CE-378

RECITATION

CE378 | HYDROLOGY AND WATER RESOURCES ENGINEERING

2013-2014 Fall Term

Recitation 1

PROBLEM 1: The values of the components of the hydrologic cycle for the Mogan Lake observed in July are given. Compute the monthly inflow to the lake from the side creeks in lt/sec unit.

- Storage at the beginning of month : 13.4 x 106 m^3 = Sbeginning
- Storage at the end of month : 12.1 x 106 m^3 = Send
- Average surface area of lake in the month : 6.3 km^2
- Total evaporation at a nearby station (from an evaporation pan) : 310 mm
- Evaporation pan correction coefficient : 0.7
- Total precipitation at a nearby station : 6 mm
- Monthly mean discharge released through the control gates (sluice gates) : 15 lt/sec
- Subsurface flow contribution : Negligible

$$\frac{dS}{dt} = I(t) - Q(t)$$

$\Delta S = I + P - E - Q - \cancel{S}$ negligible

$\Delta S = 12.1 * 10^6 - 13.4 * 10^6 = -1.3 * 10^6 \ m^3$ ⇝ change in the reservoir storage

$P = 6mm * 10^{-3} \ m/mm * 6.3 \ km^2 * 10^6 \ m^2/km^2 = 37.8 * 10^3 \ m^3$ ⇝ volume of ppt

$E = 310 * 0.7 \ mm * 10^{-3} \ m/mm * 6.3 * 10^6 \ m^2 = 13.671 * 10^5 \ m^3$ ⇝ volume of actual eva.

$Q = 15 \ lt/sn * 10^{-3} \ m^3/lt * 31 * 86400 = 40.176 * 10^3$ ⇝ volume of outflows

$-1300 * 10^3 = I + 37.8 * 10^3 - 1367.1 * 10^3 - 40.176 * 10^3$

$I = 69.476 * 10^3 \ m^3$ ⇝ inflow during July

$I = 69.476 * 10^3 \ m^3 * 10^3 \ lt/m^3 * \dfrac{1}{31 * 86400 s}$

$I = 25.94 \ lt/sn$

PROBLEM 2: A basin is given below in Figure 1. Total precipitation depths measured during a stormy day at the meteorological stations are also provided in Table 1.

a) Determine the mean areal precipitation of this day using "arithmetic mean method".
b) Determine the representative hyetograph of this basin if the rainfall mass curve of this storm is as shown in Figure 2.

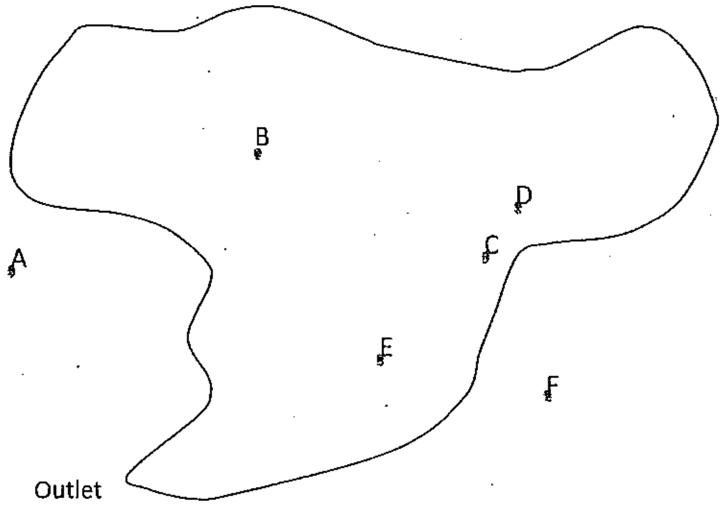


Figure 1 - Basin

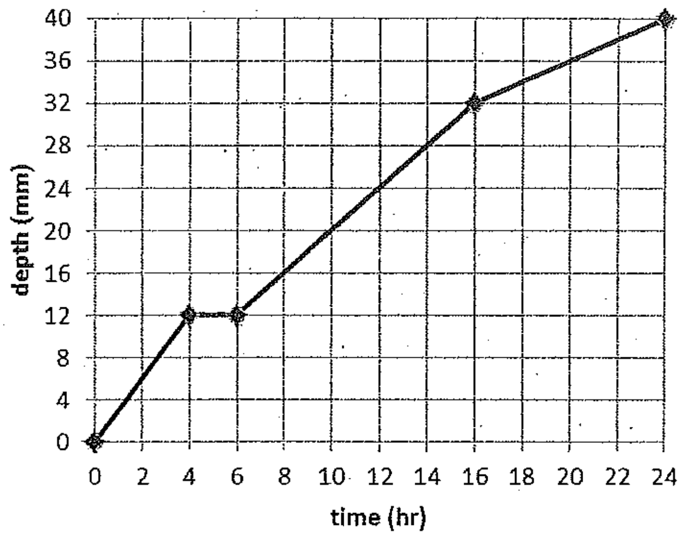


Figure 2 - Rainfall mass curve diagram for the basin

Table 1 - Total (24-hr) precipitation values

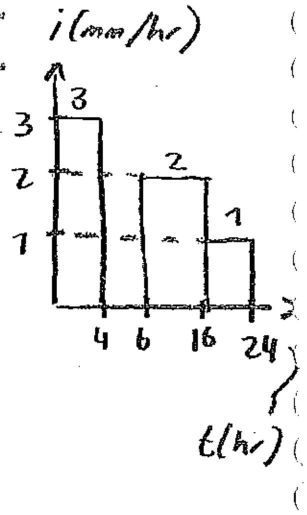
Station	Precipitation (mm)
A	16
B	35
C	48
D	53
E	24
F	45

a) $\bar{P} = \dfrac{\Sigma P_{inside}}{n_{inside}} = \dfrac{P_B+P_C+P_D+P_E}{4}$

$= \dfrac{35+48+53+24}{4} = 40mm$

b)

t(hr)	Σd(mm)	Δd(mm)	Δt(hr)	Δd/Δt = i(mm/hr)
0	0			
		12	4	3
4	12			
		0	2	0
6	12			
		20	10	2
16	32			
		8	8	1
24	40			



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Recitation 2

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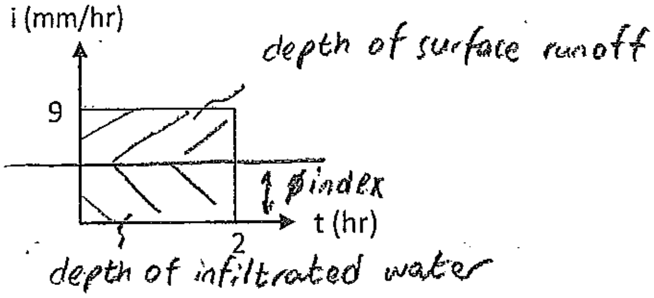
PROBLEM: A uniform storm lasting 2 hours took place over a basin. Hyetograph and total storm hydrograph is given in the figure and table below, respectively. Knowing that the depth of direct runoff is 1.2 cm and assuming the base flow is constant as 5 m³/s, determine;

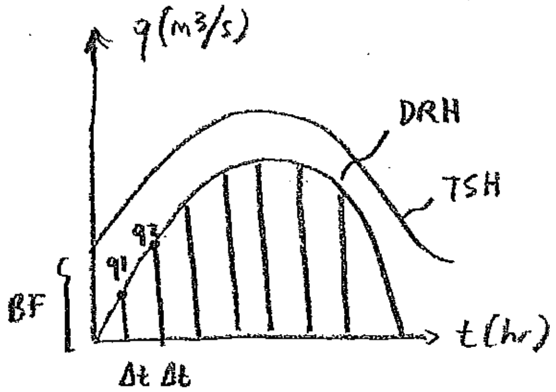
a) Φ-index,
b) the area of the basin.

a) $(9-\phi)\,2 = 12$

$\phi = 3$ mm/hr



t (hr)	TSH (m³/s)	DR (m³/s)
0	5	0
1	29	24
2	41	36
3	21.8	16.8
4	17	12
5	12.2	7.2
6	5	0
		$\Sigma q = 96$ m³/s



$$V = \frac{q_1 \Delta t}{2} + \frac{(q_1+q_2)\Delta t}{2} + \cdots + \frac{(q_{n-1}+q_n)\Delta t}{2} + \frac{q_n \Delta t}{2}$$

$$V = \Sigma q \Delta t$$

$$d = \frac{\Sigma q \Delta t}{A} \rightsquigarrow 0.012 = \frac{96 * 1 * 3600}{A}, \quad A = 28.8 \text{ km}^2$$

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Recitation 3

PROBLEM: You are given UH_3 of a basin. Determine:

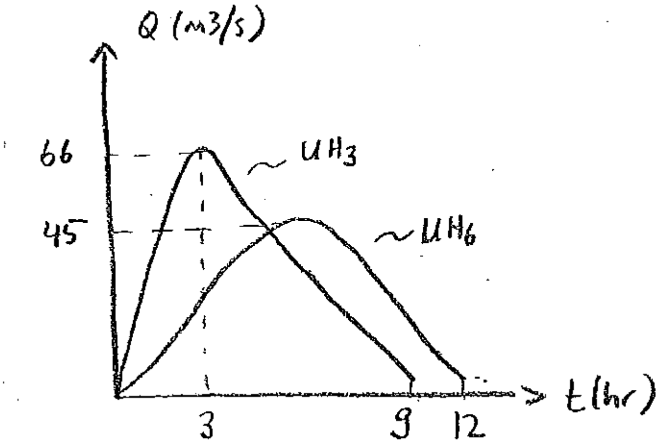
a) UH_6 of this basin using lagging method,

b) UH_2 of this basin using S-curve method,

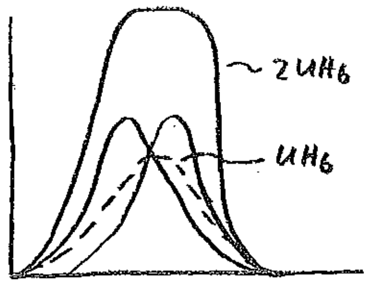
c) area of the basin,

d) the change in Q_p, t_p, and t_b with the change in the duration of excess precipitation.

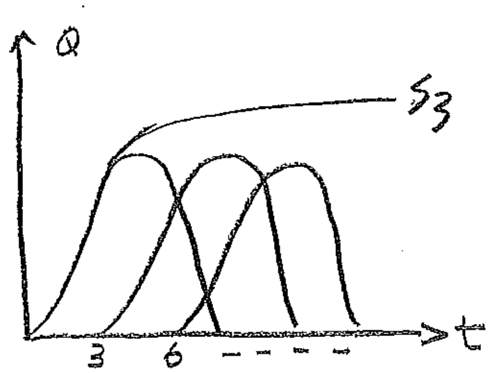
t (hr)	UH_3 (m³/s)	(*) 3hr lag UH_3	(*) $2UH_6$	UH_6 (*1/2)	(*) 6 hr lag UH_3	S_3	(2hr lag) S_3	Δ	UH_2 (*3/2)
0	0	—	0	0	—	0	—	0	0
1	22	—	22	11	—	22	—	22	33
2	50	—	50	25	—	50	0	50	75
3	66	0	66	33	—	66	22	44	66
4	54	22	76	38	—	76	50	26	39
5	34	50	84	42	—	84	66	18	27
6	24	66	90	45	0	90	76	14	21
7	14	54	68	34	22	90	84	6	9
8	6	34	40	20	50	90	90	0	0
9	0	24	24	12	66	90	90		
		14	14	7	54	⋮	⋮		
		6	6	3	34	⋮	⋮		
		0	0	0	24	⋮	⋮		
					14				
					6				
					0				



a) dUH_{tr}, $UH_3 + (3hr\ lag)UH_3 = 2UH_6$



b)

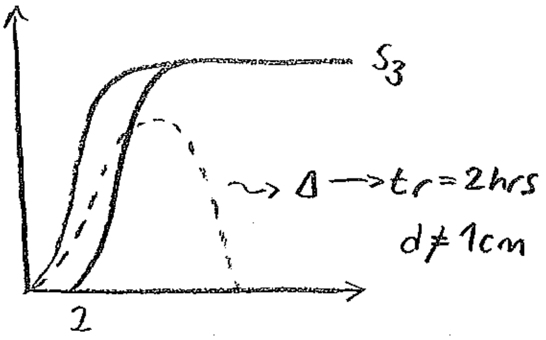


$$\left(\frac{t_b}{t_r}\right)_{UH_3} = \frac{9}{3} = 3 \rightsquigarrow \text{\# of } UH_3 \text{ is needed}$$

$$S_3 = UH_3 + (3hr\ lag)UH_3 + (6hr\ lag)UH_3$$

$$\left[S_{t_1} - (t_2 hr\ lag)S_{t_1}\right] = \Delta$$

$$\left[S_3 - (2hr\ lag)S_3\right] = \Delta$$



$$UH_{t_2} = \frac{t_1}{t_2}\Delta$$

$$UH_2 = \frac{3}{2}\Delta$$

c) $d = \frac{V}{A} \rightsquigarrow \frac{DR}{UH}$

$$d_{UH} = \frac{\Sigma q * \Delta t}{A_{basin}}$$

$$0.01\,m = \frac{270\,m^3/s * 1 * 3600s}{A_{basin}} \rightsquigarrow A_{basin} = 97\,200\,000\,m^2$$

$$= 97.2\,km^2$$

d)

	$t_r(hr)$	$Q_p(m^3/s)$	$t_p(hr)$	$t_b(hr)$
UH_2	2	75	2	8
UH_3	3	66	3	9
UH_6	6	45	6	12

* $\boxed{t_r \uparrow\ Q_p \downarrow\ t_p \uparrow\ t_b \uparrow}$

If duration elongates, peak rates decrease.

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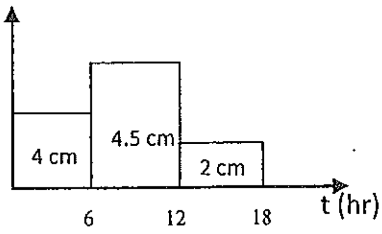
Recitation 4

PROBLEM 1: You are supposed to design a weir at the outlet of the basin given below. The design must be conducted according to the given storm hyetograph. Since there are no available recorded runoff data at the closest discharge observation station, synthetic unit hydrograph must be obtained for the basin. The characteristics of the basin are given below. Assume the baseflow equals to 15 m^3/s at the outlet of the basin.

a) Find the ordinates of the unit hydrograph that can be obtained from the given information.

b) Find the peak discharge of the design hydrograph.

