Middle East Technical University
Department of Civil Engineering
CE 363 - Soil Mechanics
PROBLEMS

NOTE. Unless otherwise stated, take γ_w = 10 kN/m^3

(a) BASIC PROPERTIES

A1. A cubic meter of soil in its natural state weighs 17.75 kN; after being dried it weighs 15.08 kN. The specific gravity of the solids is 2.70.

(a) Determine the water content, void ratio, porosity and degree of saturation for the soil as it existed in its natural state.

(b) What would be the bulk unit weight and water content if the soil were fully saturated at the same void ratio as in its natural state ?

A2. A sand with a minimum void ratio of 0.45 and a maximum void ratio of 0.97 has a relative density of 40 %.

How much will a 3 m thick stratum of this sand settle if the sand is densified to a relative density of 65 %? Assume that the sand layer is compressed in the vertical direction only, with no lateral strain.

A3. The results of sieve analysis on a soil sample are given below:

Sieve size	Percentage finer
19.1 mm	100
6.3 mm	94
2 mm	69
590 μm	32
210 μm	13
74 μm	2

(a) Plot the grain size distribution curve.

(b) Determine the percentages of gravel, sand and the fines in the sample.

(c) Determine D_{10}, D_{30}, D_{60}, C_u, C_c and comment on the gradation.

A.4. The consistency limits for a given clay were determined to be

LL = 55%, PL = 27%, SL = 20%

(a) If the specific gravity of solid particles is 2.70, and a 100 cm^3 saturated sample of this soil at its natural water content of 30% is allowed to dry, what will be its volume at a water content of 15%?
(b) What is the consistency of the soil in its natural state ?
(c) Calculate the plasticity index of the soil.
(d) Determine the liquidity index of the soil.

b) EFFECTIVE STRESS

B1. A fine sand layer of 5 m thickness lies on a 5 m clay deposit. The water table is at the ground surface. Below the clay is a rock formation. Piezometers installed in the rock show an artesian pressure, piezometric level being 3 m above the ground surface. Unit weight of the sand is 18 kN/m^3 and that of the clay 20 kN/m^3. Draw the total stress, effective stress, and pore water pressure diagrams.

B2. In a fine sand deposit of 20 m thickness, the water table is 10 m below the surface and there is a 2 m capillary zone above the water table. The drained and saturated unit weights of the sand are 17 kN/m^3 and 20 kN/m^3 respectively. Determine the total stress, effective stress, and pore pressure at 5 m, 9 m, and 15 m depths from the ground surface.

B3. A soil profile consists of 10 m of gravel overlying a clay layer of 3 m thickness, which in turn rests on a 3 m sand layer. The drained and saturated unit weights of the gravel are 18 kN/m^3 and 20 kN/m^3. Unit weights of the clay and the sand are 20 kN/m^3 and 19 kN/m^3 respectively. Below the sand layer lies impermeable bedrock. If a wide excavation of 3 m depth is to be made in a relatively short time, draw the effective, total and pore pressure diagrams before, immediately after and a long time after the excavation is made. Ground water table is 5 m below the ground surface.

B4. A clay layer of 20 m thickness with a unit weight of 20 kN/m^3 overlies a sandstone formation. Ground water table is at the ground surface. Piezometric measurements show that there is artesian pressure in the sandstone amounting to a water level of 3 m above the ground surface.

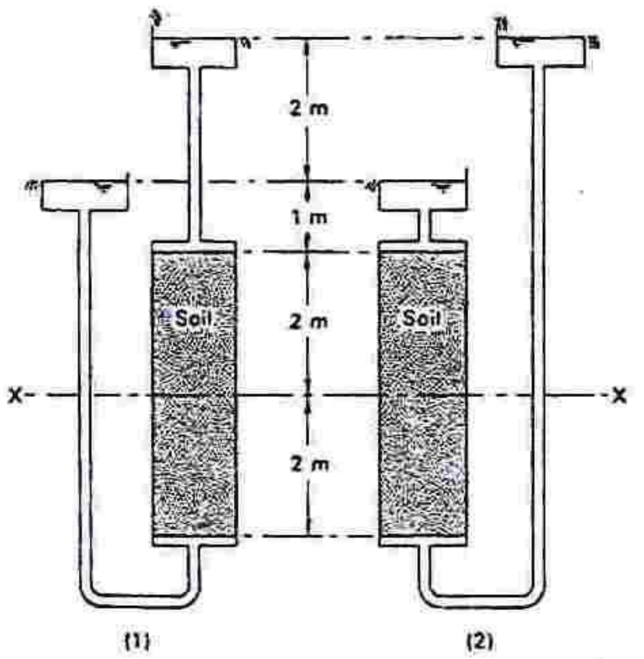
If an excavation 12 m in depth is planned in the clay layer, determine the level to which artesian water should be lowered, in order to prevent the heave of the bottom of the excavation. (Excavation is to be made in the dry.)

(c) SEEPAGE

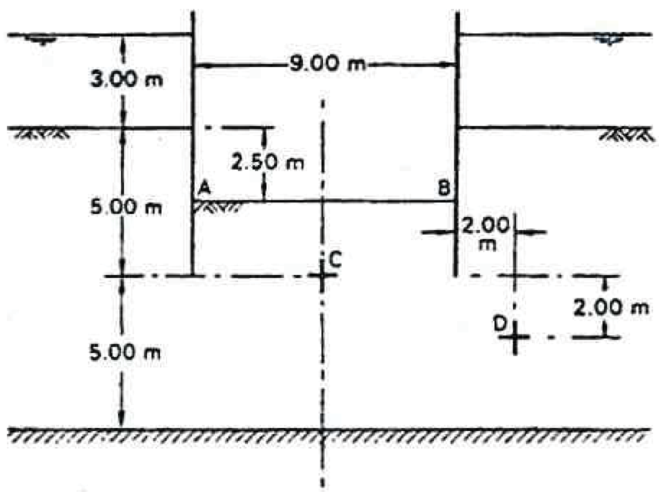
C1. The sample of soil in a permeability test is 50 mm diameter and 120 mm long. The head difference is 105 mm, and the flow is 150 ml in 5 min. Compute the coefficient of permeability in mm/s and m/year.

C.2. In a falling head permeability test on a silty soil, the sample has 75 mm diameter and 150 mm length, and the standpipe is 10 mm in diameter. A stopwatch is started when the head difference, h = 500 mm and stopped when h = 250 mm and reads 19.6 s. The test is repeated for a drop from 250 mm to 125 mm and the time is 19.4 s. Find the coefficient of permeability in mm/s.

C3. (After Craig, R.F. “Soil Mechanics”) For the following seepage situations, determine the effective normal stress on plane XX in each case (a) by considering pore water pressure, (b) by considering seepage pressure. The saturated unit weight of the soil is 20 kN/m^3.



C4. (After Craig, R.F. "Soil Mechanics") The section through a long cofferdam is shown below, the saturated unit weight of the soil being 20 kN/m^3. Determine the factor of safety against 'boiling' at the surface AB, and the values of effective vertical stress at C and D.



C.5. (a) Draw the flow net for seepage beneath a vertical - faced impervious dam, having a base width of 50 m, and resting 2 m below the surface of a 12 m thick uniform stratum of silty sand with a permeability of 4x10^{-3} mm/s and saturated unit weight 21 kN/m^3, underlain by an impervious layer.

