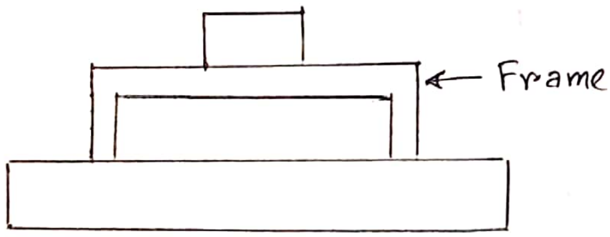
Block Type



Box Type



Wall Type



Frame Type

2012

General Criteria for Machine Foundation:

1. It should be safe against shear failure.
2. Settlement should be with in safe limit.
3. Natural frequency of foundation should be either greater than or less than the operating frequency of the machine.
4. Amplitude under service condition should be with in the permissible limit

5. Combined center of gravity of machine and foundation should lie on the same vertical plane.

6. Level of machine foundation should be lower than of Foundation.

2017 / 2012

Reinforcement and Construction Details:

1. The reinforcement in the concrete block should not be less than 25 kg/m³

2. Steel reinforceforcement around all pits and openings shall be at least equal to 0.5% to 0.75% of the cross sectional area of the pit or opening.

3. The Reinforcement shall run in all the three directions.

4. The minimum reinforcement shall be usually consists of $\boxed{\text{12 mm bar}}$ at 200 – 250 mm spacing
4 no. bar

5. The ends of all bar should be always hooked.

6. If the height of foundation blocks exceeds 1m, shrinkage reinforcement should provide in all directions.

7. The cover should be a minimum of 75 mm at the bottom and 50 mm on sides and Tops.

some definitions:

1. Vibration: The time dependent repeated motion of translation or rotational type is called vibration or Oscillation.

2. Periodic Motion: The motion which repeats its self periodically in equal time interval is called periodic motion.

3. Period: The time in which the motion repeats itself is called period or period of motion.

4. Cycle: The motion completed in period is called cycle of motion.

5. Free Vibration: The vibrations which occur under the influence of forces inherent in the system itself without any external forces is called free vibration.

2013, 2012

6. Free damped Vibration: When a viscous damper is added to the model that outputs a force that is proportional to the velocity of mass, the damping is called viscous, because it models the effects of fluid with in an object.

2013

7. Free undamped vibration: when the force of viscous damper is not proportional to the free vibrational velocity then, it will be called free undamped vibration.

2017

8. Forced Vibration: The vibration which occur under the influence of the continuous external forces is called Forced vibration.

9. Frequency: The number of cycles of motion in a unit time is known as frequency of vibration. It is usually expressed in Hz.

10. Natural Frequency: The system under free vibration, vibrates at the frequency known as The natural frequency.

2012

11. Resonance: When the frequency of existing force is equal to one of the natural frequencies of the system, the amplitude of motion becomes excessively large. This condition is known as Resonance.

12. Damping: The resistance to motion which develops due to friction and other causes is known as damping.

13. Viscous Damping: The type of damping in which the damping force is proportional to the velocity is called viscous damping. It is expressed as,

$$F = c \cdot \frac{dz}{dt}$$

where, c = Damping co-efficient

$\frac{dz}{dt}$ = velocity.

14. Non-viscous Damping: Damping models in which the dissipative forces depend on any quality other than the instantaneous generalized velocity then, it is called non-viscous damping model. and the damping is called non-viscous damping.

15. Degree of Freedom: The number of independent co-ordinates required to described the motion of a system is called the degree of freedom.

2017, 2012

16. Mass Spring System:

A system of masses which are connected by springs is called Mass-spring system. It is a classical system with several degree of freedom.

For example, A system consisting of two masses and three springs has two degree of freedom.

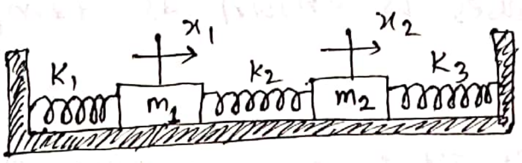


Fig. A Mass-spring system

Machine Foundation

Several Relationships:

1. $W_n = \sqrt{\frac{K}{m}}$ where, W_n = Natural Circular Frequency (rad/sec.)
   m = mass of body.
   K = spring stiffness.

2. $T = \frac{1}{f_n} = 2\pi\sqrt{\frac{m}{K}}$ where, T = Time in sec.
   f_n = Natural frequency (cycle/sec.)

3. $W_{nd} = W_n\sqrt{1-D^2}$ where, W_{nd} = damped Natural frequency (cycle/s)
   ξ or D = Damping factor

4. $D = \frac{C}{C_c}$ where, C = Damping co-efficient
   C_c = critical Damping

if $D > 1$, The system is Over Damped
$D = 1$, The system is critically Damped.
$D < 1$, The system is under damped.

5. $C_c = 2\sqrt{mK}$

6. $f_n = \frac{W_n}{2\pi}$

7. $\delta = \frac{2\pi D}{\sqrt{1-D^2}}$ where, δ = logarithmic decrement/increment

Vibration Analysis of Machine Foundation:

1. Mass: $m = m_f + m_s$ where, m_f = mass of foundation block and machine
   m_s = mass of soil

2. Spring Stiffness: $K = \frac{AE}{L}$ where, $E = 2G(1+\mu)$
   G = modulus of rigidity
   μ = poisson's ratio
   A = cross sectional area of specimen
   E = Young's modulus.

Barkan's method: $K = \frac{1.13\,E}{1-\mu^2}\sqrt{A}$ where, A = Base area of the machine, area of contact

3. Damping constants: $\delta = \frac{2\pi D}{\sqrt{1-D^2}}$

$\delta = \log\left(\frac{x_2}{x_1}\right)$ where, $\frac{x_2}{x_1}$ = Amplitude ratio

4. Natural Frequency: $\omega_n = \sqrt{\frac{K}{m}}$

$f_n = \frac{\omega_n}{2\pi}$

Barken's method: $\omega_n = \sqrt{\frac{C_u A}{m}}$

where, C_u = co-efficient of elastic uniform compression (Kg/m^3)

A = contact area of foundation with soil.

$K = C_u A$

$C_u = \frac{1.13\,E}{1-\mu^2} \times \frac{1}{\sqrt{A}}$

$Z_{max} = \frac{F_o}{m\,\omega_n^2 (1-r^2)}$

Where, Z_{max} = Maximum Amplitude

F_o = Force (N)

$r = \frac{\text{operating frequency}}{\text{Resonance frequency}} = \frac{f}{f_n} = \frac{\omega}{\omega_n}$

5. Force Vibration:

$M = \frac{1}{\sqrt{(1-r^2)^2 + 4D^2r^2}}$ where, M = magnification factor

$|F_T| = F_o M \sqrt{1+(2Dr)^2}$ F_T = Transmitted force

2018, 2016

A 3.0 Mg vertical compressor foundation system is operated at 45 Hz. The soil at the site is medium stiff clay. The C_u of which is 4.2×10^4 KN/m^3. Determine the natural frequency and magnification factor. Assume $M_s = 0.25\, m_f$. The base area of foundation is 3.0 m^2. Consider, $D=0$

Solution:

Total Mass $= m_f + m_s$

$= (3 + 0.25\times 3) = 3.75$ Mg $= 3.75\times10^3$ Kg

Given,
$C_u = 4.2\times10^4$ KN/m^3
$= 4.2\times10^7$ N/m^3
$A = 3.0$ m^2

We know,
Natural frequency, $f_n = \frac{1}{2\pi}\sqrt{\frac{C_u A}{m}}$

$= \frac{1}{2\pi}\times\sqrt{\frac{4.2\times10^7\times3.0}{3.75\times10^3}}$

$= 29.174$ Hz (Ans.)

And,
Magnification factor, $M = \frac{1}{\sqrt{(1-r^2)+4D^2r^2}}$

Here, $r = \frac{f}{f_n} = \frac{45}{29.174} = 1.54$

Thus, $M = \frac{1}{\sqrt{(1-1.54^2)^2+0}} = 0.73$

(Ans.)

2017, 2011, 2009

A machine foundation weighs 70 KN and has a spring constant $K = 13500$ KN/m. Assuming the damping co-efficient, c of the system as equal to 210 KN-sec/m. Determine (i) whether the system is over damped, Under damped or critically damped. (ii) logarithmic decrement (iii) the ratio of two successive amplitude, and (iv) damped natural frequency.

Solution: (i) $M = \frac{W}{g} = \frac{70 \times 10^3}{9.81} = 7135.6$ Kg

We know,

Damping factor, $D = \frac{C}{C_c}$ Given, $C = 210$ KN-s/m

Here, $C_c = 2\sqrt{mK} = 2 \times \sqrt{7135.6 \times 13500 \times 10^3}$

$\therefore C_c = 620743.43$ KN-s/m

$\therefore D = \frac{210 \times 10^3}{620743.43} = 0.34 < 1$

Hence, The system is under damped. (Ans.)

(ii) logarithmic decrement, $\delta = \frac{2\pi D}{\sqrt{1-D^2}} = \frac{2 \times 3.1416 \times 0.34}{\sqrt{1-(0.34)^2}}$

$\therefore \delta = 2.27$ (Ans.)

(iii) And, $\delta = \log\left(\frac{x_2}{x_1}\right)$

$\Rightarrow 2.27 = \log\left(\frac{x_2}{x_1}\right)$

$\Rightarrow \frac{x_2}{x_1} = 10^{2.27} = 186.21$ (Ans.)

(iv) Damped natural frequency, $W_{nd} = W_n\sqrt{1-D^2}$

$= \sqrt{\frac{K}{m}} \times \sqrt{1-(0.34)^2}$

$= \sqrt{\frac{13500 \times 10^3}{7135.6}} \times \sqrt{1-(.34)^2}$

$= 40.905$ rad/sec. (Ans.)

2015

In a test block of the size 1.5 m × 1.0 m × 0.75 m, resonance occurs at a frequency of 20 cycles per second in the vertical vibration. Determine the co-efficient of elastic uniform compression if the mass of oscillator is 70 Kg and the force produced by it at 15 cycles per second is 1200 N. Also compute the maximum amplitude at 15 cycles per second.

Solution:

(i) We know, $\omega_n = \sqrt{\frac{C_u A}{m}}$

Here, $\omega_n = 2\pi f_n = 2 \times 3.1416 \times 20 = 125.664$ rad/sec.

m = mass of oscillator + mass of concrete block

$= 70 + (1.5 \times 1 \times 0.75 \times 2400)$

$\therefore m = 2770$ Kg

and $A = (1.5 \times 1) = 1.5\ m^2$

Thus, $125.664 = \sqrt{\frac{C_u \times 1.5}{2770}} \Rightarrow C_u = 2.92 \times 10^7$ Kg/m^3

(Ans.)

(ii) We know, $Z_{max} = \frac{F_o}{m \omega_n^2 (1 - r^2)}$

Here, $F_o = 1200$ N

$m = 2770$ Kg, $\omega_n = 125.664$

$r = \frac{15}{20} = 0.75$

$\therefore Z_{max} = \frac{1200}{2770 \times (125.664)^2 \times (1 - 0.75^2)} = 6.27 \times 10^{-5}$ m

(Ans)

2014

The resonance of a test block 2m × 1m × 1m occured at 25 cycles/sec. in the ventical direction. The other data are as follows: weight of oscillator = 62 kg, vertical unbalanced force = 0.5 tones, unit weight of soil = 1.7 t/m3. Calculate the apparent mass of soil.

<u>solution:</u>

$$\text{weight of Soil, } W_s = \frac{4}{3}\pi\gamma \times \left[\frac{0.4775\, W_v}{4\gamma}\right]^{3/2} \text{ lb}$$

Here, W_v = weight of machine + foundation + unbalanced force

$= 62 + (2 \times 1 \times 1 \times 2400) + (0.5 \times 1000)$

$= 5362$ kg $= 11823.21$ lb   [1 kg = 2.205 lb]

$\gamma = 1.7\ t/m^3 = 106.13\ lb/ft^3$ [1 t/m³ = 62.43 lb/ft³]

Hence,

$$W_s = \frac{4}{3} \times 3.1416 \times 106.13 \times \left[\frac{0.4775 \times 11823.21}{4 \times 106.13}\right]^{3/2}$$

$= 21559.75$ lb

$= 9777.66$ kg

∴ Apparent mass of soil, $m_s = \frac{9777.66}{9.81} = 996.7$ kg-sec²/m

2012

The total mass of a machine and a rigid base foundation is 100 Kg. calculate the vertical and horizontal natural frequency. Given that, shear modulus of soil = 25, poisson's ratio = 0.30 and area of base = 6 m².

Solution:

we know, $K = \frac{1.13E}{1-\mu^2} \times \sqrt{A}$

$= \frac{1.13 \times 2G(1+\mu)}{1-\mu^2} \times \sqrt{A}$

$= \frac{1.13 \times 2 \times 25 (1+0.3)}{1-(0.3)^2} \times \sqrt{6}$

$= 197.71$ KN/m

(i) vertical natural frequency, $W_n = \sqrt{\frac{K}{m}} = \sqrt{\frac{197.71 \times 10^3}{100}}$

$= 44.465$ rad/sec.

(ii) Horizontal natural frequency, $f_n = \frac{1}{2\pi} \times W_n$

$= \frac{1}{2\pi} \times 44.465$

$= 7.077$ cycle/sec.

B.C. Punmia (Example - 28.8)

A machine having a total weight of 20000 KN has an unbalance such that it is subjected to 5000 KN at a frequency of 600 rpm. What should be the spring constant for supporting springs if the maximum force transmitted in to the foundation due to the machine is to be 500 KN. Assume, the damping can be neglected.

Solutions

$$m = \frac{20000 \times 1000}{9.81} = 2.0387 \times 10^6 \text{ kg}$$

Given, $F_o = 5000 \text{ KN} = 5000 \times 10^3 \text{ N}$; $\omega = 600 \text{ r.p.m.} = 10 \text{ r.p.s}$

$F_T = 500 \text{ KN} = 500 \times 10^3 \text{ N}$; $D = 0$

We know,

$$F_T = F_o M \times \sqrt{1 + 4D^2 r^2} = F_o M \quad [\because D = 0]$$

$$\Rightarrow F_T = F_o \times \frac{1}{\sqrt{(1-r^2)^2 + 4D^2r^2}} \quad [\because D = 0]$$

$$\Rightarrow F_T = \frac{F_o}{\sqrt{(1-r^2)^2}}$$

$$\Rightarrow \sqrt{(1-r^2)^2} = \frac{F_o}{F_T} = \frac{5000 \times 10^3}{500 \times 10^3}$$

$$\Rightarrow \sqrt{(1-r^2)^2} = 10$$

$$\Rightarrow (1-r^2)^2 = 10$$

$$\Rightarrow 1 - r^2 = 10$$

$$\therefore |r| = 3 \qquad \text{Given, } \omega = 10 \text{ r.p.s}$$

$$\left[1 \text{ Kg} = 1 \frac{N}{m} \sec^2\right]$$

We know, $r = \frac{\omega}{\omega_n} = 3 \Rightarrow \omega_n = \frac{10}{3} = 3.33$

Again, $\omega_n = \sqrt{\frac{K}{m}} \Rightarrow K = m\omega_n^2 = (2.0387 \times 10^6 \times 3.33^2) = 22.6 \times 10^6 \text{ Kg/sec}^2$

$= 22.6 \times 10^6 \text{ N/m}$

(Ans.)

Mat Foundation

What is Shallow Foundation? 15

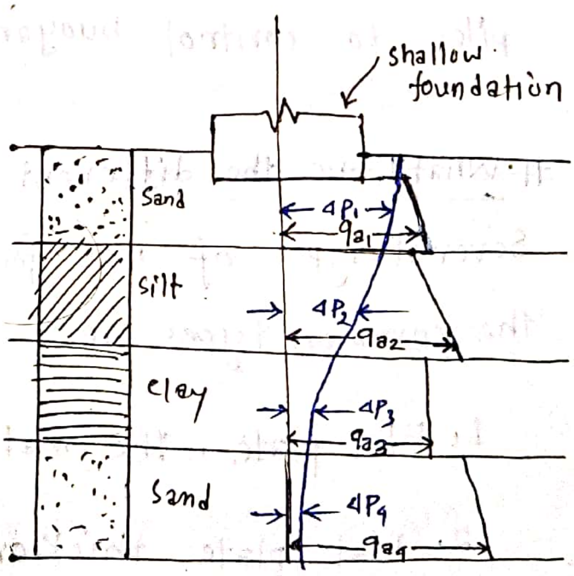
Shallow Foundation is a type of building foundation that transfers building loads to the earth very near to the surface. The depth of the shallow foundation is less than or equal to its width.

Spread footings and mat foundations generally are referred to as shallow foundations.

At what situation do you suggest shallow foundation instead of deep foundation? 15, 18

Shallow foundations such as footings and rafts are cost less and easier to construct. Shallow foundation is often selected when the soil has good bearing capacity and the structural load will not cause excessive settlement of the underlaying soil layers. They could be suggested if the following two conditions are fulfilled:

(i) The super-imposed stress (ΔP), caused by the building lies within the allowable bearing capacity of different soil strata as shown in figure:



(ii) The building could withstand the expected settlement estimated for that type of foundation.

If one or both of these two conditions can not be fulfilled, the use of deep foundation should be considered.

What is Mat foundation or raft foundation? 18, 14

A mat or raft foundation is essentially a large continuous slab supporting more than one line of columns and walls under the entire structure or a large part of the structure.

What situation do you suggest Mat foundation instead of individual footing? 17, 14, 12

Mat foundation can be suggested for following conditions:

(i) when the soil have low bearing capacity, but will have to support high column or wall loads.

(ii) when spread footings would have to cover more than half building area, use of mat foundations might be more economical.

(iii) Mats may be supported by piles, which help to reduce the differential settlement of a structure built over highly compressible soil.

(iv) Where the water table is high, mats are often placed over piles to control buoyancy.

What are the different types of raft foundation? 16, 15

Several types of mat foundations are used currently. Some of the common types are:

1. Flat plate. The mat is of uniform thickness. (Fig. a)

2. Flat plate thickened under columns. (Fig. b)

3. Beams and slab. The beams run both ways and the columns are located at the intersections of the beams. (Fig. c)

4. Flat plates with pedestals. (Fig. d)

5. Slab with basement walls as a part of the mat. The walls act as stiffeners for the mat. (Fig. e)

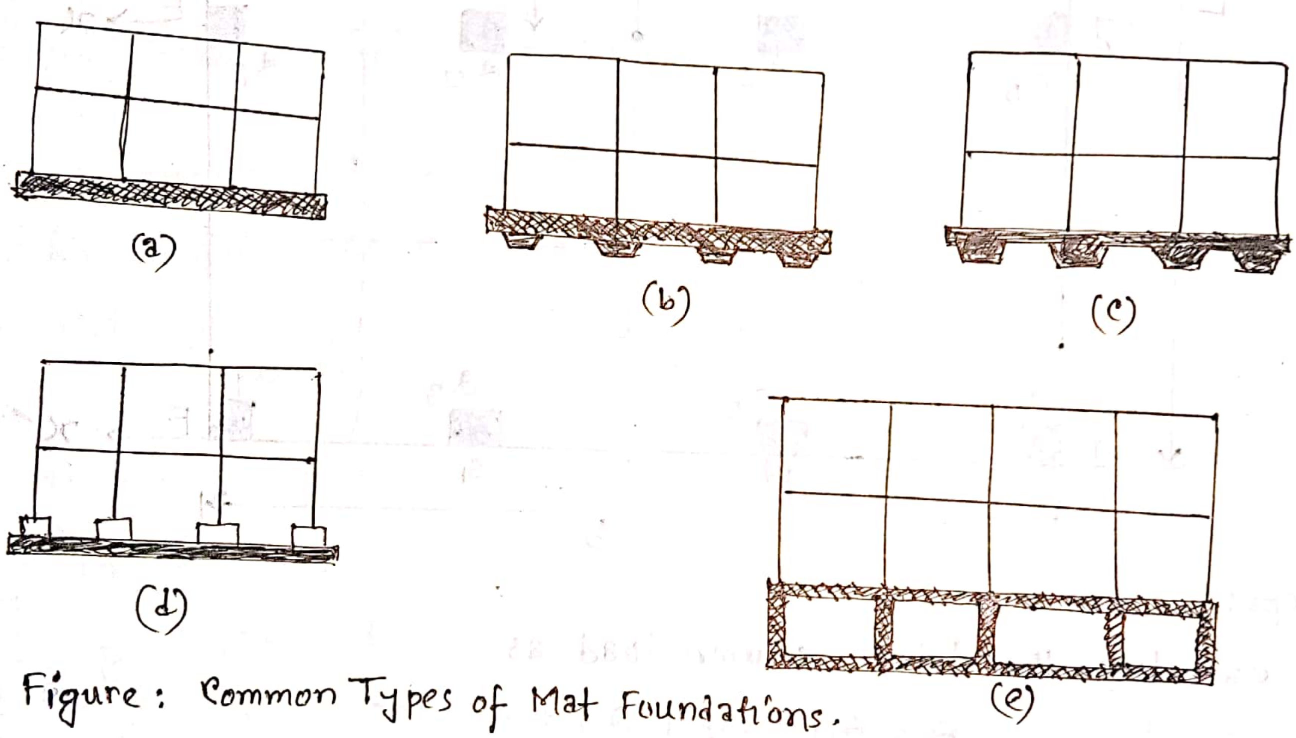


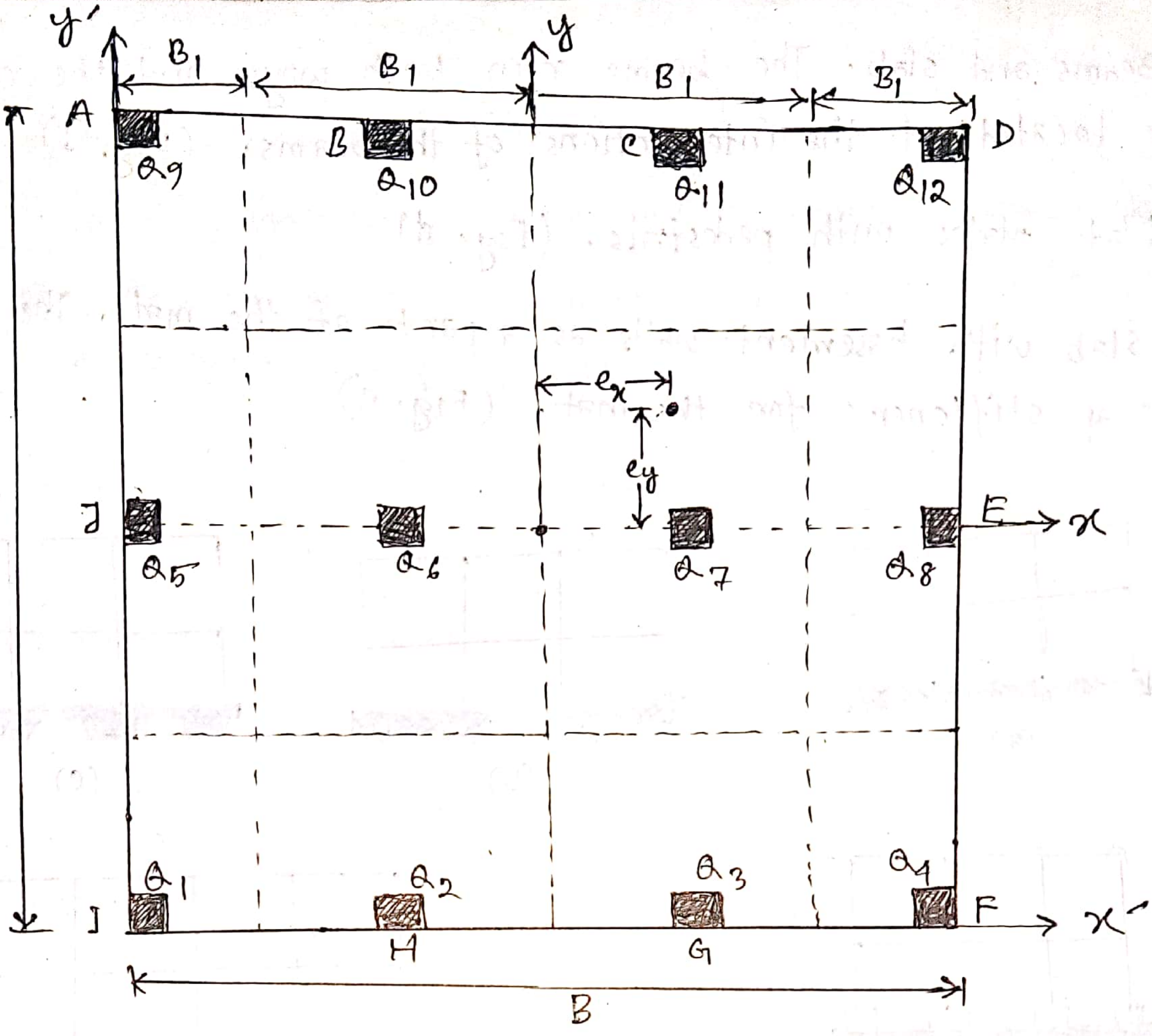
Figure: Common Types of Mat Foundations.

Discuss the design procedure of Mat foundation. 17, 16, 15, 08

The structural design of mat foundations can be carried out by two conventional methods:

1. The conventional rigid method.
2. The approximate flexible method.

The conventional rigid method of mat foundation design can be explained step by step with reference to the following figure:



Steps:

1. Calculate the total column load as

$$Q = Q_1 + Q_2 + Q_3 + \cdots$$

2. Determine the pressure on the soil, q below the mat at points A, B, C, ... by using the equation,

$$q = \frac{Q}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$$

where, $A = BL$

$$I_x = \frac{BL^3}{12}, \quad I_y = \frac{B^3 L}{12}$$

$$M_x = Q e_y, \quad M_y = Q e_x \qquad \text{Here, } e_x = x' - \frac{B}{2}$$

$$e_y = y' - \frac{L}{2}$$

x' and y' are calculated as follows:

$$x' = \frac{Q_1 x_1' + Q_2 x_2' + \cdots}{Q} \quad \text{and,} \quad y' = \frac{Q_1 y_1' + Q_2 y_2' + \cdots}{Q}$$

3. compare the values of the soil pressures with net allowable soil pressure to determine whether $q \leq q_{all(net)}$

4. Divide the mat into several strips in the x and y directions. Let the width of any strip be B_1.

5. Draw the shear V and the moment M, diagrams for each individual strip (in the x and y directions)

6. Determine the effective depth d of the mat by checking for diagonal tension shear near various columns. For critical section,

$V_c \geq U$ where, U = Factored column load
V_c = shear capacity at the column location.

7. From the moment diagrams of all strips in one direction (x or y). obtain the maximum positive and negative moments per unit width (i.e. $M_u = \frac{M}{B_1}$)

8. Determine the area of steel per unit width for positive and negative reinforcement in the x and y directions.

We have,

$$M_u = \phi A_s f_y \left(d - \frac{a}{2}\right)$$ where, $a = \frac{A_s f_y}{0.85 f_c' b}$

What is compensated Foundation? 13, 07, 16

The net pressure increase in the soil under a mat foundation can be reduced by increasing the depth D_f of the mat. This approach is generally referred to as the compensated foundation design. It is extremly useful when structures are to be built on very soft clays.

In this design, a deeper basement is made below the higher portion of the super structure, so that the net pressure increase in soil at any depth is relatively uniform.

The net average applied pressure on soil is,

$$q = \frac{Q}{A} - \gamma D_f$$

For no increase in the net pressure on soil below foundation, $q = 0$. Thus,

$$D_f = \frac{Q}{A\gamma}$$

This relation for D_f is usually referred to as the depth of a fully compensated foundation.

The factor of safety against bearing capacity failure for partially compansated foundations (i.e. $D_f < \frac{Q}{A\gamma}$) may be given as:

$$FS = \frac{q_{net(u)}}{q} = \frac{q_{net(u)}}{\frac{Q}{A} - \gamma D_f}$$

where,

$q_{net(u)}$ = Net ultimate bearing capacity

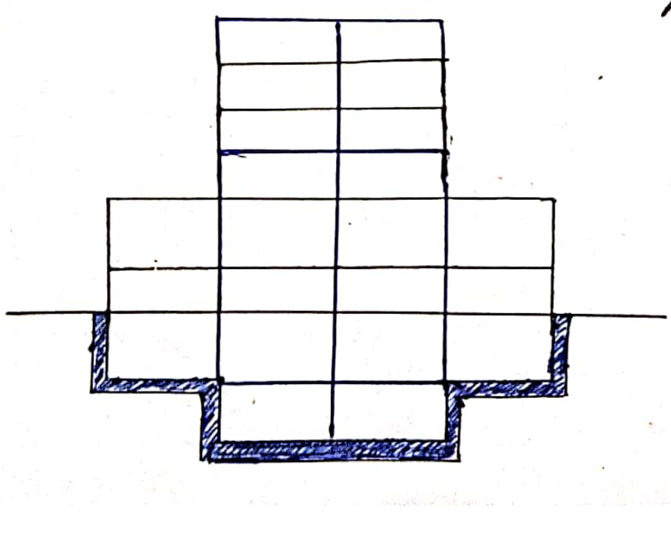


Fig. Compensated Foundation

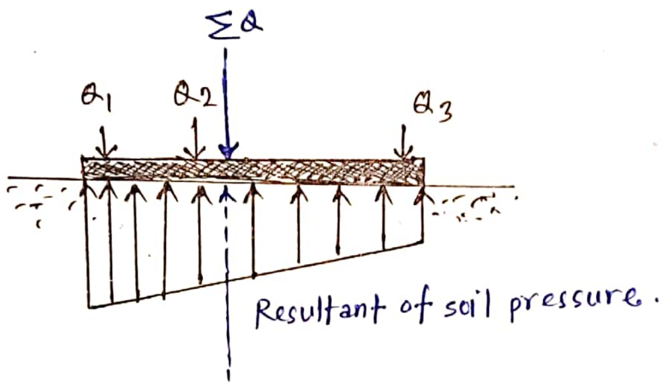
What are the design considerations of Mat Foundation? 10

In the conventional rigid method of design,

(i) The mat is assumed to be infinitely rigid.

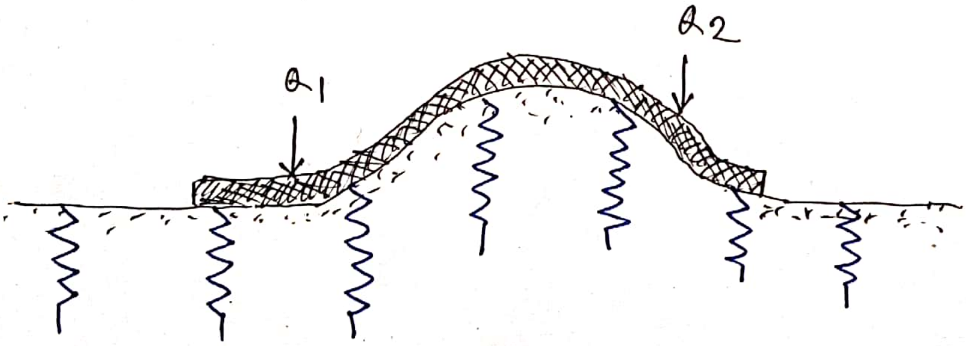
(ii) The soil pressure is distributed in a straight line.

(iii) The centroid of the soil pressure is coincident with the line of action of the resultant column loads.



In the approximate flexible method of design,

(i) The soil is assumed to be equivalent to an infinite number of elastic springs. This assumption is sometimes is referred to as the Winkler foundation.



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Mat Foundation

Bearing Capacity of Mat Foundations:

*** For Mats on saturated Clay ($\phi = 0$)**

The net ultimate bearing capacity,

$$q_{net(u)} = q_u - q = 5.14\, c_u\left(1 + \frac{0.195B}{L}\right)\left(1 + 0.4\frac{D_f}{B}\right)$$

$$q_{net(all)} = \frac{q_{net(u)}}{FS} \quad [\text{If not given, assume } FS = 3]$$

*** For Mats on granular soil ($c = 0$)**

The net allowable bearing capacity,

In SI Units:

$$q_{net(all)}\,[kN/m^2] = \frac{N_{60}}{0.08} \times \left(\frac{B+0.3}{B}\right)^2 \times F_d \left(\frac{S_e\,(mm)}{25}\right)$$

where, N_{60} = standard penetration number ; B = width (m)

$F_d = 1 + 0.33\left(\frac{D_f}{B}\right) \leq 1.33$; S_e = settlement (mm)

When the width B is large, $\left(\frac{B+0.3}{B}\right)^2 \simeq 1$

$$q_{net(all)}\,[kN/m^2] = \frac{N_{60}}{0.08} \times \left[1 + 0.33\left(\frac{D_f}{B}\right)\right] \times \left[\frac{S_e\,(mm)}{25}\right]$$

$$\leq 16.33\, N_{60}\left[\frac{S_e\,(mm)}{25}\right]$$

In English Units:

$$q_{net(all)}\,[kip/ft^2] = 0.25\, N_{60} \times \left[1 + 0.33\left(\frac{D_f}{B}\right)\right] \times [S_e\,(in)]$$

$$\leq 0.33\, N_{60}\,[S_e\,(in)]$$

*** The net pressure applied on a foundation,**

$$q = \frac{Q}{A} - \gamma D_f$$

where, Q = Dead weight of the structure and Live load.

A = Area of raft

In all cases, $q \leq q_{net(all)}$

Reference Book: Foundation Engineering — BM Das (8th Edition) / (7th Edition)

Example-8.3 (8th edition)

Determine the net Ultimate bearing capacity of a mat foundation measuring 20 m × 8 m on a saturated clay with $c_u = 85$ KN/m², $\phi = 0$ and $D_f = 1.5$ m

Solution: Given, $L = 20$ m, $B = 8$ m, $c_u = 85$ KN/m², $D_f = 1.5$ m

For Mats on saturated clay ($\phi = 0$),

$$q_{net(u)} = 5.14\, c_u \left[1 + \frac{0.195 B}{L}\right]\left[1 + 0.4\frac{D_f}{B}\right]$$

$$= 5.14 \times 85 \times \left[1 + \frac{0.195 \times 8}{20}\right] \times \left[1 + 0.4 \times \frac{1.5}{8}\right]$$

$$= 506.3 \text{ KN/m}^2$$

(Ans)

Example-8.4

What will be the net allowable bearing capacity of a Mat foundation with dimensions of 45 ft × 30 ft constructed over a sand deposit? Here, $D_f = 6.5$ ft, The allowable settlement is 2 in and the average penetration number $N_{60} = 10$

Solution: Given, $L = 45$ ft, $B = 30$ ft, $D_f = 6.5$ ft, $S_e = 2$ in, $N_{60} = 10$

For Mats on sand deposit,

$$q_{net(all)} = 0.25 N_{60} \left[1 + 0.33 \times \frac{D_f}{B}\right] \times S_e \text{ (in)}$$

$$= 0.25 \times 10 \times \left[1 + 0.33 \times \frac{6.5}{30}\right] \times 2$$

$$= 5.3575 \text{ kip/ft}^2 < 0.33 N_{60} S_e(\text{in}) \text{ (OK)}$$

(Ans)

Settlement of Mats:

For Normally Consolidated Clay,

The primary consolidation settlement under a foundation, $(\sigma_o' > \sigma_c')$

$$S_{c(p)} = \frac{C_c H}{1+e_o} \log\left(\frac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_o'}\right)$$

where,

$$C_c = 0.009\,(LL-10)$$

$$\Delta\sigma_{av}' = \frac{\Delta\sigma_t' + 4\Delta\sigma_m' + \Delta\sigma_b'}{6}$$

Influence Factor, * $I_c = \frac{2}{\pi} \times \left[\frac{m_1 n_1}{\sqrt{m_1^2+n_1^2+1}} \times \frac{m_1^2+2n_1^2+1}{(m_1^2+n_1^2)(1+n_1^2)} + \underbrace{\sin^{-1}\left(\frac{m_1}{\sqrt{1+n_1^2} \times \sqrt{m_1^2+n_1^2}}\right)}_{\text{radian mode}}\right]$

for center of the foundation

The secondary consolidation settlement,

$$S_s = C_\alpha' H \log\left(\frac{t_2}{t_1}\right)$$

Where, $C_\alpha' = \frac{C_\alpha}{1+e_p}$

* t_1 = time for completion of primary settlement
* t_2 = Total time of consolidation settlement

* $C_\alpha = \frac{\Delta e}{\log\left(\frac{t_2}{t_1}\right)}$
* $e_p = e_o - \Delta e$
* $\Delta e = C_c \log\left(\frac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_o'}\right)$

Compensated Foundation:

Factor of safety against bearing capacity failure for partially compensated foundations (i.e. $D_f < \frac{Q}{A\gamma}$) may be given as,

$$FS = \frac{q_{net(u)}}{q} = \frac{5.14 C_u \left(1 + \frac{0.195B}{L}\right)\left(1 + 0.4 \times \frac{D_f}{B}\right)}{\frac{Q}{A} - \gamma D_f}$$

Example : 8.5

A Mat foundation of dimensions 20 m X 30 m is shown in the figure. The total dead load and live load on the mat is 110 MN. The mat is placed over a saturated clay having a unit weight of 18 KN/m3 and $c_u = 140$ KN/m². Given that, $D_f = 1.5$ m. Determine the factor of safety against bearing capacity failure.

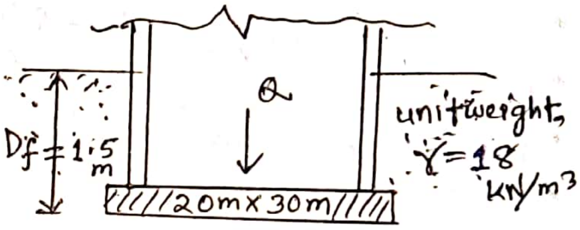
Solution: Given, $D_f = 1.5$ m, $\gamma = 18$ KN/m3

$c_u = 140$ KN/m²

B = 20 m

L = 30 m

Q = 110 MN = 110×10^3 KN



We know, $$FS = \frac{5.14\, c_u \left(1 + \frac{0.195 B}{L}\right) \times \left(1 + 0.4 \times \frac{D_f}{B}\right)}{\left(\frac{Q}{A} - \gamma D_f\right)}$$

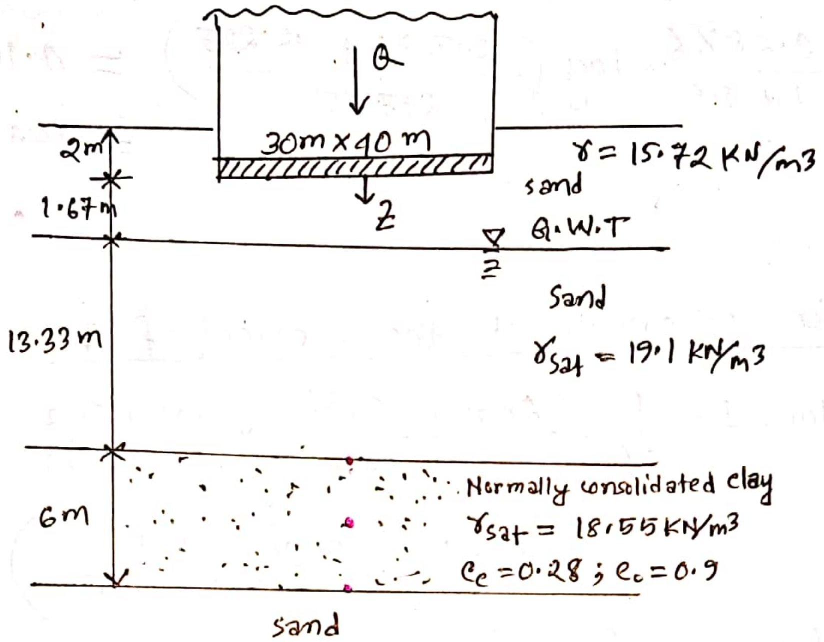
$$= \frac{5.14 \times 140 \times \left(1 + \frac{0.195 \times 20}{30}\right) \times \left(1 + \frac{1.5}{20}\right)}{\left(\frac{110 \times 10^3}{20 \times 30} - 18 \times 1.5\right)}$$

$\therefore$ FS = 5.36

(Ans.)