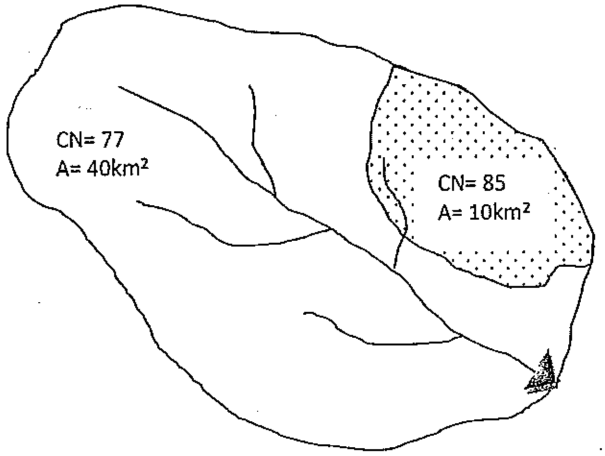


Area of the basin= 50 km^2

Main stream length= 14 km

Bed slope of the main stream= 1.4%

1m= 3.28 ft



$$Q_p = \frac{2.08A}{t_p}, \quad t_L = \frac{L^{0.8}(S+1)^{0.7}}{1900S_h^{0.5}}, \quad S = \frac{1000}{CN} - 10, \quad t_p = \frac{t_r}{2} + t_L, \quad t_b = 2.67t_p$$

$$CN_{ave} = \frac{\sum_{i=1}^{n} CN_i A_i}{\sum_{i=1}^{n} A_i}$$

$$\left.\begin{array}{l} 20\%\ CN = 85 \\ 80\%\ CN = 77 \end{array}\right\} CN_{ave} = 0.2 * 85 + 0.8 * 77 = 78.6 \quad , \quad \begin{array}{l} L = 14\ km = 45920\ ft \\ S_h = 1.4\ \% \end{array}$$

$$S = \frac{1000}{78.6} - 10 = 2.72\ in \qquad Q_p = \frac{2.08 * 50}{9} = 11.56\ m^3/s$$

$$t_L = \frac{(45920)^{0.8}(2.72+1)^{0.7}}{1900\ (1.4)^{0.5}} = 6\ hr \qquad t_b = 2.67 * 9 = 24\ hr$$

$$t_p = \frac{6}{2} + 6 = 9\ hr$$

PROBLEM 2: Stage-storage and stage-discharge relationships for an uncontrolled spillway can be calculated with the formulae $S = 523800H^{3/2}$ and $Q = 48.50H^{3/2}$, where H is the elevation (stage) of the outflow above the spillway crest (m), S is the storage (m^3) and Q is the rate of outflow (m^3/s). In order to obtain the outflow hydrograph ordinates at the outlet of the reservoir (assuming that the reservoir is full), routing procedure is to be applied following the steps given below;

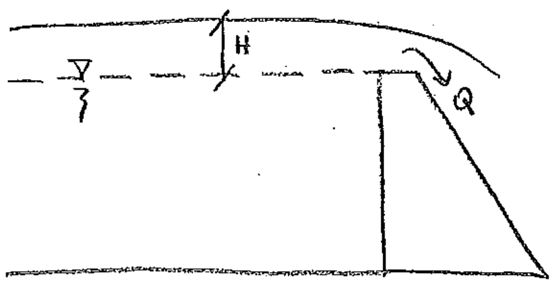
a) determine the Q vs $\frac{2S}{\Delta t} \pm Q$ relationships,

b) apply reservoir routing and obtain the outflow hydrograph,

c) find attenuation and translation.

$$\underbrace{(I_1 + I_2) + \left(\frac{2S_1}{\Delta t} - Q_1\right)}_{\text{known}} = \underbrace{\left(\frac{2S_2}{\Delta t} + Q_2\right)}_{\text{unknown}} \qquad S = Q \cdot \Delta t \qquad \frac{\Delta S}{\Delta t} = \bar{I} - \bar{Q} \qquad S = f(Q, \Delta t)$$

t (hr)	Inflow (m^3/s)	I_1+I_2	$2Q_1$	$4Q_2$	Q_2	
0	10	—	—	—	10	
2	100	110	20	130	32.50	
4	221	321	65	386	96.50	
6	342	563	193	756	189	
8	240	582	378	960	240	
10	155	395	480	875	218.75	
12	90	245	437.5	682.5	170.63	
14	44	134	341.25	475.25	118.81	
16	10	54	237.63	291.63	72.91	

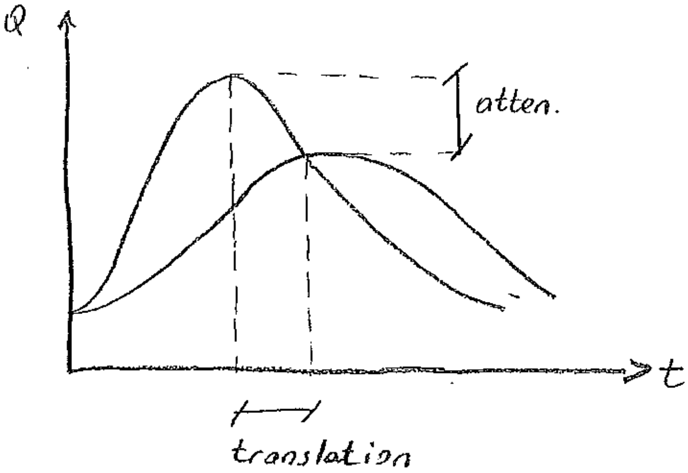


$H^{3/2} = Q/48.5$

$S = 523800\,(Q/48.5) = \underline{10800\,Q}$

Similarly

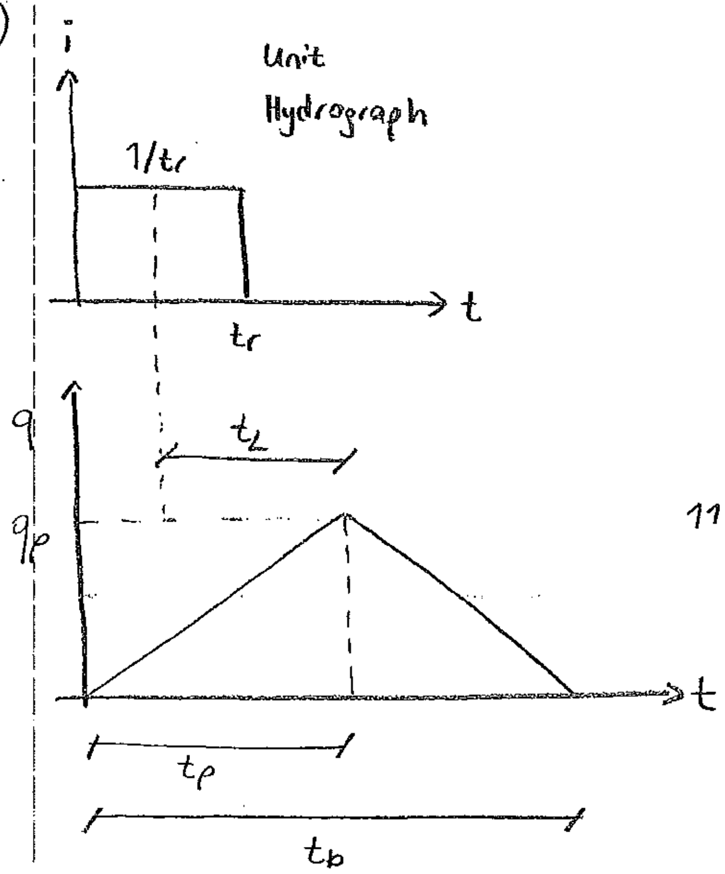
$$\frac{2S_1}{\Delta t} - Q_1 = \frac{2\,(10800Q_1)}{2 * 3600} - Q_1 = 2Q_1 \quad , \quad \frac{2S_2}{\Delta t} + Q_2 = 4Q_2$$

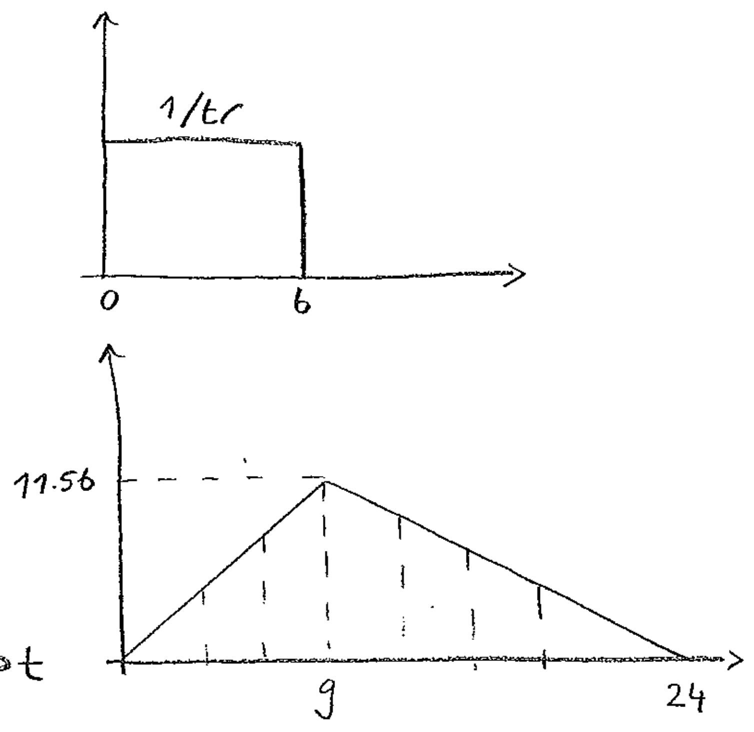


attenuation $= 342 - 240 = 102\ m^3/s$

translation $= 8 - 6 = 2\ hr$

(1)





t(hr)	UH6	4UH6	6Hr lag 4.5 UH6	12 Hr lag 2UH6	DR (m^3/s)	TSH (m^3/s)
0	0	0	—	—	0	15
3	3.85	15.41	—	—	15.41	30.41
6	7.71	30.83	0	—	30.83	45.83
9	11.56	46.24	17.34	—	63.58	78.58
12	9.25	36.99	34.68	0	71.67	86.67
15	6.94	27.74	52.02	7.71	87.47	(102.47)*
18	4.62	18.50	41.62	15.41	75.53	90.53
21	2.31	9.25	31.21	23.12	63.58	78.58
24	0.00	0	20.81	18.50	39.30	54.30
			10.40	13.87	24.28	39.28
			0	9.25	9.25	24.25
				4.62	4.62	19.62
				0	0	15

$DR = 4\,UH_6 + (6Hr\ lag)\,4.5\,UH_6 + (12\,Hr\ lag)\,2\,UH_6$

$Q_p = Q_d = 102.47\ m^3/s$

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Recitation 6

PROBLEM 1: A 60 m-high (from thalweg) concrete gravity dam will be designed. Carry out stability analysis under extreme loading. Assume that the whole dam acts as a monolithic structure. The dam is located in 4th earthquake zone.

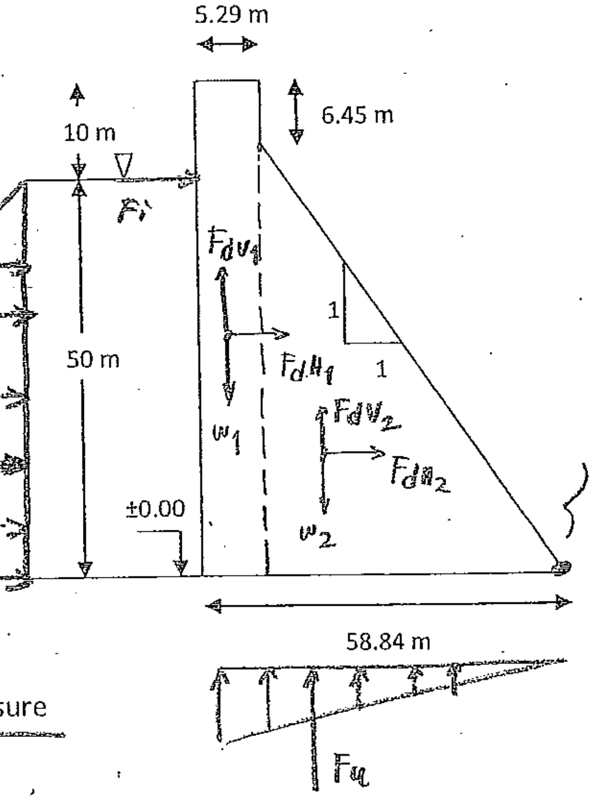
- Normal operating level: **50 m**
- Shear strength at the base level: **5 MPa**
- Compressive strength of concrete and foundation material: **30 MPa** and **50 Mpa**, respectively.
- Coefficient of friction at the base level: **0.75**
- Specific weights of concrete and water: **24 kN/m³** and **10 kN/m³**, respectively.
- Submerged specific weight of sediment accumulated in the reservoir: **11 kN/m³** with θ=32°, h_s=3 m
- Earthquake coefficients: k_h=0.1, k_v=0.05
- Ice force: **100 kN/m**
- There is no tailwater.
- * Drainage system reduces the uplift pressure by 40%.

FW

FH

hydrodinamic force

hydrostatic force



Bu noktaya göre dönme etkisi!

$$C = 0.7\left(1-\frac{\theta'}{90}\right) = 0.7$$

$$K_a = \frac{1-\sin\theta}{1+\sin\theta} = 0.31$$

32°

Extreme Loading

$FS_s > 1.0$ $\qquad \sigma_{max\ conc} < \sigma_c$

$FS_{ss} > 1.0$

$FS_o > 1.2$ $\qquad \sigma_{max\ foundation} < \frac{\sigma_f}{1.3}$

$$FS_{ss} = \frac{f\,\Sigma V + r\,A\,\tau_s}{\Sigma H} = \frac{0.75 * 31101.39 + 1 * 58.84 * 5000}{18088.73} = 17.55$$

shear & sliding

$17.55 > 1.0$ ✓

r: factor to express maximum allowable shear stress

r = 1.0 (extreme loading case)

A: area of the shear plane

τ_s = 5 MPa

Forces (kN/m)	Moment Arm about "O" (m)	Moment (kNm/m)
$W_1 = \gamma_c h b_1 = 24*60*5.29 = 7617.6 \downarrow$	$5.29/2 + 53.55 = 56.2$	428109.12 ↺
$W_2 = \gamma_c h_2 b_2 / 2 = 24*(53.55)^2*0.5 = 34411.23 \downarrow$	$53.55*(2/3) = 35.7$	1228480.91 ↺
$F_h = ½ (h_u)(\gamma_w h_u) = 0.5*50*10*50 = 12500 \longrightarrow$	$50*(1/3) = 16.67$	208375 ↻
$F_u = \phi/2\, \gamma_w h_u B = 0.6*0.5*10*50*58.84 = 8826 \uparrow$	$58.84*(2/3) = 39.23$	346243.98 ↻
$F_i = 100 \longrightarrow$	50	5000 ↻
$F_s = ½ \gamma_s h_s^2 K_a = 0.5*11*(3)^2*0.31 = 15.35 \longrightarrow$	$3*(1/3) = 1$	15.35 ↻
$F_w = 0.726\, C k \gamma_w h_u^2 = 0.726*0.7*0.1*10*50^2 = 1270.5 \longrightarrow$	$50*0.412 = 20.6$	26172.3 ↻
$F_{dh1} = k_h W_1 = 0.1*7617.6 = 761.76 \longrightarrow$	$60*0.5 = 30$	22852.8 ↻
$F_{dh2} = k_h W_2 = 0.1*34411.23 = 3441.23 \longrightarrow$	$53.55*(1/3) = 17.85$	61243.99 ↻
$F_{dv1} = k_v W_1 = 0.05*7617.6 = 380.88 \uparrow$	56.2	21405.56 ↻
$F_{dv2} = k_v W_2 = 0.05*34411.23 = 1720.56 \uparrow$	35.7	61243.99 ↻

earthquake forces

$\Sigma M_R = 1656590.03$ kNm/m

$\Sigma M_O = 752912.97$ kNm/m

$\Sigma H = 18088.73$ kN/m

$\Sigma V = 31101.39$ kN/m

$$FS_{sliding} = \frac{f \Sigma V}{\Sigma H} = \frac{0.75*31101.39}{18088.73} = 1.29 > 1.0$$

$$FS_{overturning} = \frac{\Sigma M_R}{\Sigma M_O} = 2.2 > 1 \checkmark$$

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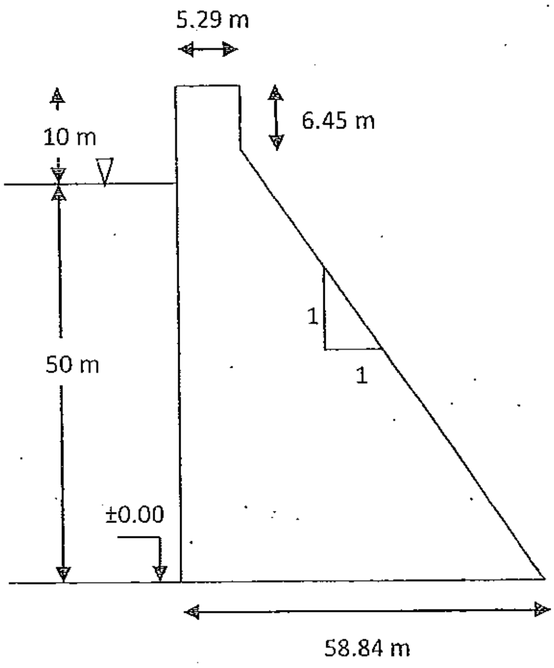
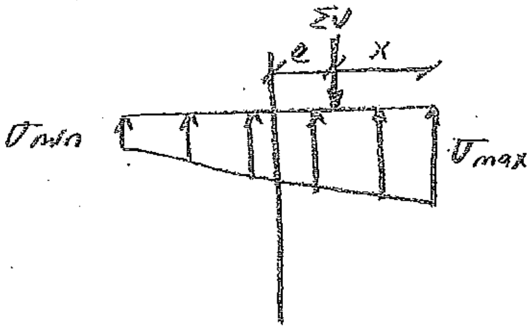


Base Pressure Distribution



$$\bar{x} = \frac{\Sigma M_r - \Sigma M_o}{\Sigma V} = 29.06 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = \frac{58.84}{2} - 29.06 = 0.36 \text{ m}$$

$$\boxed{\sigma_{max,min} = \frac{\Sigma V}{A} \pm \frac{Mc}{I}}$$

$$M = \Sigma V \cdot e = 11196.5 \text{ kNm}$$

$$C = B/2 = 29.42 \quad , \quad I = B^3/12 = \frac{58.84^3}{12} = 16976.05 \text{ m}^3$$

$$\sigma_{max,min} = \frac{31101.39}{58.84} \pm \frac{11196.5 * 29.42}{16976.05}$$

$$\sigma_{max} = 547.98 \text{ kN/m}^2 \leq \sigma_c \checkmark \quad , \quad \leq \sigma_f/1.3 \checkmark$$

$$\sigma_{min} = 509.17 \text{ kN/m}^2 > 0 \checkmark$$

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Recitation 7

PROBLEM: PROBLEM: A 200 m high dam will be constructed in a valley. There are two possible sites, 1 km apart from each other, having desirable geological formation, i.e. axis 1 and axis 2 shown in the Figure 1. The following possible alternatives will be constructed in this design.