(b) Compute the quantity of seepage in m^3/day if the length of the dam is 150 m, and the head on the dam is 20 m upstream and 5 m downstream.

(c) Determine the distribution of uplift pressure on the base of the dam.

(d) Calculate the effective normal stress at a point which is located at a depth of 6 m from the surface of silty sand layer and 4 m from the upstream face of the dam toward the dam reservoir by considering

(i) total stress and pore pressure;

(ii) effective weight and seepage pressure.

(A) Basic Properties

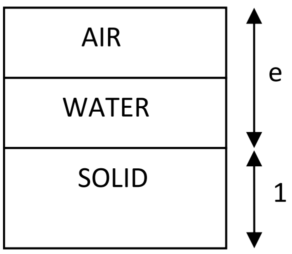
A1.

Given

W= 17.75 kN → W_d=15.08 kN (when dried)

$G_s = 2.70, \;\; V = 1m^3$

Determine w, e, n and S_r.



	VOLUME	MASS
AIR	(1-S_r)e	0
WATER	$S_r e$	$wG_s\rho_w$
SOLID		$G_s\rho_w$

a)

$$w = \frac{M_w}{M_s} = \frac{17.75-15.08}{15.08} = 0.177 \rightarrow w = 17.7\%$$

$$\gamma = \frac{G_s(1+w)\gamma_w}{(1+e)} = \frac{W}{V} = \frac{17.75}{1} = 17.75\ kN/m^3$$

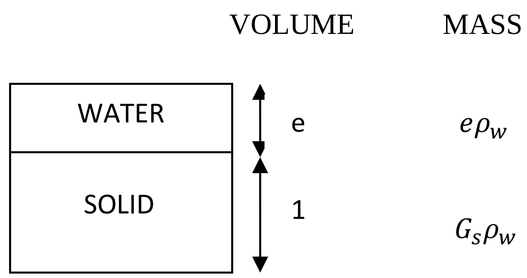
$$17.75 = \frac{2.70(1+0.177)(9.81)}{(1+e)}$$

$$void\ ratio = \ e=0.756$$

$$porosity = n = \frac{V_v}{V} = \frac{e}{1+e} = 0.4$$

$$S_r = \frac{wG_s}{e} = \frac{0.177 * 2.70}{0.667} = 0.716$$

b)



If the soil were fully saturated (no air content) at the same void ratio as in its natural state;

$$S_r = 1 \qquad e = 66.7\%$$

$\gamma_{bulk} =? \quad w =?$

$$w = \frac{M_w}{M_s} = \frac{e\rho_w}{G_s\rho_w} = \frac{e}{G_s} = \frac{0.667}{2.70} = 0.247$$

$$\gamma_{bulk} = \frac{e\gamma_w + G_s\gamma_w}{1+e} = \frac{0.667 * 9.81 + 2.70 * 9.81}{1 + 0.667} = 19.81 kN/m^3$$

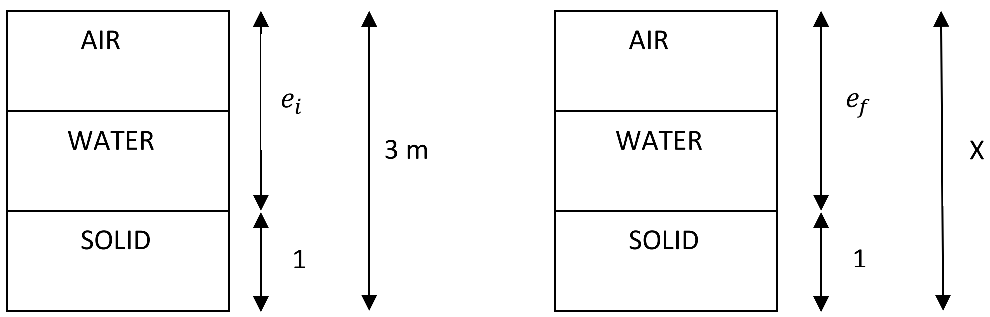
A2)

$$e_{min} = 0.45$$

$$e_{max} = 0.97$$

$$RD = \frac{e_{max} - e}{e_{max} - e_{min}} = 0.40 = \frac{0.97 - e_i}{0.97 - 0.45} \rightarrow e_i = 0.762$$

$$0.65 = \frac{0.97 - e_f}{0.97 - 0.45} \rightarrow e_f = 0.632$$



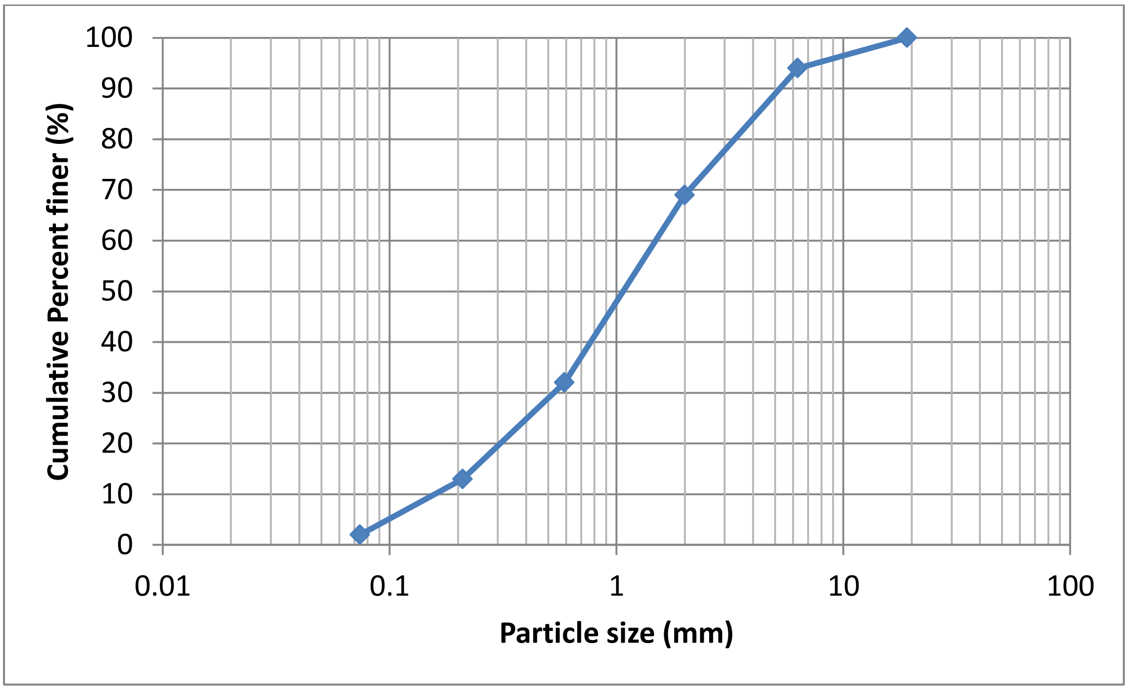
$$(1 + 0.762) \rightarrow 3m$$

$$(1 + 0.631) \rightarrow ?$$

∴ X=2.780 m, 3-2.780 = 0.220 m settles.