Example : 8.6

Consider a mat foundation 30 m X 40 m in plan as shown in Figure. The total dead load and live load on the raft 200×10^3 KN. Estimate the consolidation settlement at the center of the foundation.



Solution: For Normally Consolidated Clay,

$$S_{c(p)} = \frac{C_c H}{1+e_o} \log\left(\frac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_o'}\right)$$

Here, $C_c = 0.28$, $H = 6m$, $e_o = 0.9$

$$\sigma_o' = 15.72 \times 3.67 + (19.1 - 9.81) \times 13.33 + \frac{6}{2} \times (18.55 - 9.81)$$

$$= 207.75 \text{ KN/m}^2$$

$$q_o = \frac{200 \times 10^3}{30 \times 40} = 166.67 \text{ KN/m}^2$$

For $\Delta\sigma_{av}'$ following table to be prepared:

$$* \; I_c = \frac{2}{\pi}\left[\frac{m_1 n_1}{\sqrt{m_1^2+n_1^2+1}} \times \frac{m_1^2+2n_1^2+1}{(m_1^2+n_1^2)(n_1^2+1)} + \sin^{-1}\left(\frac{m_1}{\sqrt{m_1^2+n_1^2}\sqrt{n_1^2+1}}\right)\right]$$

$m_1 = \frac{L}{B}$	z (m)	$b = \frac{B}{2}$ (m)	$n_1 = \frac{z}{b}$	q_o (KN/m²)	* I_c	$\Delta\sigma' = q_o I_c$ (KN/m²)	$\Delta\sigma_{av}' = \frac{\Delta\sigma_t' + 4\Delta\sigma_m' + \Delta\sigma_b'}{6}$ (KN/m²)
1.33	15	15	1	166.67	0.76	$\Delta\sigma_t' = 126.67$	
1.33	18	15	1.2	166.67	0.67	$\Delta\sigma_m' = 111.67$	112.225 KN/m²
1.33	21	15	1.4	166.67	0.60	$\Delta\sigma_b' = 100$	

Hence, At the center of the foundation,

$$S_{c(p)} = \frac{0.28\times 6}{1+0.9}\times \log\left(\frac{207.75+112.225}{207.75}\right) = 0.166\ m$$

$$= 166\ mm$$

(Ans.)

For Consolidation Settlement at the $\boxed{\text{corner}}$ of the foundation:

Influence factor,

$$I = \frac{1}{4\pi}\times\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}\times\frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2}\right)$$

where, $m = \frac{B}{Z}$ and $n = \frac{L}{Z}$

when $m^2+n^2+1 < m^2n^2$, a term $\boxed{\pi}$ should be $\boxed{\text{added}}$ to that angle.

In Over consolidated Clay,

For $\sigma_0' < \sigma_0' + \Delta\sigma' \leq \sigma_c'$,

$$S_{c(p)} = \frac{C_s H}{1+e_0}\log\left(\frac{\sigma_0'+\Delta\sigma'}{\sigma_0'}\right)$$

where, $C_s = \frac{1}{5}$ to $\frac{1}{10}\ C_c$

σ_c' = Preconsolidation pressure

For $\sigma_0' + \Delta\sigma' > \sigma_c'$,

$$S_{c(p)} = \frac{C_s H}{1+e_0}\log\frac{\sigma_c'}{\sigma_0'} + \frac{C_c H}{1+e_0}\log\left(\frac{\sigma_0'+\Delta\sigma'}{\sigma_c'}\right)$$

Conventional Rigid Method: (Structural Design of Mat)

1. Total Factored Column Load, $Q_u = Q_1 + Q_2 + Q_3 + \cdots\cdots$

Here, $(Q_* = 1.4\,D.L + 1.7\,L.L)$

2. Area $= B \times L$

3. The pressure on soil, $q = \frac{Q}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$

Here, $I_x = \frac{BL^3}{12}$

$$I_y = \frac{B^3 L}{12}$$

$$x' = \frac{Q_1 x_1' + Q_2 x_2' + \cdots\cdots}{\text{Total column load}}$$

$$y' = \frac{Q_1 y_1' + Q_2 y_2' + \cdots\cdots}{\text{Total column load}}$$

$$e_x = x' - \frac{B}{2}$$

$$e_y = y' - \frac{L}{2}$$

$$M_{uy} = Q_u e_x$$

$$M_{ux} = Q_u e_y$$

3. Check whether $q \leq q_{all\,(net)}$

4. Divide the mat into several strips in x and y directions.

Let the width of any strip be B_1.

5. Draw SFD and BMD for each individual strips (in x and y direction)

$$\text{Average soil pressure, } q_{av} = \frac{q_A + q_F}{2}$$

x direction y direction

Total column load on a strip

$$\text{Average load} = \frac{q_{av} B_1 \times (B \text{ or } L) + (Q_1 + Q_2 + \ldots)}{2}$$

$$q_{av(\text{modified})} = q_{av} \times \frac{\text{Average load}}{q_{av} B_1 \times (B \text{ or } L)}$$

$$\text{Column load modification factor, } F = \frac{\text{Average load}}{(Q_1 + Q_2 + \ldots)}$$

Modified column Load are FQ_1, FQ_2 ....

6. Determine the effective depth d of the mat by checking for diagonal tension shear near various columns.

According to ACI Code 318-95, for the critical section,

$$U = b_o d \left[\phi \times (0.34) \times \sqrt{f_c'}\right]$$

In WSD,
$$v = b_o d \left(\frac{1}{6}\sqrt{f_c'}\right)$$

where, U = factored column load (bo & d are in meters)

ϕ = reduction factor = 0.85

f_c' = compressive strength of concrete at 28 days (MN/m^2)

In English units, $U = b_o d (4 \phi \sqrt{f_c'})$

In WSD,
$$v = b_o d (2\sqrt{f_c'})$$

where U is in lb, d are in inch and f_c' is in lb/in^2

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7. Obtain Maximum positive and negative moments per unit width

$$M_u = \frac{M}{B_1}$$

8. Determine the Area of steel per unit width,

$$M_u = \phi A_s f_y \left(d - \frac{a}{2}\right)$$ in WSD, $M = A_s f_s j d$

and, $$a = \frac{A_s f_y}{0.85 f_c' b}$$

Example: 8.7 (USD Method)

The plan of a mat foundation is shown in figure below. Calculate the soil pressure at points A, B, C, D, E and F. (Note: All column sections are planned to be 0.5 m × 0.5 m) All loads shown are $\boxed{\text{factored}}$ loads according to ACI 381-11 (2011)

USD method

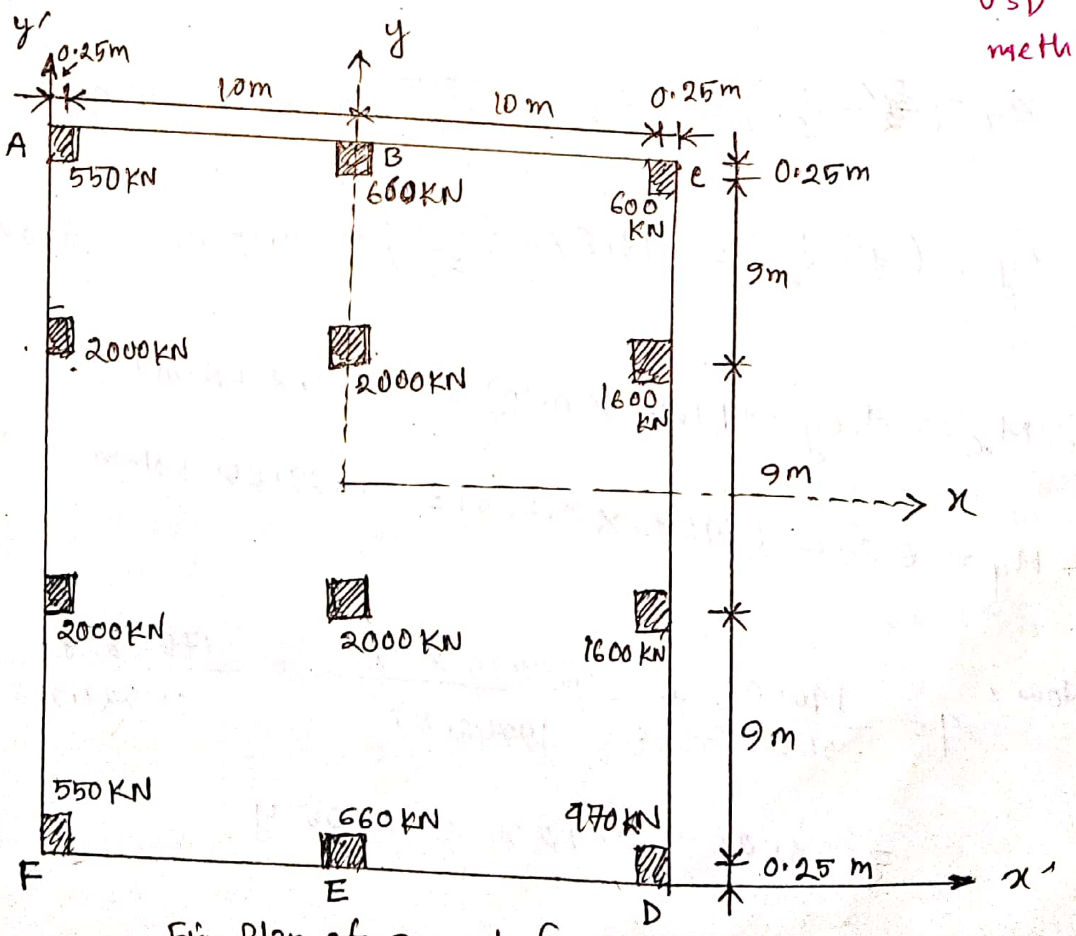

Fig. Plan of a mat foundation

<u>solution</u>: We know, $q = \frac{Q}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$

$Q = (2\times550 + 4\times2000 + 2\times660 + 2\times1600 + 600 + 470) = 14690\ KN$

Area, $A = (27.5 \times 20.5) = 563.75\ m^2$

$I_x = \frac{20.5 \times 27.5^3}{12} = 35527.995\ m^4$

$I_y = \frac{20.5^3 \times 27.5}{12} = 19742.995\ m^4$

$$x' = \frac{0.25\times(550 + 2\times2000 + 550) + 10.25\times(2\times660 + 2\times2000) + 20.25\times(470 + 2\times1600 + 600)}{14690}$$

$= 9.685\ m$

$$y' = \frac{0.25\times(470 + 660 + 550) + 9.25\times(1600 + 2\times2000) + 18.25\times(1600 + 2\times2000) + 27.25\times(600 + 660 + 550)}{14690}$$

$= 13.87\ m$

$e_x = (x' - \frac{B}{2}) = (9.685 - \frac{20.5}{2}) = -0.565\ m$ (left of the center)

$e_y = (y' - \frac{L}{2}) = (13.87 - \frac{27.5}{2}) = 0.12\ m$ (Above the center)

$\therefore M_x = Q e_y = (14690 \times 0.12) = 1762.8\ KN\text{-}m$

$M_y = Q e_x = (14690 \times 0.565) = 8299.85\ KN\text{-}m$

Now, $q = \frac{14690}{563.75} \pm \frac{8299.85 \times x}{19742.995} \pm \frac{1762.8\ y}{35527.995}$

$= 26.06 \pm 0.42x \pm 0.05y$

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There fore,

At A, $q = 26.06 + 0.42 \times 10.25 + 0.05 \times 13.75 = 31.0525 \text{ KN/m}^2$

At B, $q = 26.06 + 0.42 \times 0 + 0.05 \times 13.75 = 26.7475 \text{ KN/m}^2$

At C, $q = 26.06 - 0.42 \times 10.25 + 0.05 \times 13.75 = 22.4425 \text{ KN/m}^2$

At D, $q = 26.06 - 0.42 \times 10.25 - 0.05 \times 13.75 = 21.0675 \text{ KN/m}^2$

At E, $q = 26.06 + 0.42 \times 0.00 - 0.05 \times 13.75 = 25.3725 \text{ KN/m}^2$

At F, $q = 26.06 + 0.42 \times 10.25 - 0.05 \times 13.75 = 29.6775 \text{ KN/m}^2$

Example: 8.8

Divide the mat shown in previous question into three strips such as AGHF ($B_1 = 5.25$ m), GIJH ($B_1 = 10$ m), and ICDJ ($B_1 = 5.25$ m). Use the result of Example 8.7 and determine the reinforcement in the y direction. Here $f_c' = 20.7 \text{ MN/m}^2$, $f_y = 413.7 \text{ MN/m}^2$. Note: All column loads are factored loads.

Solution:

Determination of shear and Moment Diagram for strips:

strip AGHF:

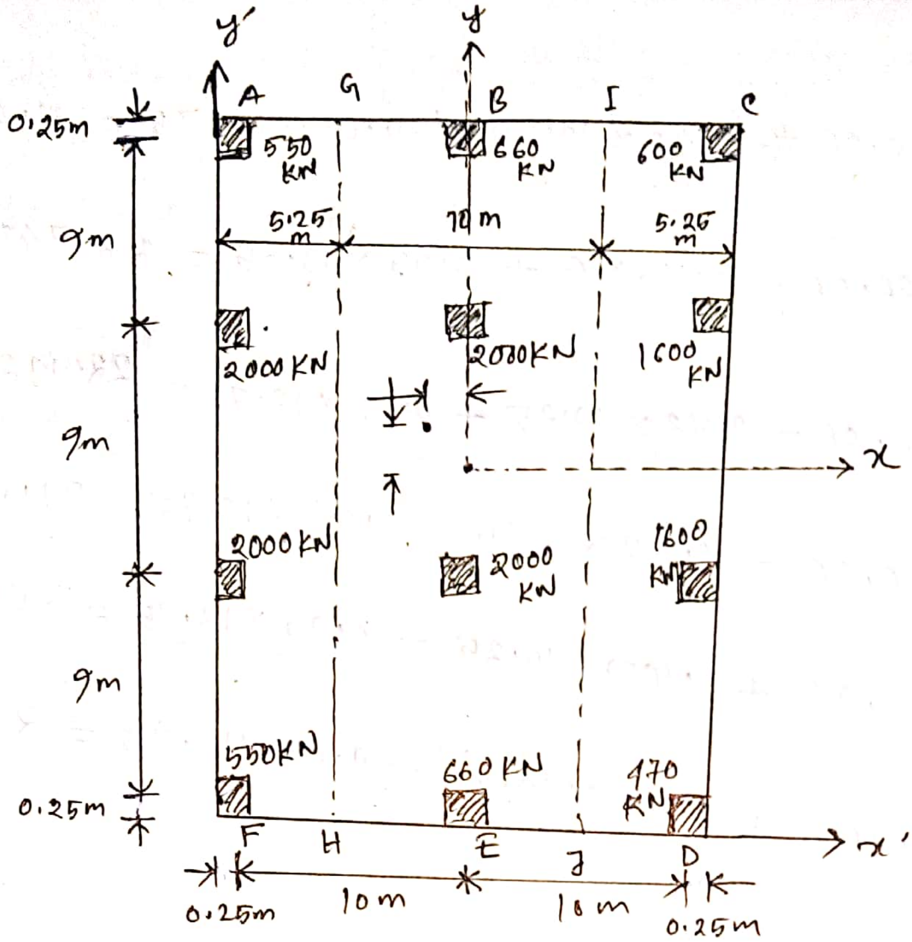
Average soil pressure, $q_{av} = \dfrac{q_{(at\,A)} + q_{(at\,F)}}{2} = \dfrac{31.0525 + 29.6775}{2}$

$= 30.365 \text{ KN/m}^2$

Total soil reaction $= q_{av} B_1 L$

$= (30.365 \times 5.25 \times 27.5)$

$= 4383.95$ KN



$$\text{Average load} = \frac{\text{Total soil reaction} + \text{column loads}}{2} = \frac{4383.95 + 5100}{2}$$

$$= 4741.975 \text{ KN}$$

So modified average soil pressure,

$$q_{av\,(modified)} = q_{av} \times \left(\frac{4741.975}{4383.95}\right) = 30.365 \times \frac{4741.975}{4383.95}$$

$$= 32.845 \text{ KN/m}^2$$

The Multiplying factor,

$$F = \frac{4741.975}{5100} = 0.93$$

The load per unit length of the beam $= B_1 q_{av\,(modified)}$

$$= (5.25 \times 32.845)$$

$$= 172.44 \text{ KN/m}$$

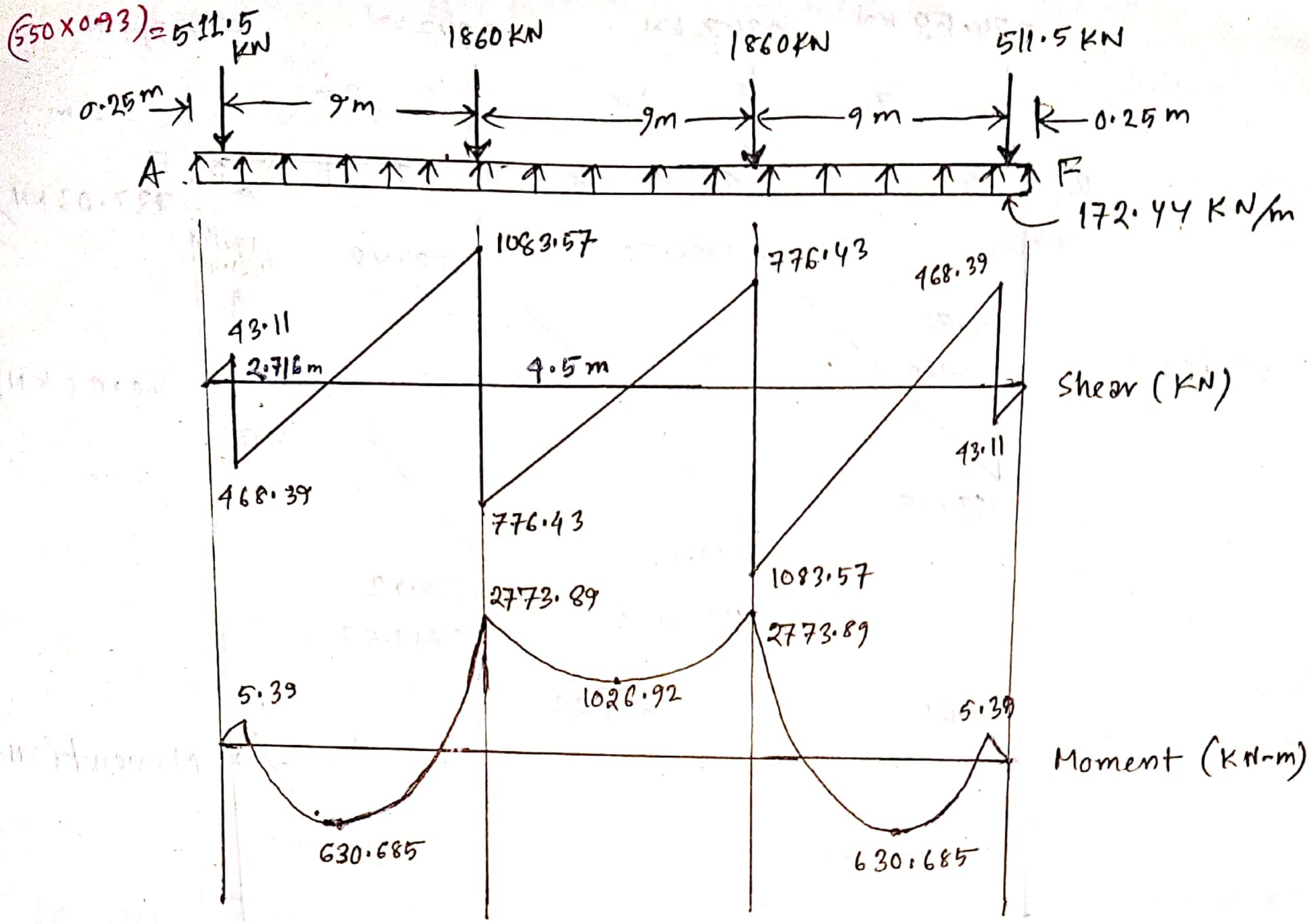


Figure. SFD & BMD for strip AGHF

Strip GIJH: In a similar manner,

$$q_{av} = \frac{26.7475 + 25.3725}{2} = 26.06 \ KN/m^2$$

Total soil reaction $= q_{av} B_1 L = (26.06 \times 10 \times 27.5) = 7166.5$ KN

Total column load $= (660 + 2000 \times 2 + 660) = 5320$ KN

$$\therefore \text{Average load} = \frac{7166.5 + 5320}{2} = 6243.25 \ KN$$

$$q_{av\,(modified)} = \frac{6243.25}{7166.5} \times 26.06 = 22.703 \ KN/m^2$$

$$F = \frac{6243.25}{5320} = 1.1735$$

The load per unit length of the beam $= q_{av\,(modified)} \times B_1$
$= (22.703 \times 10) = 227.03$ KN/m

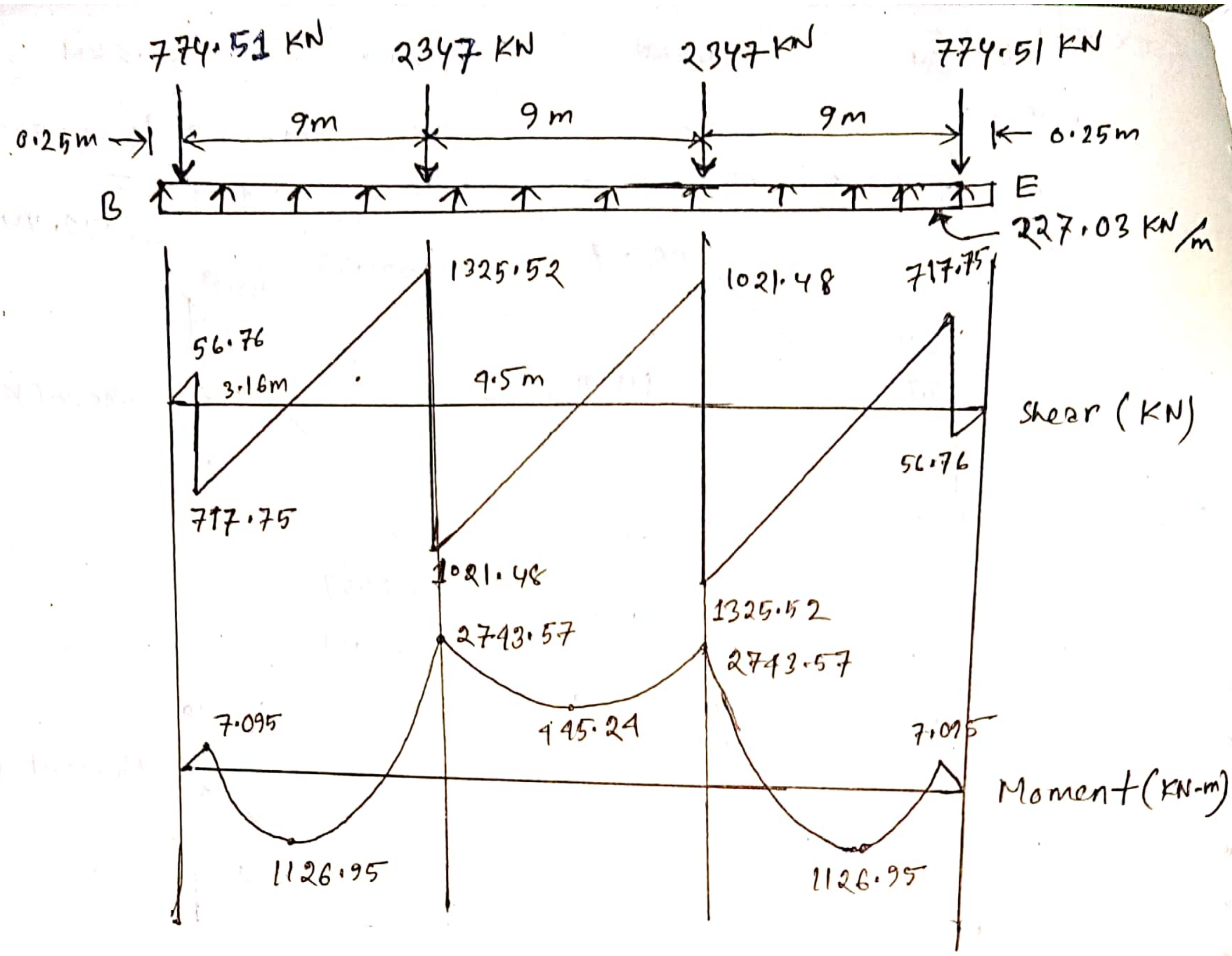


Figure: SFD & BMD for strip GIJH

<u>Strip ICDJ:</u> $q_{av} = \dfrac{22.4425 + 21.0675}{2} = 21.754 \text{ KN/m}^2$

soil reaction $= q_{av} B_1 L = (21.754 \times 5.25 \times 27.5) = 3140.73$ KN

Total column load $= (600 + 1600 \times 2 + 470) = 4270$ KN

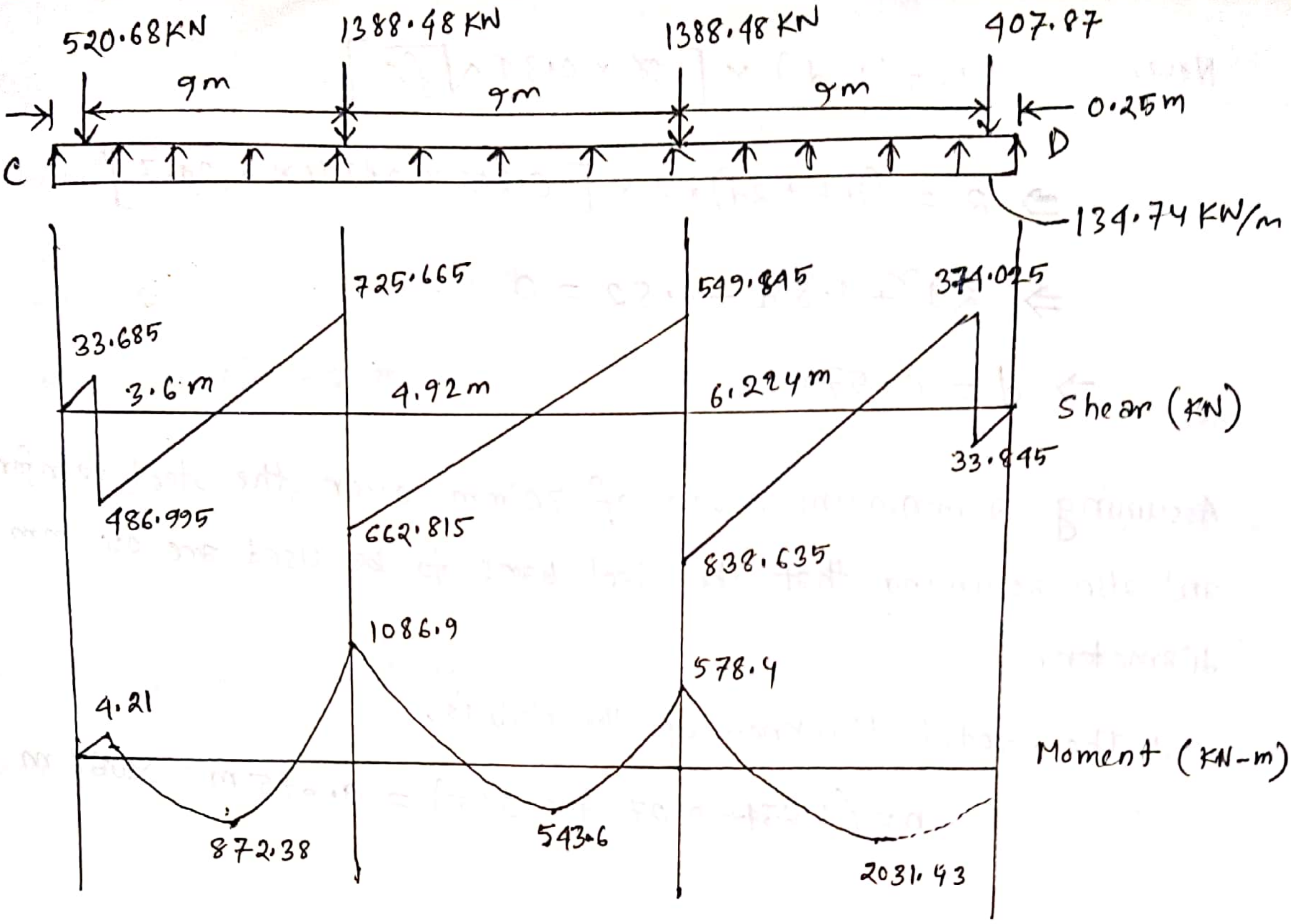
Average load $= \dfrac{3140.73 + 4270}{2} = 3705.365$

q_{av} (modified) $= 21.754 \times \left(\dfrac{3705.365}{3140.73}\right) = 25.665$

$F = \dfrac{3705.365}{4270} = 0.8678$

The load per unit length of the beam $= (25.665 \times 5.25)$

$= 134.74$ KN/m

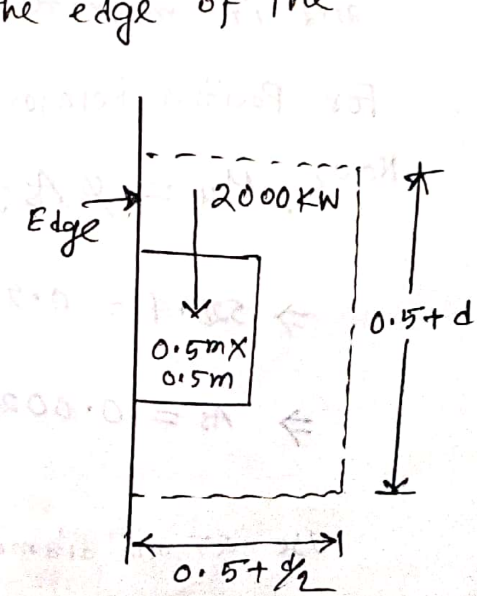


In view of the assumption of uniform soil reaction to non symmetric loading, their is discrepancy in the moment values at right column. As a result the moment diagram will not close. This is ignored since it is not the governing design moment.

Determination of thickness of Mat:

The critical section for diagonal tension shear will be at the edge of the column carrying 2000 KN of load at the edge of the mat.

$U = 2000\,KN = 2\,MN$



The critical perimeter,

$$b_o = \left(0.5+\frac{d}{2}\right)\times 2 + (0.5+d)$$

$$= 1.5+2d$$

Now, $U = (b_o d) \times [\phi \times 0.34 \sqrt{f_c'}]$

$\Rightarrow 2 = (1.5 + 2d) \times d \times [0.85 \times 0.34 \times \sqrt{20.7}]$

$\Rightarrow 2d^2 + 1.5d - 1.52 = 0$

$\Rightarrow d = 0.574$ m

Assuming a minimum cover of 76 mm over the steel reinforcement and also assuming that the steel bars to be used are 25 mm in diameter.

$\therefore$ the total thickness of the slab is,

$$h = (0.574 + 0.076 + 0.025) = 0.675 \text{ m} \simeq 0.68 \text{ m}$$

Determination of Reinforcement:

From the moment diagram, it can be seen that the maximum positive moment is located in strip AGHF and,

Its Magnitude, $M_u = \frac{2774}{B_1} = \frac{2774}{5.25} = 528.4$ KN-m/m

similarly the maximum negative moment is located in strip ICDJ and its magnitude, $M_u = \frac{2032}{B_1} = \frac{2032}{5.25} = 387.05$ KN-m/m

For Positive Reinforcement,

Now, $M_u = \phi A_s f_y (d - \frac{a}{2})$

$\Rightarrow 528.4 = 0.9 \times A_s \times (413.7 \times 1000) \times (0.68 - \frac{23.51 A_s}{2})$

$\Rightarrow A_s = 0.0022$ m²/m $= 2200$ mm²/m

Here,

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

$\Rightarrow a = \frac{A_s \times 413.7}{0.85 \times 20.7 \times 1}$

$\therefore a = 23.51 A_s$.

Use 25 mm diameter bars at $(\frac{\pi}{4} \times 25^2 \times \frac{1000}{2200}) = 223.125 \simeq 220$ mm c/c

For negative reinforcement,

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

$$\Rightarrow 387.05 = 0.9 \times A_s \times (413.7 \times 1000) \times \left(0.68 - \frac{23.51}{2} A_s\right)$$

$$\Rightarrow A_s = 0.00157 \; m^2/m = 1570 \; mm^2/m$$

use 25 mm diameter bars at $\left(\frac{\pi}{4} \times 25^2 \times \frac{1000}{1570}\right) = 312.66 \; mm$

$\simeq 300 \; mm \; c/c$

Because negative moment occurs at midway of strip ICDJ, Reinforcement should be provided. This moment is,

$$M_u = \frac{544}{5.25} = 103.62 \; KN\text{-}m/m$$

Hence,

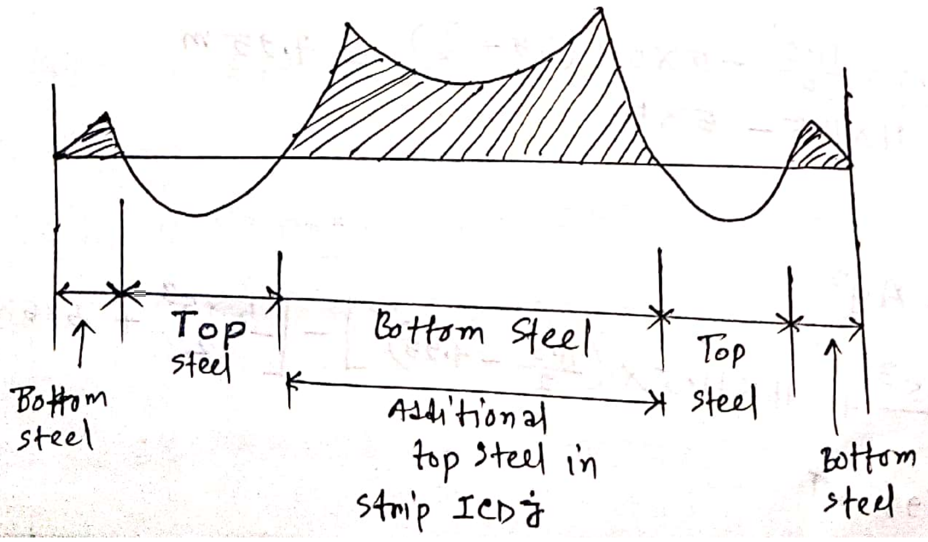
$$103.62 = 0.9 \times A_s \times (413.7 \times 1000)\left(0.68 - \frac{23.51}{2} A_s\right)$$

$$\Rightarrow A_s = 0.0004122 \; m^2/m = 412.2 \; mm^2/m$$

provide 16 mm diameter bars at $\left(\frac{\pi}{4} \times 16^2 \times \frac{1000}{412.2}\right) = 487.78 \; mm$

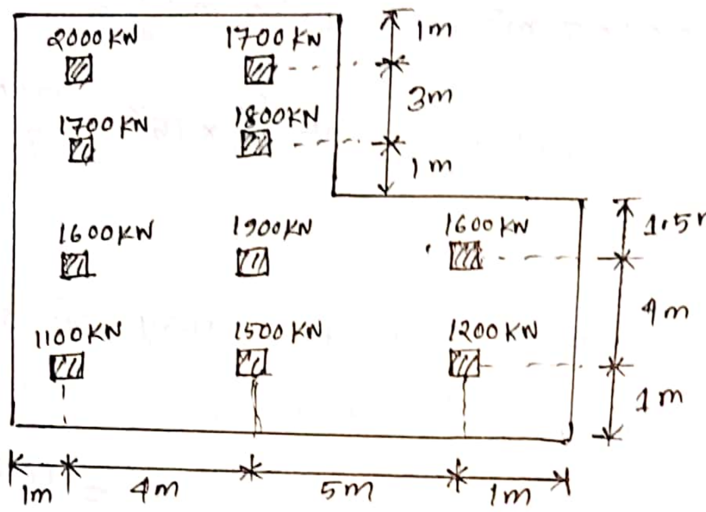
$\simeq 475 \; mm \; c/c$

General Arrangement of Reinforcement



2012

Suppose you are going to design a mat foundation as shown in figure below using the data $f_c' = 15\ MPa$ and $q_a = 15\ MPa$ and column size = 30 cm X 30 cm. Design the mat foundation.