1. Design of a concrete gravity dam using usual loading at axis 1 or axis 2. In the design take: t_c=10 m, H*=10 m, m=0, n=1, f=0.75, τ_s=5 MPa, σ_c=30 MPa, ϕ=0.6, σ_f=60 MPa, γ_c=24 kN/m^3, γ_w=10 kN/m^3, normal operating level=175 m, no tailwater, ignore silt and ice forces.

2. Design of an arch dam at axis 1 and axis 2, separately, using the simplified arch-rib analysis. In the design, consider: t_c=6 m, γ_w=10 kN/m^3, σ_{all}=6000 kN/m^2, and θ_a=133°

Compare the results according to the cross-sectional details of the designs and discuss your findings.

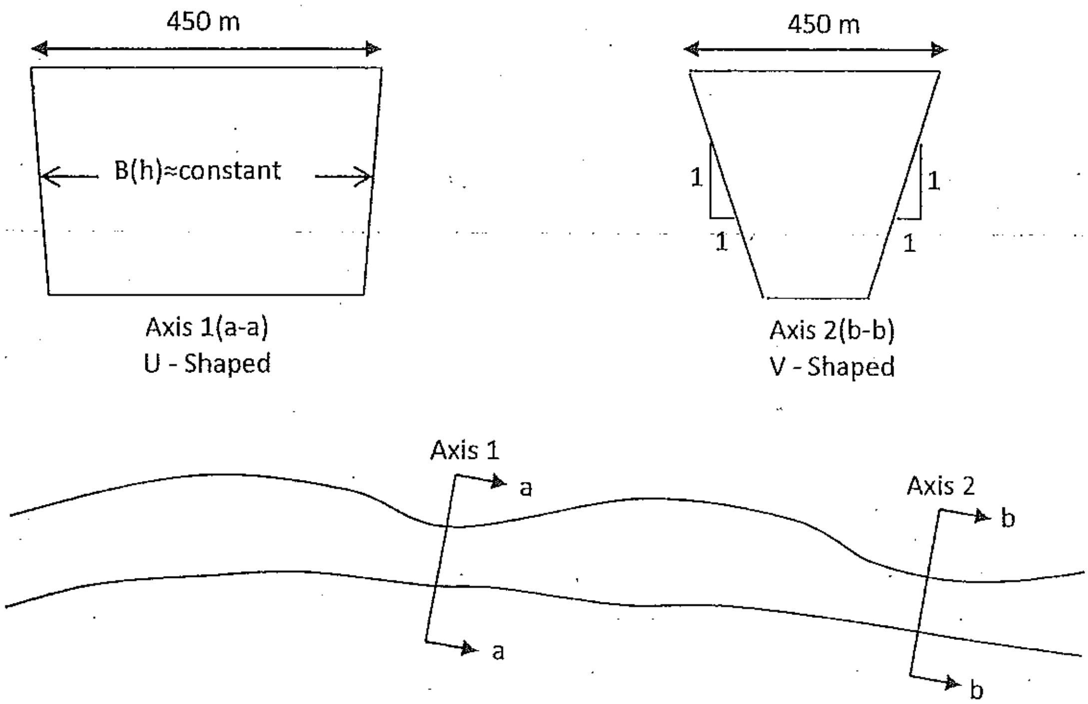
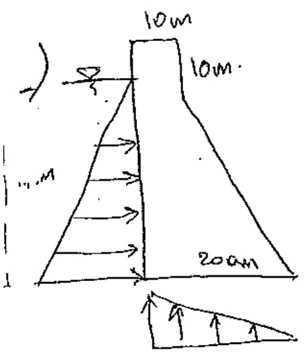


Figure 1. Cross sections of proposed dam locations



$FS_o = 2,8 > 2$ ✓ $\sigma_{max} = 2432,96 < \sigma_c/4$ ✓

$FS_s = 1,84 > 1,5$ ✓ $\sigma_{min} = 1329,04 > 0$ ✓

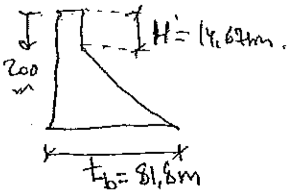
$FS_{ss} = 7,00 > 3,0$ ✓

$B = 450\,m$ $\quad r = \frac{B}{2\sin\theta/2} = \frac{450}{2\sin 133/2} = 245.35\,m$ $\quad t = \frac{\gamma h r}{\sigma_{all}} = \frac{10h \cdot 245{,}35}{6000} = 0{,}409h$

$t_b = 0{,}409 \times 200 = 81{,}8\,m$

$6 = 0{,}409 H_c$

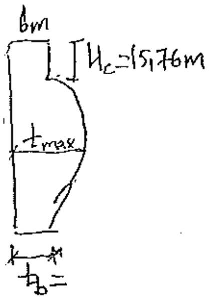
$H_c = 14{,}67\,m.$



Axis 2

$B(h) = 450 - 2h$

$r(h) = \frac{B(h)}{2\sin\theta/2} = \frac{450-2h}{2\sin\frac{133}{2}} = 245{,}37 - 1{,}09h$; $t(h) = \frac{\gamma h r}{\sigma_{all}} = \frac{10 \times h \times (245{,}35 - 1{,}09h)}{6000} = 0{,}409h - 0{,}0018h^2$



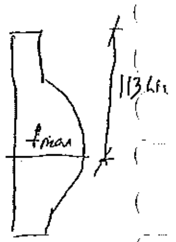
$t_b = t(200) = 0{,}409 \times 200 - 0{,}0018(200)^2 = 9{,}8\,m.$

$t_{H_c} = 6 = 0{,}409H_c - 0{,}0016H_c^2$

$H_c = 15{,}76\,m$

at t_{max} $\frac{dt}{dh} = 0$ $\quad \frac{dt}{dh} = 0{,}409 - 0{,}0036h \rightarrow h = 113{,}61\,m$

$t_{max} = 0{,}409 \times 113{,}61 - 0{,}0018(113{,}61)^2 = 23{,}23\,m.$



Selection criterion:

Dimensions of the gravity dam are independent of geometric characteristics of the valley. Arch dam's geometry strongly depends on geometry of valley. From view point of economy and safety, an arch dam to be constructed at axis 2 is preferable to the other alternative.

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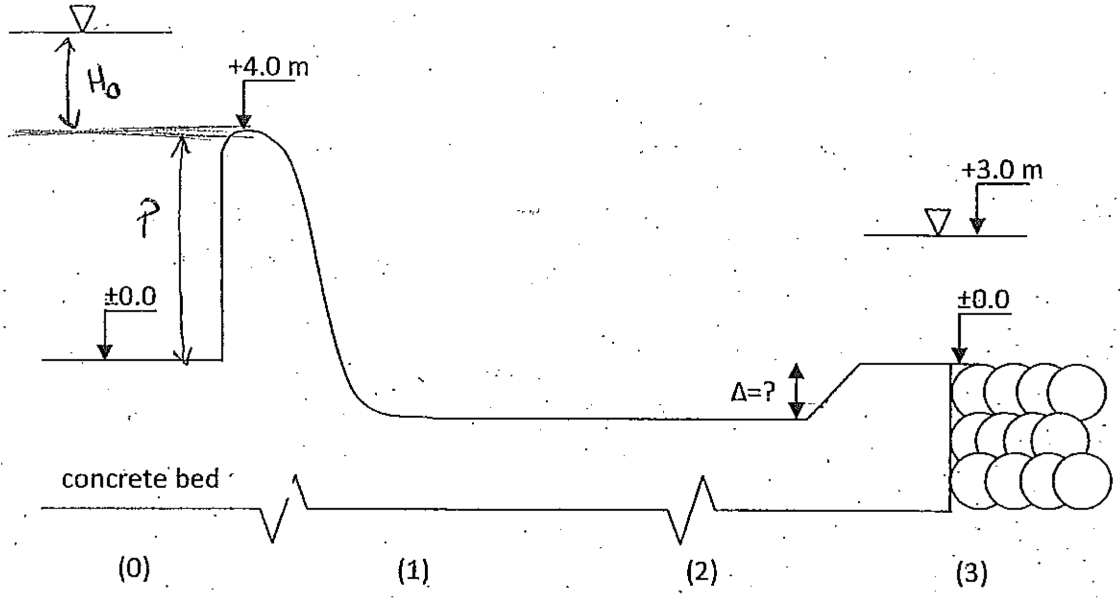
Recitation 8

PROBLEM: An uncontrolled overflow spillway, 4.0 m high and 50 m long will be designed to evacuate $Q_0 = 450\ m^3/s$. Ignore the head losses over the spillway face and at a possible end sill. Determine:

a) the design spillway head,
b) the required lowering of the river bed for the stilling basin,
c) the type and dimensions of the stilling basin.



$$y_2 = \frac{y_1}{2}\left(\sqrt{1+8F_{r1}^2}-1\right) \qquad \Delta E = \frac{(y_2-y_1)^3}{4y_1y_2}$$

a) $Q_0 = C_0 L_0 H_0^{3/2}$

Assumed H_0	P/H_0	C_0	Calculated Q_0
2.00 m	2	2.17	306.88
2.50 m	1.6	2.16	426.91
2.59	1.54	2.16	450.17 $m^3/s \simeq 450$

$E_0 = H_0 + P = 2.59 + 4 = 6.59\ m$

b) $q = \dfrac{Q_0}{L} = \dfrac{450}{50} = 9\ m^3/s/m$

$E_3 = y_3 + \dfrac{u_3^2}{2g} = 3 + \dfrac{3^2}{2 \times 9.81} = 3.46\ m$

$u_3 = \dfrac{q_3}{y_3} = \dfrac{9}{3} = 3\ m/s$

$Fr_3 = \dfrac{u_3}{\sqrt{g y_3}} = 0.55$ subcritical

$\Delta E = E_0 - E_3 = 6.59 - 3.46$
$= 3.13\ m$

$$\Delta E = \frac{\left[\frac{y_1}{2}\left(\sqrt{1+\frac{8q^2}{g y_1^3}}-1\right)-y_1\right]}{4y_1 \cdot \frac{y_1}{2}\left(\sqrt{1+\frac{8q^2}{g \cdot y_1^3}}-1\right)} = 3.13\ m \implies y_1 = 0.778\ m$$

$$Fr_1^2 = \frac{q^2}{g y_1^3} = 4.187^2$$ supercritica

$u_1 = 11.57\ m/s$

$$y_2 = \frac{0.778}{2}\left(\sqrt{1+8\times 4.187^2}-1\right) = 4.23\ m$$

$$y_2 + \frac{u_2^2}{2g} = \underbrace{y_2 + \frac{q^2}{2g\,y_2^2}}_{(2)} = \underbrace{\Delta + E_3}_{(3)}$$

$$4.23 + \frac{q^2}{2\times 9.81\times 4.23^2} = \Delta + 3.46$$

$$\boxed{\Delta = 1.00\ m}$$

c) $Fr_1 = 4.187$

$u_1 = 11.57\ m/s$

$2.5 < Fr_1 < 4.5$, $u_1 < 15\ m/s$

$\hookrightarrow$ USBR Type IV Basin

$L_{IV} = 6.1\,y_2 = 6.1\times 4.23 = 25.8\,m$

$H_{\substack{chute\\block}} = 2y_1 = 2\times 0.778 = 1.56\,m$

$H_{\substack{sill\\at\\toe}} = h_4 = y_1\,(9+Fr_1)/9 = 0.778\,(9+4.187)/9$

$= 1.14\,m$

13.12.2013

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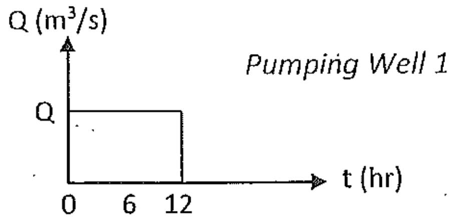
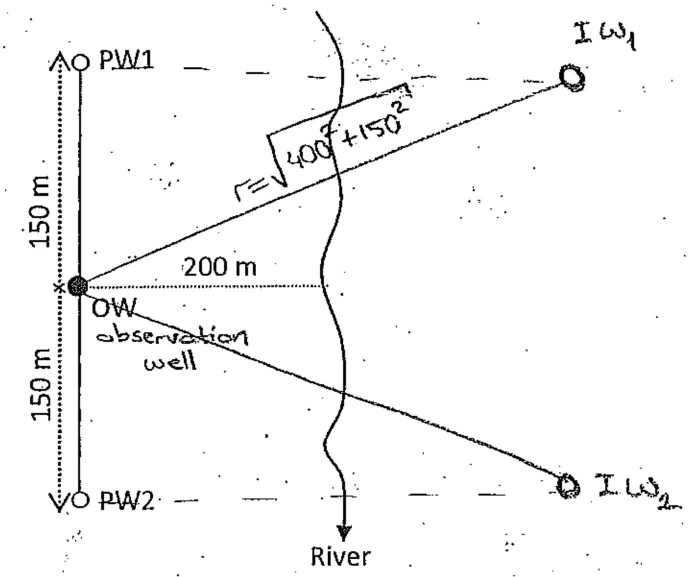
Recitation 9

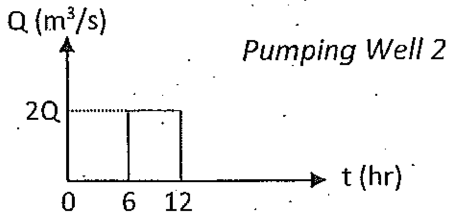
PROBLEM 1: Two pumping wells penetrating completely a confined homogenous and isotropic aquifer are located near a river as shown below. Both wells have the same radius of 0.15 m and are pumped according to the given schedules. The coefficient of storage and transmissivity for the aquifer are 0.0004 and 0.002 m²/s respectively. If the drawdown in the observation well, is not to exceed 2 m at t=12 hr, what should be the maximum values for discharges Q_1 and Q_2, where $Q_2=2Q_1$.

$$u=\frac{r^2 S}{4Tt} \qquad s=\frac{Q}{4\pi T}W(u)$$

T → transmissivity







$$u=\frac{r^2\times 0.0004}{4\times 0.002\times t\times 3600}=1.389\times 10^{-5}\frac{r^2}{t}$$

	r^2 (m²)	t (hr)	u	w(u)	Q
PW1	150	12	2.6×10^{-2}	3.1006	+Q
PW2	150	6	5.2×10^{-2}	2.4316	+2Q
IW1	427.2	12	2.11×10^{-1}	1.1836	−Q
IW2	427.2	6	4.22×10^{-1}	0.6681	−2Q

$$s=\frac{\Sigma Q.w(u)}{4\pi.T}=\frac{3.1006\times Q+2.4316\times 2Q-1.1836Q}{4\pi\times 0.002}$$

$-0.6681\ldots$

$$s=\frac{5.444Q}{4\pi\times 0.002}=2\,m$$

$$\Longrightarrow Q=9.233\times 10^{-3}\,m/s$$

$$=9.233\ lt/s$$

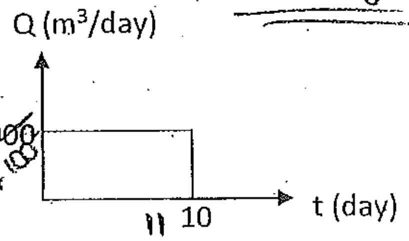
$$2Q=18.466\ lt/s$$

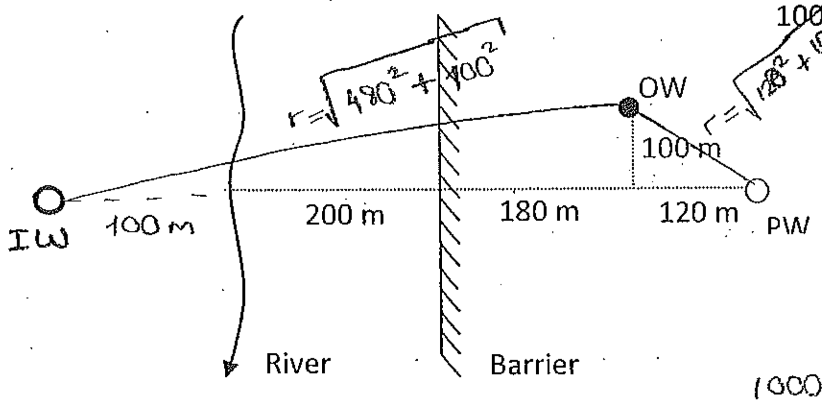
PROBLEM 2: A pumping well (PW) fully penetrates a leaky confined aquifer, which is bounded from the west by a barrier and a river boundary as shown schematically below. Aquifer characteristics are K = 14 m/day and S = 0.0001, and thickness of the aquifer (b) is 40 m. Hydraulic conductivity (K') and thickness of the overlying aquitard (b') are $5*10^{-6}$ m/day and 25 m, respectively. Determine the drawdown at the observation well (OW) 15 days after pumping starts.