A3)

a)



b)

$$100 - 69 = 31\% \ \ gravel\ (.. > 2\ mm)$$

$$67\% \ \ sand\ (0.074mm < .. < 2\ mm)$$

$$2\%\ fines\ (.. < 0.074\ mm)$$

Note: British Standards (BS) uses 0.063 mm, and American Standards (ASTM) uses 0.074 mm as the boundary of "fines". Our CE363 lab manual uses British Standards. But you may encounter both of these numbers.

c) According to the grain size distribution curve, determine; D_{10}, D_{30}, D_{60}.
$D_{10} = 0.15\ mm$, $D_{30} = 0.51\ mm$, $D_{60} = 1.5\ mm$

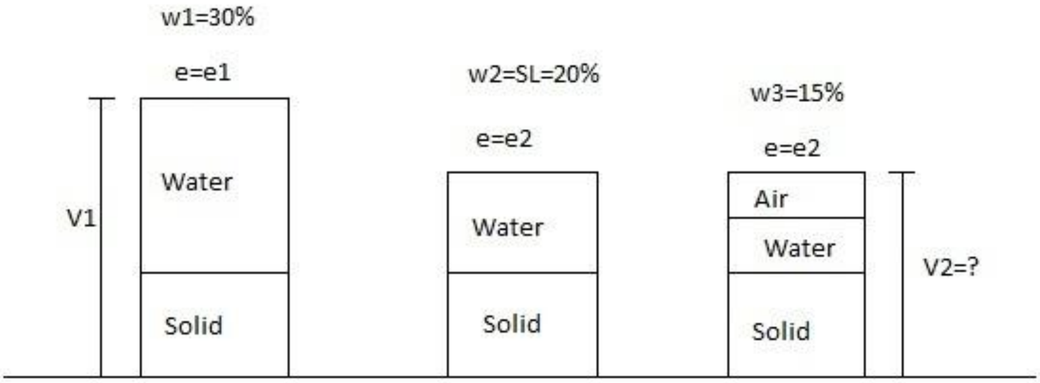
$$C_u = \frac{D_{60}}{D_{10}} = \frac{1,5}{0,15} = 10$$

$$C_C = \frac{(D_{30})^2}{D_{60} * D_{10}} = \frac{(0,51)^2}{1,5 * 0,15} = 1,16$$

According to USCS the soil is SW, well graded sand.

A4)

a)



Shrinkage Limit is the water content at which a reduction in water content does not cause a reduction in the volume of the soil mass. After reaching shrinkage limit, on further reduction in water content, air enters into the voids of soils, however volume of voids (and void ratio) remains constant as it was at shrinkage limit.

At water content of w=30% soil is saturated Sr = 100%.

At shrinkage limit (w=20%) soil is still saturated Sr=100%.

At water content (w=15%) that is lower than the shrinkage limit, air enters into soil, and soil becomes unsaturated, however void ratio is still the same as it was at shrinkage limit.

	Mass	Volume
AIR	0	0
WATER	$wG_s\rho_w$	wG_s
SOLID	$G_s\rho_w$	G_s

If we find the volume when the water content is 20%, we will find directly the volume when the water content is 15% because of the constant volume of solid particles.

$$S_r = 1 \; since \; the \; soil \; is \; saturated$$

$$w = \frac{e}{G_s}$$
$$0.30 = \frac{e}{2.70} \rightarrow e = 0.81$$

$$volume \; of \; soil = 1 + e = 1.81 \; when \; volume = 100 \; cm^3$$

$$0.20 = \frac{e}{2.70} \rightarrow e = 0.540$$

$$volume \; of \; soil = 1 + e = 1.540 \; we \; are \; asked \; to \; find \; the \; volume$$

$$make \; interpolation$$
$$100 \; cm^3 \quad \rightarrow \quad 1.81$$
$$x \quad cm^3 \quad \rightarrow \quad 1.540$$
Therefore x=85.083 cm^3

b) Depending on its water content, a soil may exist in one of the following states: liquid, plastic, semi-solid and solid.
For water content, w=30%; PL<w<LL

Therefore, Consistency of the soil sample is **PLASTIC**.

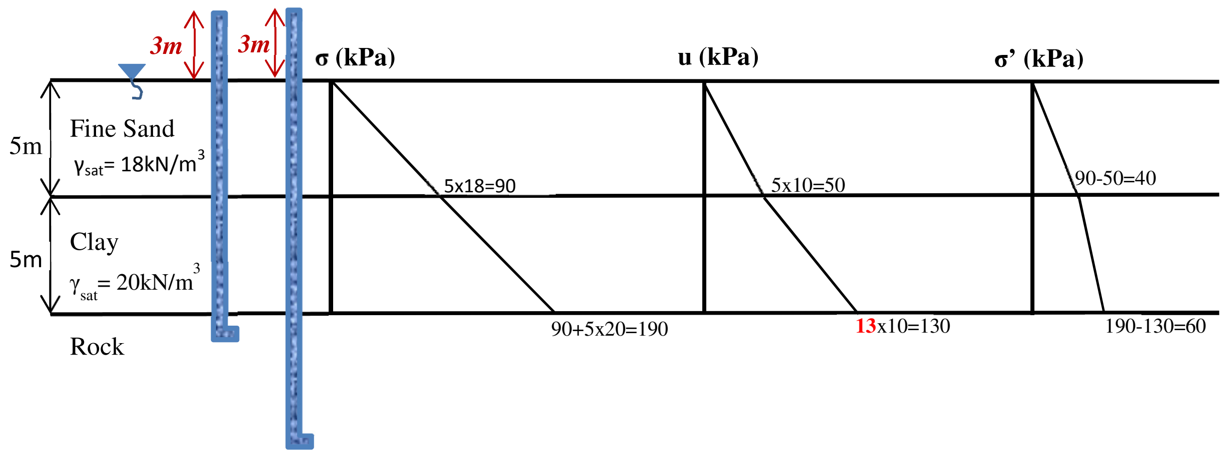
c) Plasticity Index, I_p

$$I_P = LL - PL = 55\% - 27\% = 28\%$$

d) Liquidity Index, I_L

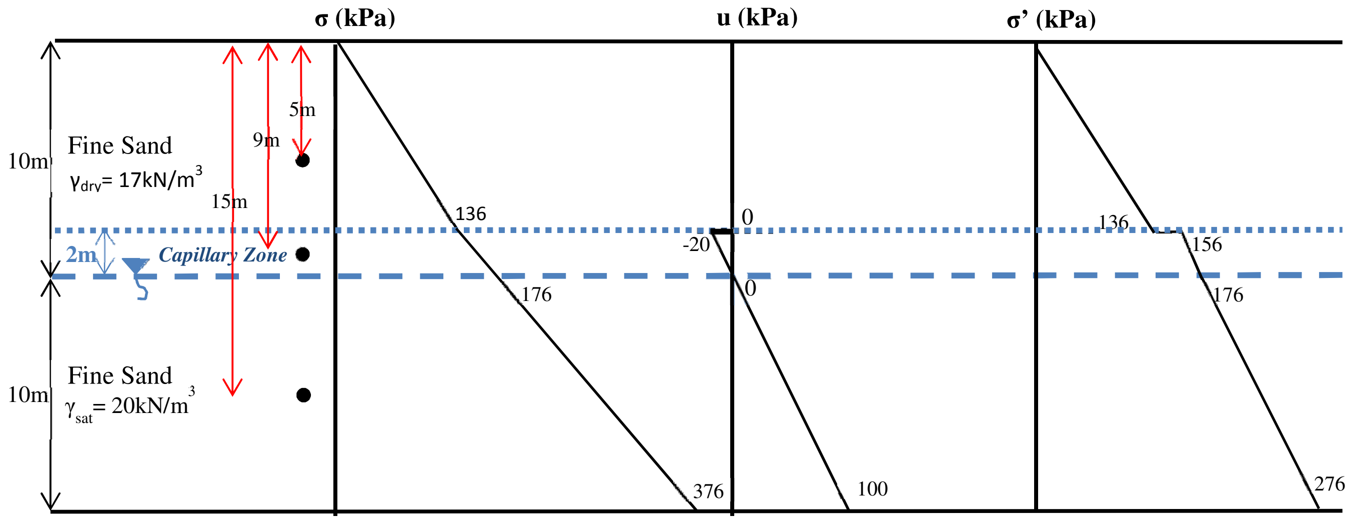
$$I_L = \frac{w - PL}{I_P} = \frac{30\% - 27\%}{28\%} = 0.107$$