Solution: Total Column load, $Q = (2000 + 1700 \times 2 + 1600 \times 2 + 1800 + 1900 + 1100 + 1500 + 1200)$

$= 16100$ KN

Area, $A = B \times L - B' \times L'$

$= (11 \times 11.5) - (5 \times 5) = 101.5\ m^2$

Area Center:

$$x = \frac{11 \times 11.5 \times \frac{11}{2} - 5 \times 5 \times (6 + \frac{5}{2})}{11 \times 11.5 - 5 \times 5} = 4.76\ m$$

$$y = \frac{11 \times 11.5 \times \frac{11.5}{2} - 5 \times 5 \times (6.5 + \frac{5}{2})}{11 \times 11.5 - 5 \times 5} = 4.95\ m$$

Moment of Inertia:

$$I_x = \frac{BL^3}{12} + Ay^2$$

$$= \left[\frac{11 \times 11.5^3}{12} + 11 \times 11.5 \times \left(\frac{11.5}{2} - 4.95\right)^2\right] - \left[\frac{5 \times 5^3}{12} + 5 \times 5 \times \left(6.5 + \frac{5}{2} - 4.95\right)^2\right]$$

$= 1012.95\ m^4$

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$$I_y = \frac{B^3 L}{12} + A x^2$$

$$= \left[\frac{11^3 \times 11.5}{12} + 11 \times 11.5 \times \left(\frac{11}{2} - 4.76\right)^2\right] - \left[\frac{5^3 \times 5}{12} + 5 \times 5 \times \left(6 + \frac{5}{2} - 4.76\right)^2\right]$$

$$= 943.04 \ m^4$$

<u>Load center:</u>

$$x' = \frac{1 \times 6400 + 5 \times 6900 + 10 \times 2800}{16100} = 4.28 \ m$$

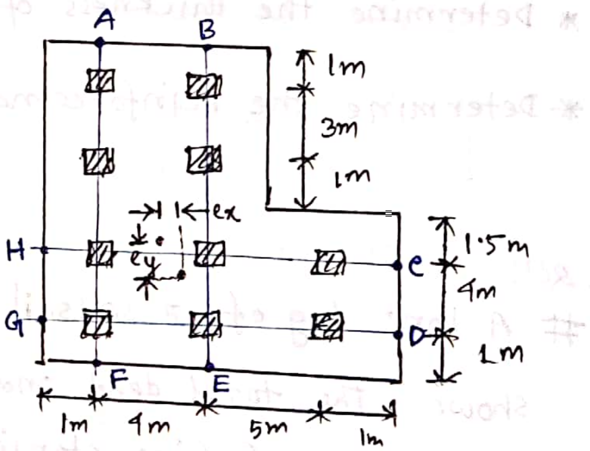
$$y' = \frac{1 \times 3800 + 5 \times 5100 + 7.5 \times 3500 + 10.5 \times 3700}{16100} = 5.86 \ m$$

$e_x = x' - x = (4.28 - 4.76) = -0.48 \ m$ (left of the area center)

$e_y = y' - y = (5.86 - 4.95) = 0.91 \ m$ (Above of the area center)

$M_y = Q e_x = (16100 \times 0.48) = 7728 \ KN\text{-}m$

$M_x = Q e_y = (16100 \times 0.91) = 14651 \ KN\text{-}m$



Now, $q = \frac{Q}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$

$$= \frac{16100}{101.5} \pm \frac{7728}{943.04} x \pm \frac{14651}{1012.95} y$$

$$= 158.62 \pm 8.195x \pm 14.464y$$

$q_A = 158.62 + 8.195 \times 3.76 + 14.464 \times 6.55$

$= 284.2 \ KN/m^2$

$q_B = 158.62 - 8.195 \times 0.24 + 14.464 \times 6.55 = 251.4 \ KN/m^2$

$q_C = 158.62 - 8.195 \times 6.24 + 14.464 \times 0.05 = 108.21 \ KN/m^2$

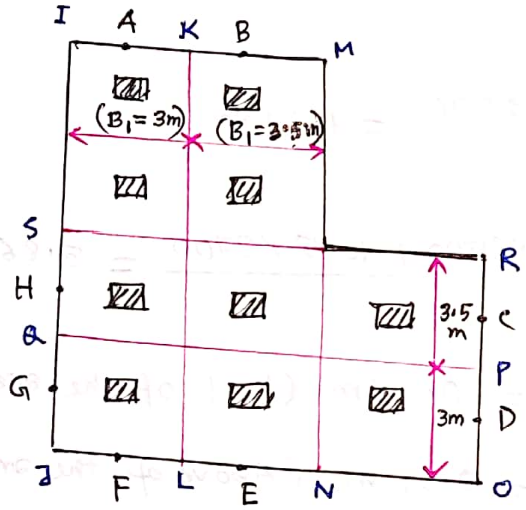
$q_D = 158.62 - 8.195 \times 6.24 - 14.464 \times 3.95 = 50.35 \ KN/m^2$

$q_E = 158.62 - 8.195 \times 0.24 - 14.464 \times 4.95 = 85.06 \ KN/m^2$

$q_F = 158.62 + 8.195 \times 3.76 - 14.464 \times 4.95 = 117.84 KN/m^2$

$q_G = 158.62 + 8.195 \times 4.76 - 14.464 \times 3.95 = 140.5 KN/m^2$

$q_H = 158.62 + 8.195 \times 4.76 + 14.464 \times 0.05 = 198.35 KN/m^2$



y direction x direction

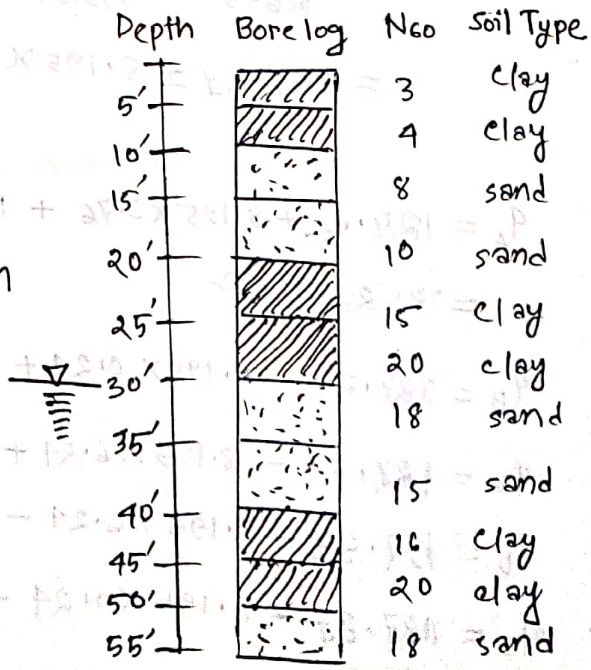
* Determine shear and Moment for strips $\boxed{\text{IAKLFJ}}$, $\boxed{\text{KBMNEL}}$, $\boxed{\text{PDOJGQ}}$ and $\boxed{\text{RCPQHS}}$

* Determine the thickness of Mat.

* Determine the reinforcement in y direction and x direction.

2013

A bore log of a subsoil exploration is shown. The total dead and live load of a column of five stories building is 90 tons. Select the type of foundation, depth and size of foundation. Make same suggestions if needed.



Solution: Assume, Type of foundation is shallow and Depth of foundation is between 10 ft to 15 ft.

Average SPT Value, $N_{60} = \frac{4+8}{2} = 6$

We know, For SPT value = 1, Allowable bearing capacity,

$q_{all} = 0.15$ Tsf

Hence, for SPT value = 6, $q_{all} = (0.15 \times 6)$ Tsf

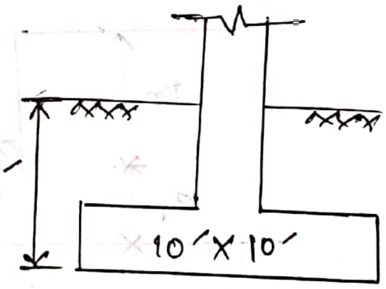
$= 0.9$ Tsf

Given, Total column Load = 90 Ton

Area required $= \frac{Q}{q_{all}} = \frac{90}{0.9} = 100$ sq. ft

So, we can select a square footing of dimension (10 ft × 10 ft)

and, depth of foundation = 12 ft



2014

Determine the net ultimate bearing capacity of mat foundation with the following field test data: (sandy soil)

Depth (m)	Field Value of N_{60}
1.5	8
3.0	12
4.5	10
6.0	8
7.5	14
9.0	12
10.5	14

Assume, $D_f = 1.5$ m, $\gamma = 16$ KN/m³ and allowable settlement = 5.0 cm, foundation size = 6.5 m × 5 m

Solution: Average SPT Value, $N_{60} = \frac{8+12+10+8+14+12+14}{7} = 11.14$

$\simeq 11$

We know, $q_{net(all)} = \frac{N_{60}}{0.08}\left[1+0.33\frac{D_f}{B}\right] \times \frac{S_e(mm)}{25}$

$= \frac{11}{0.08}\left[1+0.33 \times \frac{1.5}{5}\right] \times \frac{50}{25}$

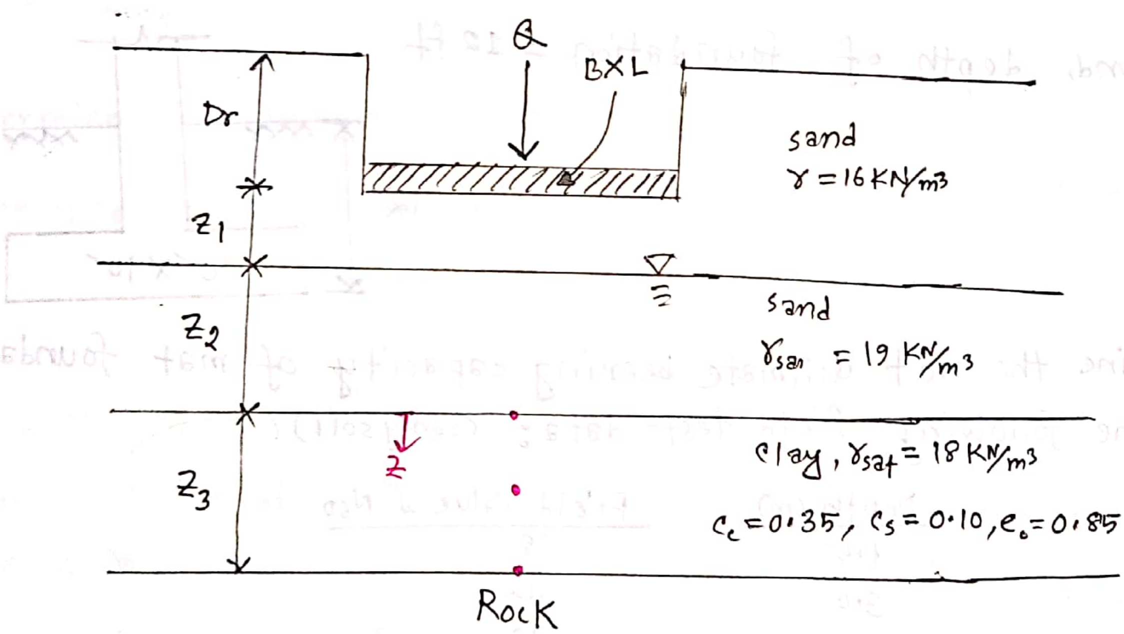
$= 302.225 \ KN/m^2$

$q_{Net(u)} = q_{net(all)} \times FS = (302.225 \times 3) = 906.675 \ KN/m^2$

(Ans.)

2014

A mat foundation is shown in figure below. Given: $L = 12m$, $B = 10m$, $D_f = 2.0m$, $Q = 30 MN$, $Z_1 = 2m$, $Z_2 = 2m$, $Z_3 = 5$ and preconsolidation pressure $\sigma_c' = 100 \ KPa$. Calculate settlement under corner of mat.



Solution: For Normally consolidated clay,

$$S_{c(p)} = \frac{C_c H}{1+e_o} \log\left(\frac{\sigma_o' + \Delta\sigma_{av}'}{\sigma_o'}\right)$$

Here, $C_c = 0.35$, $H = 5m$, $e_o = 0.85$

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$$\sigma_o' = 4\times16 + (19-9.81)\times 2 + \frac{5}{2}\times(18-9.81) = 102.855\ KN/m^2 > \sigma_c'$$

$$q_o = \frac{30\times10^3}{10\times12} = 250\ KN/m^2$$

For $\Delta\sigma_{av}'$ following table to be prepared:

$$^*I = \frac{1}{4\pi}\times\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}\times\frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2}\right)$$

z(m)	$m=\frac{B}{z}$	$n=\frac{L}{z}$	$^*I$	q_o (KN/m²)	$\Delta\sigma' = q_o I$ (KN/m²)	$\sigma_{av}' = \frac{\Delta\sigma_t' + 4\Delta\sigma_m' + \Delta\sigma_b'}{6}$ (KN/m²)
4	2.5	3	0.242	250	$\Delta\sigma_t' = 60.5$	
6.5	1.54	1.846	0.223	250	$\Delta\sigma_m' = 55.75$	55.4167
9	1.11	1.33	0.196	250	$\Delta\sigma_b' = 49.0$	

At the corner of Mat,

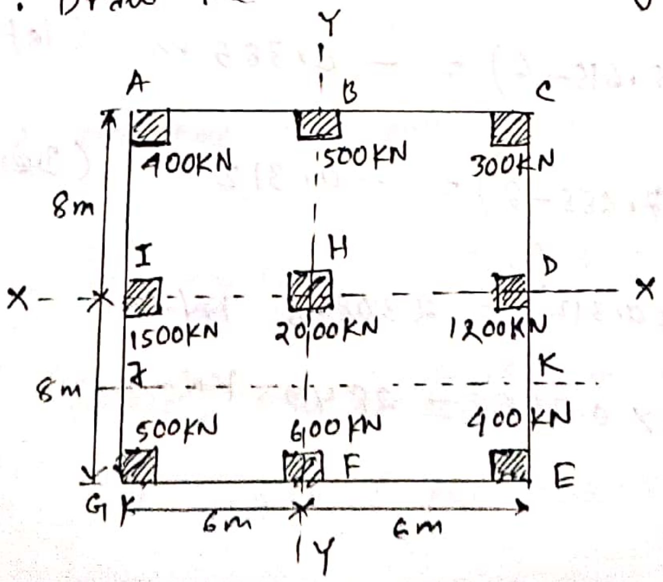
$$\text{settlement } S_{c(p)} = \frac{0.35\times5}{1+0.85}\times\log\left(\frac{102.855+55.4167}{102.855}\right)$$

$$= 0.1771\,m = 177.1\ mm$$

(Ans)

2018, 2017

The plan of a mat fundation with 9 columns are shown in figure below. Assuming that, the mat is rigid, Determine the soil pressure distribution. All the columns are of the size 0.6m × 0.6m. Draw the S.F and B.M diagrams for the strip EGJK.



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Solution: We know, $q = \frac{Q}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$

Here, $Q = (400 \times 2 + 500 \times 2 + 300 + 600 + 1200 + 1500 + 2000)$

$= 7400$ KN

$A = B \times L = (12 \times 16) = 192 \ m^2$

$I_y = \frac{16 \times 12^3}{12} = 2304 \ m^4$

$I_x = \frac{12 \times 16^3}{12} = 4096 \ m^4$

load center:

$x' = \frac{0.3 \times (400+1500+500) + 6 \times (500+2000+600) + 11.7 \times (300+1200+400)}{7400}$

$= 5.615$ m

$y' = \frac{0.3 \times (500+600+400) + 8 \times (1500+2000+1200) + 15.7 \times (400+500+300)}{7400}$

$= 7.688$ m

Area center:

$x = \frac{B}{2} = 6$ m

$y = \frac{L}{2} = 8$ m

$\therefore e_x = (x' - x) = (5.615 - 6) = -0.385$ m (left of the area center)

$e_y = (y' - y) = (7.688 - 8) = -0.312$ m (below the area center)

$M_x = Q e_y = (7400 \times 0.312) = 2308.8$ KN-m

$M_y = Q e_x = (7400 \times 0.385) = 2849$ KN-m

Now,

$$q = \frac{7400}{192} \pm \frac{2849}{2304} \times x \pm \frac{2308.8}{4096} \times y$$

$$= 38.54 \pm 1.24x \pm 0.56y$$

point	x (m)	y (m)	q (KN/m²)
A	6.0	−8.0	41.5
B	0	−8.0	34.06
C	−6.0	−8.0	26.62
D	−6.0	0	31.1
E	−6.0	8.0	35.58
F	0	8.0	43.02
G	6.0	8.0	50.46
H	0	0	0
I	6.0	0	45.98

Strip EGIK:

Average soil pressure, $q_{av} = \frac{35.58 + 50.46}{2} = 43.02$ KN/m²

Total soil reaction $= q_{av} \times B_1 \times B$

$= (43.02 \times 4 \times 12) = 2064.96$ KN

∴ Average load $= \frac{2064.96 + 1500}{2} = 1782.48$ KN

So, Modified average soil pressure,

$$q_{av\,(modified)} = 43.02 \times \frac{1782.48}{2064.96} = 37.135 \text{ KN/m}^2$$

Multiplying factor, $F = \frac{1782.48}{1500.0} = 1.18832$

The load per unit length of the beam $= q_{av\,(modified)} \times B_1$

$= (4 \times 37.135) = 148.54$ KN/m

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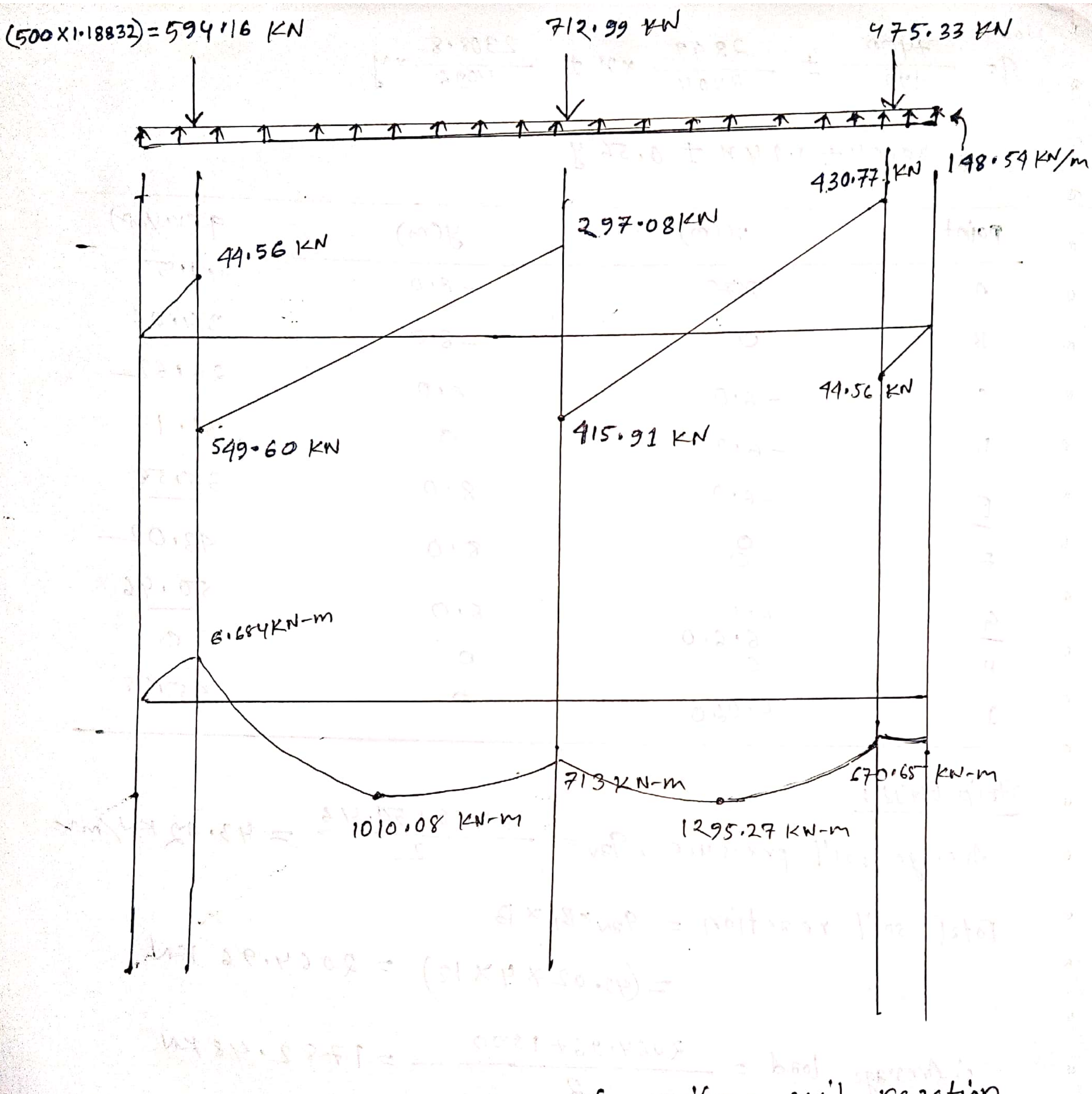

In view of the assumption of uniform soil reaction to non symmetric loading, there is discrepancy in the moment values at right column. This is ignored.

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Additional question:

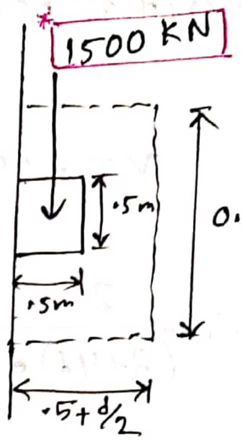
Determine the thickness of the mat.

Given, $f_c' = 20\ MN/m^2$

$f_y = 400\ MN/m^2$

* B, J, F & D এর মধ্যে সেন্টারে load Maximum.

Solution: (WSD Method)



Case-1: critical perimeter column:

critical perimeter,

$$b_o = (0.5 + d/2) \times 2 + (0.5 + d)$$

$$= 1.5 + 2d$$

$$V = 1500\ KN = 1.5\ MN$$

Now,

$$\frac{V}{b_o d} = \frac{1}{6}\sqrt{f_c'}$$

In USD, $\frac{U}{b_o d} = \frac{1}{3}\phi\sqrt{f_c'}$

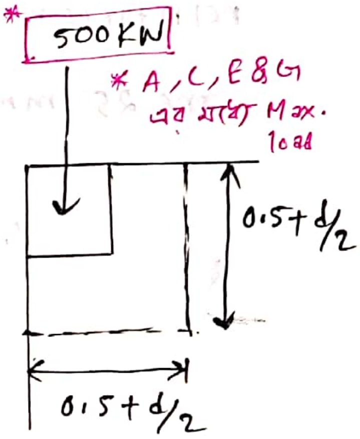
$= \phi \times 0.34\sqrt{f_c'}$

$$\Rightarrow \frac{1.5 \rightarrow (MN)}{(1.5+2d)d} = \frac{1}{6}\sqrt{20} \rightarrow (MN)$$

$$\Rightarrow 2d^2 + 1.5d - 2.0125 = 0$$

$$\therefore d = 0.696\ m \simeq 0.7\ m$$

* A, C, E & G এর মধ্যে Max. load



Case-2: critical corner column:

critical perimeter,

$$b_o = (0.5 + d/2) \times 2 = (1+d)$$

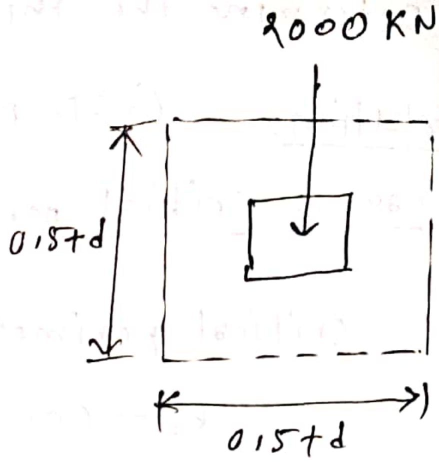
Similarly,

$$\frac{0.5}{(1+d)d} = \frac{1}{6}\sqrt{20}$$

$$\Rightarrow d^2 + d - 0.671 = 0$$

$$\therefore d = 0.46\ m \simeq 0.5\ m$$

Case-3: <u>critical middle column:</u>



$\therefore$ critical perimeter, $b_o = 4 \times (0.5+d)$

$= (2+4d)$

$V = 2000\,KN = 2\,MN$

$$\therefore \frac{2}{(2+4d)\,d} = \frac{1}{6}\sqrt{20}$$

$$\Rightarrow 4d^2 + 2d - 2.6833 = 0$$

$\therefore d = 0.61\,m \simeq 0.65\,m$

Hence, the effective thickness, $d_{eff} = 0.7\,m$

Assuming a minimum cover of 76 mm over the steel reinforcement and assuming the steel bar to be used are 25 mm in diameter.

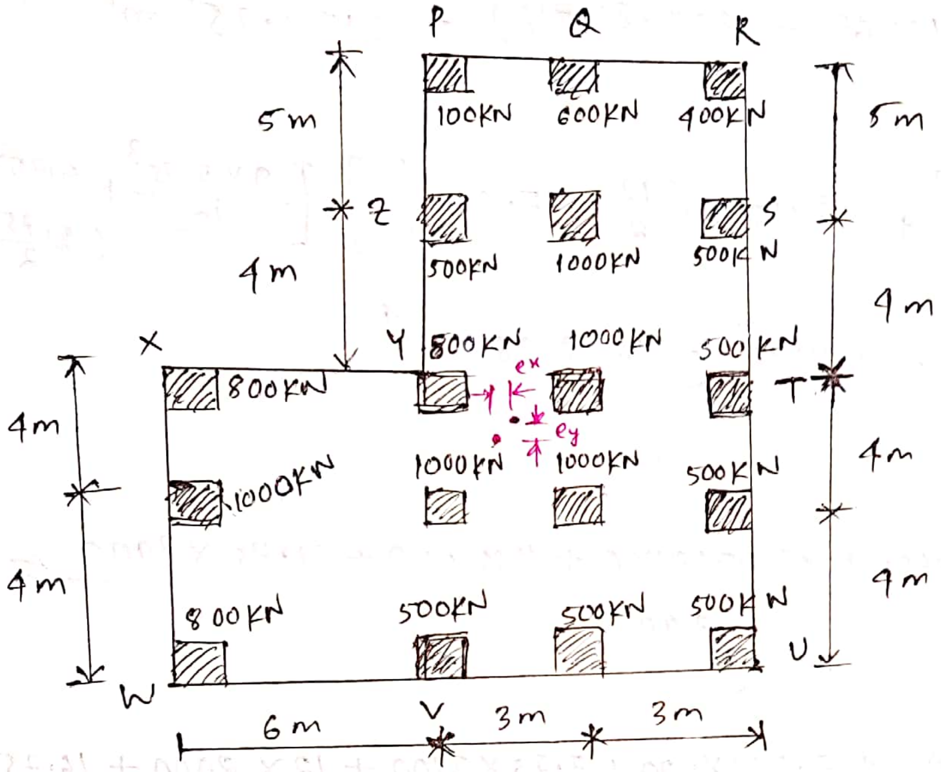
$\therefore$ The total thickness of the mat, $t = (0.7 + 0.076 + 0.025)$

$= 0.801\,m$

(Ans.)

2016

The plan of a mat foundation is shown in figure below. Determine the soil pressure at points P, Q, R, S, T, U, V, W, X, Y and Z. Assume the column size = 0.50 m × 0.50 m.



Solution: Total column load, $Q = (2400 + 4100 + 2900 + 2600)$ KN

$\therefore Q = 12000$ KN

Area, $A = (12 \times 17) - (5.75 \times 9) = 152.25\ m^2$

Area Center:

$$x = \frac{12 \times 17 \times \frac{12}{2} - 5.75 \times 9 \times \frac{5.75}{2}}{152.25} = 7.0622\ m$$

$$y = \frac{12 \times 17 \times \frac{17}{2} - 5.75 \times 9 \times \left(8 + \frac{9}{2}\right)}{152.25} = 7.1404\ m$$

Moment of Inertia:

$$I_x = \left[\frac{12\times17^3}{12} + 12\times17\times\left(\frac{17}{2} - 7.1404\right)^2\right] - \left[\frac{5.75\times9^3}{12} + 5.75\times9\times\left(8+\frac{9}{2}-7.1404\right)^2\right]$$

$$\therefore I_x = (5290.0965 - 1835.8474) = 3454.25 \ m^4$$

$$I_y = \left[\frac{17\times12^3}{12} + 12\times17\times\left(\frac{12}{2} - 7.0622\right)^2\right] - \left[\frac{9\times5.75^3}{12} + 5.75\times9\times\left(\frac{5.75}{2}-7.0622\right)^2\right]$$

$$= (2678.17 - 1049.9) = 1628.28 \ m^4$$

Load center:

$$x' = \frac{0.25\times2600 + 6.0\times2900 + 9\times4100 + 11.75\times2400}{12000} = 6.93 \ m$$

$$y' = \frac{.25\times2300 + 4\times3500 + 7.75\times3100 + 12\times2000 + 16.75\times1100}{12000}$$

$$= 6.7521 m$$

$e_x = (x'-x) = (6.93 - 7.0622) = -0.1322$ m (left of the area center)

$e_y = (y'-y) = (6.7521 - 7.1404) = -0.3883$ m (below the area center)

+ −	− −
+ +	− +

$M_y = Qe_x = (12000\times0.1322) = 1586.4$ KN-m

$M_x = Qe_y = (12000\times0.3883) = 4659.6$ KN-m

Now, $q = \frac{Q}{A} \pm \frac{M_y\, x}{I_y} \pm \frac{M_x \cdot y}{I_x} = \frac{12000}{152.25} \pm \frac{1586.4x}{1628.28} \pm \frac{4659.6y}{3454.25}$

$$\therefore q = 78.82 \pm 0.9743x \pm 1.349y$$

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Point	x (m)	y (m)	q (KN/m²)
P	1.3122	−9.8596	66.8
Q	−1.9378	−9.8596	63.63
R	−4.9378	−9.8596	60.71
S	−4.9378	−4.8596	67.45
T	−4.9378	−0.6096	73.2
U	−4.9378	7.1404	83.64
V	1.0622	7.1404	89.49
W	7.0622	7.1404	95.33
X	7.0622	−0.8596	84.54
Y	1.3122	−0.8596	78.94
Z	1.3122	−4.8596	73.54

Class Test - 13 series.

A raft 8m × 24m, is supported at a depth of 4m in sand with a value of $N_{60} = 20$ upto a great depth. What is the total load which the raft can support? What will be the total capacity if it is act as a floating foundation at this depth? Assume settlement of the mat = 50 mm and $\gamma = 20$ KN/m³

Solution: we know,

$$q_{net(all)} = \frac{N_{60}}{0.08} \times \left[1 + 0.33\frac{D_f}{B}\right] \times \frac{S_e\,(mm)}{25}$$

$$= \frac{20}{0.08} \times \left[1 + 0.33 \times \frac{4}{8}\right] \times \frac{50}{25}$$

$$= 582.5 \text{ KN/m}^2$$

Net pressure applied on a foundation,

$$q = \frac{Q}{A} - \gamma D_f \leq q_{all\,(net)}$$

so, $$\frac{Q}{A} - \gamma D_f = q_{all\,(net)}$$

$$\Rightarrow \frac{Q}{8 \times 24} - 20 \times 4 = 582.5$$

$$\Rightarrow \frac{Q}{8 \times 24} = 582.5 + 80$$

$$\therefore Q = (662.5 \times 8 \times 24) = 127200 \text{ KN}$$

$\therefore$ Total load that the raft can support is = 127200KN

For floating foundation, $q = 0$

$$\therefore \frac{Q}{A} = \gamma D_f$$

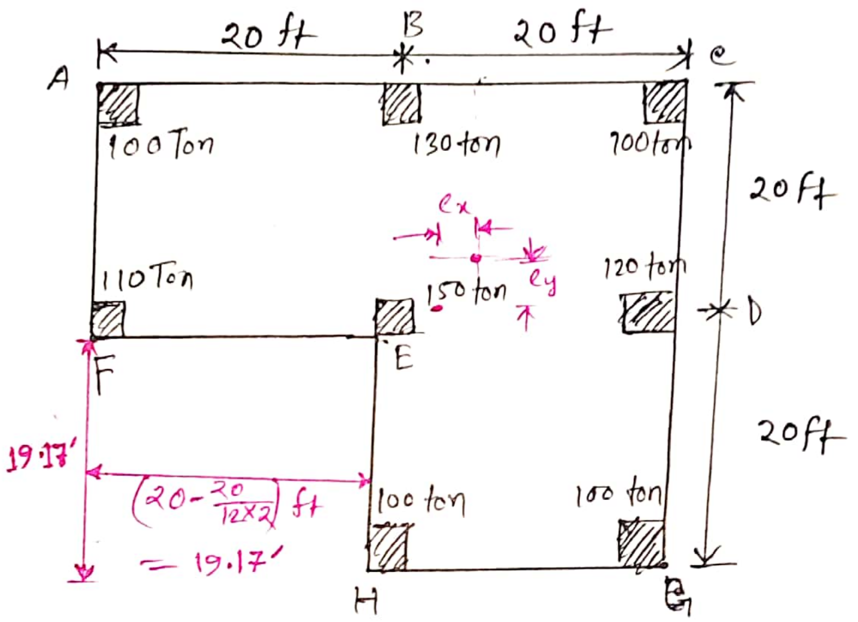
$$\Rightarrow Q = \gamma D_f \times A = (20 \times 4 \times 8 \times 24) = 15360 \text{KN}$$

$\therefore$ Total capacity = 15360KN

(Ans.)

Class Test -(15 series)

The plan of a mat foundation is shown in below. Calculate the soil pressure below the mat at points A, B, C, D, E, F, G and H. Check the mat foundation is satisfactory for bearing capacity failure. Assume Allowable bearing capacity of soil is 0.75 Ton/ft^2.
(Note: All the column sections are planned to be 20in × 20in.)



Solution:

Total load, $Q = (100 + 110 + 130 + 150 + 100 + 100 + 120 + 100) = 910$ Ton.

Area, $A = (40 \times 40 - 19.17 \times 19.17) = 1232.51\ ft^2$

Area center:

$$x = \frac{40 \times 40 \times \frac{40}{2} - 19.17 \times 19.17 \times \frac{19.17}{2}}{1232.51} = 23.105\ ft$$

$y = 23.105$ ft

$$I_x = \left[\frac{40 \times 40^3}{12} + (40 \times 40) \times \left(\frac{40}{2} - 23.105\right)^2\right] - \left[\frac{19.17 \times 19.17^3}{12} + (19.17 \times 19.17) \times \left(\frac{19.17}{2} - 23.105\right)^2\right]$$

$= 150331.52\ ft^4$

$I_y = 150331.52\ ft^4$

Load center:

$$x' = \frac{0.83 \times 210 + 20 \times 380 + 39.17 \times 320}{910} = 22.32 \text{ ft}$$

$$y' = \frac{0.83 \times 200 + 20 \times 380 + 39.17 \times 330}{910} = 22.74 \text{ ft}$$

$\therefore e_x = (x' - x) = (22.32 - 23.105) = -0.785$ ft (left of the center)

$e_y = (y' - y) = (22.74 - 23.105) = -0.365$ ft (below the center)

$M_x = Qe_y = (910 \times 0.365) = 332.15$ Ton-ft

$M_y = Qe_x = (910 \times 0.785) = 714.35$ Ton-ft

$$q = \frac{Q}{A} \pm \frac{M_y \cdot x}{I_y} \pm \frac{M_x \cdot y}{I_x} = \frac{910}{1232.5} \pm \frac{714.35}{150331.52} x \pm \frac{332.15}{150331.52} y$$

$\therefore q = 0.74 \pm 4. \times 10^{-3} x \pm 2.21 \times 10^{-3} y < q_{all} = 0.75$ ton/ft² (Given)

point	x (ft)	y (ft)	q (ton/ft²)	Remark
A	23.105	-16.895	0.8124	(not OK)
B	3.105	-16.895	0.717	(OK)
C	-16.895	-16.895	0.6224	(OK)
D	-16.895	3.105	0.67	(OK)
E	3.935	3.935	0.7674	(not OK)
F	23.105	3.735	0.86	(not OK)
G	-16.895	23.105	0.711	(OK)
H	3.935	23.105	0.81	(not OK)

Pile Foundation

What is pile foundation?

Pile foundation, a kind of deep foundation, is actually a slender structural member made of steel, concrete or wood, which are used to support the structure and transfer the load at desired depth either by end bearing or skin friction.

What are the conditions where a pile foundation is more suitable than a shallow foundation? 08,07

The following list identifies some of the conditions that require pile foundation:

(i) When one or more upper soil layers are highly compressible and too weak to support the load transmitted by the super structure.

(ii) Pile foundations are used to resist horizontal forces in addition to support the vertical loads in earth retaining structures and tall structures.

(iii) Piles are required when the soil conditions are such that a wash-out, erosion or scour of soil

(iv) When the plan of the structure is irregular relative to its out line and load transmission.

(v) When exansive and collapsible soils are present at the site of a proposed structure.

(vi) Pile foundations are required for the transmission of structural loads through deep water to a firm stratum.

Classify piles based on their functions. 11

Based on the function or action, piles may be classified as:

(i) End Bearing Piles: used to transfer loads through the pile tip to a suitable bearing stratum, passing soft soil or water.

(ii) Friction piles: Used to transfer loads to a depth in a frictional material by means of skin friction along the surface area of the pile.

(iii) Tension or uplift piles: Used to anchor structures subjected to uplift due to hydrostatic pressure or subjected to overturning moment due to horizontal forces.

(iv) Compaction piles: Used to compact loose granular soils in order to increase the bearing capacity.

(v) Anchor piles: Used to provide anchorage against horizontal pull from sheet piling water.

(vi) Fender piles: Used to protect water front structures against impact from ships or other floating objects.

(vii) Sheet piles: commonly used as bulkheads or cut-offs to reduce seepage and uplift in hydraulic structures.

(viii) Batter piles: Used to resist horizontal and inclined forces, especially in water front structures.

(ix) Laterally loaded piles: Used to support retaining walls, bridges, dams, and as fenders for harbor construction.

Write short note on: (i) Laterally Loaded pile. 18, 13, 11

(ii) Batter pile. 16, 08, 07, 06

* (iii) Negative skin friction. 13, 11

(i) Laterally Loaded pile: A vertical pile resists a lateral load by mobilizing passive pressure in the soil sorrounding it.

The degree of distribution of soil's reaction depends on:

(a) the stiffness of pile (b) the stiffness of the soil and,

(c) the fixity of the end of the pile.

In general, laterally loaded piles can be divided into two major categories:

(i) short or rigid piles (ii) Long or elastic piles.

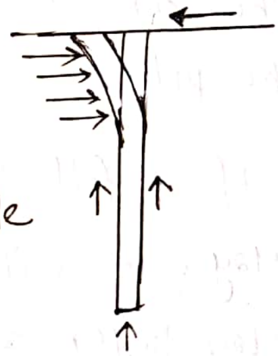


Fig. Laterally Loaded pile

(ii) Batter pile:

It is a type of pile provided when vertical pile can not resist horizontal load effectively.

It may be grouped with vertical pile to increase its lateral capacity. It is constructed at an angle to the vertical pile.

Generally for dry ground, inclination angle should be 30° and for water logged area it should be limited to 15° with the horizontal.

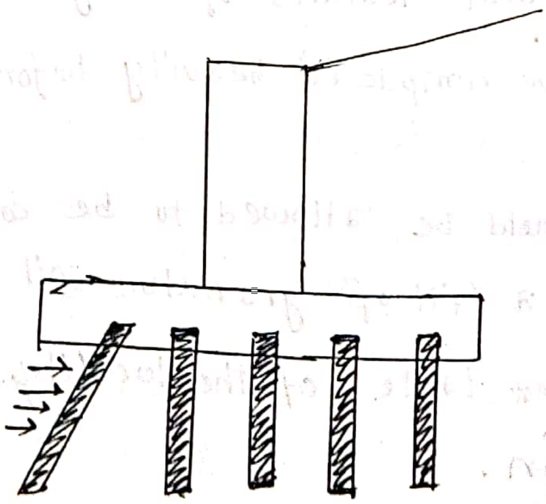


Fig. Batter pile

What is Negative skin friction? 2009, 07, 01

Negative skin friction is a downward drag force exerted on a pile by the soil surrounding it.

What are the conditions under which it may occur? 09,07,01

Negative skin friction can exist under the following conditions:

(i) If a fill of clay is placed over a granular soil layer into which a pile is driven, the fill will gradually consolidate. The consolidation process will exert a downward drag force on the pile.

(ii) If a fill of granular soil is placed over a layer of soft clay, it will induce the process of consolidation in the clay layer and thus exert a downward drag on the pile.

(iii) Lowering of the water table will increase the vertical effective stress on the soil at any depth, which will induce consolidation settlement. If a pile is located in the clay layer, it will subjected to a downward drag force.

Write down the remedial measures of negative skin friction. 09,07,01

(i) Granular soil should be compacted heavily before placing a clay fill over it.

(ii) soft clay layers should be allowed to be consolidated for long time before placing a fill of granular soil over it.

(iii) Fluctuation of water table of the locality should be considered in design.

What short note on Group Efficiency of piles. 18

In most cases, piles are used in groups to transmit the structural load to the soil. If the piles are placed close to each other, The load bearing capacity of piles will reduce. Ideally the piles in a group should be spaced so that the load-bearing capacity of the group is not less than the sum of the bearing capacity of the individual piles.