$t = 15$ days $\Rightarrow s =$

$$T = bK \qquad B^2 = \frac{Tb'}{K'} \qquad u = \frac{r^2 S}{4Tt} \qquad s = \frac{Q}{4\pi T} W(u, r/B)$$

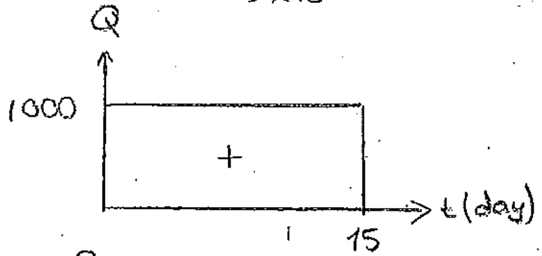
aquitard

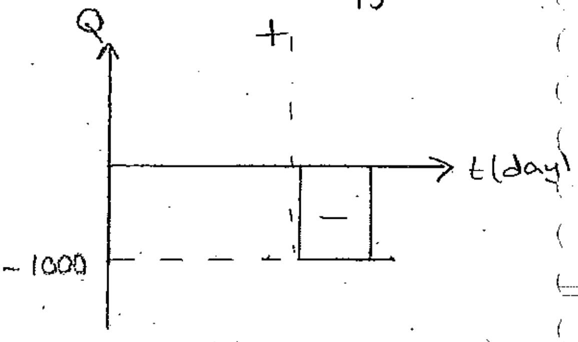




$T = b.K = 40 \times 14 = 560\ m^2/$

$B^2 = \frac{560 \times 25}{5 \times 10^{-6}} \Rightarrow B = 5291$





	r(m)	t(hr)	u	r/B
PW/1	24 400	15	7.26×10^{-5}	2.95×10^{-3}
PW/2	24 400	5	2.48×10^{-4}	2.95×10^{-3}
IW/1	240 400	15	7.15×10^{-4}	9.27×10^{-3}
IW/2	240 400	5	2.15×10^{-3}	9.27×10^{-3}

	W(u, r/B)	Q
PW/1	8.926	+1000
PW/2	7.874	−1000
IW/1	6.640	+1000
IW/2	5.533	−1000

$$s = \frac{1000}{4\pi \cdot 560} \left[(8.925 - 7.874) + (6.640 - 5.533) \right]$$

u \ r/B	0.002	0.004
7×10^{-5}	8.976	8.934
1×10^{-4}	8.623	8.594

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Recitation 10

PROBLEM 1: Analyze a gravity pipeline system that feeds the municipal water network of a town.

a) Determine the maximum discharge that can be drawn from the pipeline (satisfying all the operational requirements and limitations).

b) Using geometric extrapolation, determine until when the system meets municipal requirements of this town.

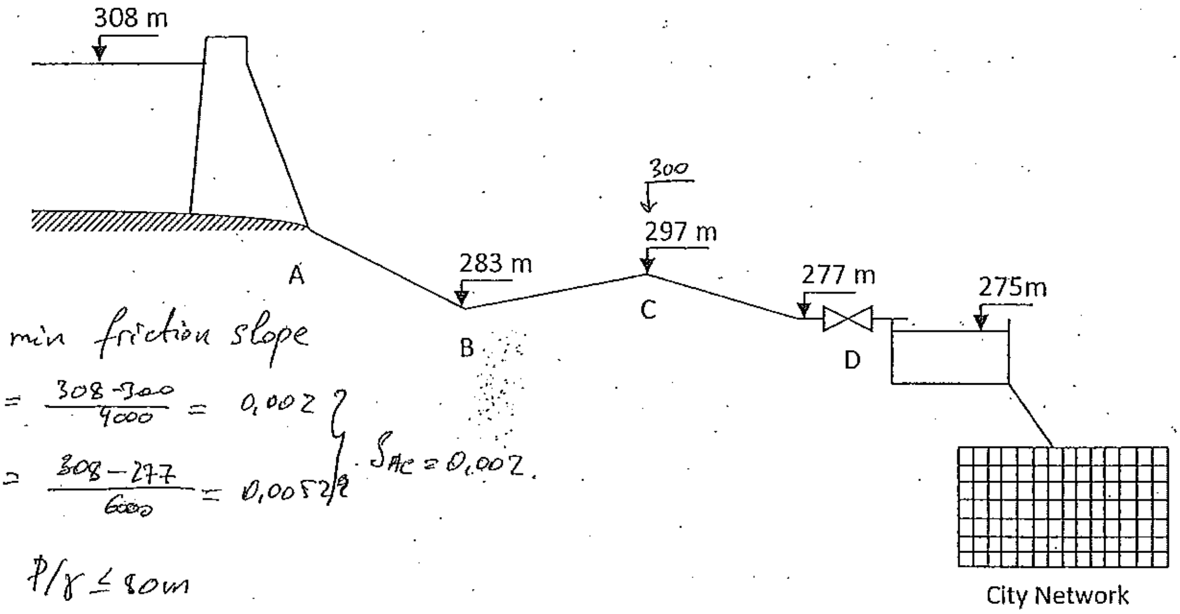
Constant reservoir level= 308 m,

For all pipe segments: f=0.02, ϕ=400 mm, L=2000 m

Operational requirements: 0.5 m/s≤ u ≤2.0 m/s 3 m≤ P/γ ≤80 m

Population: P_{2000}=23000, P_{2010}=30000, $P.F_{day}$=1.5, $P.F_{hour}$=2.5

$$h_f = \frac{8fL}{g\pi^2 D^5} Q^2$$



a) Det min friction slope

$S_{AC} = \frac{308-300}{4000} = 0.002$

$S_{AD} = \frac{308-277}{6000} = 0.0052$ } $S_{AC} = 0.002$.

$3m \le P/\gamma \le 80m$

$h_{f_{AC}} = 308-300 = 8m \rightarrow 8 = \frac{8\times 0.02\times 4000}{9.81\times\pi^2\times 0.4^5}\times Q^2 \Rightarrow Q = 0.1113\ m^3/s \rightarrow Q = 111.3\ lt/s.$

Velocity check: $u = \frac{Q}{A} = \frac{Q}{\pi D^2/4} = \frac{0.1113}{\pi\times 0.4^2/4} = 0.886\ m/s$. / Pressure check; $H_A = \left(\frac{P}{\gamma}\right) + z_B + h_{f_{AB}}$

$308 = (P/\gamma)_B + 283 + 4 \rightarrow h_{f_{AC}} = \frac{h_{f_{AB}}}{2}$.

$(P/\gamma)_B = 21m$.

b) $k_g = \frac{\ln P_2 - \ln P_1}{t_2 - t_1}$

$P_{2000} = 23000$ $P_{2010} = 30000$

$k_g = \frac{\ln 30000 - \ln 23000}{2010-2000} = 0.02657$

$Q_{avg} = \frac{Q}{PF_{day}} = \frac{111.3}{1.5} = 74.2\ lt/s$

from table 7.1 (pg 265).

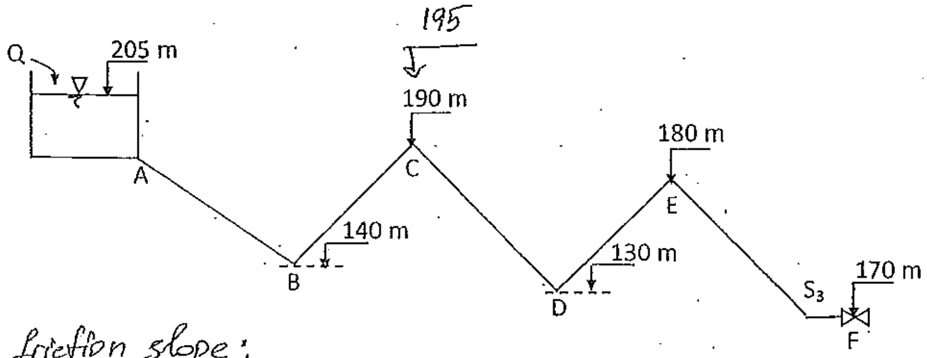
$P_n = 51884 \rightarrow Q_{ad} = 74.2\ lt/s$ (interpolation).

$\Rightarrow \ln P_n = \ln P_2 + k_g(t_n - t_2)$

$\ln 51884 = \ln 30000 + 0.02657(t_n - 2010)$.

PROBLEM 2: Determine the pipe sizes of the following transmission line shown in the figure below, which transmits $Q=1.0\ m^3/s$. Assume that $f=0.02$ for all pipes. Take $L_{AB}=L_{BC}=2000$ m and $L_{CD}=L_{DE}=L_{EF}=1000$ m. Assume that commercial pipes are available in 10 cm increment of diameter.

* Design specifications: $0.5\ m/s<u<2.0\ m/s$, $5\ m<P/\gamma<80$ m.



Determine min friction slope:

$S_{AC} = \dfrac{205-195}{4000} = \underline{0.0025} \Rightarrow$ min. slope $\qquad S_{AF} = \dfrac{205-170}{7000} = 0.005$

$S_{AE} = \dfrac{205-185}{6000} = 0.0033$

Segment AC:

$D_{AC} = \left(\dfrac{8fQ^2}{\pi^2 g\, S_{min}}\right)^{\frac{1}{5}} = \left(\dfrac{8\times 0.02\times 1}{9.81\times \pi^2\times 0.0025}\right)^{\frac{1}{5}} = 0.92\,m$ — min diameter required

↳ Choose $D_{AC} = 1\,m$

$u = \dfrac{Q}{\pi D^2/4} = \dfrac{1}{\pi\times 1^2/4} = 1.27\,m/s \rightarrow$ ok!

$H_C = H_A - f\dfrac{L}{D_{AC}}\dfrac{u^2}{2g} = 205 - 0.02\cdot\dfrac{4000}{1}\times\dfrac{1.27^2}{2\times 9.81} = \underline{198m}$

min friction slope: designing according to $H_c = 198m$ not $195m$

$S_{CE} = \dfrac{198-185}{2000} = \underline{0.0065}$ * $\qquad S_{CF} = \dfrac{198-170}{3000} = 0.0093$

Segment CE:

$D_{CE} = \left(\dfrac{8\times 0.02\times 1^2}{9.81\times\pi^2\times 0.0065}\right)^{\frac{1}{5}} = 0.76\,m \rightarrow D_{CE} = 0.8m \;/\; u_{CE} = \dfrac{1}{\pi\times 0.8^2/4} = 2m/s$

$H_E = 198 - \dfrac{0.02\times 2000}{0.8}\times\dfrac{2^2}{2\times 9.81} = 188m.$

Segment EF:

$S_{EF} = \dfrac{188-170}{1000} = 0.0018 \Rightarrow D_{EF} = \left(\dfrac{8\times 0.02\times 1^2}{9.81\times\pi^2\times 0.0018}\right)^{\frac{1}{5}} = 0.62m \rightarrow \underline{D_{EF} = 0.7m}$

$u = \dfrac{1}{\pi\times 0.7^2/4} = 2.6\,m/s$ NOT GOOD! choose $u = 2m/s = \dfrac{1}{\pi\cdot D^2/4} \Rightarrow \underline{D_{EF} = 0.8\,m}$

$H_E - f\dfrac{L}{D_{EF}}\cdot\dfrac{u^2}{2g} = Z_F + h_v \qquad 188 - \dfrac{0.02\times 1000}{0.8}\times\dfrac{2^2}{2\times 9.81} = 170 + h_v$

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Recitation 11

PROBLEM: The layout of a separate sewer system is shown in Figure 1. Design the storm and sanitary sewers between manholes 6 and 8. By investigating the topographical characteristics of the city, the flow directions in sewers are estimated as shown in Figure 1. A typical cross-section of a trench is shown in Figure 2. The design criteria for both storm and sanitary sewer systems are given below:

- Manning's roughness coefficient, N_{full} is 0.016 for all pipes and is variable.
- Maximum allowable flow velocity, u_{max} is 4 m/s.
- Minimum allowable full flow velocity, u_{min} is 0.6 m/s.
- Street slopes are 0.01

For the storm sewer design, apply the rational method and use the rainfall intensity-duration-frequency curves given in Figure 3. Consider the following data for the storm sewer system:

- Inlet times for all areas are 10 minutes.
- The flow time between two successive manholes is 2 minutes.
- T_r =25 years.
- Runoff coefficient, C for all areas are 0.7.
- Pipe sizes are available for every 50 mm increments of diameter. Take D_{min} as ϕ300.

For the sanitary sewer system, the following data are given:

- $Q_{average}$ = 165.625 lt/s
- 70% of the average daily demand returns to the sanitary sewer system.
- Groundwater infiltration is 0.4 lt/s/ha.
- Rainfall contribution is 0.5 lt/s/ha.
- Minimum depth of flow is 2 cm.
- Pipe sizes are available for every 50 mm increments of diameter. Take D_{min} as ϕ200.

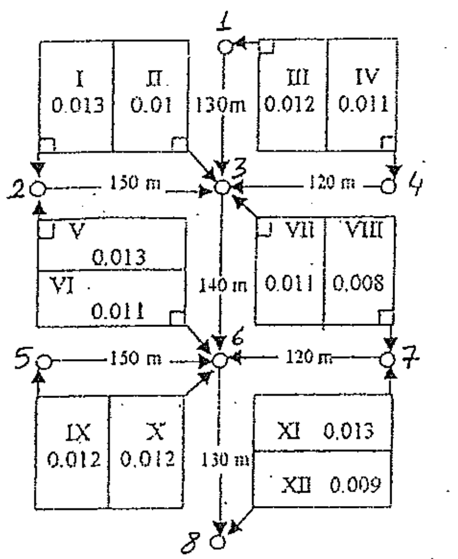


Figure 1. Layout of the system (areas in km²).

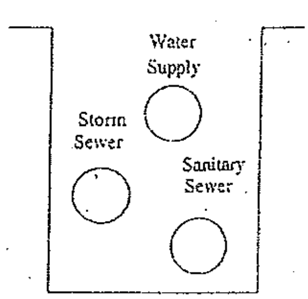


Figure 2. Bedding of pipes in the trench.

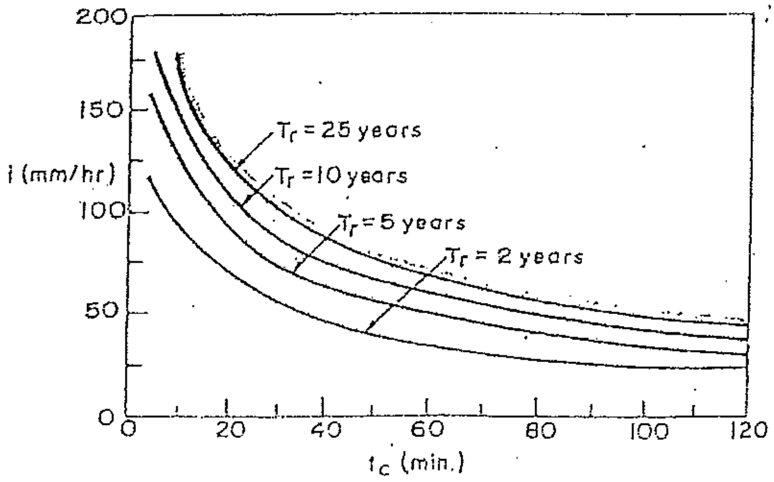


Figure 3. Rainfall intensity-duration-frequency curves.