(B1)



- A tube inserted in the artesian layer shows that the water pressure is 130 kPa at the top of the artesian layer.
- In any artesian layer water level would rise to same level if we insert another tube to another point in the artesian layer.
- In the fine sand layer pore pressure is hydrostatic (and is independent of the artesian condition below because the clay layer acts as an impermeable blanket). In the clay layer, there will be a transition, from 50 kPa at the top of clay, to 130 kPa at the bottom of clay.

(B2)

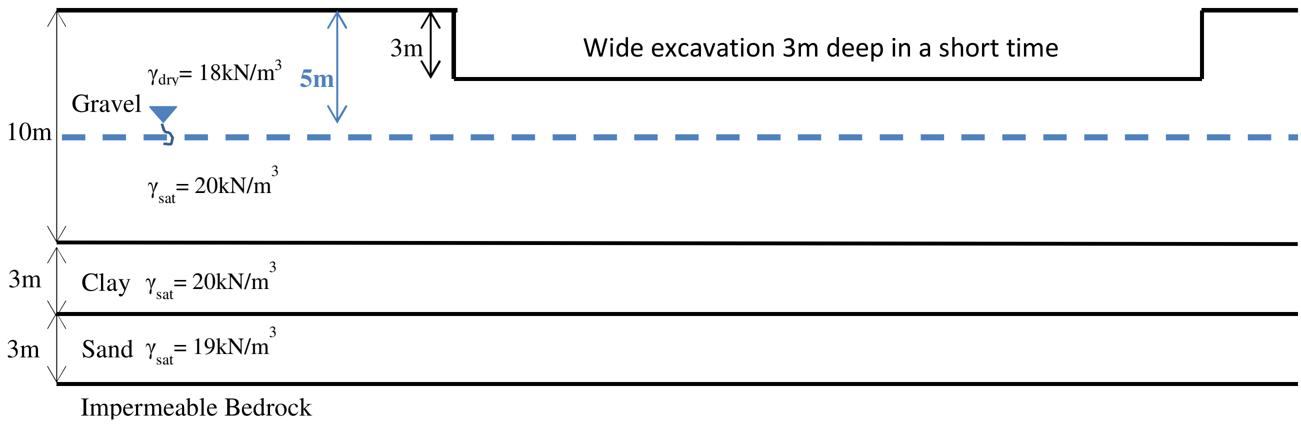


@5m →	σ=5x17=85kPa →	u=0kPa →	σ'=85-0=85kPa
@8m	σ=8x17=136kPa	u=0 and u=2x(-10)=-20kPa	σ'=136 and σ'=136-(-20)=156kPa

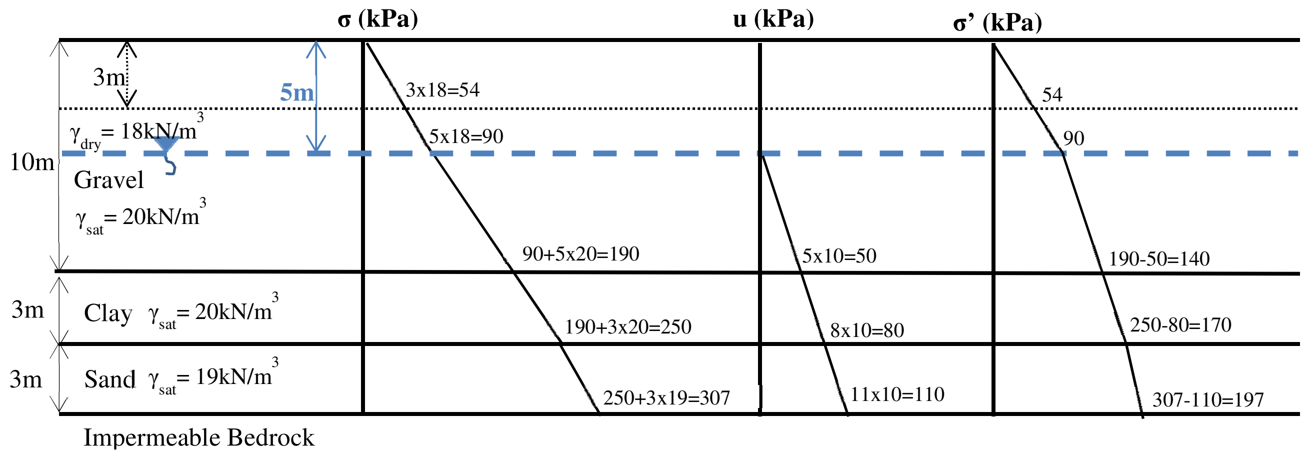
- Note that in the capillary saturation zone saturated unit weight of soil should be used.

@9m →	σ=136+1x20=156kPa →	u=1x(-10)=-10kPa →	σ'=156-(-10)=166kPa
@10m	σ=136+2x20=176kPa	u=0kPa	σ'=176-0=176kPa
@15m →	σ=176+5x20=276kPa →	u=5x10=50kPa →	σ'=276-50=226kPa
@20m	σ=176+10x20=376kPa	u=10x10=100kpa	σ'=376-100=276kPa

(B3)