The efficiency of the load bearing capacity of a group pile may be defined as, $$\eta = \frac{Q_{g(u)}}{\Sigma Q_u}$$

where, η = group efficiency

$Q_{g(u)}$ = ultimate load bearing capacity of the group pile

Q_u = ultimate load bearing capacity of each pile without the group effect.

What is pile cap? 12, 08, 05, 04

When a group of piles are used, a hard cap is necessary to spread the vertical and horizontal loads and also the over-turning moment to all the piles in the group. This cap is known as pile cap.

What situation do you use pile cap? 08, 05, 04

under following situations pile caps may be used:

(i) When more than one pile are to be driven.

(ii) When loads are to be distributed uniformly on all the piles.

(iii) when there is possibility of lateral forces.

What are the assumptions made in the design of a pile cap? 12

Assumptions made in the design of pile cape are:

(i) Pile cap is perfectly rigid.

(ii) Pile heads are hinged to the pile cap.

(iii) Piles are short and elastic columns.

(iv) Each pile carries equal ~~amount~~ amount of loads.

Write down the functions of pile cap. 06.

(i) To distribute the vertical and lateral loads.

(ii) To resist the overturning moment.

(iii) To reduce the destruction of the upper portion of piles.

(iv) Pile driving to be easy.

(v) To determine the load bearing capacity of the group piles.

How can you determine the ultimate capacity of group piles in saturated clay? 18, 16, 13

Following figure shows a group of pile in saturated clay:

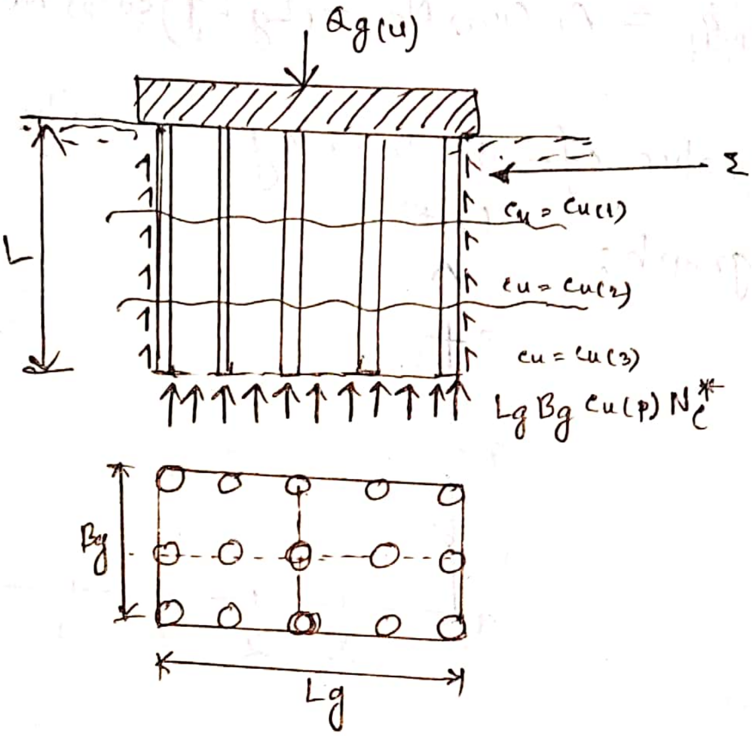


Fig. Ultimate capacity of group piles in clay

Ultimate load bearing capacity of group piles can be estimated in the following manner:

Step 1: Determine $\Sigma Q_u = n_1 n_2 (Q_p + Q_s)$

where, $Q_p = A_p [9 \, c_{u(p)}]$. Here, $c_{u(p)}$ = undrained cohesion of the clay at the pile tip.

and, $Q_s = \Sigma \alpha p c_u \Delta L$

So, $\Sigma Q_u = n_1 n_2 [9 A_p c_{u(p)} + \Sigma \alpha p c_u \Delta L]$ ------- (1)

Step 2: Determine the ultimate capacity by assuming that the piles in the group ack as a block with dimension $L_g \times B_g \times L$.

The skin resistance of the block is,

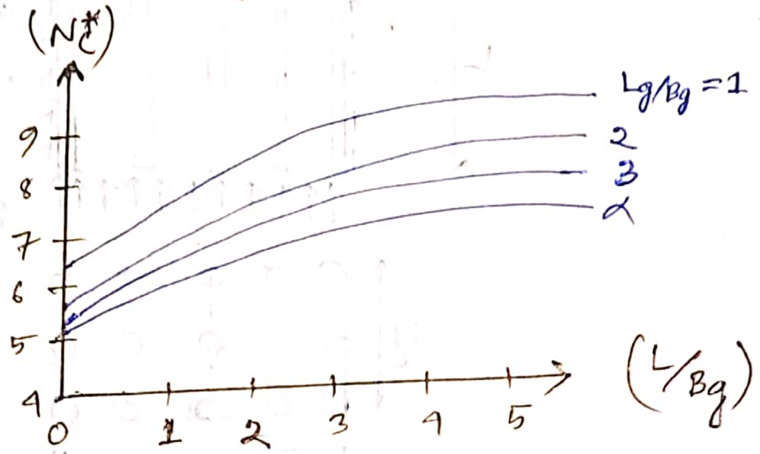
$$\Sigma P_g c_u \Delta L = \Sigma 2 (L_g + B_g)\, c_u \Delta L$$

calculate the point bearing capacity:

$$A_p q_p = A_p c_{u(p)} N_c^* = (L_g B_g)\, c_{u(p)} N_c^*$$

Obtain the value of the bearing capacity factor N_c^* from

Following graph:



Thus the ultimate load is,

$$\Sigma Q_u = L_g B_g c_{u(p)} N_c^* + \Sigma 2 (L_g + B_g)\, c_u \Delta L \quad \text{.....(2)}$$

Step 3: Compare the values obtained from eqn (1) and (II).

The lower of the two value is $Q_{g(u)}$.

Pile Foundation

Equations for estimating Pile Capacity:

The ultimate load bearing Capacity of a pile,

$$Q_u = Q_p + Q_s$$

where, Q_p = point bearing capacity of pile

Q_s = Frictional Resistance (skin-friction) derived from soil-pile interface

point bearing capacity of pile,

$$Q_p = A_p q_p = A_p (c' N_c^* + q' N_q^*)$$

where, A_p = Area of pile

c' = cohesion of soil supporting pile tip

q_p = unit point resistance

q' = effective vertical stress at the level of the pile tip

N_c^*, N_q^* = The bearing capacity factor.

Frictional Resistance of pile,

$$Q_s = \Sigma p \cdot \Delta L \cdot f$$

where, p = perimeter of the pile section

ΔL = increamental pile length over which 'p' and 'f' taken to be constant

f = unit friction resistance at any depth z.

Allowable load for each pile,

$$Q_{all} = \frac{Q_u}{FS}$$

Methods for Estimating Q_p:

Based on soil property:

BM Das. 8th ed.

(i) Meyerhof's Method:

- **For sand ($c'=0$):** $Q_p = q' A_p N_q^* \leq (\underbrace{0.5 P_a N_q^* \tan\phi'}_{q_l = \text{limiting point resistance}}) A_p$

N_q^* → depends on ϕ' (Table-9.5)

where,

$q' = \gamma L$

P_a = Atmospheric pressure = 100 KN/m² or, 2000 lb/ft²

- **For clay ($\phi = 0$):** $Q_p = N_c^* c_u A_p$ where, $N_c^* = 9$

c_u = undrained cohesion of the soil below the tip of the pile.

(ii) Vesic's Method:

- **For sand ($c'=0$):** $Q_p = \overline{\sigma_o}' A_p N_q^*$ → depends on ϕ' & I_{rr} (Table-9.7)

where, $\overline{\sigma_o}' = \left(\frac{1+2K_o}{3}\right) q'$; $K_o = 1 - \sin\phi'$ and $q' = \gamma L$

$$I_{rr} = \frac{I_r}{1 + I_r \Delta} \quad \text{where, } I_r = \frac{E_s}{2(1+\mu_s) q' \tan\phi'}$$

$E_s = m P_a$ [m = 100 to 200 (loose soil)
= 200 to 500 (Medium dense soil)
= 500 to 1000 (Dense soil)]

$\mu = 0.1 + 0.3\left(\frac{\phi' - 25}{20}\right)$ (for $25° \leq \phi' \leq 45°$)

and, $\Delta = 0.005\left(1 - \frac{\phi' - 25}{20}\right) \times \frac{q'}{P_a}$

- **For clay ($\phi' = 0$):** $Q_p = c_u A_p N_c^*$ → depends on I_{rr} (Table - 9.8) or,

where, $N_c^* = \frac{4}{3}(\ln I_{rr} + 1) + \frac{\pi}{2} + 1$

where, $I_{rr} = I_r = \frac{E_s}{3 c_u}$

The preceding value, $I_r \approx 347\left(\frac{c_u}{P_a}\right) - 33 \leq 300$

(iii) Coyle and Castello's Method:

- For sand ($c=0$): $Q_p = q' A_p N_q^*$ → depends on L/D and ϕ' (Fig. 9.15)

(iv) Janbu's Method:

- For sand ($c=0$): $Q_p = q' A_p N_q^*$
- For clay ($\phi'=0$): $Q_p = (c' N_c^* + q' N_q^*)$. where, $N_c^* = (N_q^* - 1)\cot\phi'$

➡ Based on SPT Value:

(i) Meyerhof's Method: $q_p = 0.4\, P_a N_{60} \times \frac{L}{D} \leq 4 P_a N_{60}$

where, N_{60} = Average value of SPT near the pile point (about 10 D above and 4D below the pile point)

(ii) Briaud et al.'s formula: $q_p = 19.7\, P_a (N_{60})^{0.36}$

Methods for Estimating Q_s:

➡ For Sand ($c=0$): $Q_s = \Sigma p \Delta L f$

Here, $L' \approx 15D$

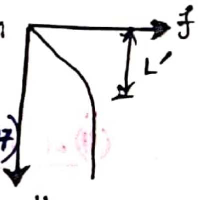
where, for $z = 0$ to L', $f = K\sigma_o' \tan\delta'$

for $z = L'$ to L, $f = f_{z=L'}$

Here, K = effective earth pressure co-efficient

σ_o' = effective vertical stress at the depth under consideration

δ' = soil-pile friction angle



Pile Type	Value of K
Bored or Jetted:	$1-\sin\phi'$
Low-displacement driven:	$(1-\sin\phi')$ to $1.4(1-\sin\phi')$
High-displacement driven:	$(1-\sin\phi')$ to $1.8(1-\sin\phi')$

* Coyle and Castello's Method: $Q_s = f_{av}\, pL = (\underbrace{K}_{\text{based on } \frac{L}{D} \,\&\, \phi' \text{ (Fig. 9.17)}} \bar{\sigma}_o' \tan\delta')\, pL$

where, $\bar{\sigma}_o'$ = Average effective overburden pressure $= \frac{\gamma L}{2}$

δ' = Soil-pile friction angle $= 0.8\phi'$

Scanned by CamScanner

Based on SPT value:

(i) Meyerhof's Formula: $Q_s = pLf_{av}$

where, for high displacement driven piles, $f_{av} = 0.02\, P_a\, (\overline{N_{60}})$

for low displacement driven piles, $f_{av} = 0.01\, P\, (\overline{N_{60}})$

(ii) Briaud et. al's Formula: $f_{av} = 0.224\, P_a\, (\overline{N_{60}})^{0.29}$

Based on CPT value:

$Q_s = \sum p(\Delta L) f$

where, $f = \alpha' f_c$ → depends on $\frac{L}{D}$

➡ For Clay ($\phi' = 0$):

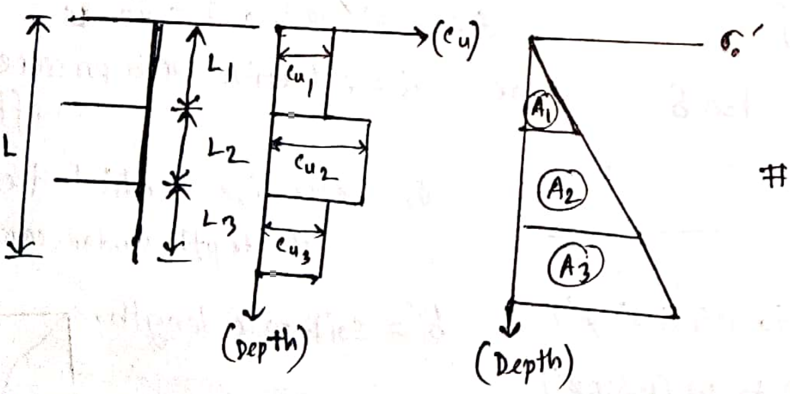
(i) λ Method: (Vijayvergiya and Focht)

$$f_{av} = \lambda(\overline{\sigma_o'} + 2c_u)$$

λ depends on L

* for layered soil, $c_u = \dfrac{c_{u_1}L_1 + c_{u_2}L_2 + \ldots}{L_1 + L_2 + \ldots}$

* for layered soil, $\overline{\sigma_o'} = \dfrac{A_1 + A_2 + \ldots}{L}$ where, $A_1, A_2 \ldots$ = areas of the vertical effective stress diagram.



\# the total frictional resistance,

$$Q_s = pLf_{av}$$

(ii) α Method:

$$f = \alpha c_u$$

α depends on $\frac{c_u}{P_a}$

(i) Sladen (1992): $\alpha = C\left(\dfrac{\overline{\sigma_o'}}{c_u}\right)^{0.45}$ where, $C = 0.4$ to 0.5 for bored piles

≥ 0.5 for driven pile

(ii) API (2007): (Randolph & Murphy)

$$f_{av} = 0.5\,(c_u\,\bar{\sigma}_o')^{0.5}$$
$$= 0.5\,(c_u)^{0.75}\,(\bar{\sigma}_o')^{0.25}$$

} whichever is larger

(iii) NGI-99 method: (Karlsrud et al.)

$$\alpha = 0.32\,(PI - 10)^{0.3} \quad (1 \geq \alpha \geq 0.2) \quad \left[\text{for } \frac{c_u}{\bar{\sigma}_o'} \leq 0.25\right]$$

$$\alpha = 0.5 \quad \left(\text{for } \frac{c_u}{\bar{\sigma}_o'} = 1\right)$$

$$\alpha = 0.5\left(\frac{c_u}{\bar{\sigma}_o'}\right)^{-0.3} c \quad \left(\text{for } \frac{c_u}{\bar{\sigma}_o'} \geq 1\right)$$

c ↓ correction factor

The ultimate side resistance, $Q_s = \Sigma f p \Delta L = \Sigma \alpha c_u \times p \times \Delta L$

(iii) β method:

$f = \beta \sigma_o'$ where, $\beta = K \tan \phi_R'$

$K = 1 - \sin \phi_R'$ (for normally consolidated clays)

$K = (1 - \sin \phi_R')\sqrt{OCR}$ (for over consolidated clay)

∴ The total frictional resistance; $Q_s = \Sigma f p (\Delta L)$

Pile Load Test:

For any load Q, Net Settlement, $S_{net} = S_t - S_e$

* one of the methods to obtain ultimate load Q_u from load-settlement plot is proposed by Davisson (1973)

The ultimate load occurs at a settlement level (S_u),

$$S_u\,(mm) = 0.012\,D_r + 0.1\left(\frac{D}{D_r}\right) + \frac{Q_u L}{A_p E_p}$$

where, Q_u in KN

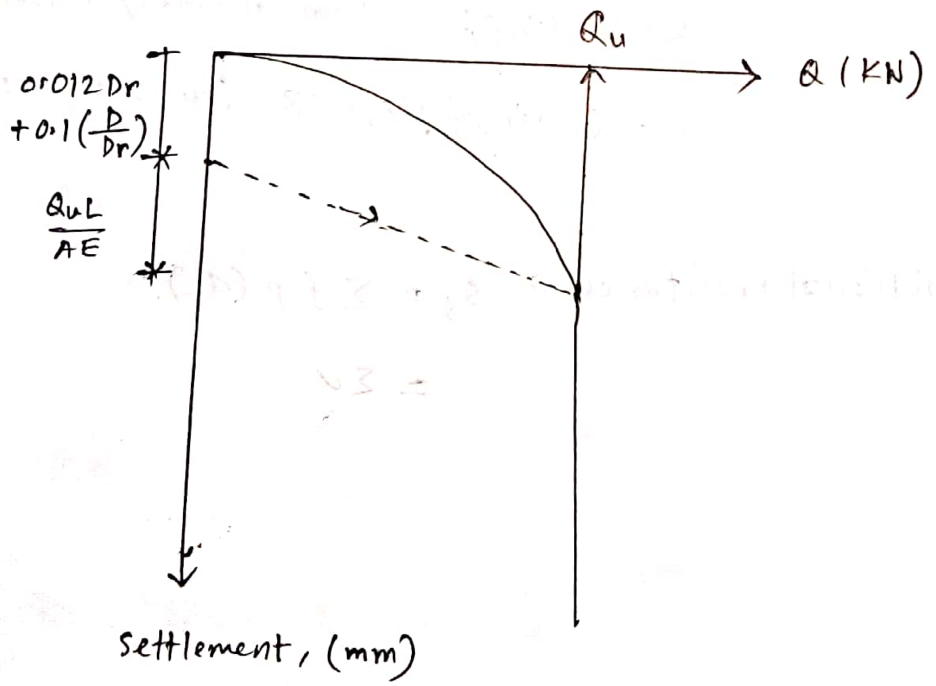
D in mm

D_r = reference pile diameter or width (= 300mm)

L = pile length (mm)

A_p = area of pile cross section (mm^2)

E_p = Young's modulus of pile material (KN/mm^2)



Elastic Settlement of Piles: (At working Load)

Total settlement of a pile under a vertical working load Q_w,

$$S_e = S_{e(1)} + S_{e(2)} + S_{e(3)}$$

where,

$S_{e(1)}$ = elastic settlement of pile

$S_{e(II)}$ = settlement of pile caused by the load at pile tip

$S_{e(III)}$ = settlement of pile caused by the load transmitted along the pile shaft

$$S_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws}) L}{A_p E_p}$$ where, $\xi = (0.5 \text{ to } 0.67)$

$$S_{e(2)} = \frac{q_{wp} D}{E_s} \times (1-\mu_s^2) I_{wp}$$ where, $q_{wp} = \frac{Q_{wp}}{A_p}$

$I_{wp} = 0.85$

- vesic proposed: $S_{e(2)} = \frac{Q_{wp} C_p}{D_b q_p}$ where, C_p = empirical co-efficient

and, $S_{e(3)} = \left(\frac{Q_{ws}}{PL}\right) \times \frac{D}{E_s} \times (1-\mu_s^2) I_{ws}$

where, $I_{ws} = 2 + 0.35\sqrt{\frac{L}{D}}$

- Vesic proposed:

$$S_{e(3)} = \frac{Q_{ws} C_s}{L q_p}$$ where $C_s = \left(0.93 + 0.16\sqrt{\frac{L}{D}}\right) \times C_p$

Negative Skin Friction:

Clay Fill over Granular Soil:

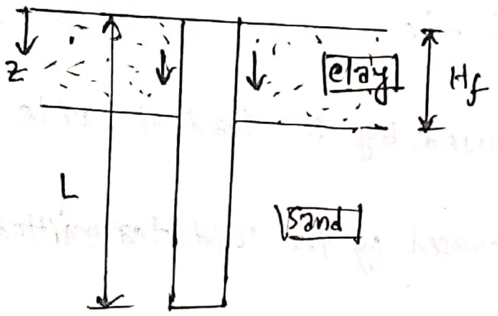
The negative skin stress on the pile,

$$f_n = K'\sigma_o' \tan\delta'$$

where, $K' = 1 - \sin\phi'$

$\sigma_o' = \gamma_f' z$

$\delta' = (0.5 - 0.7)\phi'$

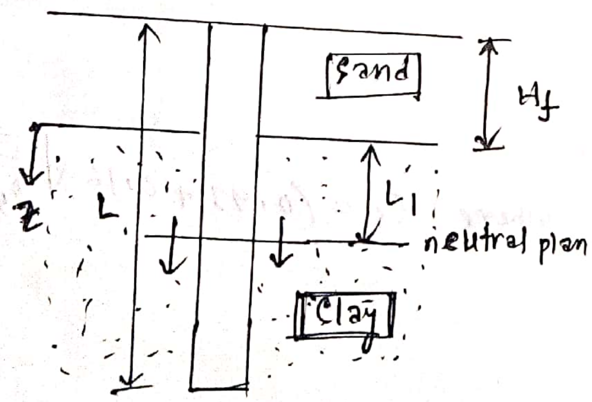


∴ The total Downward drag force on a pile is

$$Q_n = \int_0^{H_f} (pK'\gamma_f' \tan\delta')\, z\, dz = \frac{pK'\gamma_f' H_f^2 \tan\delta'}{2}$$

Granular soil over Clay:

Neutral Depth, $L_1 = \frac{(L - H_f)}{L_1} \times \left[\frac{L - H_f}{2} + \frac{\gamma_f' H_f}{\gamma'}\right] - \frac{2\gamma_f' H_f}{\gamma'}$



The Negative friction,

$$f_n = K'\sigma_o' \tan\delta'$$

where, $K' = 1 - \sin\phi'$

$\sigma_o' = \gamma_f' H_f + \gamma' z$

$\delta' = (0.5 - 0.7)\phi'$

The total downward force,

$$Q_n = \int_0^{L_1} p f_n\, dz = \int_0^{L_1} pK'(\gamma_f' H_f + \gamma' z) \tan\delta'\, dz$$

$$= (pK'\gamma_f' H_f \tan\delta') L_1 + \frac{L_1^2 pK'\gamma' \tan\delta'}{2}$$

Group Efficiency of Pile :

The efficiency of the load bearing capacity of a group pile,

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u}$$

where, $Q_{g(u)}$ = Ultimate load bearing capacity of group pile.

Q_u = Ultimate load bearing capacity of each pile without the group effect

➡ For Sand (Friction Piles)

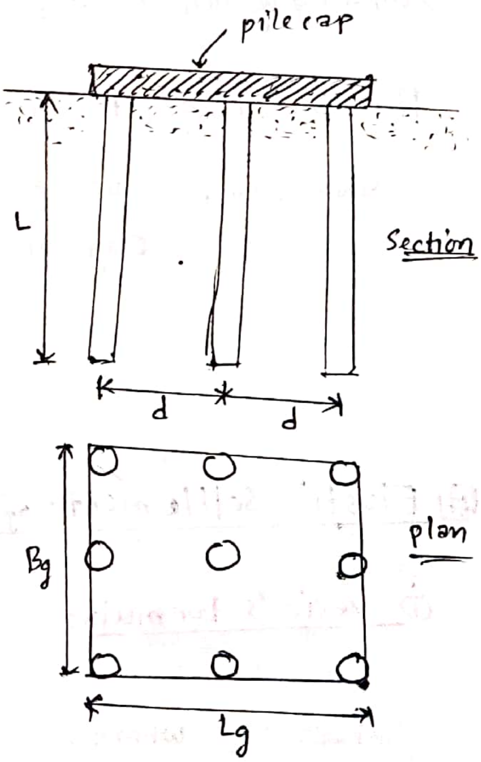
$$\eta = \frac{f_{av}[2(n_1+n_2-2)d+4D]L}{n_1 n_2 p L f_{av}} = \frac{2(n_1+n_2-2)d+4D}{p n_1 n_2}$$

Hence, $Q_{g(u)} = \frac{2(n_1+n_2-2)d+4D}{p n_1 n_2} \times \Sigma Q_u$

if, center to center spacing 'd' is large enough, $\eta > 1$. In that case the piles will behave as individual piles.

Thus, if $\eta < 1$, then $Q_{g(u)} = \eta \, \Sigma Q_u$

and, if $\eta \geq 1$, then, $Q_{g(u)} = \Sigma Q_u$



Equations for Group Efficiency of Friction piles:

① Converse – Labarre Equation:

$$\eta = 1 - \left[\frac{(n_1-1)n_2 + (n_2-1)n_1}{90 n_1 n_2}\right] \times \theta$$

where, $\theta\,(\text{deg}) = \tan^{-1}\left(\frac{D}{d}\right)$

Number of piles $= n_1 \times n_2$

(Note: $L_g \geq B_g$)

$L_g = (n_1-1)d + 2 \times \left(\frac{D}{2}\right)$

$B_g = (n_2-1)d + 2 \times \left(\frac{D}{2}\right)$

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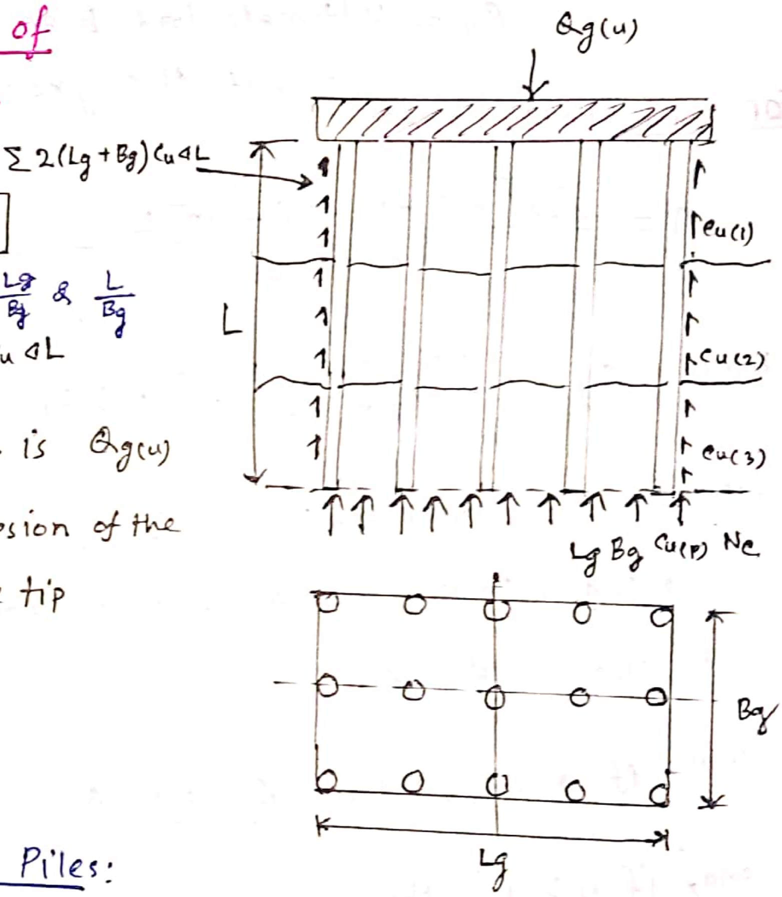
(ii) Los Angeles Group Action Equation:

$$\eta = 1 - \frac{D}{\pi d n_1 n_2} \times \left[n_1(n_2-1) + n_2(n_1-1) + \sqrt{2}\,(n_1-1)(n_2-1)\right]$$

(iii) Seiler–Keeny Equation:

$$\eta = \left\{1 - \left[\frac{11d}{7(d^2-1)}\right]\left[\frac{n_1+n_2-2}{n_1+n_2-1}\right]\right\} + \frac{0.3}{n_1+n_2}$$

Ultimate Bearing capacity of Group piles in saturated Clay:



$$\Sigma Q_u = n_1 n_2 \left[9 A_p c_{u(p)} + \Sigma \alpha p c_u \Delta L\right]$$

and

→ depends on $\frac{L_g}{B_g}$ & $\frac{L}{B_g}$

$$\Sigma Q_u = L_g B_g c_{u(p)} N_c^* + \Sigma 2(L_g + B_g) c_u \Delta L$$

The lower of the two value is $Q_{g(u)}$

where, $c_{u(p)}$ = undrained cohesion of the clay at the pile tip

Elastic Settlement of Group Piles:

(i) Vesic's Formula:

$$S_{g(e)} = \sqrt{\frac{B_g}{D}} \times S_e$$

where,

B_g = width of group pile section

D = width or diameter of each pile in the group

S_e = elastic settlement of each pile at comparable working load.

(ii) Meyerhof's Formula:

$$S_{g(e)}\ [\text{in.}] = \frac{2q\sqrt{B_g}\, I}{N_{60}}$$

where, $q = \frac{Q_g}{L_g B_g}$ (in U.S ton/ft^2)

I = influence factor $= \left(1 - \frac{L}{8B_g}\right) \geq 0.5$; L = length of embedment of piles. (ft)

In S I Units, $S_{g(e)}\ [\text{mm}] = \frac{0.96q\sqrt{B_g}\, I}{N_{60}}$; (q is in kN/m^2)

where, $I = 1 - \frac{L\,(m)}{8B_g\,(m)}$

based on cone penetration:

$$S_{g(e)} = \frac{q B_g I}{2q_c}$$

where, q_c = Average cone penetration resistance with in the seat of settlement.

⊞ Consolidation settlement of Group piles:

The consolidated settlement of each layer

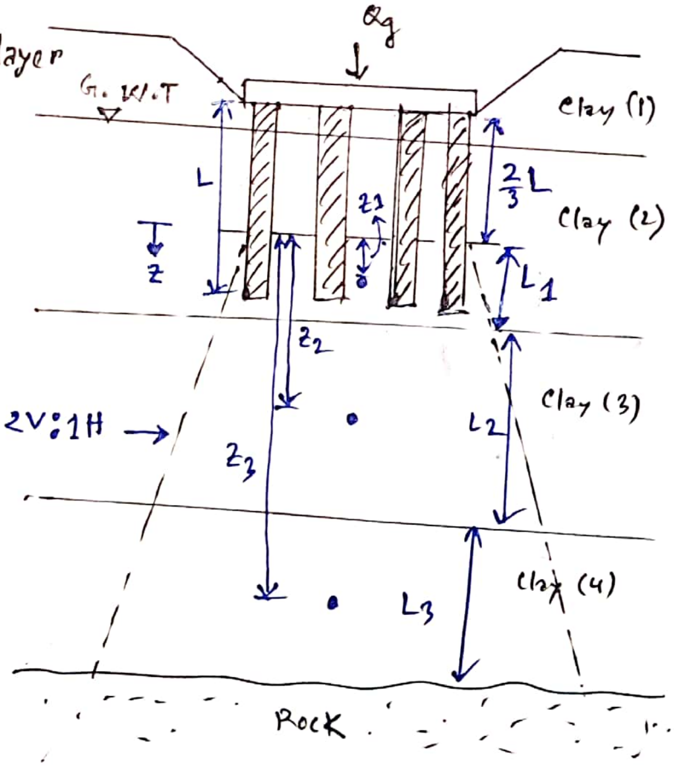
$$\Delta S_{c(i)} = \left[\frac{H_i}{1+e_{o(i)}}\right] \times \Delta e_{(i)}$$

Where,

$$\Delta e_{(i)} = C_{c(i)} \log\left[\frac{\sigma'_{o(i)} + \Delta\sigma'_{(i)}}{\sigma'_{o(i)}}\right]$$

Here,

$$\Delta\sigma'_{(i)} = \frac{Q_g}{(B_g + z_i)(L_g + z_i)}$$



∴ Total consolidation settlement,

$$\Delta S_{c(g)} = \Sigma \Delta S_{c(i)}$$

Pile Foundation (BOOK)

Example - 9.1, 9.5: (Pile in sand)

Consider a 20 m long concrete pile with a cross section of 0.407m × 0.407m fully embedded in sand. For the sand, given unit weight, $\gamma = 18\ KN/m^2$ and soil friction angle, $\phi' = 35°$. Estimate the allowable capacity Q_{all} with each of the following: (use $FS = 3$)

(a) Meyerhof's method; use, $K = 1.3$ and $\delta' = 0.8\phi'$

(b) Vesic's method; use, $K = 1.5$ and $\delta' = 0.7\phi'$

(c) Coyle and Castello's method

Solution:

(a) Meyerhof's method:

using meyerhof's formula,

$$Q_p = q' N_q^* A_p \le 0.5 P_a \tan\phi' N_q^* A_p$$

$$\Rightarrow Q_p = \gamma L \times N_q^* \times A_p$$

For $\phi' = 35°$, $N_q^* = 143$ (Table - 9.5)

$$\therefore Q_p = 18 \times 20 \times 143 \times (0.407 \times 0.407) = 8527.6\ KN$$

But, $0.5 P_a \tan\phi' N_q^* A_p = 0.5 \times 100 \times \tan 35° \times 143 \times (.407 \times .407)$

$= 829.32\ KN$

Hence, $Q_p = 829.32\ KN$

Now, $Q_s = \Sigma f p \Delta L$ where, $f = K \bar{\sigma}_o' \tan\delta'$

Here, $L' = 15D = 15 \times (.407) = 6.105\ m$

At, $z = 0$, $\sigma_o' = 0$; $\therefore f = 0$

$z = 6.105 m$, $\sigma_o' = (18 \times 6.105) = 109.89\ KN/m^2$

$\therefore f = K\bar{\sigma}_o' \tan\delta' = 1.3 \times 109.89 \times \tan(0.8 \times 35°) = 75.96\ KN/m^2$

Thus, $Q_s = \left(\frac{f_{z=0} + f_{z=6.105m}}{2}\right) \times pL' + f_{z=6.105m} \times p \times (L - L')$

$= \left(\frac{0 + 75.96}{2}\right) \times (4 \times .407) \times 6.105 + 75.96 \times (4 \times .407) \times (20 - 6.105)$

$= 2095.8\ KN$

$\therefore Q_{all} = \frac{Q_p + Q_s}{FS} = \left(\frac{829.32 + 2095.8}{3}\right) = 975\ KN$ (Ans.)

(b) Vesic's Method:

using vesic's formula,

$Q_p = \bar{\sigma}_o' N_q^* A_p$

Here, $\bar{\sigma}_o' = \left(\frac{1 + 2K_o}{3}\right) q'$ where, $K_o = 1 - \sin\phi' = (1 - \sin 35°) = 0.43$

$= \left(\frac{1 + 2 \times .43}{3}\right) \times \underset{\gamma}{18} \times \underset{L}{20}$

$= 223.2$

Now, $I_{rr} = \frac{I_r}{1 + 4 I_r}$

Here, $I_r = \frac{E_s}{2(1 + \mu_s) q' \tan\phi'}$

where, $E_s = m P_a$ Assume, $m = 250$ (medium dense soil)

$= (250 \times 100)$

$= 25000$

$\mu_s = 0.1 + 0.3\left(\frac{\phi - 25}{20}\right) = 0.1 + 0.3 \times \left(\frac{35 - 25}{20}\right) = 0.25$

$$\therefore I_r = \frac{25000}{2\times(1+.25)\times 18\times 20\times \tan 35^\circ} = 39.67$$

Again,

$$\Delta = 0.005\times\left(1-\frac{\phi-25}{20}\right)\times\frac{q'}{P_a} = 0.005\times\left(1-\frac{35-25}{20}\right)\times\frac{18\times 20}{100} = 0.009$$

$$\therefore I_{rr} = \frac{I_r}{1+\Delta I_r} = \frac{39.67}{1+39.67\times .009} = 29.23$$

From **Table-9.7**, For $I_r = 29.23$ and $\phi' = 35^\circ$

$$N_q^* = 47$$

$$\therefore Q_p = \underbrace{223.2}_{\bar{\sigma}_o'}\times\underbrace{47}_{N_q^*}\times(\underbrace{.407\times .407}_{A_p}) = 1737.7 \text{ KN}$$

Now,

$$Q_s = \left(\frac{f_{z=0}+f_{z=6.105m}}{2}\right)\times pL' + f_{z=6.105m}\times p\times(L-L')$$

Here, $f_{z=0} = 0$

$$f_{z=6.105m} = K\,\bar{\sigma}_o'\tan\delta' = 1.5\times 109.89\times\tan(0.7\times 35^\circ)$$

$$\therefore f_{z=6.105m} = 75.12 \text{ KN/m}^2$$

$$\therefore Q_s = \left(\frac{0+75.12}{2}\right)\times(4\times .407)\times 6.105 + 75.12\times(4\times .407)\times(20-6.105)$$

$$= 2072.6 \text{ KN}$$

$$\therefore Q_{all} = \frac{Q_p+Q_s}{FS} = \left(\frac{1737.7+2072.6}{3}\right) = 1270 \text{ KN}$$

(Ans.)

(c) Coyle and castello's method:

According to coyle and castello's method,

$$Q_p = q' N_q^* A_p$$

For, $\frac{L}{D} = \frac{20}{.407} = 49.14$ and $\phi' = 35°$; (Figure-9.15)

$$N_q^* = 34$$

$$\therefore Q_p = (18 \times 20) \times 34 \times (.407 \times .407) = 2027.54 \text{ KN}$$

Again, $Q_s = f_{av} \, p L$

Here, $f_{av} = K \bar{\sigma}_o' \tan \delta'$

For $\frac{L}{D} = 49.1$, $\phi' = 35°$ (From Figure -9.17)

$$K = 0.41$$

$$\therefore Q_s = 0.41 \times \underbrace{\left(\frac{20 \times 18}{2}\right)}_{\bar{\sigma}_o'} \times \tan(0.8 \times 35°) \times (4 \times 0.407) \times 20$$

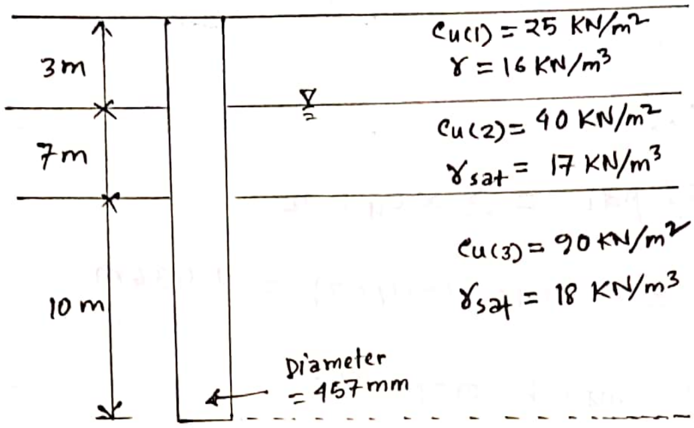
$$= 1277.66 \text{ KN}$$

$$\therefore Q_{all} = \left(\frac{Q_p + Q_s}{FS}\right) = \left(\frac{2027.54 + 1277.66}{3}\right) = 1101.73 \text{KN}$$

(Ans.)

Example - 9.2, 9.7: (Pile in Clay)

Consider a pipe pile having an outside diameter of 457 mm. The embedded length of the pile in layered saturated clay is 20 m.



(a). Estimate Q_p using Meyerhof's method and Vesic's Method

(b) Estimate Q_s by (1) the α method, (2) the λ method and (3) the β method, use $\phi_R' = 30°$ for all clay layers.

The top 10 m of clay is normally consolidated. The bottom clay layer has an OCR = 2

(c) Estimate the allowable pile capacity (Q_{all}). use FS = 4

Solution:

(a) Estimation of Q_p:

(i) Using meyerhof's method,

$$Q_p = 9\underset{\text{bottom layer}}{c_u} A_p = 9 \times 90 \times \frac{\pi}{4} \times \left(\frac{457}{1000}\right)^2 = 132.86\ KN$$

(ii) using vesic's method,

$$Q_p = c_u N_c^* A_p$$

$$I_{rr} \approx I_r = 347\left(\frac{c_u}{P_a}\right) - 33 = 347 \times \left(\frac{90}{100}\right) - 33 = 279.3$$

$$\therefore N_c^* = \frac{4}{3}(1 + \ln I_{rr}) + \frac{\pi}{2} + 1 = \frac{4}{3}(1 + \ln 279.3) + \frac{\pi}{2} + 1$$

$$\therefore N_c^* = 11.41$$

$\therefore\ Q_p = 90 \times 11.41 \times \frac{\pi}{4} \times \left(\frac{457}{1000}\right)^2 = 168.44$ KN.