Path 1 manhole 6 $t_c = 10\,min$ (Areas 6, 10)

$$Q_p = \frac{0{,}7 \times 165 \times (0{,}011 + 0{,}012)}{3{,}6} = 0{,}738\, m^3/s$$

Path 2 $t_c = 12\,min$ (Areas 6, 10, 2, 7, 8, 11, 9)

$$Q_p = \frac{0{,}7 \times 147 \times (0{,}011 + 0{,}012 + 0{,}01 + 0{,}011 + \ldots)}{3{,}6}$$

$Q_p = 2{,}2\, m^3/s$

Path 3 $t_c = 14\,min$ (Areas 6, 10, 2, 7, 8, 11, 9, 3, 1, 4, 5)

$$Q_p = \frac{0{,}7 \times 137\,(0{,}126)}{3{,}6} = 3{,}357\, m^3/s \rightarrow \text{max design}$$

$$Q = \frac{A}{n} R^{2/3} \sqrt{S_0} = \frac{0{,}312}{n} D^{8/3} \sqrt{S_0} = 3{,}357\, m^3/s \rightarrow D = 1226$$

$D_{design} = 1250$

$$Q_{full} = \frac{0{,}312}{0{,}016} (1{,}25)^{8/3} \sqrt{0{,}01} = 3{,}536\, m^3/s$$

$$\underbrace{Q_{des}/Q_{full}}_{=0{,}95} \Rightarrow \underbrace{V_{des}/V_{full}}_{1{,}04} \Longrightarrow V_{full} = \frac{Q_{full}}{A_{full}} = \ldots$$

$\ldots \times 1{,}04 = V_{des}$

$0{,}6 < V_{des} < 4$

$Q_{ave} = 165{,}625$ Lt/s
70%
GW inf = 0,4 Lt/s/ha
Rainfall cont = 0,5 Lt/s/ha
Min flow depth 2cm

Q_{design}

$(Q_{AV} \times 0{,}7 \times PF_{w}) + GW\,inf + RF\,cont$

$Q_{dry} \to Q_{avg}\,0{,}7\,PF_{dry} + GW\,inf$

$Q_{AV} = 165{,}625 \times 0{,}7 = 115{,}94$

$Q_{des} = 2{,}6 \times 115{,}94 + 504 + 163 = 312{,}78$

$Q_{dry} = 115{,}94 \times 1{,}6 + 5{,}04$

$= 190{,}54$

$$0{,}31278 = \frac{0{,}312}{0{,}016} D^{8/3} \sqrt{0{,}01} \Rightarrow D = 0{,}503$$

$\Rightarrow D = 550$ mm

$Q_{FULL} = 0{,}306\ m^3/s$

$V_{full} = 1{,}67$ m/s

$Q_{des}/Q_{FULL} = 0{,}79$ $\quad \frac{V_{des}}{V_{full}} = 0{,}98$ $\quad d/D = 0{,}75$

$V_{des} = 1{,}64$ m/s

$V_{dry}/V_{FULL} = $ ― $\quad V_{dry} > 0{,}6$?

Meria Selamoğlu
K4-206
Thursday 9.30-11.30
14.30-16.30

CE378 | WATER RESOURCES ENGINEERING

Spring 2012-2013 01.03.2013

Recitation 1

PROBLEM 1

The values of the components of the hydrologic cycle for the Mogan Lake observed in July are given. Compute the monthly inflow to the lake from the side creeks in lt/sec unit.

Storage at the beginning of month	: 13.4×10^6 m^3
Storage at the end of month	: 12.1×10^6 m^3
Average surface area of lake in the month	: 6.3 km^2
Total evaporation at a nearby station (from an evaporation pan)	: 310 mm
Evaporation pan correction coefficient	: 0.7
Total precipitation at a nearby station	: 6 mm
Monthly mean discharge released through the control gates (sluice gates)	: 15 lt/sec
Subsurface flow contribution	: Negligible

$\Delta S = (I + P) - (E + Q + \cancel{S})$

$\Delta S = (12,1 - 13,4) \times 10^6 = -1,3 \times 10^6 \ m^3$

$P = 6mm \rightarrow \underbrace{6 \times 10^{-3}}_{m} \times \underbrace{6,3 \times 10^6}_{m^2} = 37,8 \times 10^3 \ m^3$

$E = 310 \times 0,7 = 217mm \rightarrow 217 \times 10^{-3} m \times 6 \times 10^{-3} m^2 = 3,671 \times 10^5 \ m^2$

$Q = 15 lt/sec = 15 \times 10^{-3} \ m^3/s \times 31 \ days \times 86400s = 40,176 \times 10^3 \ m^3$

$-1,3 \times 10^6 m^3 = I + 37,8 \times 10^3 m^3 - (13,671 \times 10^5 + 40,176 \times 10^3 m^3)$

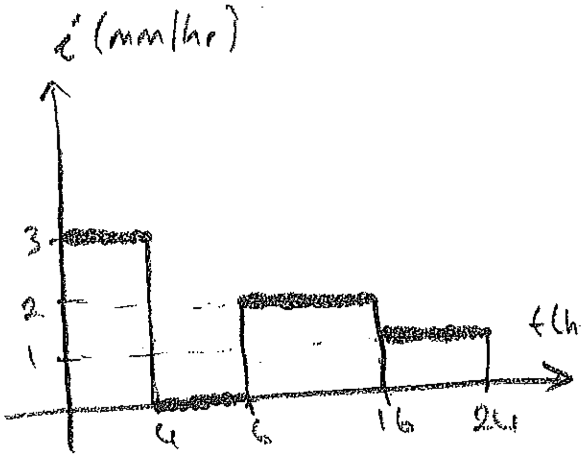
$I = 69,476 \times 10^3 m^3 \rightarrow \dfrac{69,476 \times 10^3 \times 10^3 \ lt}{31 \times 864000 \ s} = \underline{26 lt/s}$

---

2) $\bar{P} = \dfrac{P_B + P_C + P_D + P_E}{\#inside} = \dfrac{35+48+53+24}{4} = \underline{40mm}$ (a)

b)

t(hr)	Σdepth (mm)	Δd (mm)	Δt (hr)	i (mm/hr)
0	0			
		12	4	3
4	12			
		0	2	0
6	12			
		20	10	2
16	32			
		8	8	1
24	40			



PROBLEM 2

A basin is given below in Figure 1. Total precipitation depths measured during a stormy day at the meteorological stations are also provided in Table 1.

a) Determine the mean areal precipitation of this day using "arithmetic mean method".

b) Determine the representative hyetograph of this basin if the rainfall mass curve of this storm is as shown in Figure 2.

Table 1 - Total (24-hr) precipitation values

Station	Precipitation (mm)
A	16
B	35
C	48
D	53
E	24
F	45

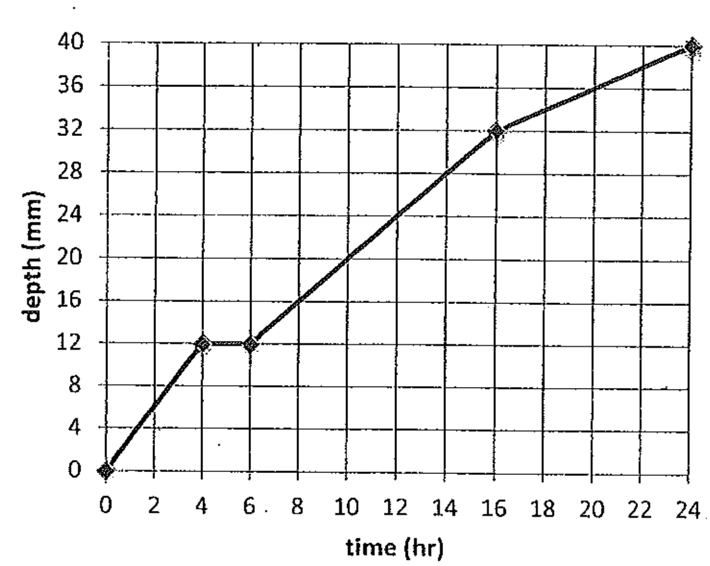


Figure 2 - Rainfall mass curve diagram for the basin

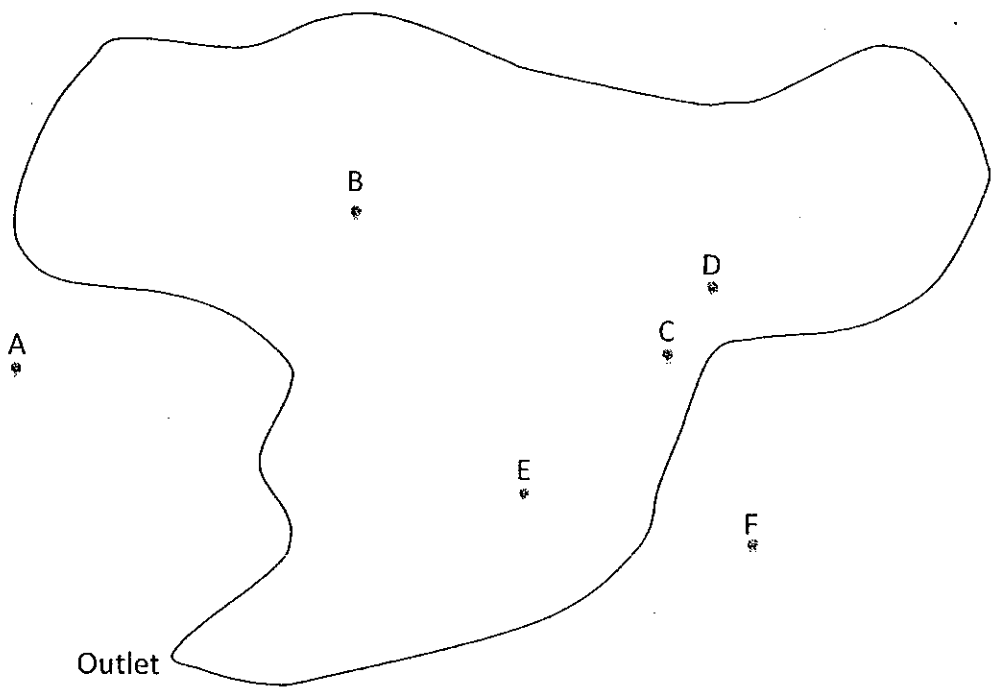


Figure 1 - Basin

Lag → katlarında
S → katı olmayanda

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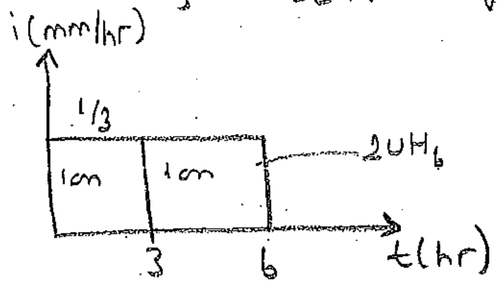
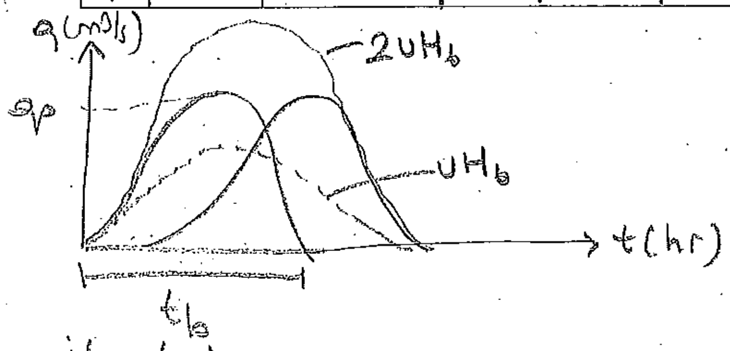


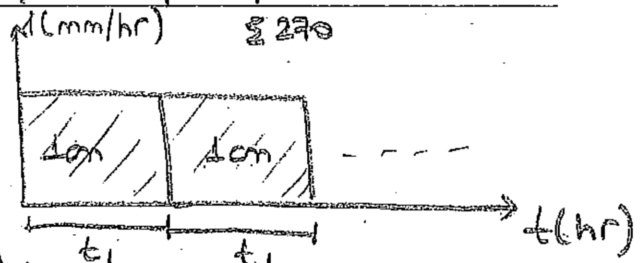
PROBLEM: You are given UH_3 of a basin. Determine:

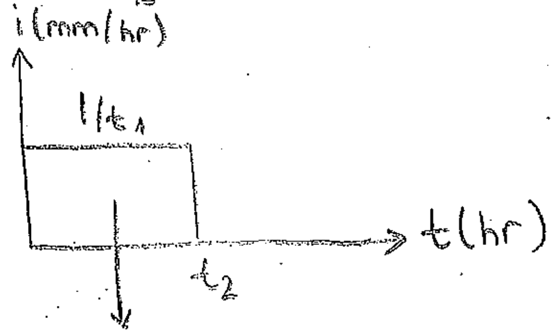
a) UH_6 of this basin using lagging method,
b) UH_2 of this basin using S-curve method,
c) area of the basin,
d) the change in Q_p, t_p, and t_b with the change in the duration of excess precipitation.

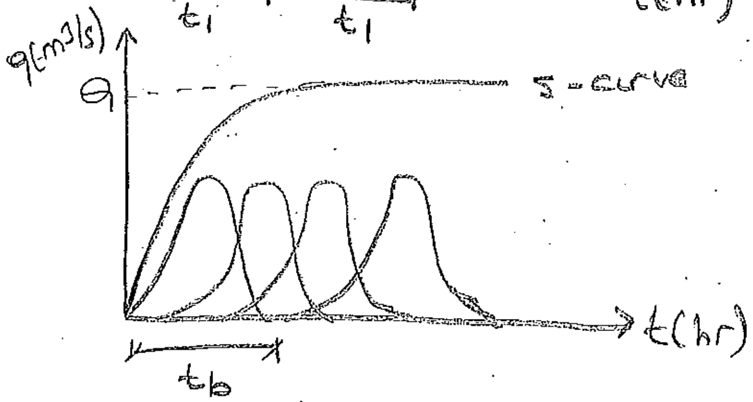
×3/2

t (hr)	UH_3 (m^3/s)	3hr lag UH_3 (m^3/s)	$2UH_6$ (m^3/s)	UH_6 m^3/s	6hr. lag UH_3	S_3	2hr lag S_3	Δ m^3/s	UH_2
0	0	–	0	0	–	0	–	0	0
1	22	–	22	11	–	22	–	22	33
2	50	–	50	25	–	50	0	50	75
3	66	0	66	33	–	66	22	44	66
4	54	22	76	38	–	76	50	26	39
5	34	50	84	42	–	84	66	18	27
6	24	66	90	45	0	90	76	14	21
7	14	54	6,8	34	22	90	84	6	9
8	6	34	40	20	50	90	90	0	0
9	0	24	24	12	66				
10	Σ270	14	14	7	54				
11		6	6	3	34				
12		0	0	0	24				
13				Σ270	14				
14					6				
15					0				









t_b/t_r

$\frac{t_2}{t_1}$ UH_{t_2} = Differential Hydrog.

$UH_2 = \frac{t_1}{t_2}$ Diff. Hydrog.

b) $\frac{t_b}{t_r} = \frac{9}{3} = 3$ UH_3 are needed to construct S_3.