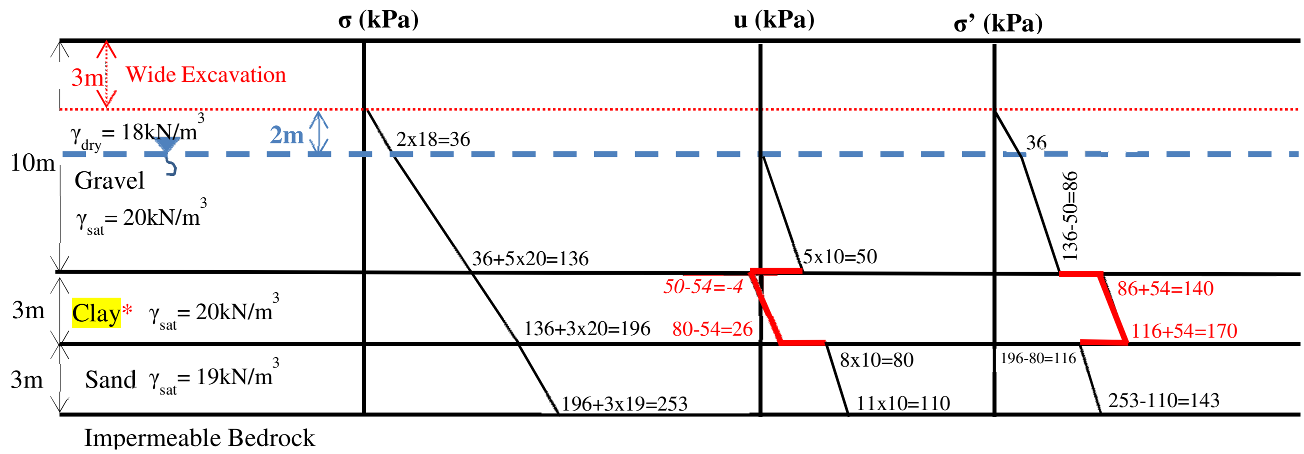
(a) Before Excavation



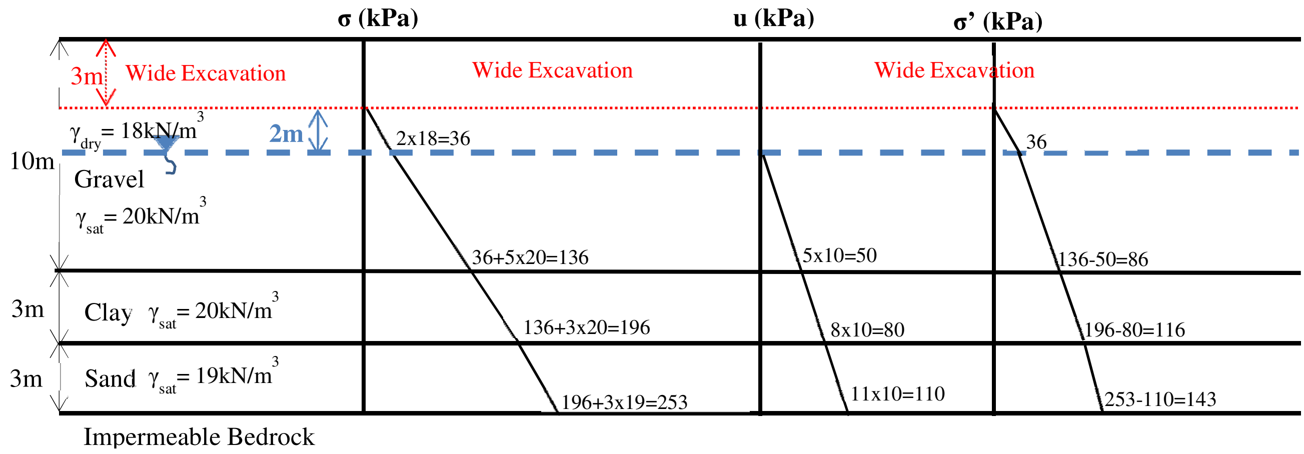
(b) Immediately After Excavation

Reduction in stress due to excavation= $\Delta\sigma$=3x18=54kPa → Δu=-54kPa in clay

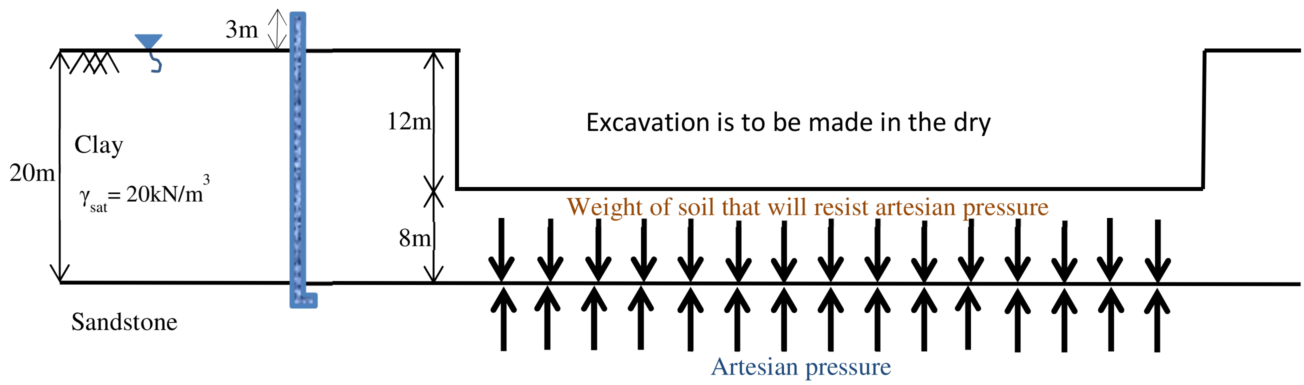
- Note that the reduction in total stress of 54 kPa can also be seen after plotting the new total stress diagram. We can see that in the total stress graph, at the top of the clay the previous value of 190 kPa became 136 kPa, and at the bottom of the clay the previous value of 250 kPa became 196 kPa, this indicates the 54 kPa reduction in total stress, which should be applied to pore pressure in the "immediately after" case in clays.
- For the "immediately after" case, the change in total stress should be applied to previous pore pressure values of clay (before excavation). For example, before excavation, the hydrostatic pore pressure at the top of the clay was 50 kPa, we apply the total stress change of -54 kPa to this value.



(c) Long After Excavation



(B4)



- Total stress at base of clay layer after excavation, $\sigma_1 = 8x20 = 160$kPa
- Max. pore pressure that can be resisted without blowing up of bottom of excavation = 160kPa

h_w=corresponding piezometric head in sand
$\gamma_w=10kN/m^3$
$h_w=160/10=16m$

- Existing piezometric head = 20+3 = 23m
- When artesian pressure is greater than pressure due to weight of soil, heave of the bottom will occur.
- Then required drop in piezometric head = 23-16 = 7m

C1) The sample of soil in a permeability test:

Diameter = D = 50 mm

Length = L = 120 mm

ΔH=105 mm

Flow = 150 ml in 5 min Hence; Q = 150 ml = 150000 mm^3, Δt= 5 min = 300 sec

q = $\frac{Q}{\Delta t}$ = 500 mm^3 / sec

Area = A = $\pi D^2/4$ = 625*π = 1963.5 mm^2

i = $\frac{\Delta H}{L}$ = 0.875

q= A*k*i

k = $\frac{q}{A*i} = \frac{500\ mm^3\ /\ sec}{1963.5\ mm^2 * 0.875} =$ 0.291 mm/sec = 9177.8 m/year

C2) In a falling head permeability test on a silty soil;