The average value of Q_p is,

$$\frac{132.86 + 168.44}{2} = 150.65 \text{ KN}$$

(b) Estimation of Q_s:

(i) α method: $Q_s = \Sigma f\, p\Delta L = \Sigma \alpha C_u\, p\Delta L$

Here, $P = \pi D = 3.1416 \times (0.457) = 1.436$ m

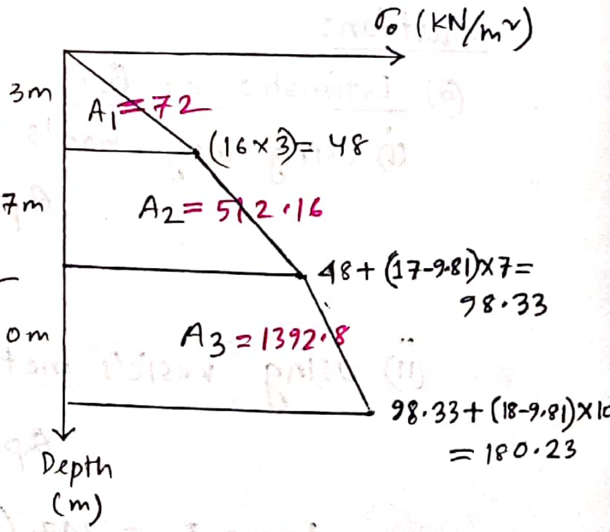
Now, following table may be prepared:

Depth (m)	ΔL (m)	cu (KN/m²)	$\frac{C_u}{P_a}$	α (Table-9.10)	$\alpha C_u p\Delta L$ (KN)
1-3	3	25	0.25	0.87	93.7
3-10	7	40	0.40	0.74	297.5
10-20	10	90	0.90	0.51	659.1

$Q_s = 1050.3$ KN

(ii) λ Method: $Q_s = f_{av}\, pL$

Where, $f_{av} = \lambda\,(\bar{\sigma}_o' + 2\,C_u)$



$$\bar{\sigma}_o' = \frac{72 + 512.16 + 1392.8}{20} = 98.85 \text{ KN/m}^2$$

$$C_u = \frac{25\times3 + 40\times7 + 90\times10}{20} = 62.75 \text{ KN/m}^2$$

For $L = 20$ m, (From Table-9.9)

$\lambda = 0.173$

$\therefore\ Q_s = 0.173 \times (98.85 + 2\times62.75) \times \pi \times 0.457 \times 20$

$= 1114.7$ KN

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(iii) β-method:

$$Q_s = \Sigma f\, p(\Delta L)$$

where, $f = \beta \bar{\sigma}_o'$; $\beta = K \tan \phi_R'$

$* K = 1 - \sin\phi_R'$ (Normally consolidated)

$* K = (1-\sin\phi_R')\sqrt{OCR}$ (overconsolidated)

Following Table may be prepared:

Depth (m)	ΔL (m)	σ_o' (KN/m^2)	$\bar{\sigma}_o'$ (KN/m^2)	β	f_{av} (KN/m^2)
1-3	3	48	$\left(\frac{0+48}{2}\right) = 24$	$(1-\sin 30^\circ)\tan 30^\circ = 0.29$	6.96
3-10	7	98.33	$\left(\frac{48+98.33}{2}\right) = 73.165$	0.29	21.22
10-20	10	180.23	$\left(\frac{98.33+180.23}{2}\right) = 139.28$	$0.29\sqrt{OCR} = 0.29\sqrt{2} = 0.41$	57.1

$$\therefore Q_s = \pi \times .457 \times (3 \times 6.96 + 7 \times 21.22 + 10 \times 57.1) = 1063\ KN$$

∴ The average value of Q_s,

$$\frac{1050.3 + 1114.7 + 1063}{3} = 1076\ KN$$

$$\therefore Q_u = Q_p + Q_s = (150.65 + 1076) = 1226.65$$

$$\therefore Q_{all} = \frac{Q_u}{FS} = \frac{1226.65}{4} = 306.66\ KN$$

(Ans.)

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Example - 9.3, 9.4: (Based on SPT value)

Consider a concrete pile that is 0.305 m × 0.305 m in cross section in sand. The pile is 12 m long. The following are the variations of N_{60} with depth:

Depth below Ground surface (m)	N_{60}
1.5	8
3.0	10
4.5	9
6.0	12
7.5	14
9.0	18
10.5	11
12	17
13.5	20
15.0	28
16.5	29
18.0	32
19.5	30
21.0	27

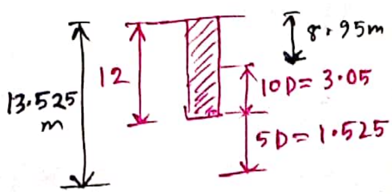
Determine the allowable load carrying capacity of the pile based on Meyerhof's method and Briaud's method. Use a Factor of safety, FS = 3

Solution:

Estimation of Q_p: The tip of the pile is 12 m below the ground surface.

For the pile D = 0.305 m. The average of N_{60} 10D above and 5D below the pile tip is,

$$N_{60} = \frac{18+11+17+20}{4} = 16.5 \simeq 17$$



(i) Meyerhof's method:

$\therefore Q_p = .4\, P_a\, N_{60} \frac{L}{D} A_p = 0.4 \times 100 \times 17 \times \left(\frac{12}{0.305}\right) \cdot (0.305 \times .305)$

$Q_p = 2488.8 \text{ KN}$

But, $4\, P_a\, N_{60}\, A_p = 4 \times 100 \times 17 \times (.305 \times .305) = 632.57 \text{ KN}$

Hence, $Q_p = 632.57 \text{ KN}$

(ii) Briaud's method:

$Q_p = 19.7\, P_a (N_{60})^{.36} A_p = 19.7 \times 100 \times (17)^{0.36} \times (.305 \times .305)$

$\therefore Q_p = 508.2 \text{ KN}$

Estimation of Q_s:

The average value of N_{60} for the sand for top 12m is

$\bar{N}_{60} = \frac{8+10+9+12+14+18+11+17}{8} = 12.375 \simeq 12$

(i) Meyerhof's method;

$Q_s = pLf_{av}$ Here, $f_{av} = 0.02\, P_a\, (\bar{N}_{60})$

$= (4 \times .305) \times 12 \times .02 \times 100 \times 12$

$= 351.36 \text{ KN}$

(ii) Briaud's method;

$Q_s = pLf_{av}$ Here, $f_{av} = 0.224\, P_a (\bar{N}_{60})^{.29}$

$= (4 \times .305) \times 12 \times .224 \times (10)^{.29} \times 100$

$= 639.4 \text{ KN}$

$\therefore$ Meyerhof's method: $Q_{all} = \frac{632.57+351.36}{3} = 327.98$ KN

and Briaud's method: $Q_{all} = \frac{508.2+639.4}{3} = 382.5$ KN

(Ans.)

Example - 9.6: (Based on CPT value)

Consider an 18-m long concrete pile (cross section: 0.305 m X 0.305 m) fully embedded in sand layer. For sand layer, the following is an approximation of the cone penetration resistance q_c (mechanical cone) and the frictional resistance f_c with depth. Estimate the allowable load that pile can carry. Use FS=3

Depth from ground surface	q_c (KN/m²)	f_c (KN/m²)
0-5	3040	73
5-15	4560	102
15-25	9500	226

Solution:

At the pile tip (at a depth of 18m),

$$q_p \simeq q_c = 9500 \text{ KN/m}^2$$

$$\therefore Q_p = A_p q_c = (.305 \times .305) \times 9500 = 883.7 \text{ KN}$$

Now, $Q_s = \Sigma p (\Delta L) f$ where, $f = \alpha' f_c$

For, $\frac{L}{D} = \frac{18}{0.305} = 59$, $\alpha' = 0.44$

$$\therefore Q_s = \underset{P}{(4\times\cdot 305)}\left[\underset{\alpha'}{\cdot 44}\times\underset{f_c}{73}\times\underset{\Delta L}{5} + \cdot 44\times 102\times 10 + \cdot 44\times 226\times \underset{\underset{L}{(18-15)}}{3}\right]$$

$$= 1107\cdot 42 \text{ KN}$$

$$\therefore Q_u = Q_p + Q_s = (883\cdot 7 + 1107\cdot 42) = 1991\cdot 12$$

$$\therefore Q_{all} = \frac{Q_u}{FS} = \frac{1991\cdot 12}{3} = 663\cdot 7 \text{ KN}$$

(Ans.)

***Example: 9.13:**

The allowable working load on a prestressed concrete pile 21-m long that has been driven into sand is 502 KN. The pile is circular in shape with D = 356 mm. skin resistance carries 350 KN of the allowable load and point bearing carries the rest. Use $E_p = 21\times10^6$ KN/m², $E_s = 25\times10^3$ KN/m², $\mu_s = 0\cdot35$ and $\xi = 0.62$. Determine the settlement of the pile.

Solution:

Total settlement of a pile under a vertical working load,

$$S_e = S_{e(1)} + S_{e(2)} + S_{(3)}$$

Here, $S_{e(1)} = \dfrac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}$

$$= \frac{(152 + 0\cdot62\times350)\times21}{\frac{\pi}{4}\times(\cdot356)^2\times 21\times10^6}$$

$$= 0\cdot00371 \text{ m}$$

$$= 3\cdot71 \text{ mm}$$

Given, $E_p = 21\times10^6$ KN/m²

$Q_T = 502$ KN

$Q_{ws} = 350$ KN

$\therefore Q_{wp} = (502-350)$

$= 152$ KN

$D = 0.356$ m

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$$S_{e(2)} = \frac{Q_{wp} D}{A_p E_s} \times (1-\mu_s^2) \times I_{wp}$$

$$= \frac{152 \times 0.356}{\frac{\pi}{4} \times (.356)^2 \times 25 \times 10^3} \times (1-0.35^2) \times 0.85$$

$$= 0.01622 m$$

$$= 16.22 mm$$

Again,

$$S_{e(3)} = \frac{Q_{ws} D}{p L E_s} \times (1-\mu_s^2) \times I_{ws}$$

Here,

$$I_{ws} = 2 + .35\sqrt{\frac{L}{D}}$$

$$= 2 + .35 \times \sqrt{\frac{21}{.356}}$$

$$= 4.69$$

$$= \frac{350 \times 0.356 \times (1-0.35^2)}{\pi \times 0.356 \times 21 \times 25 \times 10^3} \times 4.69$$

$$= 0.00087 m$$

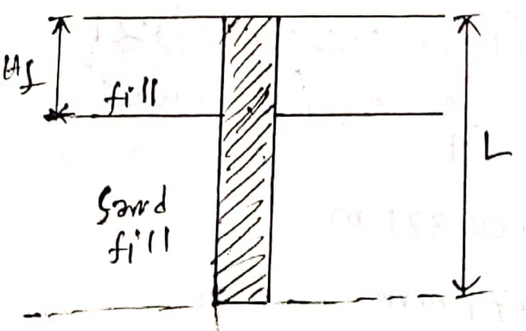
$$= 0.87 mm$$

$\therefore$ Total settlement, $S_e = (3.71 + 16.22 + 0.87) = 20.8$ mm

(Ans.)

Example-9.19:

In the following figure, $H_f = 2m$. The pile is circular in cross section with a diameter of 0.305 m. For the fill that is above the water table, $\gamma_f = 16$ KN/m3 and $\phi' = 32°$. Determine the total drag force. Use $\delta' = 0.6 \phi'$



Solution: we know, Total Drag force,

$$Q_n = p K' \gamma_f \tan\delta' \frac{H_f^2}{2}$$

Here, $p = \pi D = (3.1416 \times 0.305) = 0.96$

$K' = 1 - \sin\phi' = 1 - \sin 32^\circ = 0.47$

$\delta' = 0.6\phi = (0.6 \times 32) = 19.2^\circ$

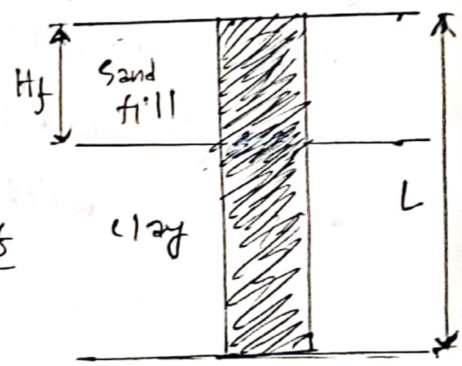
$$\therefore Q_n = 0.96 \times 0.47 \times 16 \times \tan 19.2^\circ \times \frac{2^2}{2}$$

$$= 5.03 \text{ KN.}$$

(Ans.)

Example: 9.20

In the following figure, $H_f = 2m$, diameter $= 0.305$ m, $\gamma_f = 16.5\ KN/m^3$, $\phi_{clay} = 34^\circ$, $\gamma_{sat(clay)} = 17.2\ KN/m^3$ and $L = 20$ m. The water table coincides with the top of the clay layer. Determine the downward drag force. Assume that, $\delta' = 0.6\phi'_{clay}$.



Solution:

The depth of the neutral plane,

$$L_1 = \frac{L - H_f}{L_1}\left[\frac{L - H_f}{2} + \frac{\gamma_f H_f}{\gamma'}\right] - \frac{2\gamma_f H_f}{\gamma'}$$

$$\Rightarrow L_1 = \frac{20-2}{L_1} \times \left[\frac{20-2}{2} + \frac{16.5 \times 2}{(17.2 - 9.81)}\right] - \frac{2 \times 16.5 \times 2}{(17.2 - 9.81)}$$

$$\Rightarrow L_1^2 = 242.38 - 8.93 L_1$$

$$\Rightarrow L_1 = 11.73 \text{ m}$$

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Now,

$$Q_n = pK'\tan\delta'\left(\gamma_f H_f L_1 + \gamma'\frac{L_1^2}{2}\right)$$

Here,

$$P = \pi \times D = (3.1416 \times 0.305) = 0.96$$

$$K' = (1 - \sin 34^\circ) = 0.44$$

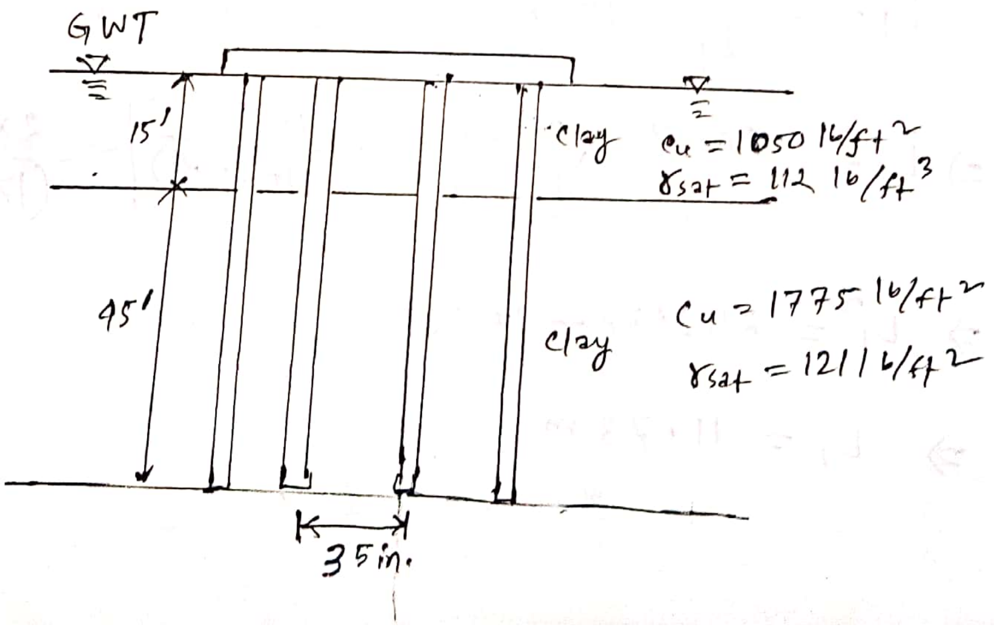
$$Q_n = 0.96 \times 0.44 \times \tan(0.6 \times 34)\left[16.5 \times 2 \times 11.73 + (17.2 - 9.81) \times \frac{11.73^2}{2}\right]$$

$\therefore Q_n = 140.7$ KN

(Ans.)

Example-9.21:

The section of a 3×4 group pile in a layer saturated clay is shown in figure below. The piles square in cross section (14 in × 14 in). The center to center spacing d, of the pile is 35 in. Determine the allowable bearing capacity of the pile group. Use FS = 4. Note that the ground water table coincides with the ground surface.



Solution:

we know,

$$\Sigma Q_u = n_1 n_2 [9 A_p c_{u(p)} + \Sigma p \alpha c_u (\Delta L)]$$

$$= n_1 n_2 [9 A_p c_{u(p)} + p \alpha_1 c_{u1} L_1 + p \alpha_2 c_{u2} L_2]$$

$$= (3 \times 4) \times \left[9 \times \left(\frac{14}{12} \times \frac{14}{12}\right) \times \underset{c_{u(b)}}{1775} + \left(4 \times \frac{14}{12}\right) \times \alpha_1 \times 1050 \times 15 + \left(4 \times \frac{14}{12}\right) \times \alpha_2 \times 1775 \times 45\right]$$

$$= 12 \times [21743.75 + 73500 \alpha_1 + 372750 \alpha_2]$$

Here,

Clay Layer-1: for, $\frac{c_{u(1)}}{p_a} = \frac{1050}{2000} = 0.525$, $\alpha_1 = 0.68$ (Table-9.10)

Clay Layer-2: for $\frac{c_{u(2)}}{p_a} = \frac{1775}{2000} = 0.89$, $\alpha_2 = 0.51$ (Table-9.10)

$$\therefore \Sigma Q_u = 12 \times [21743.75 + 73500 \times 0.68 + 372750 \times 0.51]$$

$$= 3141915 \text{ lb}$$

For pile acting as a group,

$$L_g = 3 \times 35 + 14 = 119 \text{ in} = 9.92 \text{ ft}$$

$$B_g = 2 \times 35 + 14 = 84 \text{ in} = 7 \text{ ft}$$

For $\frac{L_g}{B_g} = \frac{9.97}{7} = 1.42$ and $\frac{L}{B_g} = \frac{60}{7} = 8.57$

value of $N_c^* = 8.75$ (From figure 9.48)

$$\Sigma Q_u = L_g B_g c_u N_c^* + 2(B_g+L_g) \times [c_{u_1} L_1 + c_{u_2} L_2]$$

$$= 9.92 \times 7 \times 1775 \times 8.75 + 2 \times (9.92+7) \times [1050 \times 15 + 1775 \times 45]$$

$$= 4314440 \text{ lb} > 3141915 \text{ lb}$$

Hence, $\Sigma Q_u = 3141915$ lb

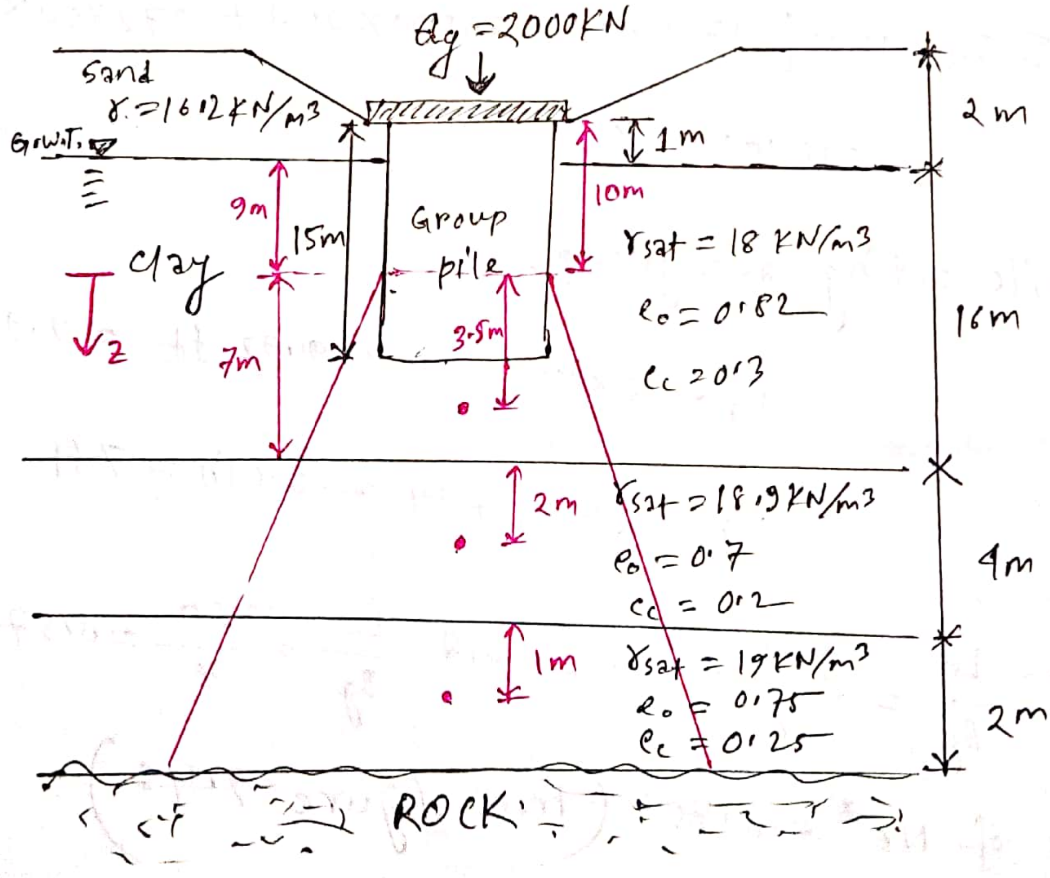
$$\Sigma Q_{all} = \frac{\Sigma Q_u}{FS} = \frac{3141915}{4} = 785478.75 \text{ lb}$$

$$\simeq 785.5 \text{ Kip.}$$

(Ans.)

Example - 9.23:

A group pile is shown in below. Determine the consolidation settlement of piles. All clays are normally consolidated. Given, $L_g = 3.3$ m, $B_g = 2.2$ m



Solution;

The stress ditribution starts at a depth of $\frac{2}{3}L = (\frac{2}{3} \times 15)$ m $= 10$ m below the top of the pile.

settlement of clay layer 1:

$$\Delta S_{c(1)} = \left[\frac{C_{c(1)} H_{c(1)}}{1+e_{o(1)}}\right] \log\left(\frac{\sigma'_{o(1)} + \Delta\sigma'_{(1)}}{\sigma'_{o(1)}}\right)$$

$$\sigma'_{o(1)} = (2 \times 16.2) + 12.5\,(18 - 9.81) = 134.8\ KN/m^2$$

$$\Delta\sigma'_{(1)} = \frac{Q_g}{(L_g + z_1)(B_g + z_1)} = \frac{2000}{(3.3+3.5) \times (2.2+3.5)}$$

$$\therefore \Delta\sigma'_{o(1)} = 51.6\ KN/m^2$$

$$\therefore \Delta S_{c(1)} = \frac{0.3 \times 7}{1+0.82} \times \log\left[\frac{134.8+51.6}{134.8}\right] = 0.1624\ m$$

$$= 162.4\ mm$$

settlement of clay layer 2:

Similarly,

$$\sigma'_{o(2)} = (2 \times 16.2) + 16 \times (18 - 9.81) + 2 \times (18.9 - 9.81) = 181.62\ KN/m^2$$

$$\Delta\sigma'_{o(2)} = \frac{2000}{(3.3+9) \times (2.2+9)} = 14.52\ KN/m^2$$

$$\therefore \Delta S_{c(2)} = \frac{0.2 \times 4}{1+0.7} \times \log\left(\frac{181.62+14.52}{181.62}\right) = 0.0157\ m = 15.7\ mm$$

settlement of layer 3 :

$$\sigma_0'(3) = 2\times(16.2) + 16\times(18-9.81) + 4\times(18.9-9.81) + 1\times(19-9.81)$$

$$= 208.99\ KN/m^2$$

$$\Delta\sigma_3' = \frac{2000}{(3.3+12)(2.2+12)} = 9.2\ KN/m^2$$

$$\Delta S_c(3) = \frac{0.25\times 2}{1+0.75} \times \log\left(\frac{208.99+9.2}{208.99}\right) = 0.0054m$$

$$= 5.4mm$$

∴ Total settlement,

$$\Delta S_c(g) = (162.4 + 15.7 + 5.4) = 183.5\ mm$$

(Ans.)

Pile Foundation

2018

A square concrete pile (30 cm side) 13 m long is driven into coarse sand ($\gamma = 18\ KN/m^3$, $N_{60} = 15$). Determine the allowable load. ($FS = 3.0$)

Solution: Given, $L = 13\ m$, $D = 30\ cm = 0.3\ m$, $N_{60} = 15$

Using Meyerhof's Method,

$$Q_p = A_p(q_p) = A_p \times \left(0.4 \times \frac{L}{D} \times P_a N_{60}\right) \leq A_p \times (4 P_a N_{60})$$

$$\therefore A_p \times \left(0.4 \times \frac{L}{D} \times P_a N_{60}\right) = (0.3 \times 0.3) \times 0.4 \times \frac{13}{0.3} \times 100 \times 15 = 2340\ KN$$

and, $A_p \times (4 P_a N_{60}) = (0.3 \times 0.3) \times 4 \times 100 \times 15 = 540\ KN$

Thus, $Q_p = 540\ KN$

$Q_s = pL f_{av}$ where, $f_{av} = 0.02\ P_a (\bar{N}_{60}) = 0.02 \times 100 \times 15$

$= 30\ KN/m^2$

$= (4 \times 0.3) \times 13 \times 30$

$= 468\ KN$

$\therefore$ The ultimate load, $Q_u = Q_p + Q_s = (540 + 468) = 1008\ KN$

$\therefore$ The allowable load, $Q_{all} = \frac{Q_u}{FS} = \frac{1008}{3} = 336\ KN$

(Ans.)

Alternative Solution:

Using Briaud's Method,

$$Q_p = A_p(q_p) = A_p \times \left[19.7\ P_a (N_{60})^{0.36}\right]$$

$$= (0.3 \times 0.3) \times 19.7 \times 100 \times (15)^{0.36}$$

$$= 470\ KN$$

$Q_s = pL f_{av}$ where, $f_{av} = 0.224\ P_a (\bar{N}_{60})^{0.29} = 0.224 \times 100 \times (15)^{0.29}$

$= 49.13\ KN/m^2$

$\therefore Q_s = (4 \times 0.3) \times 13 \times 49.13 = 766.43$ KN

$\therefore$ the ultimate load, $Q_u = Q_p + Q_s = (470 + 766.43)$

$= 1236.43$ KN

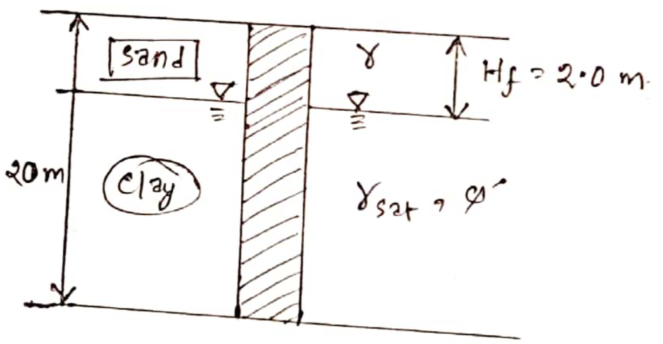
$\therefore$ The allowable load, $Q_{all} = \frac{Q_u}{FS}$

$= \frac{1236.43}{3} = 412.14$ KN

(Ans.)

2018

The pile is circular in cross section with a diameter of 0.305 m. The water table coincides with the top of the clay layer. Determine the negative skin friction. Given, $\gamma = 16$ KN/m³, $H_f = 2.0$ m, $\phi' = 35°$, $\gamma_{sat} = 17.5$ KN/m³, $L = 20$ m and $\delta' = 0.7\phi'$



Solution: The depth of neutral plan,

$$L_1 = \frac{L - H_f}{L_1} \times \left(\frac{L - H_f}{2} + \frac{\gamma_f H_f}{\gamma'}\right) - \frac{2\gamma_f H_f}{\gamma'}$$

$$\Rightarrow L_1 = \frac{20-2}{L_1} \times \left(\frac{20-2}{2} + \frac{16 \times 2}{17.5 - 9.81}\right) - \frac{2 \times 16 \times 2}{17.5 - 9.81}$$

$$\Rightarrow L_1 = \frac{236.9}{L_1} - 8.32$$

$$\Rightarrow L_1^2 + 8.32 L_1 - 236.9 = 0 \qquad \therefore L_1 = 11.784 \text{ m}$$

Now, The total Negative skin friction,

$$Q_n = P(K' \gamma_f H_f \tan\delta')L_1 + P\left(K' \times \frac{\gamma' L_1}{2} \times \tan\delta'\right) \times L_1$$

Here,

$K' = 1 - \sin\phi' = 1 - \sin 35° = 0.43$

$\delta' = 0.7\phi' = 0.7 \times 35° = 24.5$

$P = \pi D = (3.1416 \times 0.305) = 0.958\ m$

Hence,

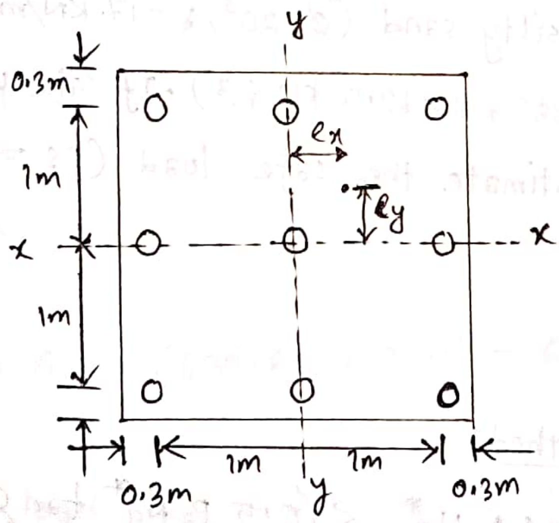
$$Q_n = (0.958 \times 0.43 \times 16 \times 2 \times \tan(24.5) \times 11.784) + \left(0.958 \times 0.43 \times (17.5 - 9.81) \times \frac{11.784^2}{2} \times \tan 24.5\right)$$

$\therefore Q_n = 171.03\ KN$

(Ans.)

2017

A pile group consisting of 9 piles is subjected to a load of 4 MN with eccentricity $e_x = 0.3m$ and $e_y = 0.4m$. Determine the maximum load carried in an indivi ual pile.



Solution: For Biaxial Bending Moment,

The force 'P' in any pile, $P = \frac{\Sigma V}{n} \pm \frac{M_y \cdot x}{\Sigma x^2} \pm \frac{M_x \cdot y}{\Sigma y^2}$

Here, $\Sigma V = 4 MN = 4 \times 10^3 KN$, $e_x = 0.3 m$ & $e_y = 0.4 m$

$$M_y = \Sigma V \times e_x = (4000 \times 0.3) = 1200 \text{ KN-m}$$

and, $M_x = \Sigma V \times e_y = (4000 \times 0.4) = 1600 \text{ KN-m}$

$\Sigma x^2 = 6 \times (1)^2 = 6 \ m^2$; $\Sigma y^2 = 6 \times (1)^2 = 6 \ m^2$

∴ The maximum load carried in an individual pile,

$$P = \frac{\Sigma V}{n} + \frac{M_y \cdot x}{\Sigma x^2} + \frac{M_x \cdot y}{\Sigma y^2}$$

$$= \frac{4000}{9} + \frac{1200 \times 1.0}{6} + \frac{1600 \times 1.0}{6}$$

$$= 911.11 \text{ KN}$$

(Ans.)

2016

A concrete pile 40 cm diameter, 9.0 m long is driven through a 6 m thick layer of silty sand ($\phi = 20°, \gamma = 17 KN/m^3$) overlying a dense layer of sand ($\phi' = 35°, \gamma = 19.5 KN/m^3$). If the water table is at the ground surface, estimate the safe load (FS = 3). Take K = 0.1 and $\delta = 0.7\phi$

Solution:

Using Meyerhof's Method:

Point load, $Q_p = q' A_p N_q^* \leq (0.5 \, P_a N_q^* \tan\phi) A_p$

At the base of pile, $q' = 6 \times (17 - 9.81) + 3 \times (19.5 - 9.81)$

$$= 72.21 \ KN/m^2$$

$A_p = \frac{\pi}{4} D^2 = \frac{\pi}{4} \times \left(\frac{40}{100}\right)^2 = 0.126 \ m^2$

For $\phi' = 35°$, $N_q^* = 143$ (from Table 9.5) [Book - B.M Das - 8th ed.]

Now, $q' A_p N_q^* = 72.21 \times 0.126 \times 143 = 1301.08$ KN

and $(0.5 \, P_a N_q^* \tan\phi') A_p = 0.5 \times 100 \times 143 \times \tan 35° \times 0.126 = 630.82$ KN

$\therefore Q_p = 630.82$ KN

skin friction, $Q_s = \Sigma p \Delta L f$ where, $f = K \sigma_o' \tan$, $(z = 0$ to $L')$
$= f_{L'=z}$ $(z = L'$ to $L)$

The unit friction increases with depth more or less linearly upto a depth L'.

$L' \approx 15D$ $\therefore L' = (15 \times 0.4) = 6$ m

At $z = 0$ m, $\sigma_o' = 0$ KN/m² ; $f = 0$ KN/m²

$* f = K \sigma_o' \tan \delta'$

$z = 6$m, $\sigma_o' = (17 - 9.81) \times 6 = 43.14$ KN/m² ; $f = 0.1 \times 43.14 \times \tan\left(\frac{.7 \times}{20}\right)$
$= 1.076$ KN/m²

Thus,

$$Q_s = \frac{(f_{z0} + f_{z=6})}{2} \times pL' + f_{z=6} \times p(L - L')$$

$$= \frac{(0 + 1.076)}{2} \times (\pi \times 0.40) \times 6 + 1.076 \times (\pi \times 0.40) \times (9 - 6)$$

$= 8.11$ KN

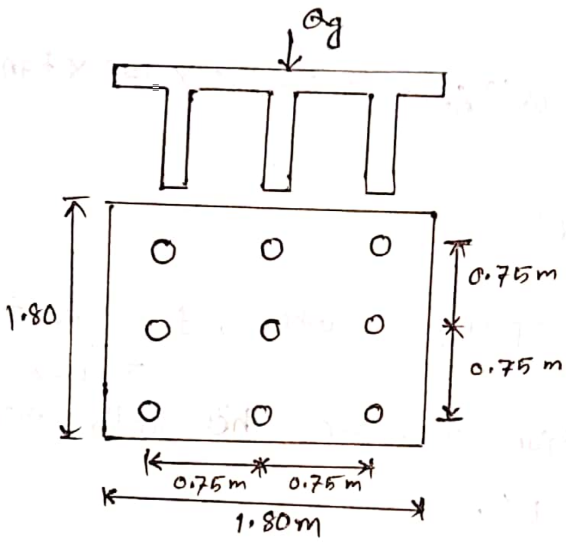
$\therefore Q_u = Q_p + Q_s = (630.82 + 8.11) = 638.93$ KN

$\therefore$ The safe load, $Q_{all} = \frac{Q_u}{FS} = \frac{638.93}{3} = 212.98$ KN

(Ans.)

2016, 2009, 2008

A pile group consists of 9 friction piles of 30 cm diameter and 10 m depth driven in clay ($c_u = 100$ kN/m², $\gamma = 20$ kN/m³) as shown in figure below. Determine the safe load for the group (FS = 3, $\alpha = 0.6$)



Solution:

Individual Action: $\Sigma Q_u = n_1 n_2 \left[9 A_p c_{u(p)} + \alpha_s p\, c_{u(1)} L_1\right]$

$= (3\times3)\times\left[9\times\frac{\pi}{4}\times(.30)^2\times100 + 0.6\times\pi\times0.3\times100\times10\right]$

$= 5661.95$ KN

Group Action:

$L_g = (2\times0.75) + 0.3 = 1.8$ m

$B_g = (2\times0.75) + 0.3 = 1.8$ m

For, $\frac{L_g}{B_g} = 1$ & $\frac{L}{B_g} = \frac{10}{1.8} = 5.56$; $N_c^* = 9$ (from graph) $\boxed{\text{Fig-9.48}}$ (8th ed.) BM Das

$\Sigma Q_u = L_g B_g c_{u(p)} N_c^* + 2\times(L_g + B_g)\, c_u L$

$= 1.8\times1.8\times9\times100 + 2\times(1.8+1.8)\times100\times10$

$= 10116$ KN

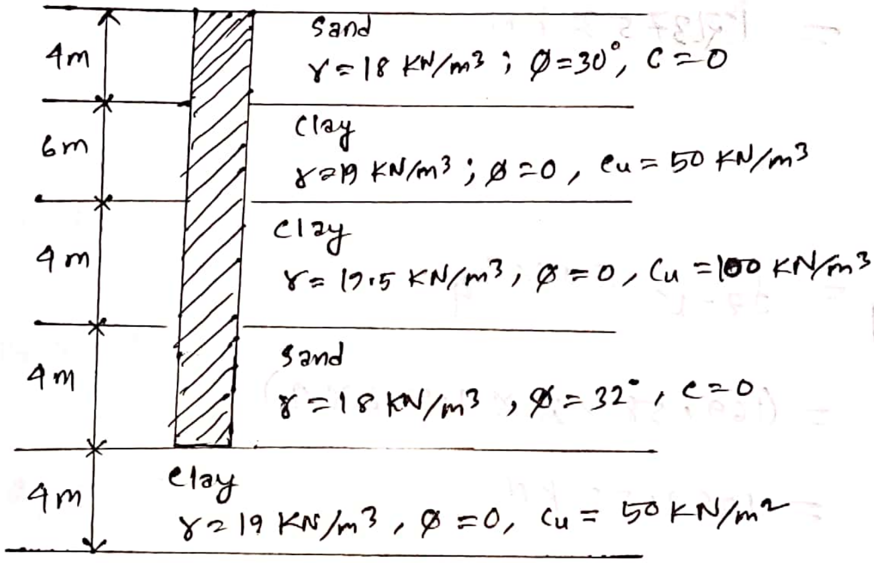
$\therefore Q_{g(u)} = 5661.95$ KN

$\therefore$ The safe load, $Q_{all} = \frac{Q_{g(u)}}{FS} = \frac{5661.95}{3} = 1887.32$ KN

(Ans.)

2017

Estimate the ultimate Bearing capacity of the single pile as shown in figure below. Dia of pile = 20 in. Type of pile = Driven pile. Assume $\delta = 0.7\phi$. Necessary graph will be supplied.