$S_3 = UH_3 + (3hr\ lag)\ UH_3 + (6hr\ lg)\ UH_3$

$\Delta = S_3 - (2hr\ lg)\ S_3$

$UH_2 = \frac{3}{2}\ \Delta$

c) $d_{UH} = \frac{\Sigma q_{UH}\ \Delta t}{A}$

$A = 97{,}2\ km^2$

$0{,}01 = \frac{270 \times 1 \times 3600}{A}$

d)

t_r (hr)	Q_p (m^3/s)	t_p (hr)	t_b (hr)
2	75	2	8
3	66	3	9
6	45	6	12

As $t \uparrow$ $Q \downarrow$ $t_p \uparrow$ $t_b \uparrow$

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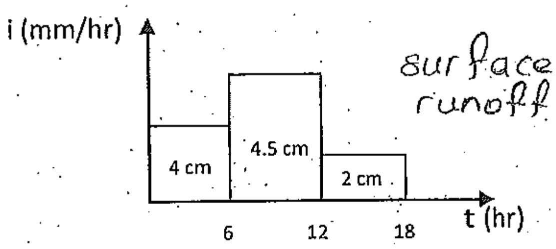
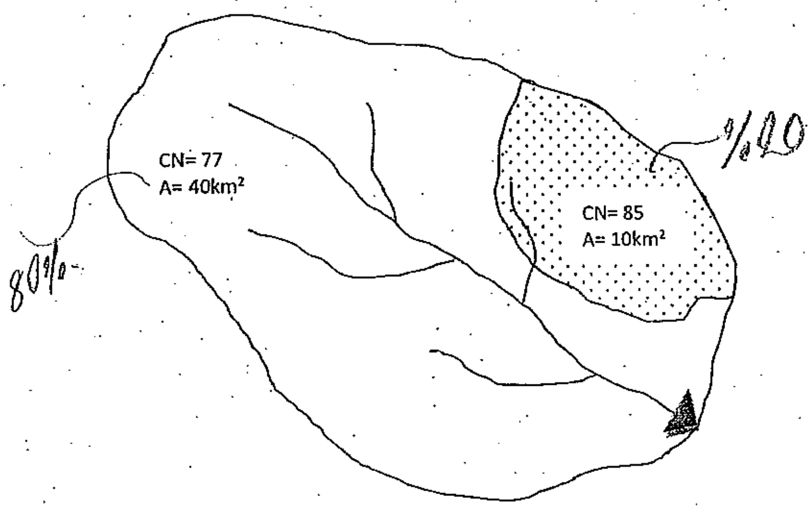
PROBLEM: You are supposed to design a weir at the outlet of the basin given below. The design must be conducted according to the given storm hyetograph. Since there are no available recorded runoff data at the closest discharge observation station, synthetic unit hydrograph must be obtained for the basin. The characteristics of the basin are given below. Assume the baseflow equals to 15 m^3/s at the outlet of the basin.

a) Find the ordinates of the unit hydrograph that can be obtained from the given information.

b) Find the peak discharge of the design hydrograph.

Area of the basin= 50 km^2
Main stream length= 14 km
Bed slope of the main stream= 1.4%
1m= 3.28 ft

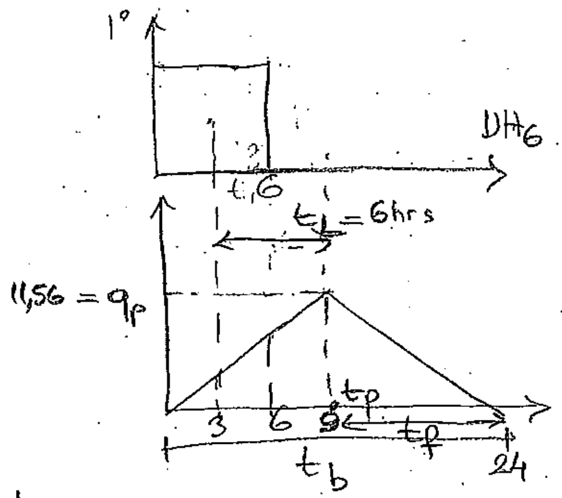




20% → CN = 85
80 % → CN = 77
$CN_{ave} = 0,2 \times 85 + 0,8 \times 77 = 78,6$

$$Q_p = \frac{2.08A}{t_p}, \quad t_L = \frac{L^{0.8}(S+1)^{0.7}}{1900 S_h^{0.5}}, \quad S = \frac{1000}{CN} - 10, \quad t_p = \frac{t_r}{2} + t_L, \quad t_b = 2.67 t_p$$

$$CN_{ave} = \frac{\sum_{i=1}^{n} CN_i A_i}{\sum_{i=1}^{n} A_i}$$



$L = 14000 m = 45920 ft$

$S_h = 1,4\%$

$S = \frac{100}{78,6} - 10 = 2,72$ in

$t_L = \frac{(45920)^{0,8} \cdot (2,72 \times 1)^{0,7}}{1900 \cdot (1,4)^{0,5}} = 6$ hr

$t_b = 2,67 \times 9 = 24$ hrs

$t_p = \frac{6}{2} + 6 = 9$ hrs

$\frac{2,08 \times 50}{} = 11,56\ m^3/s$

Forces (kN/m)	Moment Arm about "O" (m)	Moment (kNm/m)
$W_1 = \gamma_c h b_1$ $24 \times 60 \times 5.29 = 7617.6$ (↓)	$53.55 + 5.29/2 = 56.2$	428109.12 ↺
$W_2 = \gamma_c h_2 b_2 / 2$ $24 \times 53.55^2/2 = 34411.23$ (↓)	$53.55 \times 2/3 = 35.7$	1228480.91 ↺
$F_h = ½ (h_u)(\gamma_w h_u)$ $\frac{1}{2} \cdot 50(10 \times 50) = 12500$ (→)	$50 \times 1/3 = 16.67$	208375 ↻
$F_u = \phi/2\ \gamma_w h_u B$ $\frac{0.6}{2} \times 10 \times 50 \times 58.84 = 8826$ (↑)	$58.84 \times 2/3 = 39.23$	346243.98 ↻
F_i 100 (→)	50	5000 ↻
$F_s = ½ \gamma_s h_s^2 K_a$ $\frac{1}{2} \times 11 \times 3^2 \times 0.31 = 15.35$ (→)	$3 \times 1/3 = 1$	15.35 ↻
$F_w = 0.726\ C\ k_h \gamma_w h_u^2$ horizontal force $0.726 \times 0.7 \times 0.1 \times 10 \times 50^2 = 1270.5$ (→)	$0.412 \times 50 = 20.6$	26172.3 ↻
$F_{dh1} = k_h W_1 = 0.1 \times 7617.6 = 761.76$ (→)	$60 \times 1/2 = 30$	22852.8 ↻
$F_{dh2} = k_h W_2 = 0.1 \times 34411.23 = 3441.12$ (→)	$53.55 \times 1/3 = 17.85$	61423.99 ↻
$F_{dv1} = k_v W_1 = 0.05 \times 7617.6 = 380.88$ (↑)	56.2	21405.46 ↻
$F_{dv2} = k_v W_2 = 0.05 \times 34411.23 = 1720.56$ (↑)	35.7	61423.99 ↻

Resisting

overtu

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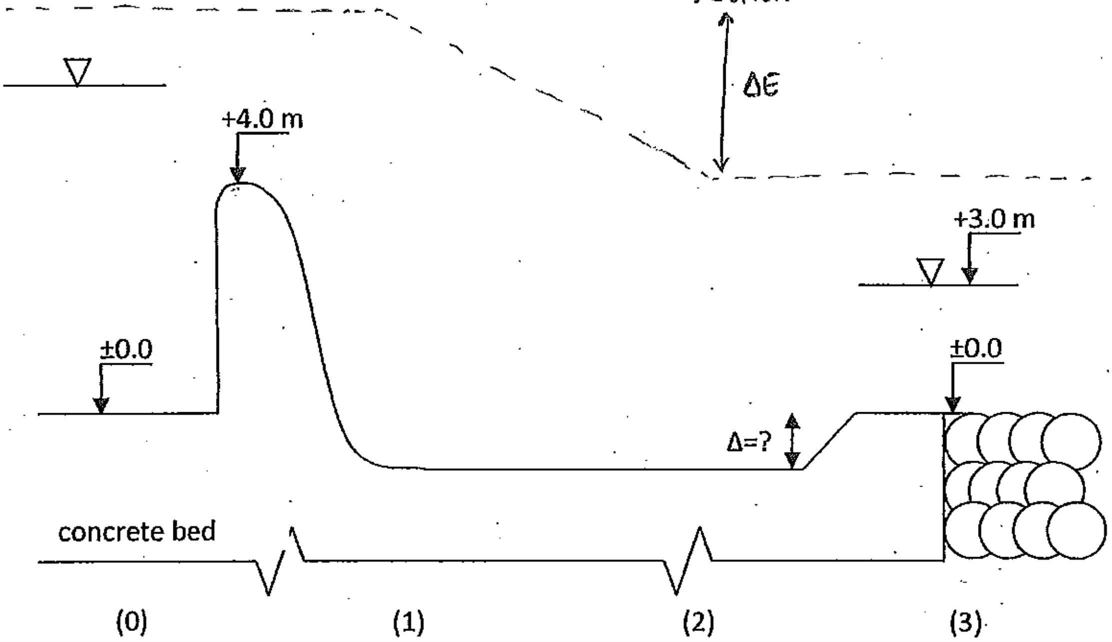
Recitation 7

PROBLEM: An uncontrolled overflow spillway, 4.0 m high and 50 m long will be designed to evacuate $Q_0 = 450\ m^3/s$. Ignore the head losses over the spillway face and at a possible end sill. Determine:

* Ignore downstream effects on spillwa flow

a) the design spillway head,
b) the required lowering of the river bed for the stilling basin,
c) the type and dimensions of the stilling basin.

* All sections are rectangular in cross section



$P = 4\,m$

$$y_2 = \frac{y_1}{2}\left(\sqrt{1+8F_{r1}^2}-1\right)$$

$$\Delta E = \frac{(y_2-y_1)^3}{4y_1y_2}$$

$Q_0 = C_0 L H_0^{3/2}$ $\qquad E_0 = P + H_0$

Ass. H_0	P/H_0	C_0	Q_0
2.00	2	2.17	306
2.50	1.6	2.16	426.91
2.59	1.58	2.16	450.17

$H_0 = 2.59$

$E_0 = P + H_0 = 4 + 2.59 = 6.59\,m$

b) $q = \frac{Q}{L} = \frac{450}{50} = 9\,m^3/s/m$

$u_3 = \frac{q_3}{y_3} = \frac{9}{3} = 3\,m/s$

$E_3 = y_3 + \frac{u_3^2}{2g}$

$= 3 + \frac{3^2}{2\times 9.81} = 3.46\,m$

$\Delta E = E_0 - E_3$

$= 6.59 - 3.46$

$= 3.13\,m$

$$\Delta E = \frac{\left[\frac{\ }{2}\left(\sqrt{1+\frac{\ }{g y_1^3}}-1\right)-y_1\right]}{4y_1 \cdot \frac{y_1}{2}\left(\sqrt{1+\frac{8q^2}{gy_1^3}}-1\right)} = 3{,}13$$

$$y_1 = 0.778\,m$$

$$Fr_1 = \frac{y_1}{\sqrt{gy_1}} \quad ; \quad F_1^2 = \frac{q^2}{gy_1^3} = \frac{q^2}{9.81 \times (0.778)^3} = 4.187.$$

$$\Rightarrow u_1 = 11.57\,m/s$$

$$y_2 = \frac{0.778}{2}\left(\sqrt{1+8\cdot(4.187)^2}-1\right) = 4.23\,m$$

$$y_2 + \frac{u_2^2}{2g} = \Delta + y_3 + \frac{u_3^2}{2g}$$

$$E_2 = \Delta + E_3$$

$$4.23 + \frac{q^2}{2 \times 9.81 \times 4.23^2} = \Delta + 3.46$$

$\Delta = 1.00\,m$ → min required lowering of the stilling basin.

c) $Fr_1 = 4.187$

$u_1 = 11.57\,m/s$

$2.5 < Fr < 4.5$ $\quad u_1 < 15\,m/s$

USBR Type IV basin

Dimensions : length of the still basin

$$L_w = 6.1\,y_2$$
$$= 6.1 \times 4{,}23$$
$$= 25.8\,m$$

height of the chute blocks $= 2y_1 = 2 \times 0{,}778 = 1.56\,m$

height of the sill at toe $= y_1(9+Fr_1)/9 = 0.778(9+4.187)/9$
$= 1.14\,m$

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Recitation 8

PROBLEM: An aquifer system composed of an unconfined aquifer, an aquitard, and a leaky confined aquifer is shown below. The hydraulic conductivities for the unconfined aquifer, the aquitard and the leaky confined aquifer are $K_{ua}=8x10^{-5}$ m/s, $K_{aqt}=2x10^{-6}$ m/s and $K_{lca}=4x10^{-6}$ m/s, respectively. All three aquifers are isotropic and homogeneous. Assume groundwater table elevation stays horizontal. Answer the following questions for this aquifer system:

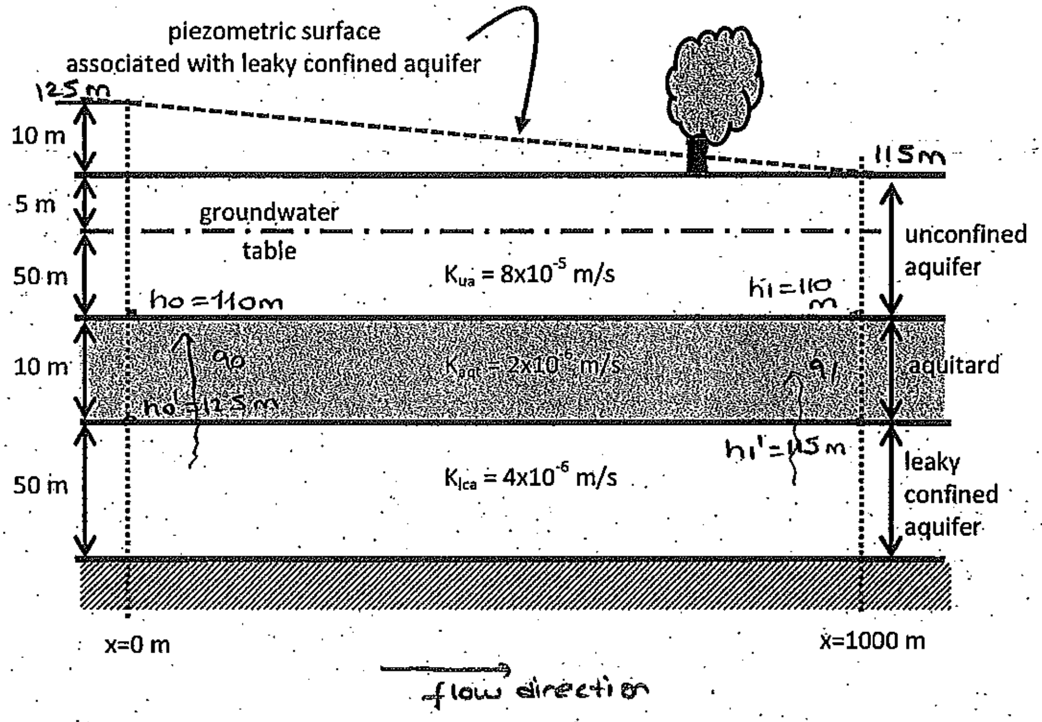
a) Calculate the horizontal specific discharge in the unconfined aquifer and mark its direction on the figure.

b) Calculate the horizontal specific discharge in the confined aquifer and mark its direction on the figure.

c) Calculate the vertical specific discharge through the aquitard at x=0 m and x=1000 m and mark their directions on the figure.

d) Calculate the total rate of leakage per unit width of the aquifer between x=0 m and x=1000 m.