Length = L = 150 mm

<u>SAMPLE</u> => Diameter = D = 75 mm Hence; A = 4417.9 mm^2

<u>THE STANDPIPE</u> => Diameter = d = 10 mm Hence; a = 78.5 mm^2

CASE-1	***CASE-2***
h_0=500 mm	h_1=250 mm
h_1=250 mm	h_2=125 mm
Δt_1=19.6 sec	Δt_2=19.4 sec

$$k_1 = \frac{a * L}{A * \Delta t_1} \ln \frac{h_0}{h_1} = \frac{78.5 * 150}{4417.9 * 19.6} \ln \frac{500}{250} = 0.0943\, mm/sec$$

$$k_2 = \frac{a * L}{A * \Delta t_2} \ln \frac{h_1}{h_2} = \frac{78.5 * 150}{4417.9 * 19.4} \ln \frac{250}{125} = 0.0952\, mm/sec$$

$$k = \frac{k_1 + k_2}{2} = 0.0947\, mm/sec$$

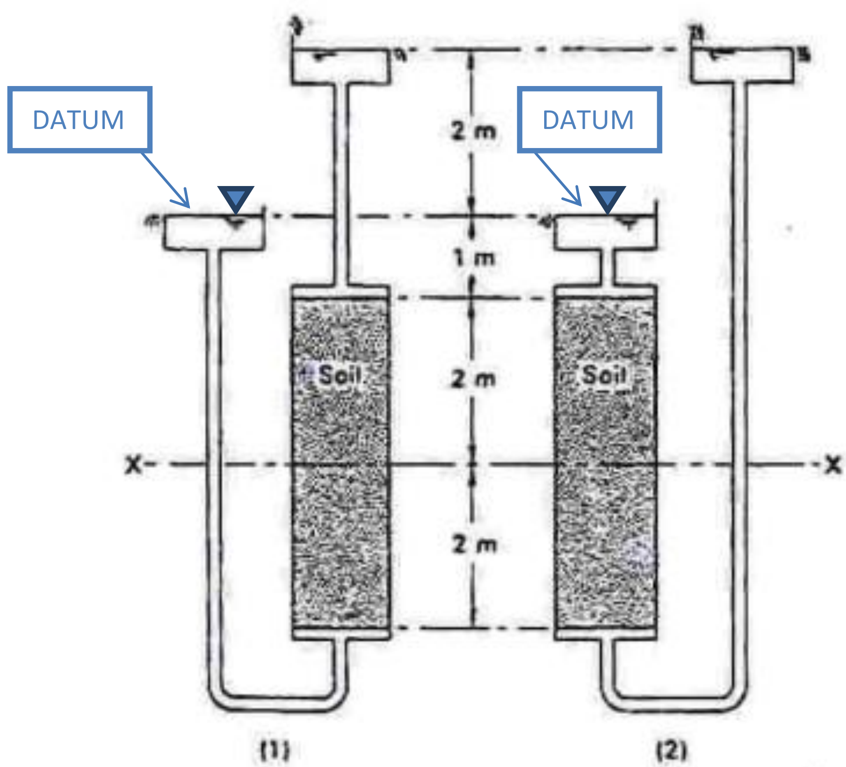
C3) **(a)** the effective normal stress on plane XX by considering pore water pressure;

Using Method 1: using total stress and pore pressure

To find the pore pressure: Consider the total head equation:

$h = z + u/\gamma_w$

where z = elevation head with respect to a selected datum , and γ_w = unit weight of water



Case (1) :

Total head at the top of the soil specimen: $H_{top\ of\ the\ soil} = -1 + 3 = +2\ m$

Total head at the bottom of the soil specimen: $H_{bottom\ of\ the\ soil} = -5 + 5 = 0\ m$

$H_{x-x\prime} = 1$ (total head varies linearly in the direction of flow, for a given soil)

$h = z + u/\gamma_w$ => $1 = -3 + u/\gamma_w$ => $u = 40$ kPa

$\sigma_{xx}' = \sigma - u$

$\sigma_{xx}' = [20*2+3*10] - 40$

$\sigma_{xx}' = 30$ kPa

Case (2) :

$H_{top\ of\ the\ soil} = -1 + 1 = 0\ m$

$H_{bottom\ of\ the\ soil} = -5 + 7 = +2\ m$

$H_{x-x\prime} = 1$

$h = z + u/\gamma_w$ => $1 = -3 + u/\gamma_w$ => $u = 40$ kPa

$\sigma_{xx}' = \sigma - u$

$\sigma_{xx}' = [20*2+1*10] - 40$

$\sigma_{xx}' = 10$ kPa

(b) the effective normal stress on plane XX by considering seepage pressure.

Using Method 2: using buoyant unit weight and seepage force method

$\sigma_{xx}' = z\ [\gamma' \pm i\gamma_w]$

Case (1) :

where "z" is the thickness of soil above the point considered, line XX. If the seepage is downward, as in this problem, the ± sign should be 'positive'.

$i = \Delta h/\Delta L = 2/4 = 0.5$

σ_{xx}' = 2 [**10** + 0.5*10]

σ_{xx}' = 30 kPa

<u>Case (2) :</u>

where "z" is the thickness of soil above the point considered, line XX. If the seepage is upward, as in this problem, the ± sign should be 'negative'.

i = Δh/ΔL = 2/4 = 0.5

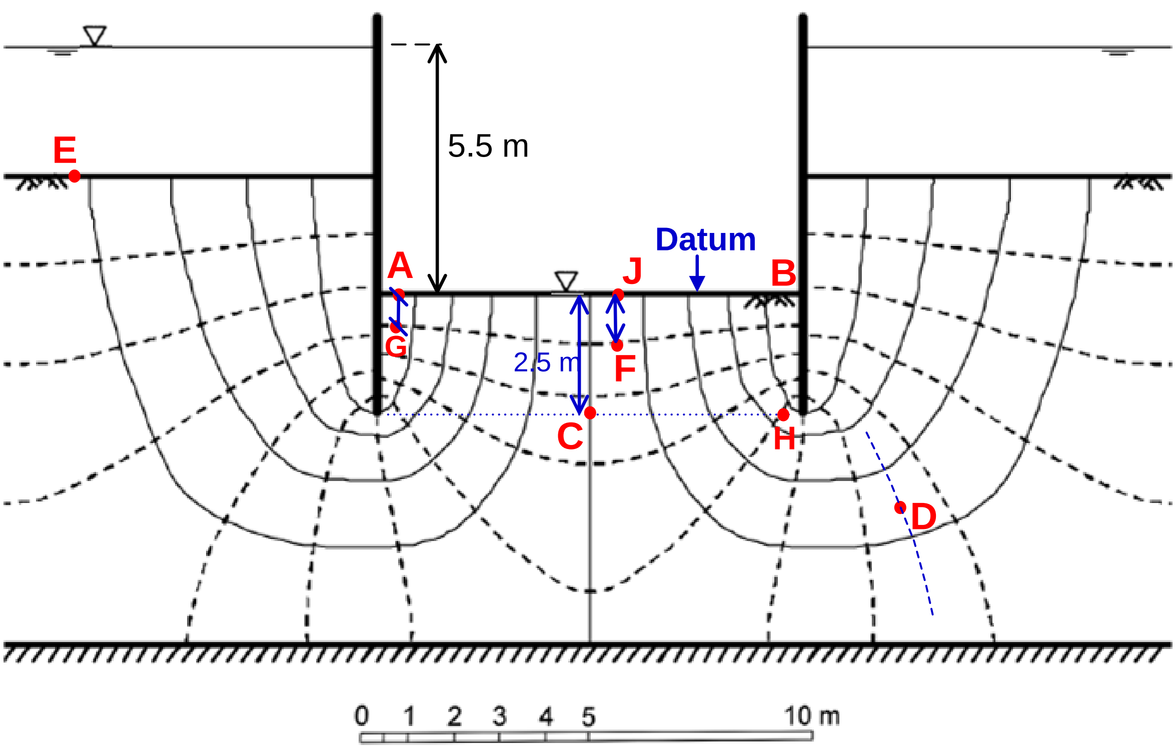
σ_{xx}' = 2 [**10** - 0.5*10]

σ_{xx}' = 10 kPa

<u>Note:</u> In one-dimensional flow problems, you can check your effective stress calculations by solving the problem in two ways, since you should obtain exactly the same result by both methods.