20 in = 50.8 cm

Solution:

End bearing, $Q_p = A_p C_u N_c^*$

$$= \frac{\pi}{4} \times (0.508)^2 \times 50 \times 9$$

$$= 91.2076\ KN/m^2$$

Skin friction, $Q_s = Q_{s_1} + Q_{s_2} + Q_{s_3} + Q_{s_4}$

For layer 1: we know, $Q_{s_1} = \Sigma f p \Delta L$ where, $f = K\sigma_0' \tan\delta'$

Here, $L' = 15D$

$$= (15 \times 0.508) = 7.62\ m$$

At $z = 0$, $\sigma_0' = 0$, $f = 0$

$z = 4m$ $\sigma_0' = \gamma z_1 = (18 \times 4) = 72\ KN/m^2$

$\therefore f_{z=4} = 1.4(1 - \sin 30^\circ) \times 72 \times \tan(0.7 \times 30^\circ) = 19.35\ KN/m^2$

$z = 7.62m$, $\sigma_0' = 18 \times 4 + (7.62 - 4) \times 19 = 169.58\ KN/m^2$

$$\therefore Q_{s_1} = \left(\frac{f_{z=0} + f_{z=4m}}{2}\right) \times p \times L_1$$

$$= \left(\frac{0 + 19.35}{2}\right) \times \pi \times 0.508 \times 4)$$

$$= 61.7626 \text{ KN}$$

<u>layer 4:</u>

$$Q_{s_4} = f_{z=L'} \times p \times L_4$$

$$= (169.58 \times \pi \times 0.508 \times 4)$$

$$= 1082.55 \text{ KN}$$

<u>For layer 2 & 3:</u>

For $\frac{c_{u_2}}{p_a} = \frac{50}{100} = 0.5$, $\alpha = 0.68$

$\frac{c_{u_3}}{p_a} = \frac{100}{100} = 1$, $\alpha = 0.48$

$$\therefore Q_{s_2} + Q_{s_3} = \pi \times 0.508 \times (0.68 \times 50 \times 6 + 0.48 \times 100 \times 4)$$

$$= 631.9894 \text{ KN}$$

$$\therefore \text{Total } Q_s = (61.7626 + 1082.55 + 631.9894) \text{ KN}$$

$$= 1776.302 \text{ KN}$$

$\therefore$ Total ultimate capacity, $Q_u = Q_p + Q_u$

$$= (91.2076 + 1776.302)$$

$$= 1867.5096 \text{ KN}$$

(Ans.)

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2015

A 10 m long concrete pile, 30 cm diameter is driven into a medium dense sand ($\phi = 30°$, $\gamma = 20\ KN/m^3$). Estimate the safe load if FS = 3.0

Solution:

Using Meyerholf's Method,

$Q_p = q' A_p N_q^*$ Here, For $\phi' = 30°$, $N_q^* = 56.7$

$= (20 \times 10) \times \frac{\pi}{4} \times (.30)^2 \times 56.7$

$= 801.58\ KN \leq (0.5\ P_a N_q^* \tan\phi')A_p$

$\therefore (0.5\ P_a N_q^* \tan\phi)A_p = 0.5 \times 100 \times 56.7 \times \tan 30 \times \frac{\pi}{4} \times (.3)^2$

$= 115.7\ KN$

Thus, $Q_p = 115.7\ KN$

Now, $Q_s = \Sigma P \Delta L f$ where, $f = K \sigma_0' \tan\delta'$

$L' = 15D = (15 \times 0.3) = 4.5\ m$

$K = 1.4\ (1 - \sin\phi') = 1.4 \times (1 - \sin 30) = 0.7$ (for low displacement driven pile)

At $z = 0$, $\sigma_0' = 0$ $\therefore f = 0$

At $z = 4.5m$, $\sigma_0' = (20 \times 4.5) = 90\ KN/m^2$ $\therefore f = 0.7 \times 90 \times \tan(0.8 \times 30)$
$= 28.05\ KN/m^2$

Now, $Q_s = \left(\frac{f_{z=0} + f_{z=L'}}{2}\right) \times pL' + f_{z=L'} \times p \times (L - L')$

$= \left(\frac{0 + 28.05}{2}\right) \times (\pi \times 0.3) \times 4.5 + 28.05 \times (\pi \times 0.3) \times (10 - 4.5)$

$= 204.9\ KN$

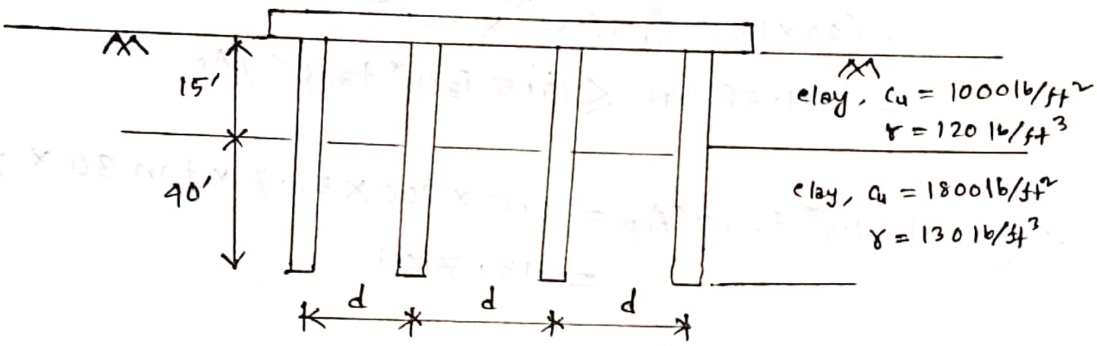
$\therefore Q_u = Q_p + Q_s = (115.7 + 204.9) = 320.6\ KN$

$\therefore$ The safe load, $Q_{all} = \frac{Q_u}{FS} = \frac{320.6}{3} = 106.87\ KN$

(Ans.)

2015

A section of a 3X4 group pile in layered saturated clay is shown in figure below. The piles are square in cross section (15 in x 15 in). The center to center spacing of piles is 36 in. Determine the allowable load bearing capacity of pile, if FS = 4



Solution:

Individual Action:

$$\Sigma Q_u = n_1 n_2 [9 A_p C_{u(p)} + \alpha_1 C_{u(1)} P L_1 + \alpha_2 C_{u(2)} P L_2]$$

Here, $L_g = (36\times3 + 2\times7.5) = 123$ in ; $B_g = (36\times2+15) = 87$ in

$A_p = (15\times15) = 225 \, in^2 = 1.5625 \, ft^2$

For, $\frac{C_{u(1)}}{P_a} = \frac{1000}{2000} = 0.5$; $\alpha_1 = 0.68$ (From Table-9.10)

$\frac{C_{u(2)}}{P_a} = \frac{1800}{2000} = 0.9$; $\alpha_2 = 0.51$

Hence, $\Sigma Q_u = (3\times4)\times[9\times1.5625\times1800 + 0.68\times4\times(\frac{15}{12})\times1000\times15 + 0.51\times4\times(\frac{15}{12})\times1800\times40]$

$= 3118950$ lb $= 3118.95$ Kips.

Group Action: $L_g = \frac{123}{12} = 10.25'$, $B_g = \frac{87}{12} = 7.25'$

For $\frac{L_g}{B_g} = 1.4$ & $\frac{L}{B_g} = \frac{55}{7.25} = 7.6$; $N_c^* = 8.7$ (from Graph) Fig-9.48

Now,

$$\Sigma Q_u = L_g B_g C_{u(p)} N_c^* + \Sigma 2 (L_g + B_g) C_u (\Delta L)$$

$$= (10.25 \times 7.25) \times 1800 \times 8.7 + 2 \times (10.25 + 7.25) \times 1000 \times 15 + 2 \times (10.25 + 7.25) \times 1800 \times 40$$

$= 4208733.75$ lb

$= 4208.73$ Kips.

Thus, $Q_{g(u)} = 3118.95$ Kips.

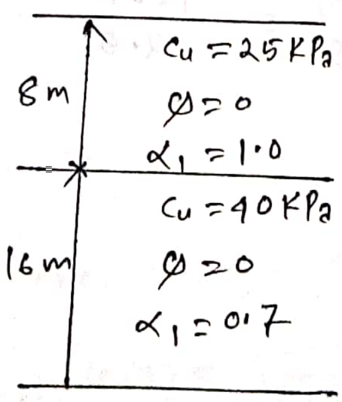
$\therefore$ The allowable load bearing Capacity, $Q_{all} = \frac{Q_{g(u)}}{FS} = \frac{3118.95}{4}$ Kip

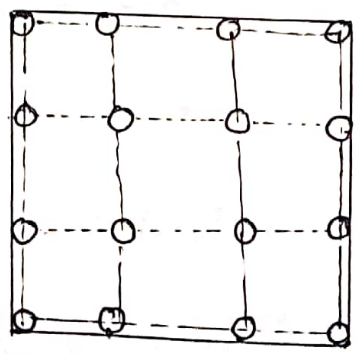
$= 779.74$ Kips.

(Ans.)

2014, 2011

A group of 16 piles (4 in each row) was installed in a layered clay soil as shown in figure below. The diameter of each pile is 500 mm and their c/c distance is 1 m. The length of pile group is 18 m. Estimate the safe load capacity of the group with FS = 2.5





Solution:

Individual Action:

$$\Sigma Q_u = n_1 n_2 \left[9 A_p C_{u(p)} + \Sigma \alpha C_u P \Delta L\right]$$

$$= (4\times4)\times\left[9\times\frac{\pi}{4}(.5)^2\times40 + 1.0\times25\times(\pi\times0.5)\times8 + 0.7\times40\times(\pi\times0.5)\times10\right]$$

$$= 13194.72 \text{ KN}$$

Group Action:

$$L_g = B_g = (3\times1+0.5) = 3.5\,m$$

For, $\frac{L_g}{B_g} = 1$ & $\frac{L}{B_g} = \frac{18}{3.5} = 5.14$; $N_c^* = 9$ (from graph)

$$\Sigma Q_u = L_g B_g C_{u(p)} N_c^* + \Sigma 2(L_g + B_g) C_u \Delta L$$

$$= \left[3.5\times3.5\times40\times9 + 2\times(3.5+3.5)\times25\times8 + 2\times(3.5+3.5)\times40\times10\right]$$

$$= 12810 \text{ KN}$$

Thus, $Q_{g(u)} = 12810$ KN

∴ The safe load capacity of the group,

$$Q_{all} = \frac{Q_{g(u)}}{FS} = \frac{12810}{2.5} = 5124 \text{ KN}$$

(Ans.)

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2013

The allowable working load on a prestressed concrete pile 20 m long that has been driven in to sand is 500 KN. The pile is circular in shape with D = 350 mm. Skin friction carries 300 KN of the allowable load, and point bearing carries the rest. Use $E_p = 21 \times 10^6$ KN/m², $E_s = 25 \times 10^3$ KN/m², $\mu_s = 0.35$ and $\xi = 0.62$. Determine the settlement of the pile.

Solution:

We know, Total elastic settlement of a pile,

$$S_e = S_{e(1)} + S_{e(2)} + S_{e(3)}$$

$$S_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws}) L}{A_p E_p}$$

Here, $A_p = \frac{\pi}{4} \times (.35)^2 = 0.0962 \text{ m}^2$, $Q_{ws} = 300$ KN

$$Q_{wp} = (500 - 300) = 200 \text{ KN}$$

$$\therefore S_{e(1)} = \frac{(200 + 0.62 \times 300) \times 20}{0.0962 \times 21 \times 10^6} = 0.00382 \text{ m} = 3.82 \text{ mm}$$

$$S_{e(2)} = \frac{q_{wp} D}{E_s} \times (1 - \mu_s^2) I_{wp}$$

$$= \frac{200}{.0962} \times \frac{0.35}{25 \times 10^3} \times (1 - 0.35^2) \times 0.85$$

$$= 0.02171 \text{ m} = 21.71 \text{ mm}$$

And, $S_{e(3)} = \frac{Q_{ws}}{pL} \times \frac{D}{E_s} \times (1 - \mu_s^2) I_{ws}$

Here, $p = \pi \times (.35) = 1.1$ m ; $I_{ws} = 2 + 0.35\sqrt{\frac{L}{D}} = 2 + 0.35\sqrt{\frac{20}{.35}}$

$\therefore I_{ws} = 4.65$

$$\therefore S_{e(3)} = \frac{300}{1.1 \times 20} \times \frac{0.35}{25 \times 10^3} \times (1 - 0.35^2) \times 4.65 = 0.00078 \text{ m} = 0.78 \text{ mm}$$

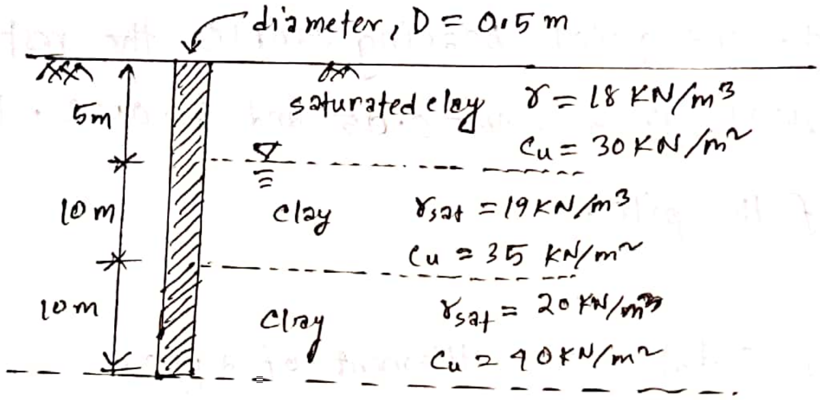
$\therefore$ Total settlement, $S_e = S_{e(1)} + S_{e(2)} + S_{e(3)} = (3.82 + 21.71 + 0.78) = 26.31$ mm (Ans.)

2013

\# A driven pile in clay is shown in figure below.

(i) Calculate the point bearing capacity.

(ii) Calculate the skin resistance using α and λ method.



Solution:

(i) Using Meyerhof method,

point Bearing, $Q_p = N_c^* \, C_u \, A_p$

$$= 9 \times 40 \times \frac{\pi}{4} \times (0.5)^2$$

$$= 70.686\ KN$$

(Ans.)

(ii) Skin friction:

(a) λ method:

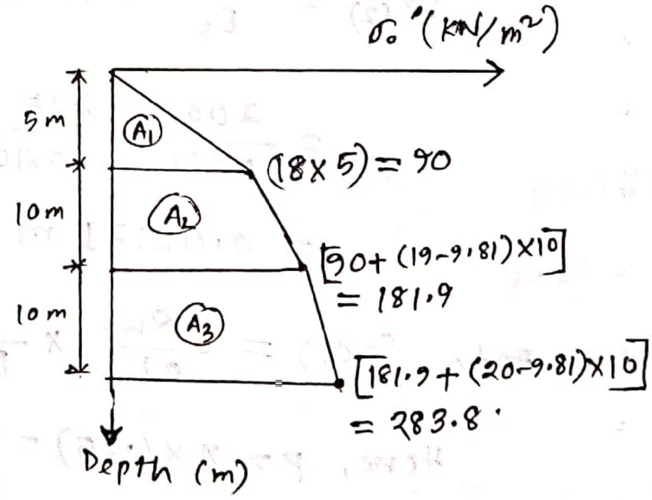
$$C_u = \frac{30\times5 + 35\times10 + 40\times10}{5+10+10} = 36\ KN/m^2$$

$$\sigma_o' = \frac{A_1 + A_2 + A_3}{L}$$

Here $A_1 = (\frac{1}{2}\times5\times90) = 225$

$A_2 = \frac{1}{2}\times(90+181.9)\times10 = 1359.5$

$A_3 = \frac{1}{2}\times(181.9+283.8)\times10 = 2328.5$



$$\therefore \bar{\sigma_o}' = \frac{225+1359.5+2328.5}{25} = 156.52$$

$\therefore f_{av} = \lambda\,(\bar{\sigma_o}' + 2C_u)$ for $L = 25\,m$; $\lambda = 0.15$ (from Table)

Table - 9.9 (8th ed.)

BM Das.

$\therefore f_{av} = 0.15 \times (156.52 + 2 \times 36) = 34.278 \ KN/m^2$

Thus, $Q_s = p L f_{av} = \pi \times (0.5) \times 25 \times 34.278 = 1346.1 \ KN$

(Ans.)

(b) α method:

$$f = \alpha C_u$$

* layer 1: For $\frac{C_{u(1)}}{P_a} = \frac{30}{100} = 0.3$; $\alpha_1 = 0.82$ (from Table 9.10) BM Das (8th ed.)

$\therefore f_1 = (0.82 \times 30) = 24.6 \ KN/m^2$

* layer 2: For, $\frac{C_{u(2)}}{P_a} = \frac{35}{100} = 0.35$; $\alpha_2 = 0.78$

$\therefore f_2 = (0.78 \times 35) = 27.3 \ KN/m^2$

* layer 3: For $\frac{C_{u(3)}}{P_a} = \frac{40}{100} = 0.4$; $\alpha_3 = 0.74$

$\therefore f_3 = (0.74 \times 40) = 29.6 \ KN/m^2$

Thus,

$Q_s = \Sigma p(\Delta L) f = (\pi \times 0.5) \times [24.6 \times 5 + 27.3 \times 10 + 29.6 \times 10]$

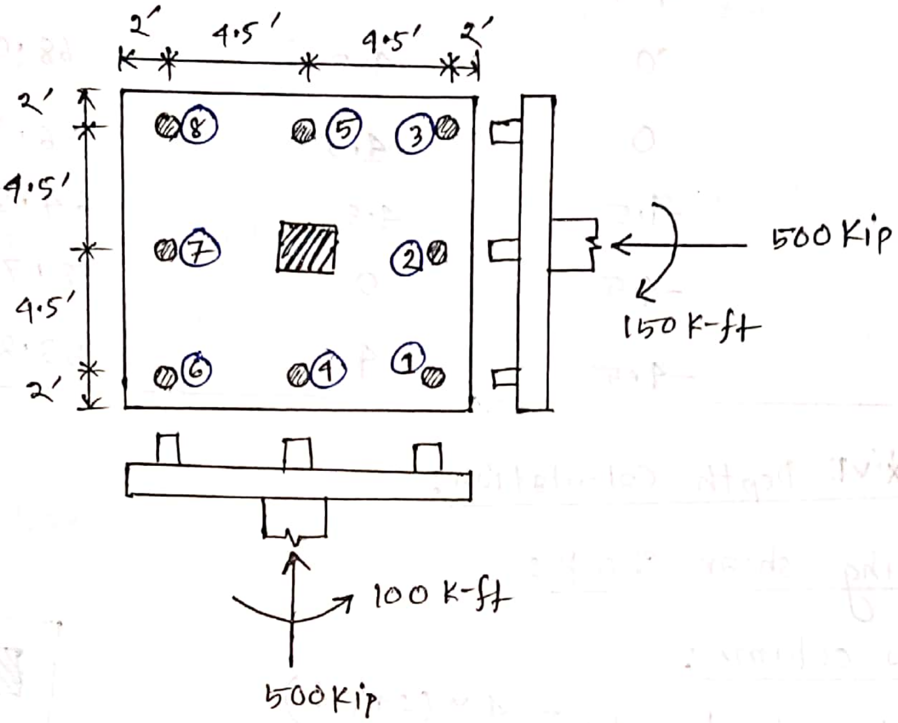
$\therefore Q_s = 1086.9936 \simeq 1087 \ KN$

(Ans.)

Pile - Cap

2018

A pile cap is shown in figure below. Design the pile cap with details reinforcement using. $f_c' = 3.0$ ksi and $f_y = 60$ ksi. Use pile diameter = 18" and column size = 24" × 24"



Solution: <u>vertical load carried by each load:</u>

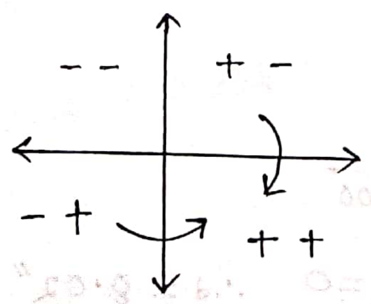
$$P_n = \frac{\Sigma V}{n} \pm \frac{M_y \cdot x}{\Sigma x^2} \pm \frac{M_x \cdot y}{\Sigma y^2}$$

$\Sigma V = 500$ Kip ; $n = 8$, $M_y = 100$ K-ft , $M_x = 150$ K-ft

$\Sigma x^2 = 6 \times 4.5^2 = 121.5$ ft^2 , $\Sigma y^2 = 6 \times 4.5^2 = 121.5$ ft^2

Thus, $P_n = \dfrac{500}{8} \pm \dfrac{100}{121.5} x \pm \dfrac{150}{121.5} \pm y$

$\therefore P_n = 62.5 \pm 0.823 x \pm 1.235 y$



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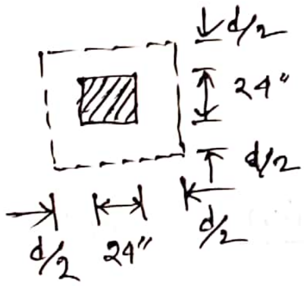
Pile No	x (ft)	y (ft)	Pn (Kip)
1	4.5	4.5	71.761
2	4.5	0	66.2035
3	4.5	-4.5	60.646
4	0	4.5	68.0575
5	0	-4.5	56.9425
6	-4.5	4.5	64.354
7	-4.5	0	58.7965
8	-4.5	-4.5	53.239

Effective Depth Calculation:

① Punching shear check:

(a) below column:

critical perimeter, $b_o = 4 \times (24 + d)$



critical section

Now, $\dfrac{\Sigma V}{b_o d} = 2\sqrt{f_c'}$

$$\Rightarrow \frac{500 \times 10^3}{4 \times (24+d) \times d} = 2 \times \sqrt{3000}$$

$$\Rightarrow 24d + d^2 - 1141.09 = 0$$

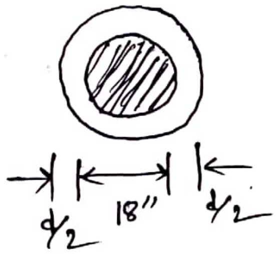
$$\Rightarrow d = 23.85'' \simeq 24 \text{ in.}$$

in $\boxed{\text{USD}}$

$$\frac{V_u}{b_o d} = 4\,\phi\sqrt{f_c'}$$

(b) above pile: (এইটা না check করলেও হবে)

critical perimeter, $b_o = \pi \times (18 + d)$



critical section

Thus, $\dfrac{P_{max}}{b_o d} = 2\sqrt{f_c'}$

$$\frac{71.761 \times 10^3}{\pi(18+d) \times d} = 2\sqrt{3000}$$

$$\Rightarrow d^2 + 18d - 208.52 = 0 \qquad \therefore d = 8.02'' \simeq 8.25''$$

(11) Beam shear check:

Maximum load strip: (in x-x direction)

$(1,4,6) = (71.761 + 68.0575 + 64.354) = 204.2\ K$

$(2,7) = (66.2035 + 58.7965) = 125\ K$

$(3,5,8) = (60.646 + 56.9425 + 53.239) = 170.83\ K$

(in Y-Y direction)

$(1,2,3) = (71.761 + 66.2035 + 60.646) = 198.61\ K$

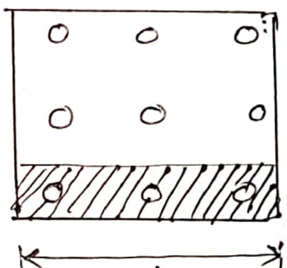
$(4,5) = (68.0575 + 56.9425) = 125\ K$

$(6,7,8) = (64.354 + 58.7965 + 53.239) = 176.39\ K$

$\therefore$ Maximum load in a strip $= 204.2\ K$

Now, $\frac{V}{bd} = 1.1\sqrt{f_c'}$

in USD: $\frac{V_u}{bd} = 2\phi\sqrt{f_c'}$



critical section

$\Rightarrow \frac{204.2 \times 10^3}{(13 \times 12) \times d} = 1.1\sqrt{3000}$

$\Rightarrow d = 21.73'' \simeq 22''$

Hence, Effective Depth, $d_{eff} = 24$ in.

$\therefore$ Total thickness of pile cap, $t = (24 + 6 + 2 + \frac{10}{2 \times 8})$
$= 32.625$ in.

Reinforcement Calculation:

$n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{57000\sqrt{3000}} = 9.28 \simeq 9$

$r = \frac{f_s}{f_c} = \frac{0.4 \times 60}{0.45 \times 3} = 17.78$

$K = \frac{n}{n+r} = \frac{9}{9+17.78} = 0.336$

$j = 1 - \frac{K}{3} = 0.888$

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$$M_{x-x} = (V_{max})_{x-x} \times \left(4.5 - \frac{24}{2 \times 12}\right) = (204.2 \times 3.5) = 714.7 \text{ K-ft}$$

$$M_{y-y} = (V_{max})_{y-y} \times \left(4.5 - \frac{24}{2 \times 12}\right) = (198.61 \times 3.5) = 695.135 \text{ K-ft}$$

$$\therefore As_{x-x} = \frac{M_{x-x}}{f_s \times j d} = \frac{714.7 \times 12}{0.4 \times 60 \times 0.888 \times 24} = 16.77 \text{ in}^2$$

provide 14 # 10 bar

and,

$$As_{y-y} = \frac{M_{y-y}}{f_s \times j d} = \frac{695.135 \times 12}{0.4 \times 60 \times 0.888 \times 24} = 16.31 \text{ in}^2$$

Provide 13 # 10 bar

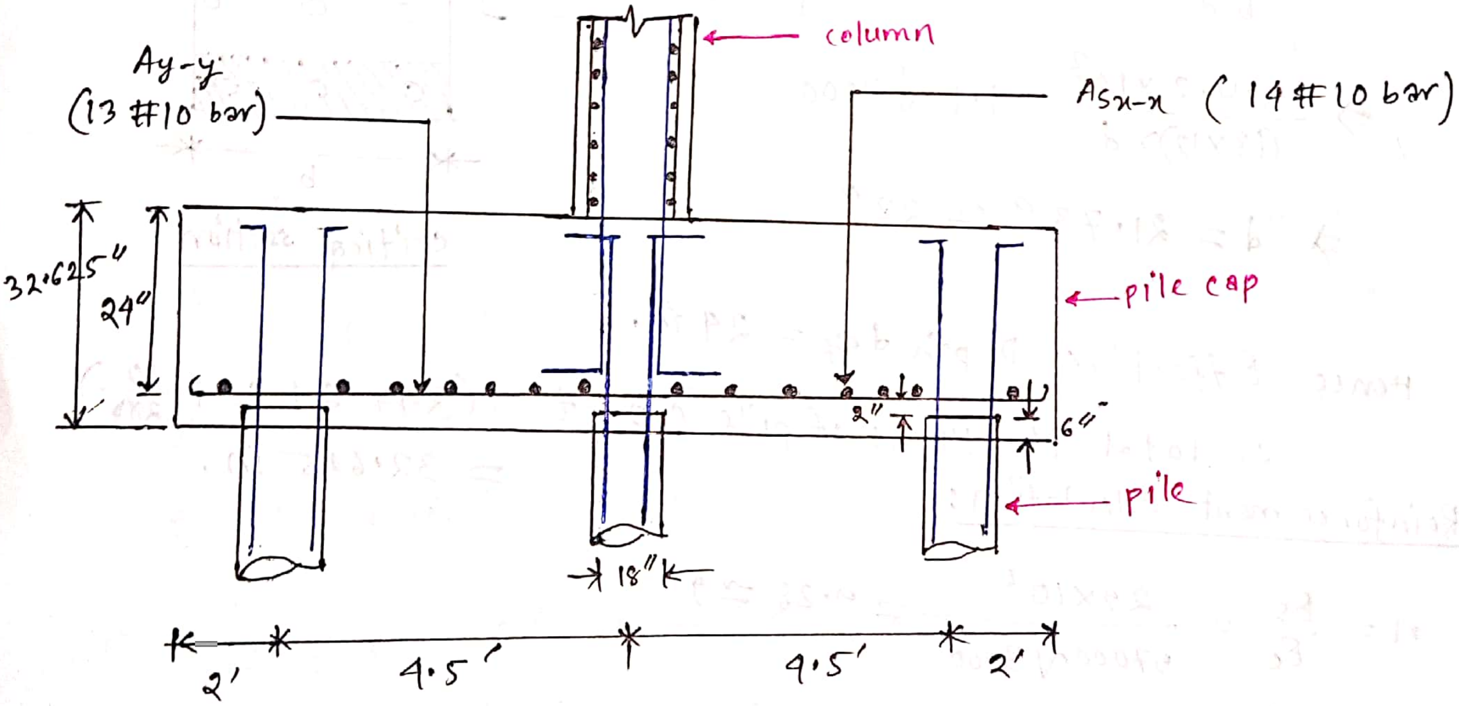
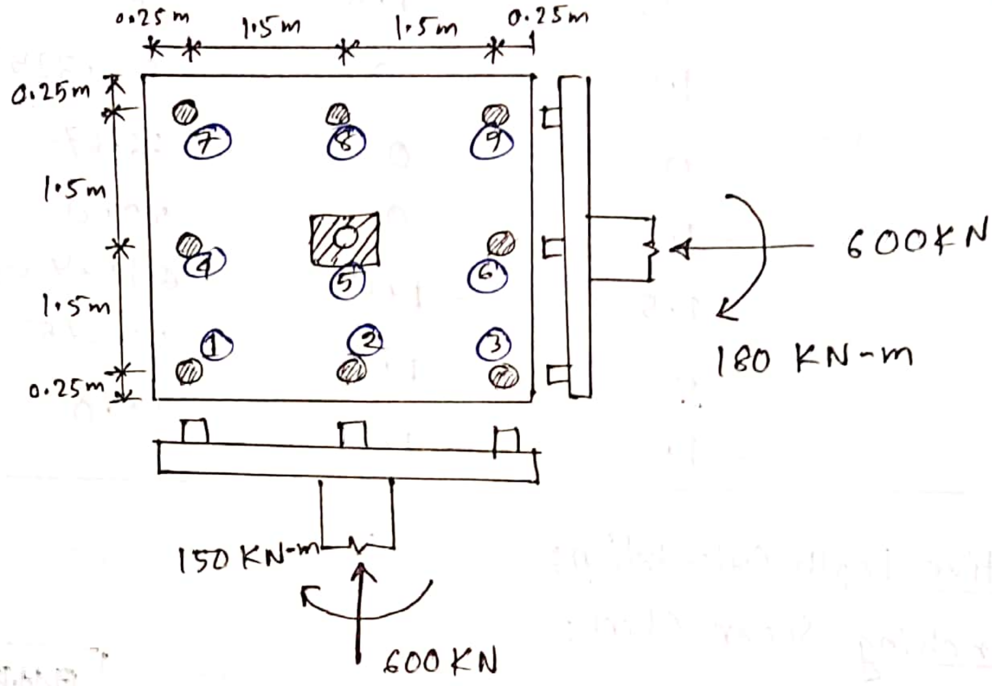


Fig. Reinforcement Details of pile Cap.

2017

A pile cap is shown in figure below. Design the pile cap using USD method. Column size = 500 mm × 400 mm. Pile Dia = 380 mm. Assume, $f_c' = 20$ MPa and $f_y = 380$ MPa.



Solution: vertical load carried by each pile:

$$P_u = \frac{\Sigma V_u}{n} \pm \frac{M_{uy} \cdot x}{\Sigma x^2} \pm \frac{M_{ux} \cdot y}{\Sigma y^2}$$

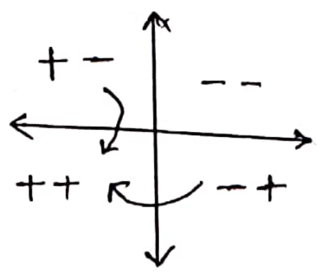
Here, $\Sigma V_u = 600$ KN, $n = 9$, $M_{uy} = 150$ KN-m, $M_{ux} = 180$ KN-m

$\Sigma x^2 = 6 \times (1.5)^2 = 13.5\ m^2$; $\Sigma y^2 = 6 \times (1.5)^2 = 13.5\ m^2$

Hence,

$$P_u = \frac{600}{9} \pm \frac{150}{13.5} \times x \pm \frac{180}{13.5} \times y$$

$$\Rightarrow P_u = 66.67 \pm 11.11 x \pm 13.33 y$$

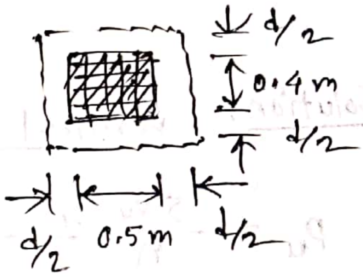


Pile No.	x (m)	y (m)	P_u (KN)
1	1.5	1.5	103.33
2	0	1.5	86.665
3	−1.5	1.5	70.0
4	1.5	0	83.335
5	0	0	66.67
6	−1.5	0	50.0
7	1.5	−1.5	63.34
8	0	−1.5	46.675
9	−1.5	−1.5	30.0

Effective Depth Calculation:

① Punching Shear Check:

(a) Below Column:



critical section

Critical perimeter, $b_o = 2\times(0.5+d+0.4+d)$

$= 1.8+4d$

Now,

$$\boxed{\frac{\Sigma V_u}{b_o d} = \frac{1}{3}\phi\sqrt{f_c'}} \longrightarrow \text{in MPa (USD)}$$

WSD

$$\boxed{\frac{V}{b_o d} = \frac{1}{6}\sqrt{f_c'}}$$

$$\Rightarrow \frac{600\times10^{-3}}{(1.8+4d)\,d} = \frac{1}{3}\times0.75\times\sqrt{20}$$

$$\Rightarrow 4d^2 + 1.8d - 0.537 = 0$$

$\therefore d = 0.205$ m

⑪ Beam shear check: (Maximum load strip)

in X-X direction,

$(1,2,3) = (103.33 + 86.665 + 70) = 259.995 \simeq 260$ KN

$(4,5,6) = (83.335 + 66.67 + 50) = 200$ KN

$(7,8,9) = (63.34 + 46.675 + 30) = 140.015$ KN

in Y-Y direction,

$(1,4,7) = (103.33 + 83.335 + 63.34) = 250$ KN

$(2,5,8) = (86.665 + 66.67 + 46.675) = 200$ KN

$(3,6,9) = (70 + 50 + 30) = 150$ KN

∴ Maximum strip load, $V_{u(max)} = 260$ KN

Now,

$$\frac{V_u}{bd} = \frac{2}{11} \phi \sqrt{f_c'}$$

in WSD

$$\boxed{\frac{V}{bd} = \frac{1}{11}\sqrt{f_c'}}$$

$$\Rightarrow \frac{260 \times 10^{-3}}{3.5 \times d} = \frac{2}{11} \times 0.75 \times \sqrt{20}$$

$\therefore d = 0.122$ m

Hence, Effective depth, $d_{eff} = 0.205$ m $\simeq 205$ mm

∴ Total thickness of pile cap, $t = (205 + 150 + 50 + \frac{25}{2})$ mm $= 417.5$ mm

Reinforcement Calculation:

$M_{u(x-x)} = 260 \times (1.5 - \frac{0.4}{2}) = 338$ KN-m

$M_{u(y-y)} = 250 \times (1.5 - \frac{0.5}{2}) = 312.5$ KN-m in x-x direction:

factor: $R_n = \frac{M_{u(x-x)}}{0.9 \times bd^2} = \frac{338}{0.9 \times 3.5 \times (0.205)^2} = 2553.28 \ KN/m^2$

$m = \frac{f_y}{0.85 f_c'} = \frac{380}{0.85 \times 20} = 22.35$

Reinforcement Ratio: $\rho_w = \frac{1}{m}\left[1 - \sqrt{1 - \frac{2m R_n}{f_y}}\right]$

$= \frac{1}{22.35} \times \left[1 - \sqrt{1 - \frac{2 \times 22.35 \times 2553.28}{380 \times 1000}}\right]$

$= 0.00732$

Minimum reinforcement check: $\rho_{min} = \frac{4}{3}\rho_w = \frac{4}{3} \times 0.00732$

$\therefore \rho_{min} = 0.00976 > \rho_{min} = 0.002$

Thus,

$A_{s(x-x)} = \rho bd = (0.00976 \times 3.5 \times 0.205) = 0.0070028 \ m^2$

$= 7002.8 \ mm^2$

Provide 15 # 25 mm bar

in Y-Y Direction:

$R_n = \frac{M_{u(y-y)}}{0.9 \times bd^2} = \frac{312.5}{0.9 \times 3.5 \times (0.205)^2} = 2360.65 \ KN/m^2$

$m = 22.35$

$\therefore \rho_w = \frac{1}{22.35} \times \left[1 - \sqrt{1 - \frac{2 \times 22.35 \times 2360.65}{380 \times 1000}}\right] = 0.00672$

$\rho_{min} = \frac{4}{3} \times 0.00672 = 0.00896 > \rho_{min} = 0.002$

Thus,

$A_{s(y-y)} = (0.00896 \times 3.5 \times 0.205) = 0.0064288 \ m^2$

$= 6428.8 \ mm^2$

provide 14 # 25 mm bar

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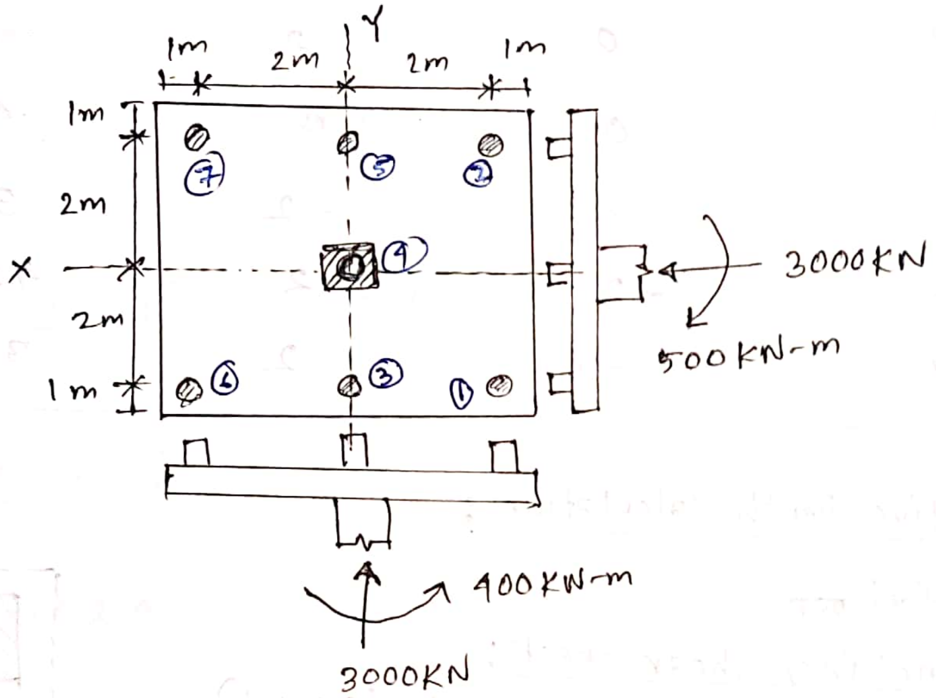
2016

A pile cap is shown in figure below. Design the pile cap with details reinforcement using $f_c' = 25\ N/mm^2$, $f_y = 400\ N/mm^2$, column size $= 0.50 \times 0.50$ m, pile dia. $= 500$ mm.