NOTE: Specific discharge is discharge per unit area.



$$Q = -k \cdot A \cdot \frac{dh}{dl}$$
$$q = -k \cdot \frac{dh}{dl}$$
Darcy's law $\Rightarrow$ a) $q = -k \frac{dh}{dt}$

$$\left.\frac{dh}{dt}\right|_{\text{unconf. aquif.}} = 0 \Rightarrow \text{therefore,}$$
$$q_{un} = 0.$$

b) $q = -k \cdot \frac{dh}{dx} = -4 * 10^{-6} \frac{m}{s} \frac{-10}{1000} (h_2 - h_1) \Rightarrow q_{conf} = 4 * 10^{-8}$ m/s

c) $q_0 = -2 * 10^{-6} \frac{110 - 125}{10} = 3 * 10^{-6}$ m/s $(\uparrow)$

$q_1 = -2 * 10^{-6} \frac{110 - 115}{10} = 1 * 10^{-6}$ m/s $(\uparrow)$

d) Eqn for piezometric surface associated with aquitard

$x = 0 \quad h = 125$ m
$x = 1000 \quad h = 115$ m

$h = ax + b$
$h = 125 - 0{,}01x$

$$\Delta h = h\Big|_{\text{confined}} - h\Big|_{\text{leaky confined}} = 110 - 125 + 0{,}01x \Rightarrow \Delta h = 0{,}01x - 15$$

$dQ = q\,dA$

$$q = -k \frac{\Delta h}{\Delta x} = -2 * 10^{-6} \frac{0{,}01x - 15}{10}$$

$$q = -2 * 10^{-7} (0{,}01x - 15)$$

$$Q = \int_0^{1000} -2 * 10^{-7} (0{,}01x - 15)\, dx$$

$$Q = -2 * 10^{-7} \left[0{,}01 \frac{x^2}{2} - 15x \right]\Bigg|_0^{1000} \Rightarrow Q = 2 * 10^{-3} \text{ m}^3/\text{s} / \text{m}$$

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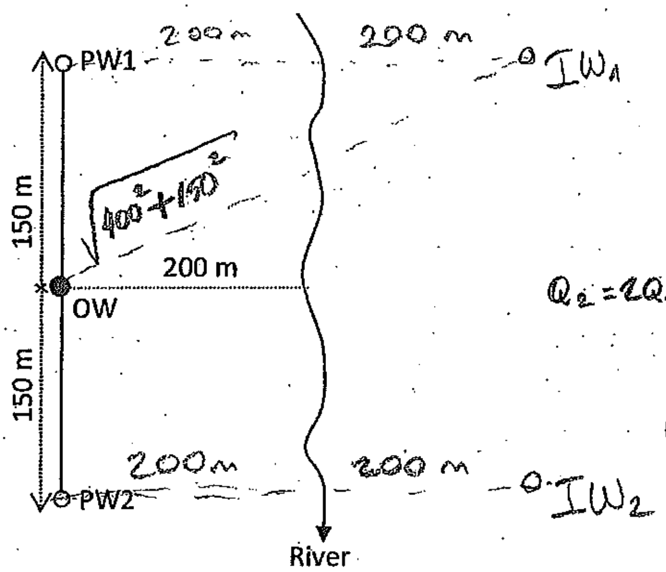
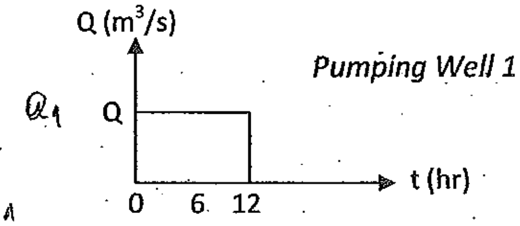
Recitation 9

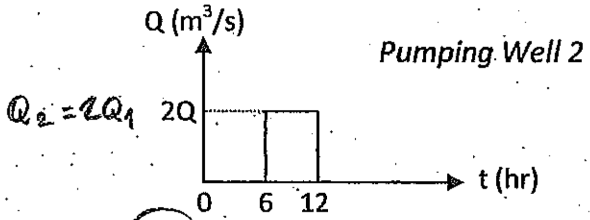
PROBLEM 1: Two pumping wells penetrating completely a confined homogenous and isotropic aquifer are located near a river as shown below. Both wells have the same radius of 0.15 m and are pumped according to the given schedules. The coefficient of storage and transmissivity for the aquifer are 0.0004 and 0.002 m²/s respectively. If the drawdown in the observation wall is not to exceed 2 m at t=12 hr, what should be the maximum values for discharges Q_1 and Q_2, where $Q_2=2Q_1$.

$$u=\frac{r^2 S}{4Tt} \qquad s=\frac{Q}{4\pi T}W(u)$$

mirror image.







I

	r(m)	t(hr)	u
PW_1	150	12	$2.6 \cdot 10^{-2}$
PW_2	150	6	$5.2 \cdot 10^{-2}$
IW_1	427,2	12	$2.11 \cdot 10^{-1}$
IW_2	427,2	6	$4.22 \cdot 10^{-1}$

I) tablonun devamı:

W(u)	Q
3.1006	Q_1 (+)
2,4316	$2Q_1$ (+)
1,1836	Q_1 (−)
0,6681	$2Q_1$ (−)

PW1

$$u=\frac{150^2 \, 0{,}0004}{4 \cdot 0{,}002 \cdot 12 \cdot 3600}=2.6 \cdot 10^{-2}$$

$$2=s=\frac{\Sigma Q \cdot W(u)}{4\pi T}=\frac{\left[Q_1 \cdot 3{,}1006+2Q_1 \cdot 2{,}4316-Q_1 \cdot 1{,}1836 - 2Q_1 \cdot 0{,}6681\right]}{4\pi \cdot 0{,}002}$$

u vs. W değerlerini gösteren tablodan linear interpolation yapılarak W(u) well func. bulundı

$\therefore Q_1 = 9{,}233$ l/s

$Q_2 = 2Q_1 = 18{,}466$ l/s

River → recharge boundary → decrease the drawdown

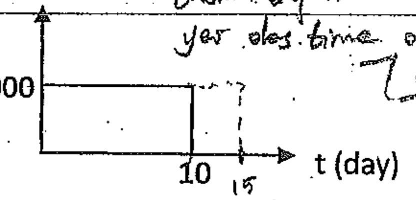
Ω işaretlerine bakarken ⇒ recharge veya barrier şeklinde bir boundary olabilir. drawdowna (+) katkı

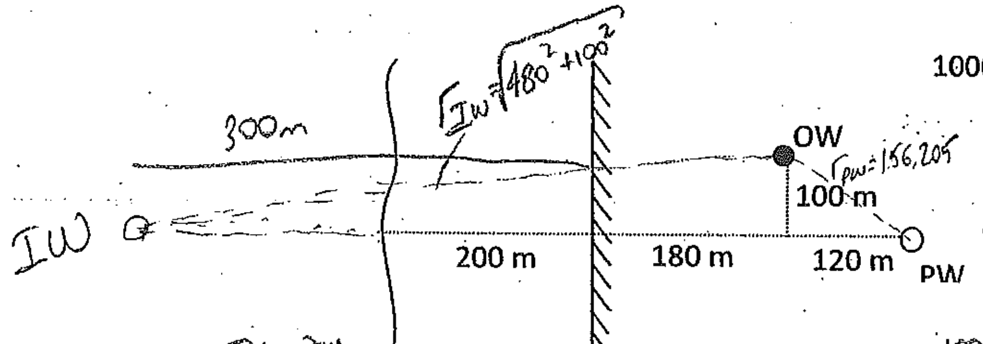
$r_{PW} = \sqrt{100^2 + 120^2} = 156{,}205$

PROBLEM 2: A pumping well (PW) fully penetrates a leaky confined aquifer, which is bounded from the west by a barrier and a river boundary as shown schematically below. Aquifer characteristics are K = 14 m/day and S = 0.0001, and thickness of the aquifer (b) is 40 m. Hydraulic conductivity (K') and thickness of the overlying aquitard (b') are $5*10^{-6}$ m/day and 25 m, respectively. Determine the drawdown at the observation well (OW) 15 days after pumping starts.

grafiğin başladığı yer önemli değil, ama bittiği yer obs. time olmalı lazım

$$T = bK \qquad B^2 = \frac{Tb'}{K'} \qquad u = \frac{r^2 S}{4Tt} \qquad s = \frac{Q}{4\pi T} W(u, r/B)$$



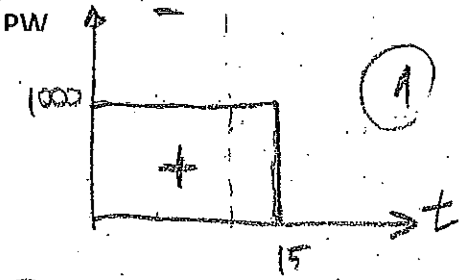


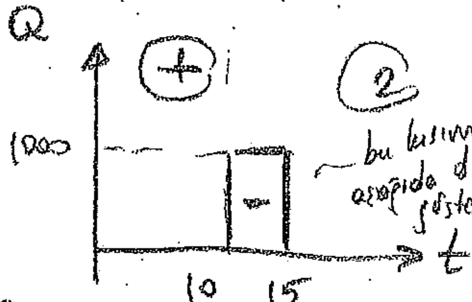
$T = bK = 40 \cdot 14 = 560\ m^2/day$

There is no effect of river on the drawdown of the pumping well

$$B^2 = \frac{T \cdot b'}{K'} = \frac{560 \cdot 25}{5 \cdot 10^{-6}}$$

$= 52915$ m.





	r(m)	t(day)	u	r/B	W(u,r/B)	Q
PW 1	156,205	15	$7{,}26 \cdot 10^{-5}$	$2{,}95 \cdot 10^{-3}$	8,926	1000
PW 2	"	5	$2{,}18 \cdot 10^{-4}$	"	7,874	−1000
IW 1	490,306	15	$7{,}15 \cdot 10^{-4}$	$9{,}27 \cdot 10^{-3}$	6,64	1000
IW 2	"	5	$2{,}15 \cdot 10^{-3}$	"	5,533	−1000

$$s = \frac{1000}{4\pi 560}\left[\underbrace{8{,}926 - 7{,}874}_{PW} + \underbrace{6.64 - 5{,}533}_{IW}\right] = 0.3\ldots$$

$$PW\,1 \Rightarrow u = \frac{r^2 S}{4Tt} = \frac{156{,}205^2 \cdot 0{,}0001}{4 \cdot 560 \cdot 15} = 7{,}26 \cdot 10^{-5}$$

$$s = \frac{Q}{4\pi T} W(u, r/B) \qquad r/B = 0{,}00295$$

u \ r/B	0.002	0,00295	0,004
$7 \cdot 10^{-5}$	8,976		8,934
$10 \cdot 10^{-5} = 10^{-4}$	8,623		8,594

$\Rightarrow x_1$, $\Rightarrow x_2$) $x_3 = 8{,}926$

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Recitation 10

→ Design sorularında galls.

PROBLEM: Analyze a gravity pipeline system that feeds the municipal water network of a town.

a) Determine the maximum discharge that can be drawn from the pipeline (satisfying all the operational requirements and limitations).

b) Using geometric extrapolation, determine until when the system meets municipal requirements of this town.

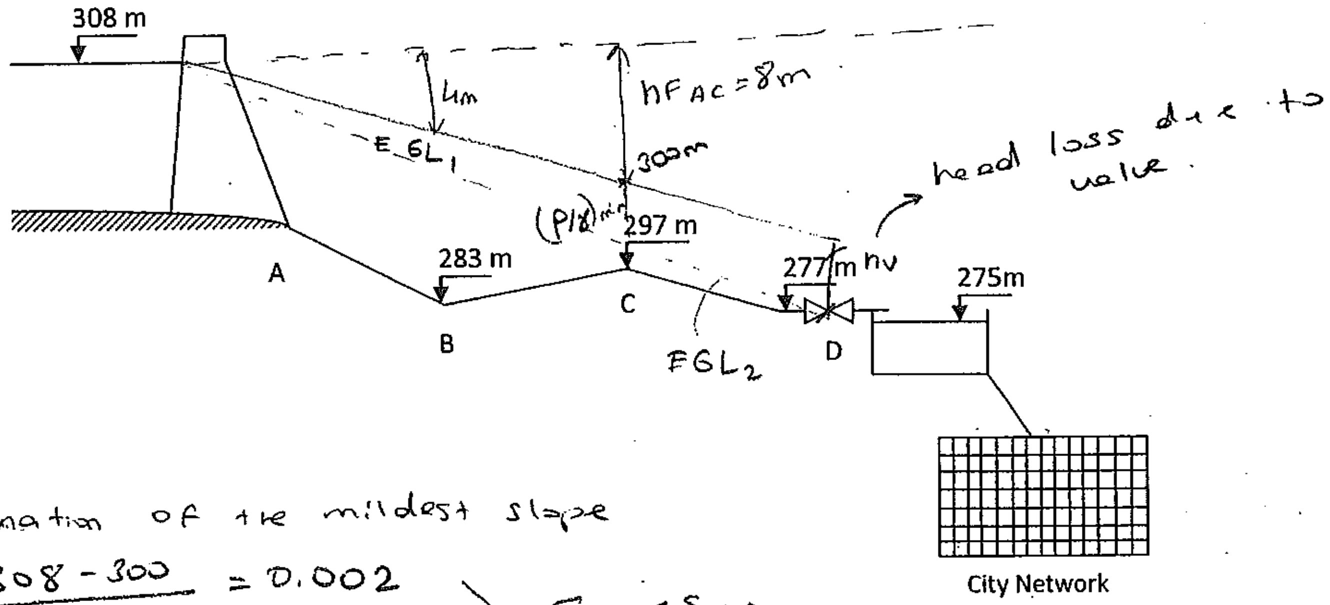
Constant reservoir level= 308 m,

For all pipe segments: f=0.02, ϕ=400 mm, L=2000 m

Operational requirements: 0.5 m/s≤ u ≤2.0 m/s 3 m≤ P/γ ≤80 m

Population: P_{2000}=23000, P_{2010}=30000, $P.F_{day}$=1.5, $P.F_{hour}$=2.5

$$h_f = \frac{8fL}{g\pi^2 D^5}Q^2$$



a) Determination of the mildest slope

$$AC = \frac{308-300}{4000} = 0.002$$

$$AD = \frac{308-277}{6000} = 0.005167$$

$S_{AC} < S_{AD}$

$$hf = \frac{8fL}{g\pi^2 D^5}Q^2 \qquad 8 = \frac{8\times0.02\times4000}{9.81\times\pi^2\times0.4^5}Q^2 \rightarrow Q=0.1113\ m^3 = 111.3\ lt/s$$

Velocity check

$$u = \frac{Q}{A} = \frac{4Q}{\pi D^2} = \frac{4\times0.1113}{\pi\times0.4^2} = 0.886\ m/s \quad \text{o.k.}$$

Pressure check

Energy eqn btw A and B

$$H_A = \left(\frac{P}{\gamma}\right)_B + z_B + h_{fA-B} \qquad 308 = \left(\frac{P}{\gamma}\right)_B + 283 + 4 \quad \left(\frac{P}{\gamma}\right)_B = 21m \quad \text{o.k.}$$

$$\ln P_n = \ln P_2 + k_g \underbrace{(t_n - t_2)}_{\Delta t}$$

$$k_g = \frac{\ln P_2 - \ln P_1}{t_2 - t_1} \qquad k_g = \frac{\ln 30000 - \ln 23000}{2010 - 2000} = 0{,}0265$$

$$\underbrace{Q_{ad}}_{\text{average daily demand}} = \frac{Q}{(PF)_{day}} = \frac{111.3 \ lt/s}{1.5} = 74.2 \ lt/s$$

From Table 7.1 on page 265 Yanez, A.M (2006) Applied Water Resources Eg.

Population	Average Demand
⋮	⋮

$P_n = 51884$

$$\ln 51884 = \ln 30000 + 0.02657 \ \Delta t$$

$$\Delta t \cong 20 \text{ years}$$

System meets municipal requirements until 2030.

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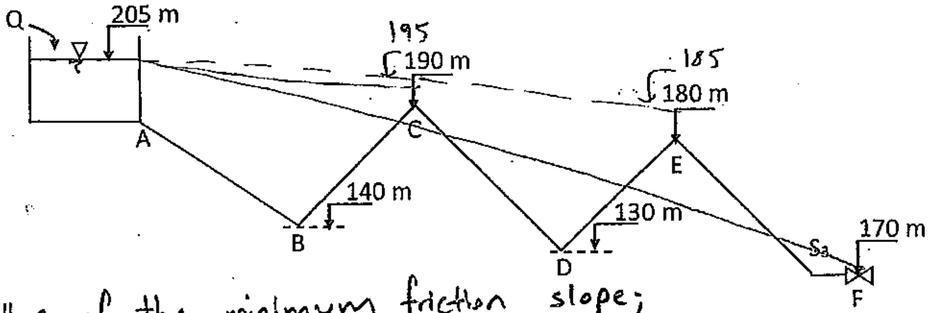
Recitation 11

PROBLEM: Determine the pipe sizes of the following transmission line shown in the figure below, which transmits Q=1.0 m³/s. Assume that f=0.02 for all pipes. Take $L_{AB}=L_{BC}=2000$ m and $L_{CD}=L_{DE}=L_{EF}=1000$ m. Assume that commercial pipes are available in 10 cm increment of diameter.
Design specifications: 0.5 m/s<u<2.0 m/s, 5 m<P/γ<80 m.