C4)

Flow net is drawn as in the figure below. From the flownet, the number of flow channels N_f = 10, and the number of equipotential drops N_d = 11.



The head loss in this system, (i.e. the head loss between the first and the last equipotential lines) is 5.5 m. This can be easily seen as the difference between the water levels. (Or it can be calculated by writing the total head of points J and point E). Therefore,

To determine the factor of safety against boiling at the surface 'AB':

$$F.S._{boiling} = i_{cr} / i$$

$$i_{cr} = (\gamma_{sat} - \gamma_w) / \gamma_w = \gamma' / \gamma_w = (20\text{-}10)/10 = 1$$

We should find "exit hydraulic gradient" close to surface A-B. For this, we can use two methods: consider one 'square' of the flownet next to the surface A-B, or we can consider a length of soil from surface A-B till the depth of penetration of sheet pile wall.

Considering one 'square' of the flownet next to the surface A-B:

The head loss between adjacent equipotentials is = total head loss / number of equipotential drops = h / N_d = 5.5 / 11 = 0.5

We should use the length of one square in the direction of flow. We can see that the length of one square is not constant near the surface A-B. We can measure from the scaled drawing that the distance JF is = 1 m and the distance AG is = 0.75 m.

- Hydraulic gradient "i" between points J and F is = 0.5 / 1 m = 0.5 . Therefore; $F.S._{boiling} = i_{cr} / i = 1 / 0.5 = 2.0$
- Hydraulic gradient "i" between points A and G is = 0.5 / 0.75 m = 0.67 . Therefore; $F.S._{boiling} = i_{cr} / i = 1 / 0.67 = 1.49$
- In conclusion, boiling does not occur at surface A-B since everywhere near the surface $F.S._{boiling} > 1.0$. The factor of safety against boiling is in the range of 1.49 to 2.0.

Considering a length of soil from surface A-B till the depth of penetration of sheet pile wall (this methods gives "an average exit hydraulic gradient"):

The length of soil from surface A-B till the depth of penetration of sheet pile wall is 2.5 m.

- The head loss between the surface A-B and the point C is 2.2 drops. In between adjacent equipotentials the head loss is = h / N_d = 5.5 / 11 = 0.5 m. Therefore the head loss between the surface A-B and the point C is = 2.2×0.5 = 1.1 m.
  Hydraulic gradient "i" is = 1.1 / 2.5 m = 0.44. Therefore $F.S._{boiling} = i_{cr} / i = 1/0.44 = 2.27$.
- The head loss between the surface A-B and the point H is 4 drops. In adjacent equipotentials the head loss is h / N_d = 5.5 / 11 = 0.5 m. Therefore the head loss between the surface A-B and the point H is = 4×0.5 = 2.0 m.

Hydraulic gradient "i" is = 2.0 / 2.5 m = 0.8. Therefore $F.S._{boiling} = i_{cr} / i = 1/0.8 = 1.25$.

- In conclusion, boiling does not occur at surface A-B. The factor of safety against boiling is in the range 1.25 to 2.27.

Find effective stresses at points C and D. (At point C, if the flow can be assumed as 1D-flow, find effective stress at point C also by using buoyant unit weight and seepage force method).

Effective stress at point D:

$\sigma' = \sigma - u$

Total stress at point D = 3x10 + 7 x 20 = 170 kPa.
To find pore pressure at point D we need to know total head at that point.
Assuming datum as the tailwater elevation (i.e. at the surface A-B):
Total head at surface AB = $h = z + u/\gamma_w = 0 + 0 = 0$
The total head at point D = total head at surface AB + 6.5 drops x (head loss between two equipotentails)= 0 + 6.5 x (5.5/11) = 3.25 m.
$h = z + u/\gamma_w \rightarrow 3.25 = (-4.5) + u/\gamma_w \rightarrow u = 77.5$ kPa

$\sigma' = \sigma - u = 170 - 77.5 = 92.5$ kPa

Effective stress at point C:

Method 1: $\sigma' = \sigma - u$

Total stress at point C = 2.5 x 20 = 50 kPa.

To find pore pressure at point C we need to know total head at that point.
Assuming datum as the tailwater elevation (i.e. at the surface A-B):
Total head at surface AB = $h = z + u/\gamma_w = 0 + 0 = 0$
The total head at point C = total head at surface AB + 2.2 drops x (head loss between two equipotentails)= 0 + 2.2 x (5.5/11) = 1.1 m.
$h = z + u/\gamma_w \rightarrow 1.1 = (-2.5) + u/\gamma_w \rightarrow u = 36$ kPa

$\sigma' = \sigma - u = 50 - 36 = 14$ kPa

Method 2: Since flow between point C and surface AB can be assumed as 1-dimensional flow, we can use the second method, "*using buoyant unit weight and seepage force method".*

$\sigma' = z\,[\gamma' \pm i\,\gamma_w]$

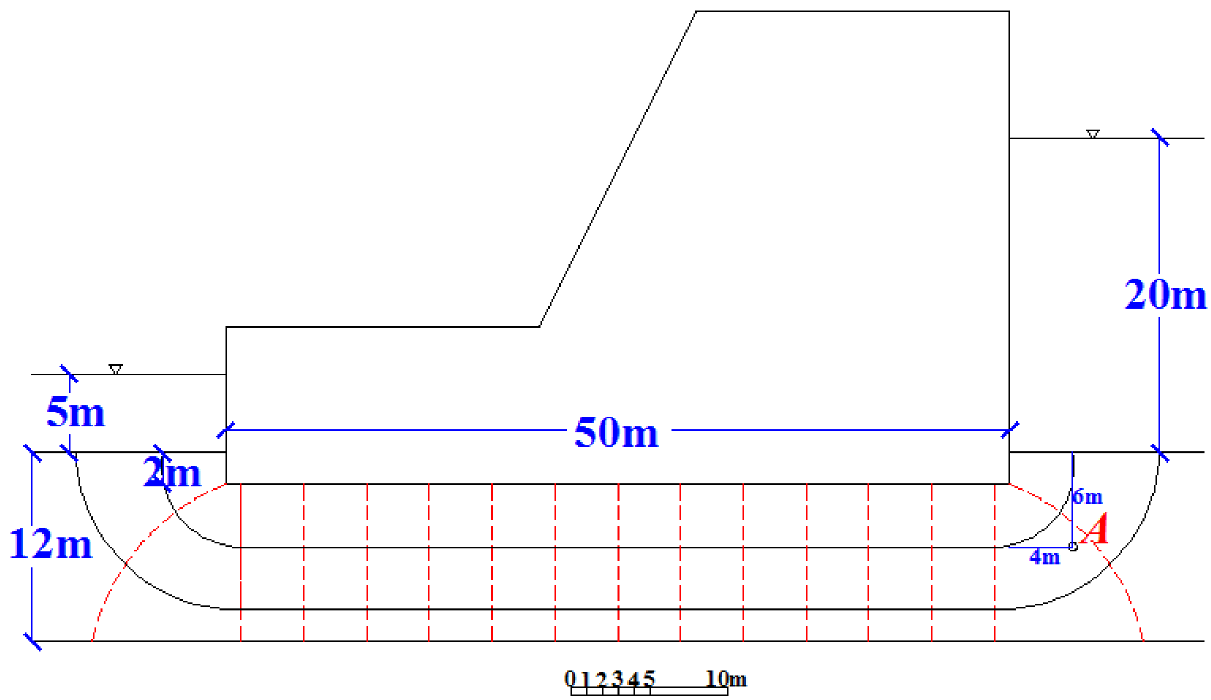
where "z" is the thickness of soil above the point considered. For the hydraulic gradient, i, we can use the length 2.5 m, and consider the head loss between point C and the surface AB:

$$i = \frac{(2.2 \times \frac{5.5}{11})}{2.5\ \text{m}} = 0.44$$

$\sigma' = 2.5\,[(20\text{-}10) - 0.44 \times 10] = 14$ kPa

C5)

a)



b)