Solution: Load carried by each Pile:

$$P_n = \frac{\Sigma V}{n} \pm \frac{M_y x}{\Sigma x^2} \pm \frac{M_x y}{\Sigma y^2}$$

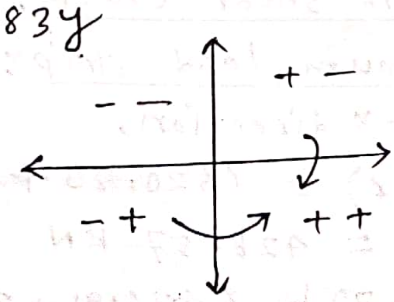
Here, $\Sigma V = 3000$ KN, $n = 7$

$M_y = 400$ KN-m, $M_x = 500$ KN-m

$\Sigma x^2 = 4 \times 2^2 = 16\ m^2$, $\Sigma y^2 = 6 \times 2^2 = 24\ m^2$

$$\therefore P_n = \frac{3000}{7} \pm \frac{400}{16} \times x \pm \frac{500}{24} y$$

$$= 428.57 \pm 25x \pm 20.83y$$



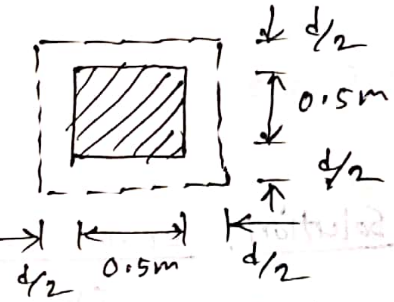
Pile No	x (m)	y (m)	Pn (KN)
1	2	2	520.23
2	2	-2	436.91
3	0	2	470.23
4	0	0	428.57
5	0	-2	386.91
6	-2	2	420.23
7	-2	-2	336.91

Effective Depth Calculation:

~~critical~~ per

(I) Punching shear check:

critical perimeter, $b_o = 4 \times (0.5 + d)$



critical section

Now, $\dfrac{\Sigma V}{b_o d} = \dfrac{1}{6}\sqrt{f_c'}$ (in MPa)

$$\Rightarrow \frac{3000 \times 10^{-3}}{4 \times (0.5+d) \times d} = \frac{1}{6} \times \sqrt{25}$$

$$\Rightarrow d^2 + 0.5d - 0.9 = 0$$

$\therefore d = 0.731$ m $\simeq 0.735$ m

(II) Beam shear check:

Maximum load strip:

in x-x direction,

(1,3,6) = (520.23 + 470.23 + 420.23) = 1410.69 KN.

(4) = 428.57 KN

(2,5,7) = (436.91 + 386.91 + 336.91) = 1159.92 KN in Y–Y direction,

$(1,2) = (520.23 + 436.91) = 957.14$ KN

$(3,4,5) = (470.23 + 428.57 + 386.91) = 1285.71$ KN (c.g)

$(6,7) = (420.23 + 336.91) = 757.14$ KN

$\therefore$ Maximum load in a strip $= 1410.69$ KN

Now,

$$\frac{V}{bd} = \frac{1}{11}\sqrt{f_c'} \quad \text{(in MPa)}$$

$$\Rightarrow \frac{1410.69 \times 10^{-3}}{6 \times d} = \frac{1}{11} \times \sqrt{25}$$

$$\Rightarrow d = 0.52 \text{ m}$$

Hence, Effective depth, $d_{eff} = 0.735$ m $= 735$ mm

$\therefore$ Total thickness, $t = (735 + 150 + 50 + \frac{25}{2})$ mm

$= 947.5$ mm

<u>Reinforcement calculation:</u>

$$n = \frac{E_s}{E_c} = \frac{2 \times 10^5}{4700\sqrt{25}} = 8.51 \simeq 8$$

$$r = \frac{f_s}{f_c} = \frac{0.4 \times 400}{0.45 \times 25} = 14.22$$

$$\therefore K = \frac{n}{n+r} = \frac{8}{8 + 14.22} = 0.36$$

$$j = 1 - \frac{K}{3} = 0.88$$

$$M_{x-x} = 1410.69 \times (2 - \frac{0.5}{2}) = 2468.71 \text{ KN-m}$$

$$M_{y-y} = 957.14 \times (2 - \frac{0.5}{2}) = 1675 \text{ KN-m}$$

$$\therefore As_{x-x} = \frac{2468.71}{(0.4 \times 400 \times 1000) \times 0.88 \times 0.735} = 0.024\ m^2$$

$$= 24000\ mm^2$$

provide 49 # 25 mm bar.

$$As_{y.y} = \frac{1675}{(0.4 \times 400 \times 1000) \times 0.88 \times 0.735} = 0.0162\ m^2$$

$$= 16200\ mm^2$$

provide 34 # 25 mm bar

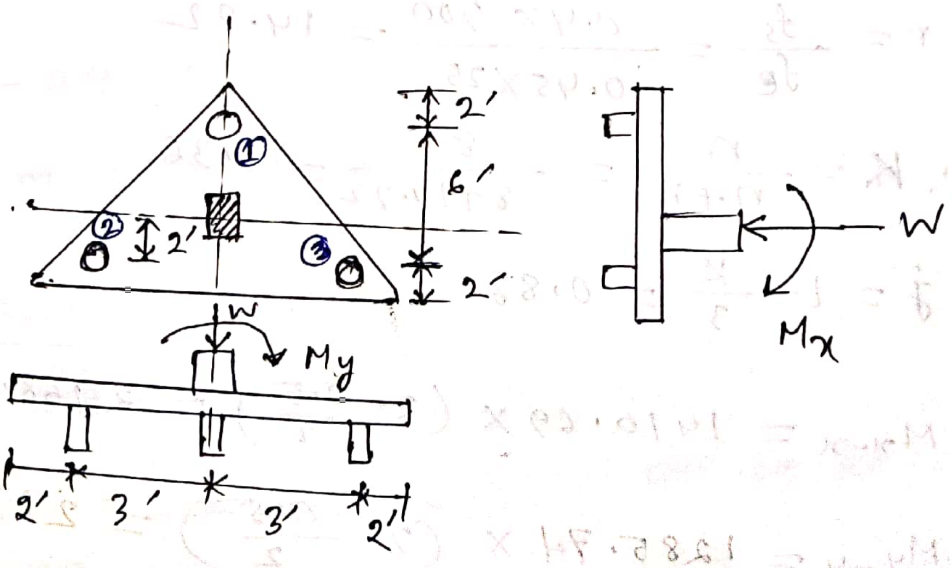
Fig. Reinforcement Details (same before)

2013

A pile cap is subjected to axial load and moment as shown in figure below. Design the pile cap using $f_c' = 3000$ psi, $f_y = 60000$ psi, column size = 15 in x 30 in, Dia of pile = 18 in.

Note that, column load, $W = 240$ Kip.

$M_y = 100$ K-ft, $M_x = 80$ K-ft



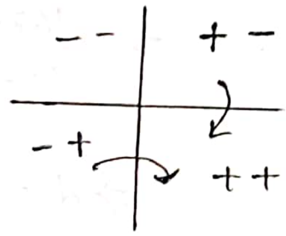
Solution: <u>Vertical load carried by each pile:</u>

$$P_n = \frac{\Sigma V}{n} \pm \frac{M_y x}{\Sigma x^2} \pm \frac{M_x y}{\Sigma y^2}$$

$\Sigma V = 240$ kip, $n = 3$, $M_y = 100$ K', $M_x = 80$ K'

$\Sigma x^2 = 2 \times 3^2 = 18$ ft^2, $\Sigma y^2 = 2 \times 2^2 + 1 \times 4^2 = 24$ ft^2

$$\therefore P_n = \frac{240}{3} \pm \frac{100}{18} x \pm \frac{80}{24} y$$



$$= 80 \pm 5.56\, x \pm 3.33\, y$$

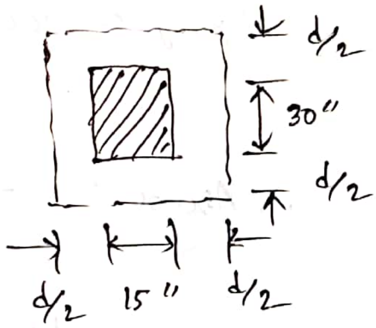
Pile no.	(x) (ft)	(y) (ft)	P_n (K-ft)
1	0	-4	66.68
2	-3	2	66.96
3	3	2	100.32

<u>Effective depth calculation:</u>

① <u>punching shear check:</u>

critical perimeter, $b_o = 2 \times (15 + d + 30 + d)$

$$= 4d + 90$$



Now,

$$\frac{\Sigma V}{b_o d} = 2\sqrt{f_c'}$$

$$\Rightarrow \frac{240 \times 1000}{(4d + 90) \times d} = 2 \times \sqrt{3000}$$

$$\Rightarrow 4d^2 + 90d - 2190.89 = 0$$

$\therefore d = 14.717$ in. $\simeq 14.75$ in.

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(11) Beam shear check: (2, 3).

Maximum load on a strip, $V_{max} = (100.32 + 66.96)$
$= 167.28\,K$

Hence,

$$\frac{V}{bd} = 1.1\sqrt{f_c'}$$

$$\Rightarrow \frac{167.28 \times 1000}{(10 \times 12) \times d} = 1.1\sqrt{3000}$$

$\Rightarrow d = 23.14$ in $\simeq 24$ in.

Thus, effective depth, $d_{eff} = 24$ in

$\therefore$ Total thickness of pile cap, $t = (24 + 6 + 2 + \frac{5}{2 \times 8})$
$= 32.3125$ in.

$$M_{x-x} = 167.28 \times (2 - \frac{30}{2 \times 12}) = 125.46\ K\,ft$$

$$M_{y-y} = 100.32 \times (3 - \frac{15}{2 \times 12}) = 238.26\ K\text{-}ft$$

$$\therefore As_{x-x} = \frac{125.46 \times 12}{0.4 \times 60 \times 0.888 \times 24} = 2.94\ in^2$$

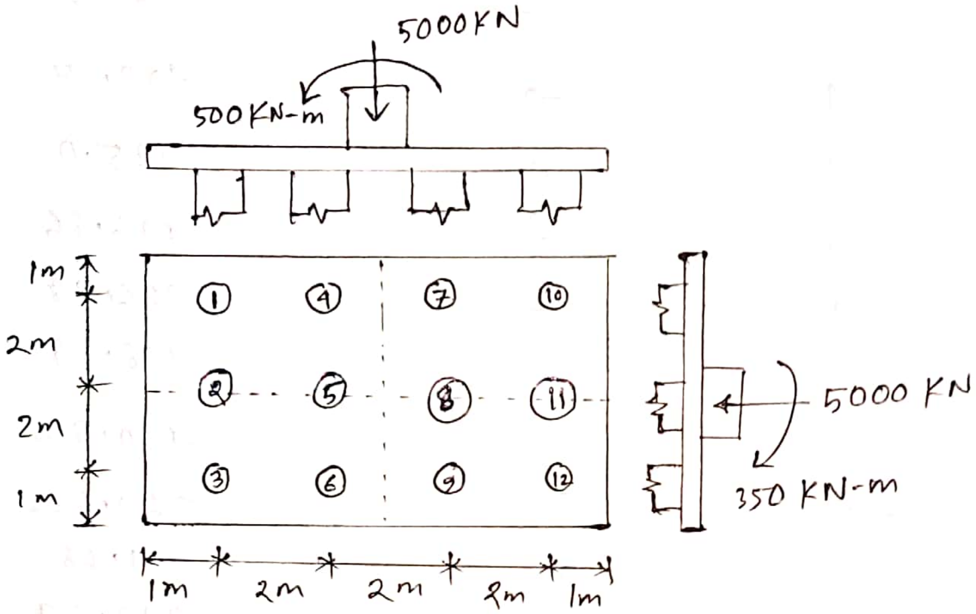
provide 10 # 5 bar

$$As_{y-y} = \frac{238.26 \times 12}{0.4 \times 60 \times 0.888 \times 24} = 5.6\ in^2$$

provide 13 # 6 bar

2012, 2011

A pile cap is subjected to axial load and moments as shown in figure below. Design the pile cap using $f_c' = 18$ MPa and $f_y = 400$ MPa. The size of column is 30 cm × 40 cm and diameter of each pile is 40 cm. Use WSD Method.



Solution:

vertical load caried by each pile:

$$P_n = \frac{\Sigma V}{n} \pm \frac{M_y \cdot x}{\Sigma x^2} \pm \frac{M_x \cdot y}{\Sigma y^2}$$

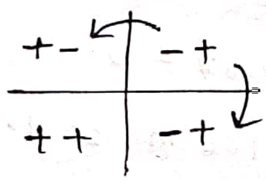
Here, $\Sigma V = 5000$ KN, $n = 12$, $M_y = 500$ KN-m, $M_x = 350$ KN-m

$$\Sigma x^2 = 6 \times (1)^2 + 6 \times (3)^2 = 60 \text{ m}^2$$

$$\Sigma y^2 = 8 \times 2^2 = 32 \text{ m}^2$$

Thus, $$P_n = \frac{5000}{12} \pm \frac{500 \times x}{60} \pm \frac{350 \times y}{32}$$

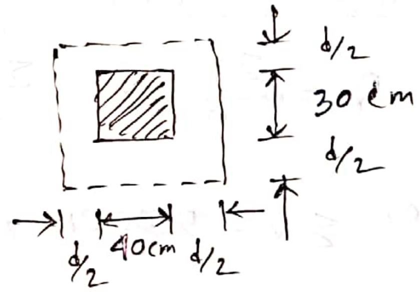
$$= 416.67 \pm 8.33\,x \pm 10.9375\,y$$



Pile No	x (m)	y (m)	Pn (KN)
1	3	-2	419.8
2	3	0	441.66
3	3	2	463.52
4	1	-2	403.14
5	1	0	425.0
6	1	2	446.86
7	-1	-2	386.48
8	-1	0	408.34
9	-1	2	430.2
10	-3	-2	369.82
11	-3	0	391.68
12	-3	2	413.54

Effective Depth Calculation:

① Punching Shear check:



∴ Critical perimeter,

$$b_o = 2\times(0.3+d) + 2\times(.4+d)$$

$$= 1.4 + 4d$$

Now, $\frac{\Sigma V}{b_o d} = \frac{1}{6}\sqrt{f_c'}$

$$\Rightarrow \frac{5000\times 10^{-3}}{(1.4+4d)\times d} = \frac{1}{6}\times\sqrt{18}$$

$$\Rightarrow 1.4d + 4d^2 = 7.07$$

$$\Rightarrow 4d^2 + 1.4d - 7.07 = 0$$

$$\therefore d = 1.166\ m \simeq 1.2\ m.$$

(ii) Beam shear check:

in x-x direction:

$(1,4,7,10) = (419.8 + 403.14 + 386.48 + 369.82) = 1579.24$ KN

$(2,5,8,11) = (441.66 + 425 + 408.34 + 391.68) = 1666.68$ KN

$(3,6,9,12) = (463.52 + 446.86 + 430.2 + 413.54) = 1754.12$ KN

in y-y direction:

$(1,2,3) = (419.8 + 441.66 + 463.52) = 1324.98$ KN

$(4,5,6) = (403.14 + 425 + 446.86) = 1275$ KN

$(7,8,9) = (386.48 + 408.34 + 430.2) = 1225.02$ KN

$(10,11,12) = (369.82 + 391.68 + 413.54) = 1175.04$ KN

∴ Maximum load on a strip = 1754.12 KN

Now,

$$\frac{V}{bd} = \frac{1}{11}\sqrt{f_c'}$$

$$\Rightarrow \frac{1754.12 \times 10^{-3}}{8 \times d} = \frac{1}{11}\sqrt{18}$$

$\Rightarrow d = 0.91$ m

Thus, effective depth, $d_{eff} = 1.2$ m

∴ total thickness, $t = (1.2 + 0.15 + 0.05 + 0.025) = 1.425$ m

Reinforcement calculation:

$$n = \frac{E_s}{E_c} = \frac{2\times10^5}{4700\sqrt{f_c'}} = \frac{2\times10^5}{4700\sqrt{18}} = 10.03 \simeq 10$$

$$r = \frac{f_s}{f_c} = \frac{0.4\times400}{.45\times18} = 19.753$$

$$\therefore k = \frac{n}{n+r} = \frac{10}{10+19.753} = 0.336$$

$$\therefore j = 1 - \frac{k}{3} = \left(1 - \frac{0.336}{3}\right) = 0.888$$

Now, Moment along X axis,

$$M_{x-x} = 1754.12 \times \left(2 - \frac{.40}{2}\right) = 3157.416 \text{ KN-m}$$

Moment along Y axis,

$$M_{y-y} = 1324.98 \times \left(3 - \frac{0.30}{2}\right) = 3776.2 \text{ KN-m}$$

$$\therefore A_{x-x} = \frac{M_{x-x}}{f_s j d} = \frac{3157.416}{0.4\times400\times10^3\times.888\times1.35} = 0.0165 m^2$$

$$= 16500 \text{ mm}^2$$

provide 34 # 25 mm bar

$$A_{y-y} = \frac{M_{y-y}}{f_s j d} = \frac{3776.2}{0.4\times400\times1000\times.888\times1.35} = 0.020 m^2$$

$$= 20000 \text{ mm}^2$$

Provide 41 # 25 mm bar.

Drilled Shaft

07, 05, 04, 03, 02

What is Drilled Shaft?

The term drilled shaft is used to refer a hole drilled or excavated to the bottom of a structure's foundation and then filled with concrete. It is cast-in-place pile generally having a diameter of about 2.5 ft or more, with or without steel reinforcement and with or without an enlarged bottom.

Write down the advantages of Drilled Shaft. 07, 05, 03, 02, 04

The use of drilled shaft foundations has several advantages:

1. A single drilled shaft may be used instead of a group piles and the pile cap.
2. constructing drilled shafts in despites of dense sand and gravel is easier than driving piles.
3. Drilled Shaft may be constructed before grading operation are completed.
4. The ground vibration produced during driving of piles by hammer may cause damage to near by structures, which can be avoided by the use of drilled shaft.
5. Piles driven into clay soils may produce ground heaving and cause previously driven piles to move laterally. This does not occur during the construction of drilled shaft.
6. There is no hammer noise during construction of drilled shaft.
7. Because the base of a drilled shaft can be enlarged, it provides more resistance.

8. Drilled shaft have high resistance to lateral loads.

9. The surface over which the base of drilled shaft is constructed can be visually inspected.

07, 05, 04, 03, 02

What are the short comings / disadvantages of drilled shaft?

There are also a couple of drawbacks to the use of drilled shaft construction:

(1) The concreting operation may be delayed by bad weather and always need close supervision.

(2) Deep excavations for drilled shafts may induce substantial ground loss and damage to nearby structures.

(3) Strict supervison of drilling operation is required in drilled piers.

(4) Load tests in the case of drilled piers are difficult.

How can you determine load bearing capacity of drilled shaft in granular soil? Discuss. 2015, 09, 07, 06

Estimation of Q_p: For a drilled shaft with its base located on granular soil ($c' = 0$), The net ultimate load bearing capacity at the base can be obtained as,

$$Q_{p(net)} = A_p \left[q' (N_q - 1) F_{qs} F_{qd} F_{qc} \right]$$

The magnitude of $Q_{p(net)}$ also can be reasonably estimated from a relationship based on the analysis of Berezantzev et. al (1961), That can be expressed as,

$$Q_{p(net)} = A_p q'(\omega N_q^* - 1) \quad \text{where, } N_q^* = 0.21 e^{0.17\phi'}$$

bearing capacity factor.

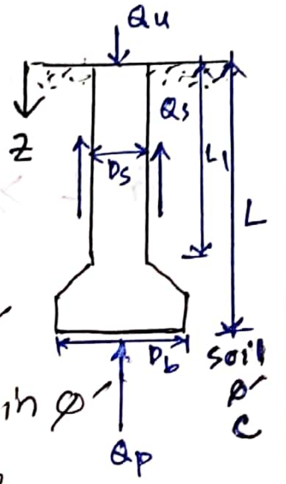
$$\omega = f \times \left(\frac{L}{D_b}\right)$$

correction factor.

Estimation of Q_s:

The frictional resistance at ultimate load, Q_s, developed in a drilled shaft may be calculated as,

$$Q_s = \int_0^{L_1} p f \, dz$$



where, p = shaft meter $= \pi D_s$

f = unit frictional (or skin) resistance $= K \sigma_o' \tan\delta'$

K = earth pressure co-efficient $\approx K_o = 1 - \sin\phi'$

σ_o' = effective vertical stress at any depth z.

Thus,

$$Q_s = \int_0^{L_1} p f \, dz = \pi D_s (1 - \sin\phi') \int_0^{L_1} \sigma_o' \tan\delta' \, dz$$

The value of σ_o' will increase to a depth about $15 D_s$ and will remain constant there fter.

Allowable net Load, $Q_{all(net)}$:

As appropriate factor of safety should be applied to the ultimate load to obtain the net allowable load,

$$Q_{all(net)} = \frac{Q_{p(net)} + Q_s}{FS}$$

How can you determine the load bearing capacity of drilled shaft in clay? Discuss. 2013, 09, 07, 06

<u>Estimation of Q_p:</u> For saturated clays with $\phi = 0$, the bearing capacity factor N_q is equal to unity, the net ultimate bearing capacity at the base can be obtained as,

$$Q_{p(net)} \simeq A_p \, c_u \, N_c \, F_{cs} \, F_{cd} \, F_{ce}$$

where, c_u = undrained cohesion

For $L \geq 3D_b$ we can Re-write the equation,

$$Q_{p(net)} = A_p \, c_u \, N_c^*$$

where, $N_c^* = N_c \, F_{cs} \, F_{cd} \, F_{ce}$

$$= 1.33\left[(\ln I_r) + 1\right]$$

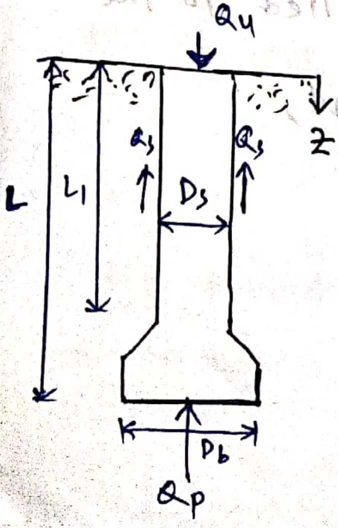
I_r = soil rigidity index

$$= \frac{E_s}{3c_u}$$

For $\frac{L}{D_b} < 3$ (O'Neill and Reese, 1999),

$$Q_{p(net)} = A_p\left\{\frac{2}{3}\left[1 + \frac{1}{6}\left(\frac{L}{D_b}\right)\right]\right\} c_u N_c^*$$

<u>Estimation of Q_s:</u> The expression for the skin resistance of drilled shafts in clay is,



$$Q_s = \sum_{L=0}^{L=L_1} \alpha^* \, c_u \, p \, \Delta L$$

where, p = shaft perimeter

$$= \pi D_s$$

$$\alpha^* = 0.21 + 0.25\left(\frac{p_a}{c_u}\right) \leq 1$$

p_a = atmospheric pressure

$= 100 \text{ kN/m}^2$ ($\simeq 2000$ lb/ft^2)

conservatively, we ~~a~~ assume that, $\alpha^* = 0.4$

Estimation of Allowable net load, $Q_{all(net)}$:

An appropriate factor of safety should be applied to the ultimate load to obtain the net allowable load,

$$Q_{all(net)} = \frac{Q_{p(net)} + Q_s}{FS}$$

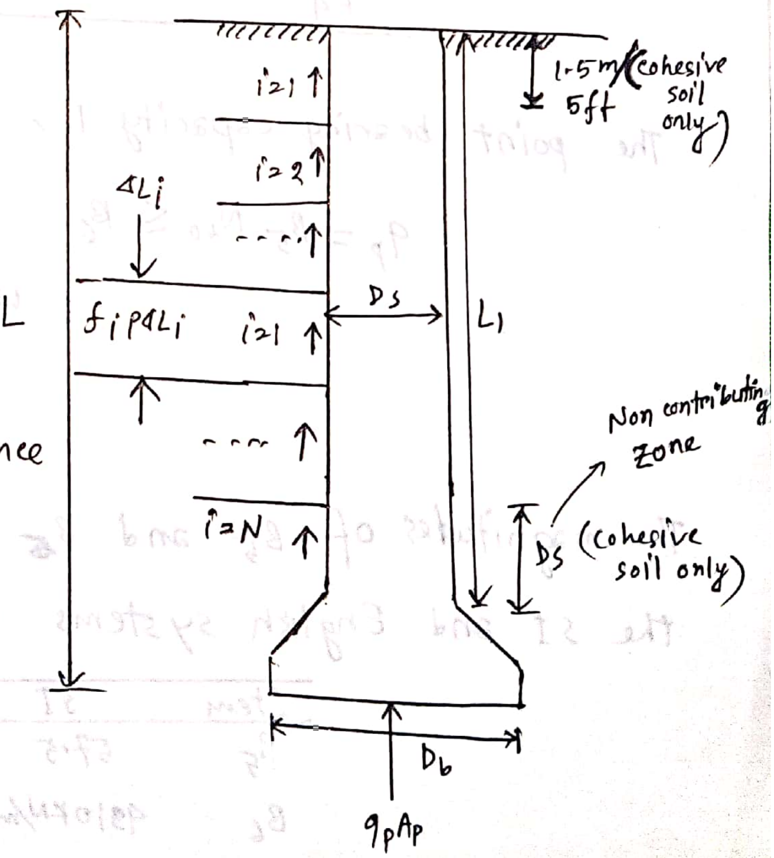
How can you determine the load bearing capacity of drilled shaft on settlement? Discuss. 2017, 14

Reese and O'Neill (1989) proposed a method for calculating the load bearing capacity of drilled shaft that is based on settlement. The method is applicable to the following ranges:

1. Shaft Diameter, D_s = 0.52 to 1.2 m (1.7 to 3.93 ft)
2. Bell depth, L = 4.7 to 30.5 m (15.4 to 100 ft)
3. Field standard penetration resistance, N_{60} = 5 to 60
4. concrete slump = 100 to 225 mm (4 to 9 in.)

Reese and O'Neill's procedure gives,

$$Q_{u(net)} = \sum_{i=1}^{N} f_i p \Delta L_i + q_p A_p$$



where,

f_i = ultimate unit shearing resistance in layer i

p = perimeter of shaft $= \pi D_s$

q_p = unit point resistance

A_p = Area of base $= \frac{\pi}{4} D_b^2$

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in sand: $f_i = \beta_1 \sigma'_{ozi} < \beta_2$ where, σ'_{ozi} = vertical effective stress at the middle of layer i

$$\beta_1 = \beta_3 - \beta_4 z_i^{0.5} \quad (\text{for } 0.25 \leq \beta_1 \leq 1.2)$$

The units for f_i, z_i and σ'_{ozi} and the magnitude of β_2, β_3 and β_4 in the SI and English systems are

Item	SI	English
f_i	KN/m^2	K/ft^2
z_i	m	ft
σ'_{ozi}	KN/m^2	K/ft^2
β_2	$192\ KN/m^2$	$4\ K/ft^2$
β_3	1.5	1.5
β_4	0.244	0.135

The point bearing capacity is,

$$q_p = \beta_5 N_{60} \leq \beta_6 \quad [\text{for } D_b < 1.27\text{ m } (50\text{ in})]$$

where, N_{60} within a distance of $2D_b$ below the base of the drilled shaft

The magnitudes of β_5 and β_6 and the unit of q_p in the SI and English systems are given here,

Item	SI	English
β_5	57.5	1.2
β_6	$4310\ KN/m^2$	$90\ K/ft^2$
q_p	KN/m^2	Kip/ft^2

a_p

If $D_b \geq 1.27\,m$ (50 in.) Excessive settlement may occur, In that case, q_p may be replaced by q_{pr}.

SI Units:
$$q_{pr} = \frac{1.27}{D_b(m)} \times q_p$$

English Units:
$$q_{pr} = \frac{50}{D_b(in.)} \times q_p$$

Rollings et al. (2005) have modified the following equation $(\text{for } 0.25 \leq \beta_1 \leq 1.2)$ as follows: for gravelly sands, $\beta_1 = \beta_3 - \beta_4 z_i^{0.5}$

* For sand with 25 to 50% gravel,

$$\beta_1 = \beta_7 - \beta_8 z_i^{0.75} \quad (\text{for } 0.25 \leq \beta_1 \leq 1.8)$$

* For sand with more than 50% gravel,

$$\beta_i = \beta_9 e^{-\beta_{10} z_i} \quad (\text{for } 0.25 \leq \beta_1 \leq 3.0)$$

The magnitude of β_7, β_8, β_9 and β_{10} and the unit of z_i in the SI and English systems are given here,

Item	SI	English
β_7	2.0	2.0
β_8	0.15	0.062
β_9	3.4	3.4
β_{10}	-0.085	-0.026
z_i	m	ft

In Clay: The unit skin resistance can be given as,

$$f_i = \alpha_i^* \, c_{u(i)}$$

The following values are recommended for α_i^*:

$\alpha_i^* = 0$ for the top 1.5 m (5 ft) and bottom 1 Diameter, D_s, of the drilled shaft

$\alpha_i^* = 0.55$ elsewhere

The expression for q_p (point load per unit area) can be given as:

$$q_p = 6\,c_{ub}\left(1 + 0.2\frac{L}{D_b}\right) \le 9\,c_{ub} \le 40\,P_a$$

Where, c_{ub} = Average undrained cohesion within a vertical distance of $2D_b$ below the base.

If D_b is large, Excessive settlement will occur at the ultimate load per unit area, q_p. Thus For $D_b > 1.91$ m (75 in), q_p is replaced by,

$$q_{pr} = F_r\, q_p$$ where,

$$F_r = \frac{2.5}{\psi_1 D_b + \psi_2} \le 1$$

Relationships for ψ_1 and ψ_2:

Item	SI	English
ψ_1	$\psi_1 = 2.78\times10^{-4} + 8.26\times10^{-5}\left(\frac{L}{D_b}\right) \le 5.9\times10^{-4}$	$\psi_1 = 0.0071 + 0.0021\left(\frac{L}{D_b}\right) \le 0.015$
ψ_2	$\psi_2 = 0.065\,[c_{ub}\,(kN/m^2)]^{0.5}$ $(0.5 \le \psi_2 \le 1.5)$	$\psi_2 = 0.45\,[c_{ub}\,(k/ft^2)]^{0.5}$ $(0.5 \le \psi_2 \le 1.5)$
D_b	mm	in.

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What is the basic difference between drilled shaft and caisson?

2017, 16

Drilled Shaft	Caisson
(i) Drilled shaft is a type of deep foundation, which consists of a cylindrical column of large diameter to support and transfer large super-imposed loads to firm strata below.	(i) Caisson are watertight structures made up of wood, steel or reinforced concrete built above the ground level and then sunken into the ground.
(ii) The types of Drilled shaft are straight shaft and Belled shaft.	(ii) The types of caissons are box, open, pneumatic, floating etc.
(iii) Drilled shaft is inserted down to the bedrock.	(iii) Caisson is putting a box into under water and pouring it with concrete.
(iv) Drilled shaft has a footing	(iv) Caisson does not have a footing.

What are the conditions in which a drilled shaft is more suitable than a caisson? 2017, 16

Drilled Shaft

Estimation of Load-Bearing Capacity:

The ultimate load bearing capacity of a drilled shaft,

$$Q_u = Q_p + Q_s$$

where, Q_u = ultimate load

Q_p = Ultimate load at the base

Q_s = frictional (skin) resistance

The ultimate base load,

$$Q_p = A_p \left(c' N_c F_{cs} F_{cd} F_{cc} + q' N_q F_s F_{qd} F_{qc} \right) \text{ (Neglecting } N_\gamma\text{)}$$

where, c' = cohesion

N_c, N_q, N_γ = bearing capacity factor

$F_{cs}, F_{qs}, F_{\gamma s}$ = shape factors

$F_{cd}, F_{qd}, F_{\gamma d}$ = depth factors.

$F_{cc}, F_{qc}, F_{\gamma c}$ = compressibility factors.

$F_{cc}, F_{qc}, F_{\gamma c}$ = effective unit weight of soil at the base of the shaft

γ' = effective unit weight of soil at the base of the shaft

q' = effective vertical stress at the base of the shaft

A_p = Area of the base $= \frac{\pi}{4} D_b^2$

Frictional Resistance of drilled shaft,

$$Q_s = \Sigma p \cdot \Delta L \cdot f$$

where, p = perimeter of shaft section

ΔL = increamental shaft length over which 'p' and 'f' taken to be constant

f = unit friction resistance at any depth

Drilled shaft in Granular Soil: ($c' = 0$)

- Estimation of Q_p: For drilled shaft on a granular soil,

$$\text{(1)}\quad Q_{p(net)} = A_p\left[q'(N_q - 1)\, F_{qs}\, F_{qd}\, F_{qc}\right]$$

where, $F_{qs} = 1 + \tan\phi'$

$$F_{qd} = 1 + C\,\underbrace{\tan^{-1}\left(\frac{L}{D_b}\right)}_{\text{radian}}$$

and, $N_q = \tan^2\left(45 + \frac{\phi'}{2}\right)e^{\pi\tan\phi'}$

Here, $c = 2\tan\phi'(1 - \sin\phi')^2$

- According to Chen and Kulhawy (1994), $\boxed{F_{qc}}$ can be calculated in the following manner:

Step 1: calculate the critical rigidity index as,

$$I_{cr} = 0.5\exp\left[2.85\cot\left(45 - \frac{\phi'}{2}\right)\right]$$

Step 2: Calculate the reduced rigidity index as,

$$I_{rr} = \frac{I_r}{1 + I_r\Delta}$$

where,

I_r = soil rigidity index

$$= \frac{E_s}{2(1 + \mu_s)\,q'\tan\phi'}$$

And,

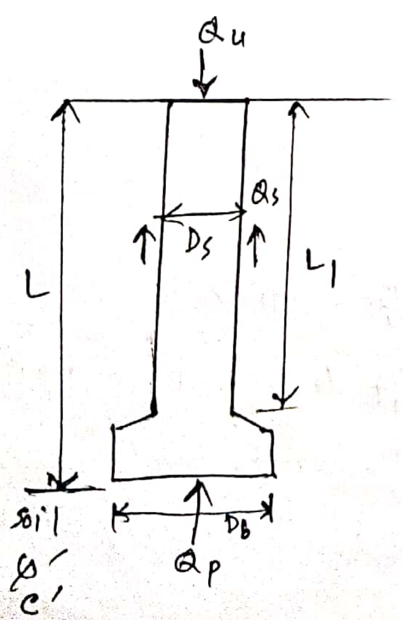
$$\Delta = n\frac{q'}{P_a}$$

$$n = 0.005\left(1 - \frac{\phi' - 25}{20}\right)$$

in which, E_s = drained modulus of elasticity $= m P_a$

P_a = atmospheric pressure ($\simeq 100\ kN/m^2$ or $2000\ lb/ft^2$)

$$m = \begin{cases} 100 \text{ to } 200 \text{ (loose soil)} \\ 200 \text{ to } 500 \text{ (medium dense soil)} \\ 500 \text{ to } 1000 \text{ (dense soil)} \end{cases}$$



μ_s = poisson's ratio of soil

$$= \left[0.1 + 0.3\left(\frac{\phi' - 25}{20}\right)\right]$$

(for $25° \le \phi' \le 45°$)

Step 3: If $I_{rr} \geq I_{cr}$ then $F_{qc} = 1$

However, if $I_{rr} < I_{cr}$, then

$$F_{qc} = \exp\left\{ (-3.8 \tan \phi') + \left[\frac{(3.07 \sin \phi')(\log_{10} 2 I_{rr})}{1 + \sin \phi'} \right] \right\}$$

② The magnitude of $Q_{p(net)}$ also can be resonably estimated from a relationship based on the analysis of Berezantzev et al. (1961) that can be expressed as –

$$Q_{p(net)} = A_p q' (\omega N_q^* - 1)$$

where, N_q^* = bearing capacity factor $= 0.21 e^{0.17\phi'}$

ω = correction factor $= f(L/D_b)$ (Table – 10.3)

- Estimation of Q_s:

The frictional resistance at ultimate load, Q_s, developed in a drilled shaft may be calculated as –

$$Q_s = \int_0^{L_1} p f \, dz$$

where,

p = shaft perimeter $= \pi D_s$

f = unit frictional (or skin) resistance $= K \sigma_o' \tan \delta'$

K = earth pressure co-efficient $\simeq K_o = 1 - \sin \phi'$

σ_o' = effective vertical stress at any depth z

Thus,

$$Q_s = \int_0^{L_1} p f \, dz = \pi D_s (1 - \sin \phi') \int_0^{L_1} \sigma_o' \tan \delta' \, dz$$

* the value of σ_o' will increase to a depth about $15 D_s$ and will remain constant

* $\frac{\delta'}{\phi'} = 1$ (for cast-in-concrete & good construction)

- **Allowable Net Load, $Q_{all(Net)}$**

The net allowable load,

$$Q_{all(net)} = \frac{Q_{p(net)} + Q_s}{FS}$$

Load-Bearing Capacity Based on Settlement: (Sandy soil)

On the basis of a database of 41 loading tests, Reese and O'Neil (1989) proposed a method for calculating the load-bearing capacity of drilled shaft that is based on settlement. The method is applicable to the following ranges:

1. shaft diameter; $D_s = 0.52$ to 1.2 m (1.7 to 3.93 ft)
2. Bell depth: $L = 4.7$ to 30.5 m (15.4 to 100 ft)
3. Field Standard penetration resistance: $N_{60} = 5$ to 60
4. Concrete slump $= 100$ to 225 mm (4 to 9 in)

Reese and O'Neill's procedure,

$$Q_{u(net)} = \sum_{i=1}^{N} f_i \, p \, \Delta L_i + q_p A_p$$

where, f_i = ultimate unit shearing resistance in layer i

p = perimeter of the shaft $= \pi D_s$

q_p = unit point resistance

A_p = area of the base $= \left(\frac{\pi}{4} D_b^2\right)$

Here, $f_i = \beta_1 \sigma'_{ozi} < \beta_2$ where, σ'_{ozi} = vertical effective stress at the middle of layer i

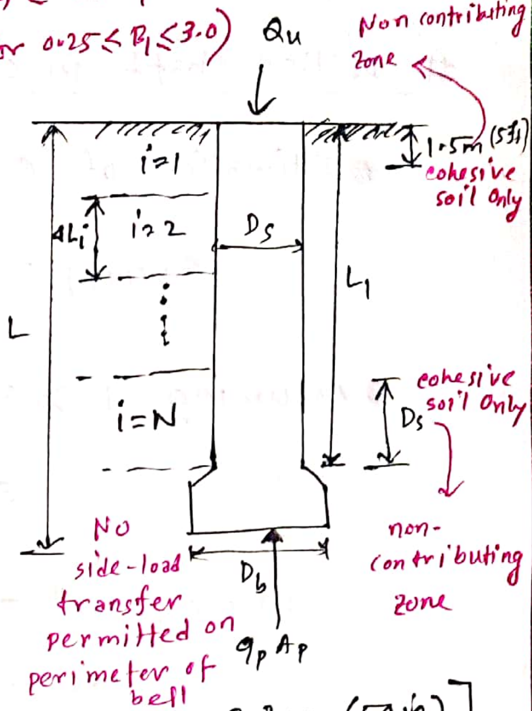
Rollins et. al. (2005) $\beta_1 = \beta_3 - \beta_4 z_i^{0.5}$ (for $0.25 \le \beta_1 \le 1.2$)

* for sand with 25 to 50% Gravel, $\beta_1 = \beta_7 - \beta_8 z_i^{0.75}$ (for $0.25 \le \beta_1 \le 1.8$)

* For sand with more than 50% Gravel, $\beta_1 = \beta_9 e^{-\beta_{10} z_i}$

(for $0.25 \le \beta_1 \le 3.0$)

Item	SI	English
f_i	KN/m^2	Kip/ft^2
z_i	m	ft
σ'_{ozi}	KN/m^2	Kip/ft^2
β_2	$192\ KN/m^2$	$4\ Kip/ft^2$
β_3	1.5	1.5
β_4	0.244	0.135



The point bearing capacity,

$$q_p = \beta_5 N_{60} \le \beta_6 \quad [\text{for } D_b < 1.27\text{m } (50\text{in})]$$

where, N_{60} = field penetration number within a distance of $2D_b$ below the base of the drilled shaft

Item	SI	English
β_5	57.5	1.2
β_6	$4310\ KN/m^2$	$90\ kip/ft^2$
q_p	KN/m^2	kip/ft^2

Item	SI	English
β_7	2.0	2.0
β_8	0.15	0.062
β_9	3.4	3.4
β_{10}	-0.085	-0.026

if $D_b \ge 1.27\text{m}$ (50 in), excessive settlement may occur.

In that case, q_p may be replaced by q_{pr}

SI Units: $$q_{pr} = \frac{1.27}{D_b(m)} \times q_p$$

English Units: $$q_{pr} = \frac{50}{D_b(in)} \times q_p$$

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Drilled shaft in clay : ($\phi = 0$)

- **Estimation of Q_p:** For $\phi = 0$, $N_q = 1$. Thus,

$$Q_{p(net)} = A_p c_u N_c F_{cs} F_{cd} F_{cc}$$ where, c_u = undrained cohesion

- Assuming $L \geq 3D_b$, we can rewrite the equation;

$$Q_{p(net)} = A_p c_u N_c^*$$

where, $N_c^* = N_c F_{cs} F_{cd} F_{cc} = 1.33[(\ln I_r)+1]$ (for $\frac{L}{D_b} > 3$)

Here, soil rigidity index, $I_r = \frac{E_s}{3c_u}$

c_u/p_a	$E_s/3c_u$	N_c^*
0.25	50	6.5
0.5	150	8.0
≥ 1.0	250–300	9.0

- For $L/D_b < 3$ (O'Neil and Reese, 1999)

$$Q_{p(net)} = A_p \left\{ \frac{2}{3}\left[1 + \frac{1}{6}\left(\frac{L}{D_b}\right)\right] \right\} c_u N_c^*$$ where, $N_c^* = 9$

- **Estimation of Q_s:** Kulhawy and Jackson (1989)

$$Q_s = \sum_{L=0}^{L=L_1} \alpha^* c_u p \Delta L$$ where $\alpha^* = 0.21 + 0.25\left(\frac{p_a}{c_u}\right) \leq 1$

so, conservatively we can assume, $\alpha^* = 0.4$

Load bearing capacity based on settlement: (clay)

The net ultimate load,

$$Q_{ult(net)} = \sum_{i=1}^{n} f_i \, p \, \Delta L_i + q_p A_p$$

where, $f_i = \alpha_i^* \, c_{u(i)}$

Here, $\alpha_i^* = 0$ for top 1.5 m (5 ft) and bottom 1 dia. of Drilled shaft

$\alpha_i^* = 0.55$ elsewhere

$$q_p = 6 \, c_{ub} \left(1 + 0.2 \frac{L}{D_b}\right) \leq 9 c_{ub} \leq 40 P_a$$

where,

c_{ub} = Average undrained cohesion with in a vertical distance of $2D_b$ below the base

P_a = Atmospheric pressure.

if D_b is large, excessive settlement may occur

for $D_b > 1.91\,m$ (75 in), q_p may be replaced by,

$$q_{pr} = F_r \, q_p \quad \text{where,} \quad F_r = \frac{2.5}{\psi_1 D_b + \psi_2} \leq 1$$

item	SI Unit	English unit
ψ_1	$\psi_1 = 2.78 \times 10^{-4} + 8.26 \times 10^{-5} \times \left(\frac{L}{D_b}\right) \leq 5.9 \times 10^{-4}$	$\psi_2 = 0.0071 + 0.0021 \left(\frac{L}{D_b}\right) \leq 0.015$
ψ_2	$\psi_2 = 0.065 \, [c_{ub} \, (kN/m^2)]^{0.5}$	$\psi_2 = 0.45 \, [c_u \, (K/ft^2)]^{0.5}$
D_b	mm	in

Drilled shaft extending into Rock:

- Procedure of Reese and O'Neil (1988, 1989):

step 1: calculate the ultimate side resistance

$$* f\,(\mathrm{lb/in^2}) = 2.5\, q_u^{0.5} \le 0.15\, q_u$$

$$* f\,(\mathrm{KN/m^2}) = 6.564\, q_u^{0.5}\,(\mathrm{KN/m^2}) \le 0.15\, q_u\,(\mathrm{KN/m^2})$$

step 2: calculate the ultimate bearing capacity,

$$Q_u = \pi D_s L f$$

step 3: calculate the settlement, S_e of the shaft at the top of the rock socket,

$$S_e = S_{e(s)} + S_{e(b)}$$

where,

$S_{e(s)}$ = elastic compression of drilled shaft within the socket (assuming no side resistance)

$S_{e(b)}$ = settlement at base

Here

$$S_{e(s)} = \frac{Q_u L}{A_c E_c}$$

and,

$$S_{e(b)} = \frac{Q_u I_f}{D_s E_{mass}}$$

where,

Q_u = ultimate load

A_c = cross sectional area $= \frac{\pi}{4} D_s^2$

E_c = Young's Modulus of concrete and reinforcing steel

E_{mass} = Young's Modulus of rock mass

I_f = elastic influence co-efficient depends on $\frac{E_c}{E_{mass}}$ & $\frac{L}{D_s}$

The magnitude of E_{mass} can be taken as,

$$\frac{E_{mass}}{E_{core}} \simeq 0.0266\,(RDQ) - 1.66$$

where, RDQ = Rock quality disignation in (%)

However, unless the socket is very long,

$$s_e \simeq s_{e(b)} = \frac{Q_u I_f}{D_s E_{mass}}$$

Step 4: If $s_e < 10$ mm ($\simeq 0.4$ in), $Q_u = \pi D_s L f$

If $s_e \geq 10$ mm (0.4 in), then go to step 5

Step 5: If $s_e \geq 10$ mm (0.4 in)

$$Q_u = 3A_p \left[\frac{3 + \frac{c_s}{D_s}}{10 \times \left(1 + 300\frac{\delta}{c_s}\right)^{0.5}} \right] \times q_u$$

* for $c_s > 305$ mm (12 in.) and $\delta < 5$ mm (0.2 in)

where, c_s = spacing of discontinuities

δ = thickness of individual Discontinuity

q_u = unconfined compression strength of the rock beneath the base of the socket

- Procedure of Zhang and Einstein (1998):

$$Q_{u(net)} = Q_p + Q_s = q_p A_p + f p L$$

where, $Q_p\,(MN) = q_p A_p = \left[4.83\left(q_u\ MN/m^2\right)^{0.51}\right] A_p\,(m^2)$

and, $Q_s\,(MN) = f p L = \left[0.4\left(q_u\ MN/m^2\right)^{0.5}\right] \times \left[\pi\ D_s\,(m)\right] \times \left[L\,(m)\right]$

* for smooth socket

$Q_s\,(MN) = f p L = \left[0.8\left(q_u\ MN/m^2\right)^{0.5}\right] \times \left[\pi\ D_s\,(m)\right] \times \left[L \times (m)\right]$

* for rough socket

Settlement of Drilled shaft at working load:

Total settlement of shaft,

$$S_e = S_{e(1)} + S_{e(2)} + S_{e(3)}$$

Here,

$$S_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})}{A_p E_p} \times L \quad ; \quad \xi = (0.5 \text{ to } 0.67)$$

$$S_{e(2)} = \frac{Q_{wp}}{A_p} \times \frac{D_b}{E_s} \times (1 - \mu_s^2) I_{wp} \quad ; \quad I_{wp} = 0.85$$

According to vesic,

$$S_{e(2)} = \frac{Q_{wp} C_p}{D_b q_p} \quad ; \quad q_p = C_{u(b)} N_c^*$$

$$S_{e(3)} = \frac{Q_{ws}}{PL} \times \frac{D_s}{E_s} \times (1 - \mu_s^2) \times I_{ws} \quad ; \quad I_{ws} = 2 + 0.35\sqrt{\frac{L}{D_s}}$$

According to vesic,

$$S_{e(3)} = \frac{Q_{ws} C_s}{L q_p} \quad \text{where,} \quad C_s = \left(0.93 + 0.16\sqrt{\frac{L}{D_s}}\right) C_p$$

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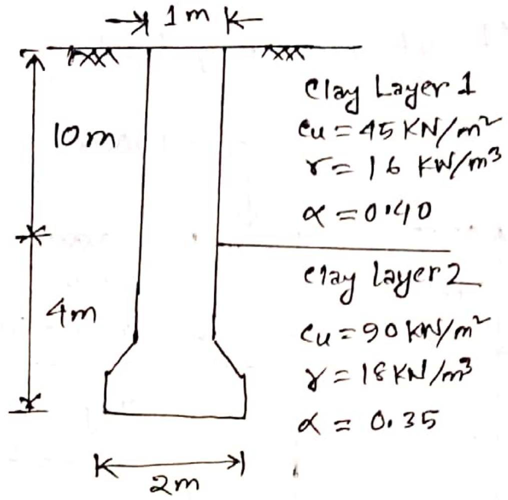
Drilled Shaft

2018 (Example- 10.4)

Determine allowable load for the drilled shaft constructed in a clay deposite as shown in figure below. Use factor of safety = 3.0



Solution:

The net ultimate point bearing capacity,

$Q_{p(net)} = A_p\, c_u\, N_c^*$ Here, $\frac{L}{D_b} = \frac{14}{2} = 7 > 3$

Now,

$\frac{c_{u(2)}}{P_a} = \frac{90}{100} = 0.9$; from table for $\frac{c_u}{P_a} = 0.9$, $N_c^* = 8.8$ (interpolation)

or, $N_c^* = 1.33\left[(\ln I_r) + 1\right]$; $I_r = \frac{E_s}{3c_u} \rightarrow$ (৫৩৪ থাকলে

$\therefore Q_{p(net)} = A_p\, c_u\, N_c^* = \frac{\pi}{4} \times (2)^2 \times 90 \times 8.8 = 2488.15$ KN

Now, The ultimate skin resistance,

$Q_s = \Sigma \alpha^* c_u\, p\, \Delta L$ where, $\alpha^* = 0.4$

$p = \pi D_s = \pi \times (1)$
$= 3.1416$ m

$= 0.4 \times 3.1416 \times \left[(10 \times 45) + (90 \times 4)\right]$

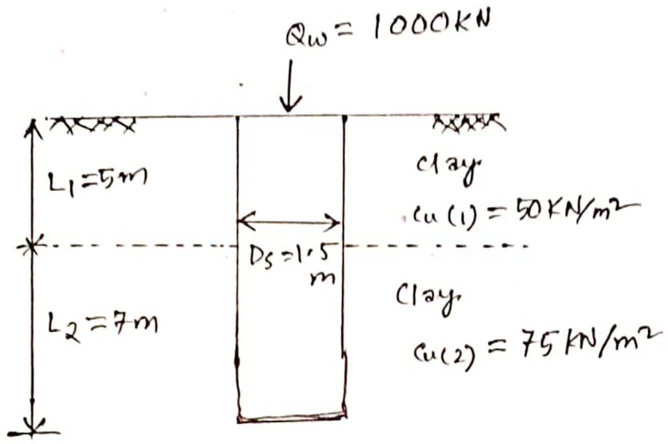
$= 1017.88$ KN

$\therefore$ The Net allowable load, $Q_{all(net)} = \frac{Q_{p(net)} + Q_s}{FS} = \frac{2488.15 + 1017.88}{3}$

$\therefore Q_{all(net)} = 1168.68$ KN (Ans.)

2017 (Example - 10.6)

A drilled shaft without a bell as shown in figure below. Assume that $E_p = 20\times10^6\ KN/m^2$, $C_p = 0.03$, $\xi = 0.65$, $\mu_s = 0.3$, $E_s = 12000 KN/m^2$ and $Q_{ws} = 0.8 Q_w$. Estimate the total settlement of the drilled shaft at working load.



Solution:

Total settlement of the shaft at working load,

$$S_e = S_{e(1)} + S_{e(2)} + S_{e(3)}$$

Here, $S_{e(1)} = \dfrac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}$

Now, $Q_{ws} = 0.8 \times Q_w = (0.8 \times 1000) = 800\ KN$

$\therefore Q_p = (1000 - 800) = 200\ KN$

So, $S_{e(1)} = \dfrac{[200 + (0.65 \times 800)] \times (5+7)}{\frac{\pi}{4} \times (1.5)^2 \times 20\times10^6} = 2.44 \times 10^{-4}\ m$

$\therefore S_{e(1)} = 0.2445\ mm$

Then, $S_{e(2)} = \dfrac{Q_{wp}\, C_p}{D_b\, q_p} = \dfrac{200 \times 0.03}{1.5 \times C_{u(b)} \times N_c^*} = \dfrac{200 \times 0.03}{1.5 \times 75 \times 9}$

$= 5.926 \times 10^{-3}\ m$

$= 5.926\ mm$

Again.

$$S_{e(3)} = \left(\frac{Q_{ws}}{pL}\right) \times \left(\frac{D_s}{E_s}\right) (1-\mu_s^2)\, I_{ws}$$

Here,

$$I_{ws} = 2 + 0.35\sqrt{\frac{L}{D_s}} = 2 + 0.35\sqrt{\frac{12}{1.5}} = 2.99$$

$$\therefore S_{e(3)} = \left(\frac{800}{3.1416 \times 1.5 \times 12}\right) \times \left(\frac{1.5}{12000}\right) \times (1 - 0.3^2) \times 2.99$$

$$= 4.812 \times 10^{-3}\ m = 4.812\ mm$$

$\therefore$ The total settlement, $S_e = S_{e(1)} + S_{e(2)} + S_{e(3)}$

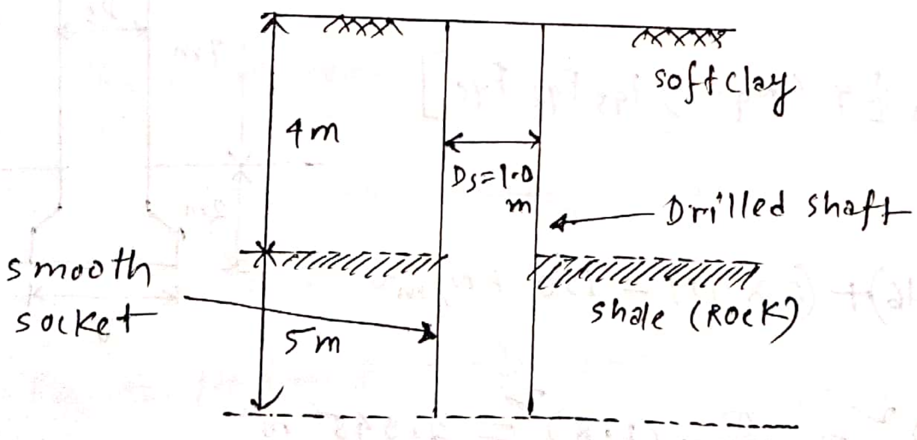
$$= (0.2445 + 5.926 + 4.812)\ mm$$

$$= 10.9825\ mm$$

(Ans.)

2016 (Example- 10.9)

A drilled shaft extendinding into a shale (rock) formation as shown in figure below. For the intact rock cores, given, $q_u = 4.0\ MN/m^2$. Calculate the allowable load bearing capacity of drilled shaft. Use $FS = 3.0$ Assume a smooth socket for side resistance.



<u>Solution:</u> Using Zhang and Einstein method.

$$Q_p = A_p \times \left[4.83\,(q_u)^{0.51}\right] = \frac{\pi}{4} \times 1^2 \times \left[4.83 \times (4)^{0.51}\right]$$

$$\therefore Q_p = 7.693 \text{ MN}$$

$$Q_s = \left[0.4 \times (q_u)^{0.5}\right] \times \left[\pi \times D_s\right] \times L$$

L → shale layer

$$\therefore Q_s = \left[0.4 \times (4)^{0.5}\right] \times (3.1416 \times 1) \times 5 = 12.5664 \text{ MN}$$

Hence, $Q_{u(net)} = Q_p + Q_s = (7.693 + 12.5664)$ MN

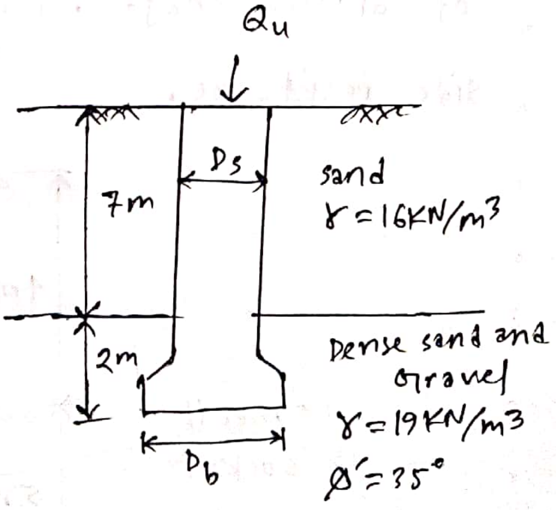
$$= 20.26 \text{ MN}$$

$$\therefore Q_{all} = \frac{Q_{u(net)}}{FS} = \frac{20.26}{3.0} = 6.7533 \text{ MN}$$

(Ans.)

2015 (Example – 10.1 & 10.2)

A soil profile is shown is figure below. A point bearing drilled shaft with a bell is placed in a layer of dense sand and gravel. Determine the allowable load the drilled shaft could carry. Given $D_s = 1.0$ m, $D_b = 1.8$ m, $F.S = 3.0$, $\phi' = 35°$ and $E_s = 500$ Pa.



<u>Solution:</u>

we know,

$$Q_{p(net)} = A_p \times q' \times (\omega N_q^* - 1)$$

Here,

$$q' = (7 \times 16) + (2 \times 19) = 150 \text{ KN/m}^2$$

$$A_p = \frac{\pi}{4} D_b^2 = \frac{\pi}{4} \times (1.8)^2 = 2.545 \text{ m}^2$$

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$$N_q^* = 0.21\, e^{0.17\phi'} = 0.21 \times e^{0.17 \times 35} = 80.5882$$

For,

$$\frac{L}{D_b} = \frac{9}{1.8} = 5 \quad \text{and}, \quad \phi' = 35^\circ$$

from Table - 10.3 (B.M. Das)

the value of $w = \dfrac{0.804 + 0.822}{2} = 0.813$

so, $Q_{p(net)} = 2.545 \times 150 \times [0.813 \times 80.5882 - 1]$

$= 24629.8254$ KN

Now,

Assume, $L_1 \simeq L$

$$Q_s = \pi D_s (1 - \sin\phi') \int_0^{L_1} \sigma_0' \tan\delta' \, dz$$

$$= 3.14176 \times 1 \times (1 - \sin 35^\circ) \times \tan(0.8 \times 35^\circ) \times \int_0^{L_1} \gamma z \, dz$$

$$= 0.7123 \times \left[\int_0^7 16 z\, dz + \int_7^9 19 z \, dz \right] = 513.2 \text{ KN}$$

$$\therefore Q_{all} = \frac{24629.8254 + 513.2}{3} = 8381 \text{ KN}$$

(Ans.)

alternate solution:

We Know, $Q_{p(net)} = A_p \left[q' (N_q - 1) \, F_{qs} \, F_{qd} \, F_{qc} \right]$

Here,

$A_p = 2.545 \ m^2$

$q' = 150 \ KN/m^2$

From Table 10.1, (BOOK - BM Das)

For $\phi' = 35^\circ$, $N_q = 33.30$

$F_{qs} = 1.70$

→ Exam-এ Table Provide করলে OK. নাহলে Manually বের করতে হবে।

$$N_q = \tan^2\left(45 + \frac{\phi'}{2}\right) e^{\pi \tan\phi'}$$

$$= \tan^2\left(45 + \frac{35}{2}\right) e^{\pi \tan 35}$$

$$= 33.13$$

$$F_{qs} = 1 + \tan\phi' = (1 + \tan 35) = 1.7$$

$F_{qd} = 1 + C \tan^{-1}\left(\frac{L}{D_b}\right)$

$= 1 + 0.255 \tan^{-1}\underbrace{\left(\frac{9}{1.8}\right)}_{\text{radian mode}}$

$= 1.35$

Here,

$C = 2 \tan \phi' (1 - \sin\phi')^2$

$= 2 \tan(35)° \times (1 - \sin 35)^2$

$= 0.255$

$I_{cr} = 0.5 \exp\left[2.85 \cot\left(45 - \frac{\phi'}{2}\right)\right]$

$= 0.5 \exp\left[2.85 \cot\left(45 - \frac{35}{2}\right)\right]$

$= 119.3$

Now, $I_{rr} = \dfrac{I_r}{1 + I_r \Delta}$

Here, $\Delta = n \times \dfrac{q'}{P_a}$

$I_r = \dfrac{E_s}{2(1+\mu_s)q' \tan\phi'}$

Hence,

$n = 0.005\left(1 - \frac{\phi' - 25}{20}\right)$

$= 0.005\left(1 - \frac{35 - 25}{20}\right)$

$= 0.0025$

$\therefore \Delta = 0.0025 \times \frac{150}{100} = 0.00375$

and,

for dense soil, $m = 500$

$E_s = m P_a$

$= (500 \times 100) = 50000 \text{ KN/m}^2$

$\mu_s = 0.1 + 0.3 \times \left(\frac{\phi' - 25}{20}\right)$

$= 0.1 + 0.3 \times \left(\frac{35 - 25}{20}\right)$

$= 0.25$

2

Now,

$$I_r = \frac{50000}{2\times(1+0.25)\times 150\times \tan 35} = 190.42$$

$$\therefore I_{rr} = \frac{190.42}{1+190.42\times 0.00375} = 111.092$$

$I_{rr} < I_{cr}$ Hence,

$$F_{qc} = \exp\left\{(-3.8\tan\phi') + \left[\frac{(3.07\sin\phi')(\log_{10} 2I_{rr})}{1+\sin\phi'}\right]\right\}$$

$$= \exp\left\{(-3.8\times\tan 35) + \frac{3.07\sin 35\times\log_{10}(2\times 111.092)}{1+\sin 35}\right\}$$

$$= e^{-0.035}$$

$\therefore F_{qc} = 0.966$

Hence,

$$Q_{p(net)} = 2.545\times[150\times(33.3-1)\times 1.70\times 1.35\times 0.966]$$

$$= 27336.4 \text{ KN}$$

Here, $L_1 \simeq (7+2) = 9$ m

Now,

$$Q_s = \pi D_s(1-\sin\phi')\int_0^{L_1}\sigma_o'\tan\delta'\,dz$$

$$= 3.1416\times 1\times(1-\sin 35^\circ)\tan(0.8\times 35^\circ)\times\int_0^{L_1}\gamma z\,dz$$

$$= 0.7123\times\left[\int_0^7 16z\,dz + \int_7^9 19z\,dz\right] = 513.2 \text{ KN}$$

$$\therefore Q_{all} = \frac{27336.4+513.2}{3} = \frac{27849.6}{3} = 9283.2 \text{ KN}$$

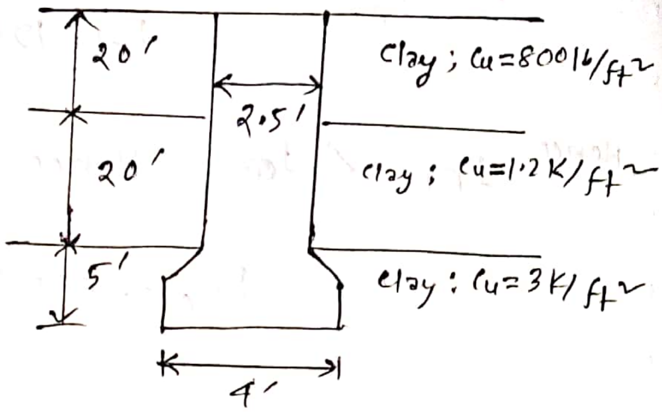
(Ans.)

2014 (Example-10.5)

A drilled shaft in a cohesive soil is shown in figure below. Use Reese and O'Neil's method to determine

(i) the ultimate capacity

(ii) The load bearing capacity for an allowable settlement of 0.5 inch.



Solution:

(i) According to Reese and O'Neil's procedure,

$$Q_{u(net)} = \sum_{i=1}^{N} f_i\, p\, \Delta L_i + q_p A_p$$

Here,

$$f_i = \alpha_i^* c_{u(i)}$$

From figure, $\Delta L_1 = (20-5)\ ft = 15\ ft$

$\Delta L_2 = (20-2.5) = 17.5\ ft$

$$\therefore \sum f_i\, p\, \Delta L_i = 0.55 \times 0.8 \times (3.1416 \times 2.5) \times 15 + 0.55 \times 1.2 \times (3.1416 \times 2.5) \times 17.5$$

$$= 142.55\ \text{Kips.}$$

Now,

$$q_p = 6\,c_{ub}\left(1 + 0.2 \times \frac{L}{D_b}\right) = 6 \times 3 \times \left(1 + 0.2 \times \frac{45}{4}\right) = 58.5\ K/ft^2$$

Check: $q_p = 9\,c_{ub} = (9 \times 3) = 27\ K/ft^2 < 58.5\ K/ft^2$

Hence, we should use, $q_p = 27\ K/ft^2$

$$\therefore q_p A_p = 27 \times \frac{\pi}{4} \times (4)^2 = 339.3\ \text{Kip.}$$

$\therefore Q_{ult} = \Sigma \alpha_i^* c_{u(i)} p \Delta L_i + q_p A_p$

$= (142.55 + 339.3) = 481.85$ kip. (Ans.)

(ii)

Now,

$$\frac{\text{Allowable settlement}}{D_s} = \frac{0.5}{2.5 \times 12} = 0.1667 = 1.67\%.$$

From Fig. 10.16, (B.M. das - 8th ed.)

$$\frac{\text{side-load transfer}}{\text{ultimate sideload transfer, } (\Sigma f_i p \Delta L_i)} \approx 0.9$$

Thus, side-load transfer $= (0.9 \times 142.55) = 128.295$ kip.

Again,

$$\frac{\text{Allowable settlement}}{D_b} = \frac{0.5}{4 \times 12} = 0.0104 = 1.04$$

From fig. 10.17,

$$\frac{\text{End Bearing}}{\text{ultimate end Bearing, } q_p A_p} = 0.65$$

$\therefore$ End Bearing $= (0.65 \times 339.3) = 220.545$ kip.

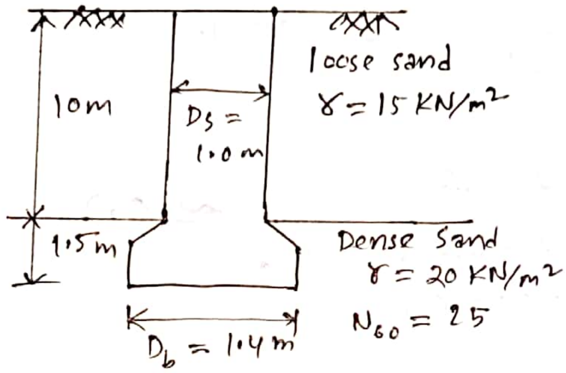
$\therefore$ The total load $= (128.295 + 220.545)$ kip.

$= 348.84$ kip.

(Ans.)

2006, 2013, 2008 (Example – 10.3)

A drilled shaft is shown in figure below. The uncorrected average standard penetration number (N_{60}) with in the distance of $2D_b$ below the base of the shaft is 25. Determine the ultimate load carrying capacity of drilled shaft.



Solution;

From Reese and O'Neil's procedure,

$$Q_{u(net)} = \sum_{i=1}^{N} f_i\, p\, \Delta L_i + q_p A_p$$

Here, $f_i = \beta_i\, \sigma'_{ozi} < 192\ KN/m^2$ and $\beta_i = 1.5 - (0.244) \times (z_i)^{0.5}$.

we can divide the loose sand layer into two layers, each having 5m thickness. Now the following table can be prepared:

Layer No.	Depth of middle of layer (z_i)	β_i	$\sigma'_{ozi} = \gamma z_i$	f_i (KN/m^2)
1	2.5	1.1142	37.5	41.7825
2	7.5	0.832	112.5	93.6

Thus, $\Sigma f_i\, p\, \Delta L_i = \pi \times (1) \times [41.7825 \times 5 + 93.6 \times 5]$

$= 2126.6\ KN$

Then,

$$q_p = \beta_5 N_{60} \leq (\beta_6 = 4310\ KN/m^2)$$

$$= 57.5\ N_{60}$$

$$= (57.5 \times 25)\ KN/m^2$$

But, $D_b > 1.27\,m$

$$\therefore q_p = 1437.5\ KN/m^2$$

Hence, $$q_{pr} = \frac{1.27}{D_b(m)} \times q_p = \frac{1.27}{1.4} \times 1437.5$$

$$\therefore q_{pr} = 1304.02\ KN/m^2$$

Now, $$q_{pr} A_p = 1304.02 \times \frac{\pi}{4} \times (1.4)^2 = 2007.4\ KN$$

$\therefore$ The ultimate load carring capacity,

$$Q_{ult(net)} = \sum f_i p \Delta L_i + q_{pr} A_p$$

$$\therefore Q_{ult(net)} = (2126.6 + 2007.4) = 4134\ KN$$

(Ans.)

Additional Quection :

* Determine the load carrying capacity for settlement of 12mm

Solution: $$\frac{\text{Allowable settlement}}{D_s} = \frac{12}{1 \times 1000} = 0.012 = 1.2\%$$

From figure 10.12, for 1.2% of normalize settlement, the normalized load is 0.98.

$$\therefore \text{side load tranfer} = (0.98 \times 2126.6)\ KN$$

$$= 2084.07\ KN$$

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Again,

$$\frac{\text{Allowable settlement}}{D_b} = \frac{12}{1.4 \times 1000} = 0.0086 = 0.86\%$$

from Figure 10.11, The normalize load is 0.3

$\therefore$ The base load is $= (0.3 \times 2007.4) = 602.22$ KN

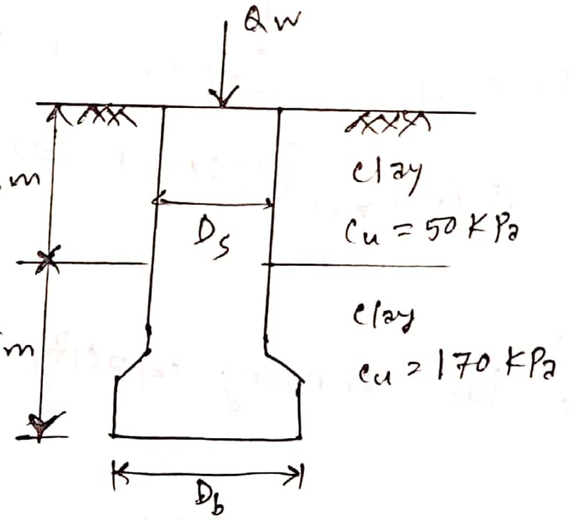
$\therefore$ The total load carrying capacity,

$$Q = (2084.07 + 602.22) = 2686.29 \text{ KN}$$

(Ans.)

2012, 2009

A drilled shaft with a bell to be considered as shown in the figure below. Given $f_c' = 20$ MPa, $Q_w = 1.2$ MN and F.S = 2.5. Design the shaft and bell.



Solution: Ultimate load, $Q_u = F.S \times Q_w = (2.5 \times 1.2)$ MN

$\therefore Q_u = 3$ MN

Diameter of the drilled shaft,

$$D_s = 2.257\sqrt{\frac{Q_u}{f_c'}} = 2.257\sqrt{\frac{3}{20}}$$

$\therefore D_s = 0.874\text{m} \simeq 0.9\text{m}$ (let)

Now,

$$Q_s = \sum_{i=1}^{N} f_i \, P \, \Delta L_i \qquad \text{Here } f_i = \alpha_i^{*} \, C_{u(i)}$$

$$p = \pi \times D_s = (3.1416 \times 0.9)$$

$$= \sum_{i=1}^{N} \alpha_i^{*} \, C_{u(i)} \, P \, \Delta L_i \qquad = 2.83 \text{ m}$$

$$= 0.4 \times 2.83 \times [(50 \times 6) + (170 \times 5)]$$

$$\therefore Q_s = 1301.8 \text{ KN}$$

and, $\frac{L}{D_b} > 3$ (let)

$$\therefore Q_p = A_p \, C_u \, N_c^{*}$$

$$= \frac{\pi}{4} (D_b)^2 \times 170 \times 9$$

$$Q_p = 1201.662 \, D_b^2$$

Here, $N_c^{*} = 1.33 [(\ln I_r) + 1]$

for, $\frac{C_{u(2)}}{P_a} = \frac{170}{100} = 1.7$; $I_r = \frac{E_s}{3 C_u} = 300$

$$\therefore N_c^{*} = 1.33 [\ln (300) + 1]$$

$$= 8.92 \simeq 9$$

Thus, $Q_u = Q_p + Q_s$

$$\Rightarrow (3 \times 1000) = 1201.662 \, D_b^2 + 1301.8$$

$$\Rightarrow 1201.662 \, D_b^2 = 3000 - 1301.8$$

$$\Rightarrow D_b^2 = \frac{3000 - 1301.8}{1201.662} = 1.4132$$

$$\therefore D_b = 1.19 \text{ m} \simeq 1.2 \text{ m}$$

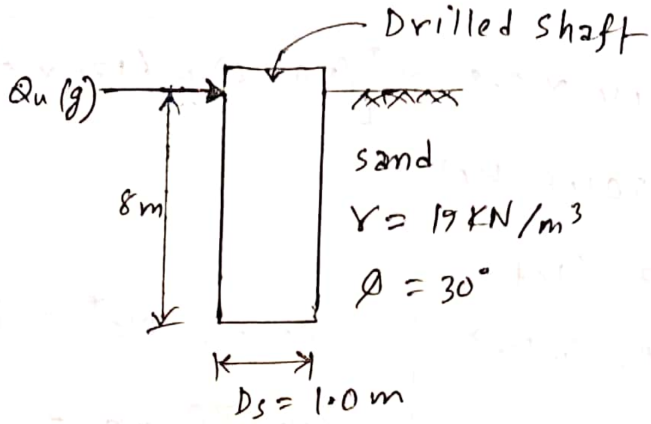
Thus, the bell of the drilled shaft, $D = 1.2$ m

and the shaft of the drilled, $D_s = 0.9$ m

(Ans)

2010, 2009

A drilled shaft in a sand is shown in figure below. Determine the ultimate lateral load $Q_u(g)$ applied at the ground surface. Assume, $E_s = 35 \times 10^3$ KN/m^2 ; $E_p = 20.7 \times 10^6$ KN/m^2, $N_q = 33$; $K_{br} = 10$



Solution:

The ultimate lateral load,

$$Q_{u(g)} = 0.12\, \gamma\, D_s\, L_e^2\, K_{br} \leq 0.4\, P D_s L_e$$

Now,

$$K_r = \frac{E_p I_p}{E_s L^4}$$

$$I_p = \frac{\pi D^4}{64} = \frac{\pi}{64} \times (1)^4 = 0.0491 \text{ m}^4$$

$$\therefore K_r = \frac{20.7 \times 10^6 \times 0.0491}{35 \times 10^3 \times (8)^4} = 0.0071 < 0.01,$$

Hence this is a flexible drilled shaft

Thus,

$$\frac{L_e}{L} = 1.65\, K_r^{0.12} = 1.65 \times (0.0071)^{0.12} = 0.911$$

$$\therefore L_e = 0.911 \times 8 = 7.288$$

$$Q_{u(g)} = 0.12\, \gamma\, D\, L_e^2\, K_{br}$$

$$= 0.12 \times 19 \times 1 \times (7.288)^2 \times 10$$

$$= 1211.021 \text{ KN} < 2221.681 \text{ KN (ok)}$$

And, $0.4\, P D_s L_e = 0.4 \times 40\, N_q \tan\phi \times D_s \times L_e$

$= 0.4 \times 40 \times 33 \times \tan 30 \times 1 \times 7.288$

$= 2221.681$ KN

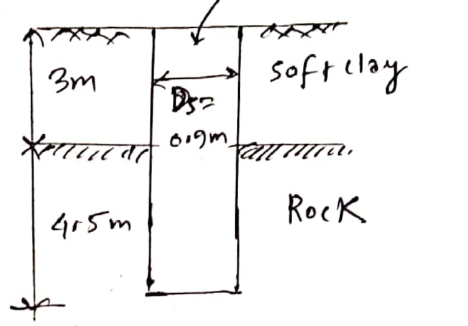
Thus, $Q_{u(g)} = 1211.021$ KN (Ans)

Example- 10.8

A drilled shaft extending into rock as shown in Figure below.

Let, $L = 4.5$, $D_s = 0.9m$, q_u (rock) $= 72450 \, KN/m^2$, q_u (concrete) $= 20700 \, KN/m^2$, $E_c = 20.7 \times 10^6 \, KN/m^2$, RQD (rock) $= 80\%$, E_{core} (rock) $= 2.48 \times 10^6 \, KN/m^2$, $C_s = 457$ mm, and $\delta = 3.81mm$. Estimate the allowable load-bearing capacity of drilled shaft. Use F.S = 3

Use the Reese and O'Neil's Method.



Solution: According to Reese and O'Neil,

Step-1: $f(KN/m^2) = 6.564 \, q_u^{0.5} \leq 0.15 \, q_u$

Since, q_u (concrete) $< q_u$ (rock),

use, q_u (concrete) in the equation.

$$\therefore f = 6.564 \times (20700)^{0.5} = 944.4 \, KN/m^2$$

step-2: $Q_u = \pi D_s L f = (3.1416 \times 0.9 \times 4.5 \times 944.4)$

$$\therefore Q_u = 12016.055 \, KN$$

Step-3: $$S_e = \frac{Q_u L}{A_c E_c} + \frac{Q_u I_f}{D_s E_{mass}}$$

For, RDQ $= 80\%$; $\frac{E_{mass}}{E_{core}} = 0.0266(RDQ) - 1.66$

$= 0.0266 \times 80 - 1.66$

$= 0.468$

$\therefore E_{mass} = 0.468 \, E_{core} = 0.468 \times 2.48 \times 10^6$

$\therefore E_{mass} = 1160640 \, KN/m^2$

For, $\frac{E_c}{E_{mass}} = \frac{20.7 \times 10^6}{1160640} = 17.835$ and $\frac{L}{D_s} = \frac{4.5}{0.9} = 5$

From table 10.6, $I_f = 0.35$.

Hence,
$$S_e = \frac{12016.055 \times 4.5}{\frac{\pi}{4} \times (0.9)^2 \times 20.7 \times 10^6} + \frac{12016.055 \times 0.35}{0.9 \times 1160640}$$

$$\therefore S_e = 8.132 \times 10^{-3}\ m = 8.132\ mm < 10\ mm$$

$$\therefore Q_u = \pi D_s L f = 12016.055\ KN$$

$$\therefore Q_{all} = \frac{Q_u}{FS} = \frac{12016.055}{3} = 4005.352\ KN$$

(Ans.)

* Practice all the example in Book (B.M Das - 8th ed.)
(Example - 10.1 to 10.9)