L = 150 m, k= $4*10^{-3}$ mm/s

$N_f = 2.5$, $N_d = 16$, $N_f / N_d = 0.15625$

$\Delta h = h/N_d = 15/16 = 0.9375$

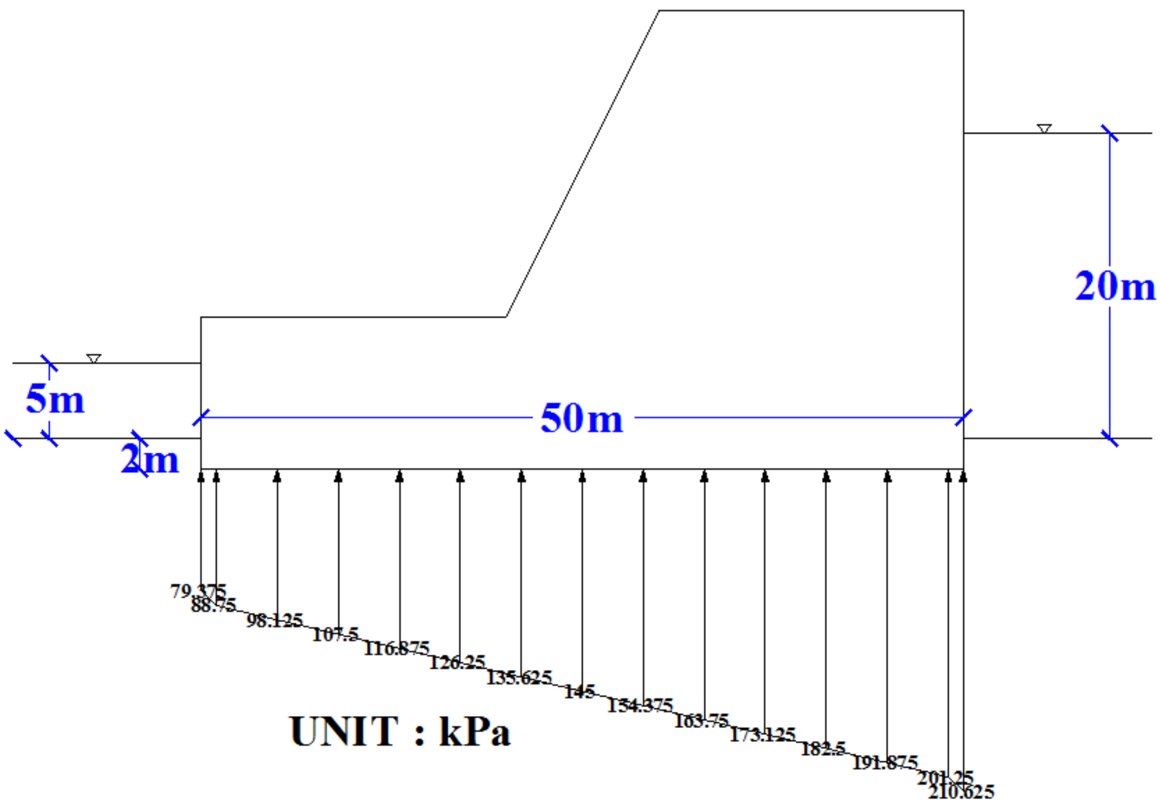
$q = k \times h \times N_f / N_d = 4 \times 10^{-6} \times 15 \times 0.15625 = 9.375 \times 10^{-6}\ m^2 / s$

1 day = 86400 sec

$q = 9.375 \times 10^{-6} \times 86400 = 0.81\ m^2 / s$

$Q = 0.81 \times 150 = 121.5\ m^3 / s$

c)



$u = h_p \times \gamma_w$

$u_1 = [2 + 5 + \Delta h] \times \gamma_w = 79.375$ kPa

$u_2 = [2 + 5 + 2\Delta h] \times \gamma_w = 88.75$ kPa

$u_3 = [2 + 5 + 3\Delta h] \times \gamma_w = 98.125$ kPa

$u_4 = [2 + 5 + 4\Delta h] \times \gamma_w = 107.5$ kPa

$u_5 = [2 + 5 + 5\Delta h] \times \gamma_w = 116.875$ kPa

$u_6 = [2 + 5 + 6\Delta h] \times \gamma_w = 126.25$ kPa

$u_7 = [2 + 5 + 7\Delta h] \times \gamma_w = 135.625$ kPa

$u_8 = [2 + 5 + 8\Delta h] \times \gamma_w = 145$ kPa

$u_9 = [2 + 5 + 9\Delta h] \times \gamma_w = 154.375$ kPa

$u_{10} = [2 + 5 + 10\Delta h] \times \gamma_w = 163.75$ kPa

u_{11} = [2 + 5 + 11Δh] x γ_w = 173.125 kPa

u_{12} = [2 + 5 + 12Δh] x γ_w = 182.5 kPa

u_{13} = [2 + 5 + 13Δh] x γ_w = 191.875 kPa

u_{14} = [2 + 5 + 14Δh] x γ_w = 201.25 kPa

u_{15} = [2 + 5 + 15Δh] x γ_w = 210.62 kPa

d)

i) $\sigma'_A = \sigma - u$

$\sigma_A = 20\,\gamma_w + 6\,\gamma_{sat}$ = 20x10 + 6X21 = 326 kN/m^2

$u_A = h_{PA}\,\gamma_w$ = [20-1.2 Δh + 6] γ_w = 248.75 kN/m^2

σ'_A = 326 – 248.75 = 77.25 kN/m^2

ii) $w_{sub} = volume \times \gamma_{sub} = h \times \gamma_{sub}$ = 6 x (21-10) = 66 kN/m^2

Flow taking place in soil above point A can be considered as 1-dimensional downward flow. Therefore to find hydraulic gradient, we can consider the head loss between the first equipotential line to an imaginary equipotential line passing from point A. The number of equipotential drops from the first equipotential line to an imaginary equipotential line passing from point A is 1.2.

$F_{seep} = i\,\gamma_w \times volume$ = ((1.2 Δh)/6) x 10 x 6 = 11.25 kN/m^2

$\sigma' = (w_{sub} + P_{seep})$/Area = 77.25 / 1 = 77.25 kN/m^2

OR

$\sigma'_A = z(\gamma' + i\,\gamma_w)$ = 6x[(21-10)+ 1.2* Δh/6*10]= 77.25 kN/m^2