Determination of the minimum friction slope;

$$S_{AC} = \frac{205-(190+5)}{4000} = 2.5\times10^{-3}$$

$$S_{AE} = \frac{205-(180+5)}{6000} = 3.33\times10^{-3}$$

$$S_{AF} = \frac{205-170}{7000} = 5\times10^{-3}$$

$\therefore S_{min} = S_{AC}$

$$h_f = \frac{8fL}{g\pi^2D^5}Q^2 \rightarrow D_{AC} = \left(\frac{8\times0.02\times1^2}{9.81\times\pi^2\underbrace{(2.5\times10^{-3})}_{h_f/L = S}}\right)^{1/5} = 0.92m.$$

Select $\underline{\underline{D_{AC} = 1.0m}}$.

$$U_{AC} = \frac{Q}{A_{AC}} = \frac{1}{\pi 1^2/4} = 1.27\,m/s \quad ok \checkmark$$

$$H_C = H_A - f\frac{L}{D_{AC}}\frac{U_{AC}^2}{2g} = 205 - \frac{0.02\times4000}{1}\frac{(1.27)^2}{2\times9.81} = 198m.$$

~~Diameter of the segment~~

$$S_{CE} = \frac{198-185}{2000} = 6.5\times10^{-3}$$

$$S_{CF} = \frac{198-170}{3000} = 9.33\times10^{-3}$$

$\therefore S_{min} = S_{CE}$

$$D_{CE} = \left(\frac{8\times0.02\times1^2}{9.81\times\pi^2(6.5\times10^{-3})}\right)^{1/5} = 0.76m. \rightarrow \text{Select } \underline{\underline{D_{CE} = 0.80m}}.$$

$$u_{CE} = \frac{1}{\pi\, 0.8^2/4} = 2\ m/s \quad \text{ok} \checkmark$$

$$H_E = H_C - f\frac{L}{D_{CE}}\frac{u_{CE}^2}{2g} = 198 - \frac{0.02 \times 2000}{0.8}\frac{2^2}{2 \times 9.81} = 188\ m.$$

$$S_{EF} = \frac{188-170}{1000} = 18 \times 10^{-3}$$

$$D_{EF} = \left(\frac{8 \times 0.02 \times 1^2}{9.81\pi^2 (18 \times 10^{-3})}\right)^{1/5} = 0.62\ m. \rightarrow \text{Select } D_{EF} = 0.70\ m.$$

$$u_{EF} = \frac{1}{\pi\, 0.7^2/4} = 2.6\ m/s > 2\ m/s \quad \text{not ok} \times$$

$\therefore$ Take $u = 2.0\ m/s$

$$Q = u\,\pi D^2/4 \Rightarrow D_{EF} = 0.8\ m.$$

→ Headloss of the valve

$$H_E - f\frac{L}{D_{EF}}\frac{u_{EF}^2}{2g} = z_F + h_v$$

$$188 - 0.02\,\frac{1000}{0.8}\,\frac{2^2}{2 \times 9.81} = 170 + h_v$$

$$h_v = 13\ m.$$

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Recitation 2

PROBLEM: Determine the areal mean precipitation for a 2 hour storm that took place over a basin of 376 km^2 by using following methods:

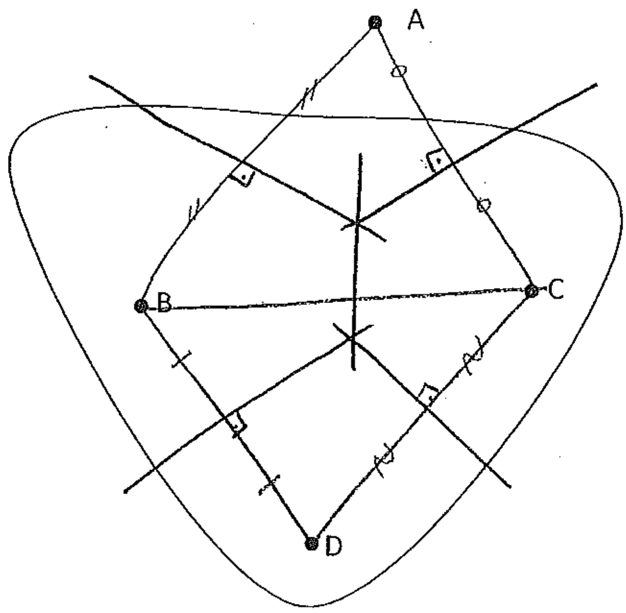
a. Arithmetic Mean Method,
b. Thiessen Polygons Method (draw polygons),
c. Isohyetal Map Method (draw approximate isohyets at 2 mm intervals and determine average precipitation and area between isohyets).

Table 1 - Thiessen Polygon Data

Station	Precipitation (mm)	Polygon Area (km^2)
A	24	25
B	19	141.1
C	20	105.6
D	16	104.3

Table 2 - Isohyetal Map Data

Isohyets	Precipitation (mm)	Area (km^2)
>22		15
22-20		111
20-18		130
18-16		100
<16		20

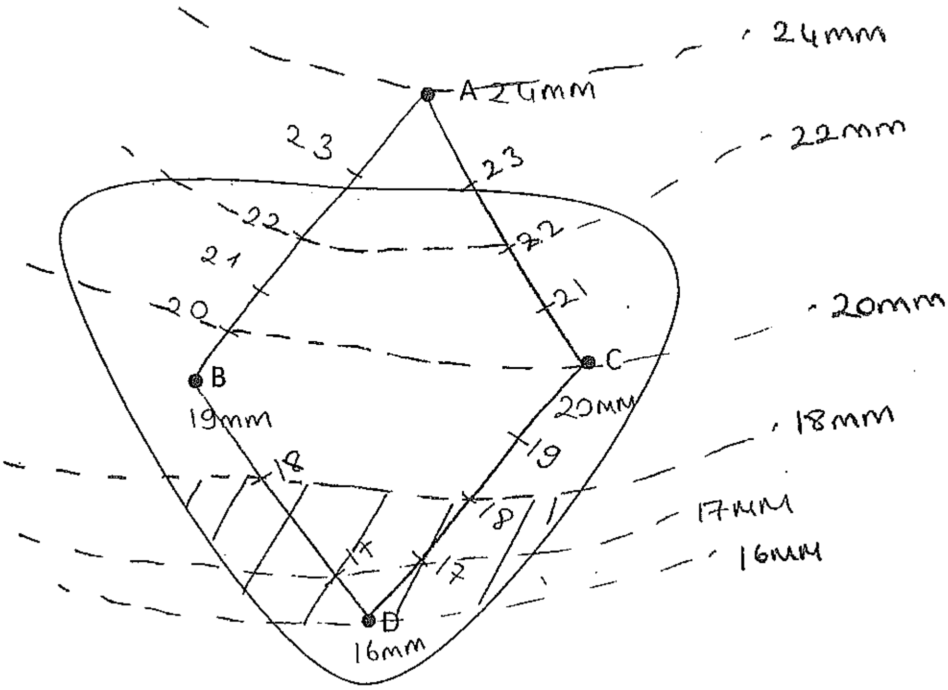


a) $P_{ave} = \dfrac{\sum_{i=1}^{n} P_i}{n} = \dfrac{19+20+16}{3} = 18.33\,mm$

(B, C, D stations)

b) $P_{ave} = \dfrac{\sum_{i=1}^{n} a_i P_i}{\sum a_i} = \dfrac{(25*24)+(141.1*19)+(105.6*20)+(104.3*16)}{376} = 18.78\,mm$

(burada A istasyonunu da gözönünde bulunduruyoruz) c)



$$P_{ave} = \frac{\sum_{i=1}^{n} a_i P_i}{\sum a_i} = \frac{(15*22.4)+(111*21)+(130*19)+(100*17)+(20*15.6)}{376}$$

$P_{ave} = 19.01\text{mm}$

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Recitation 3

PROBLEM: A uniform storm lasting 2 hours with intensity 11 mm/hr took place over a basin, which has a drainage area of 21.59 km². Using the given total storm hydrograph and assuming that the base flow is constant as 3 m³/s:

a) Determine the volume of surface runoff.
b) Determine the depth of surface runoff
c) Determine the Φ-index.
d) Determine the total depth of infiltrated water.

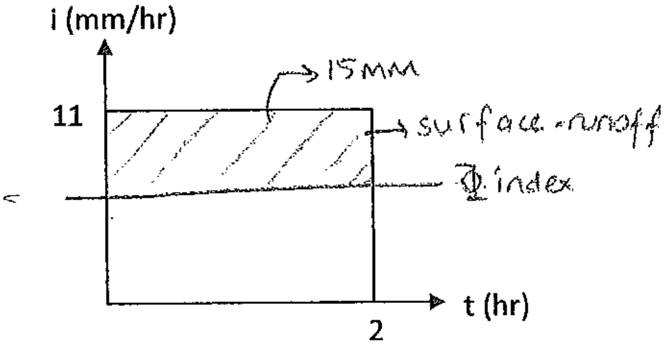

Figure 1. Hyetograph of the given storm

Table 1. Total storm hydrograph

t (hr)	TSH (m³/s)	Surface Runoff (m³/s) (Direct run-off)
0	3	0
1	12	9
2	30	27
3	33	30
4	21	18
5	9	6
6	3	0
		+ Σ 90 m³/s

a) Volume of surface run-off $= \Sigma Q \cdot \Delta t$ ⟶ dt = 1h integral'den dolayı dt ile çarpmalıyız.

$= 90 * 3600(sn)$

$= 324 * 10^3 m^3$

b) $d_{DR} = \frac{\Sigma Q . \Delta t}{A} = \frac{324*10^3}{21.59*10^6} = 0.015 m = 15 mm$

c) $(11 - \Phi) * 2 = 15 \longrightarrow \Phi = 3.5$ mm/hr

d) $3.5 * 2 = 7$ MM

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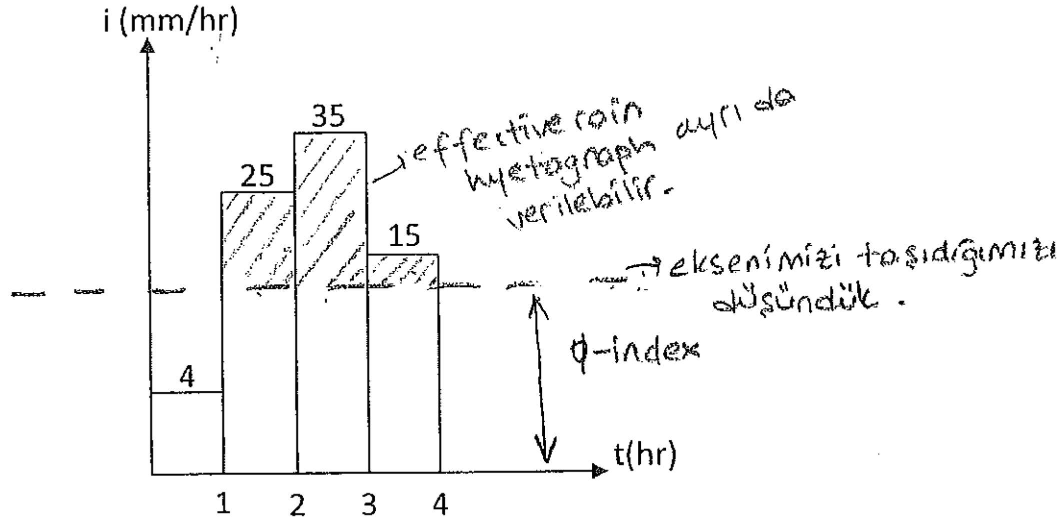
Recitation 4

PROBLEM: Total hyetograph and corresponding total storm hydrograph (TSH), which occurred over a basin of size 38.16 km^2 are given below. Base flow is assumed to be constant as 5 m^3/s for this storm. Determine the following;

a) Depth of surface runoff.
b) Φ-index.
c) Surface runoff equation in terms of unit hydrographs and lag times.
d) UH_2 of this basin.



t (hr)	TSH (m^3/s)	DR (m^3/s)	$2UH_1$	1hour lag $3UH_1$	2 hour lag UH_1	UH_1	1hr lag UH_1	$2UH_2$	UH_2
0	5	0	0	—	—	0	—	0	0
1	29	24	24	0	—	12	0	12	6
2	77	72	36	36	0	18	12	30	15
3	113	108	42	54	12	21	18	39	19.5
4	120	115	34	63	18	17	21	38	19
5	107	102	30	51	21	15	17	32	16
6	91	86	24	45	17	12	15	27	13.5
7	70	65	14	36	15	7	12	19	9.5
8	46	41	8	21	12	4	7	11	5.5
9	24	19	0	12	7	0	4	4	2
10	9	4		0	4		0	0	0
11	5	0			0				

$\Sigma q = 636$

$2UH_2 = UH_1 + (1hr\ lag)UH_1$

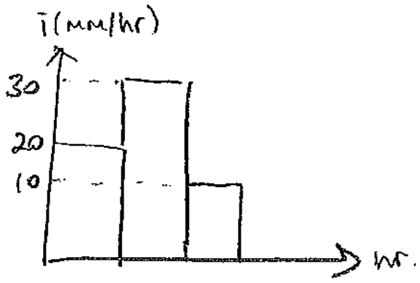
a) $d = \frac{\Sigma q \Delta t}{A}$ $\quad d = \frac{636 * 1 * 3600}{38.16 \times 10^6} = 0{,}06\,m = 6\,cm$

b) $(25-\phi)*1 + (35-\phi)*1 + (15-\phi)*1 = 60$

ϕ index $= 5\,mm/hr$

c) $2\,UH_1$ + (1hr lagged) $3\,UH_1$ + (2hr lagged) UH_1 = DR.

$\underset{\text{2cm}}{2}\ UH_{\underset{\text{1hour}}{1}}$



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Recitation 5

PROBLEM 1: The S-curve obtained from UH_2 of a basin is given below. Derive UH_3 by S-curve technique and determine the basin area.

Time (hr)	S-curve (m^3/s)	3 hour lag S_2	Difference	UH_3
0	0	—	0	0
1	10	—	10	6,67
2	25	—	25	16,67
3	44	0	44	29,33
4	65	10	55	36,67
5	82	25	57	38,00
6	96	44	52	34,67
7	107	65	42	28,0
8	116	82	34	22,67
9	122	96	26	17,33
10	126	107	19	12,67
11	128	116	12	8
12	129	122	7	4,67
13	130	126	4	2,67
14	130	128	2	1,33
15	130	129	1	0,67
		130	0	0

$$UH_{t_2} = \frac{t_1}{t_2}\left[S_{t1} - (t_2 \text{ hour lag}) S_{t_1}\right] \qquad \Sigma = 260.02 \rightarrow \Sigma q$$

$$UH_3 = \frac{2}{3}\underbrace{\left[S_2 - (3 \text{ hour lag})\, S_2\right]}_{\text{Difference}}$$

$$d_{DR} = \frac{\Sigma q \cdot \Delta t}{A} \qquad 0.01 = \frac{260.02 * 3600}{A} \qquad A = 93.6\ km^2$$

[unit hydrograph'da depth 1cm'dir!]

PROBLEM 2: An uncontrolled spillway will be designed for a small dam. The design flood hydrograph of the reservoir is given below. The storage – outflow relationship is approximated to a linear relationship:

$S = 7200 * Q$ where; Q: outflow, (m^3/s); S: storage, (m^3)

Using the routing equation;

$$(I_1 + I_2) + \left(\frac{2S_1}{\Delta t} - Q_1\right) = \left(\frac{2S_2}{\Delta t} + Q_2\right)$$

$\Delta t = 3600s$

$S = 7200 . Q$

$\frac{2S_1}{\Delta t} = 4Q$

$\frac{2S}{\Delta t} - Q = 3Q$

$\frac{2S}{\Delta t} + Q = 5Q$.

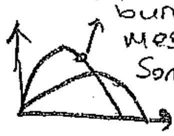
Determine;

a) The peak rate and time to peak of outflow hydrograph.
b) Attenuation and translation of outflow hydrograph.
c) The maximum storage for this reservoir.
d) The volume of flood storage gained in the reservoir during the rising stage.

Time (hr)	Inflow (m^3/s)	I_1+I_2	$\frac{2S_1}{\Delta t} - Q_1$ [3Q]	$\frac{2S_2}{\Delta t} + Q_2$ [5Q]	Q_2	I – Q
0	100	100	—	—	100	100–100
1	150	250	3*100	250+300	110	150–110
2	250	400	3*110	400+330	146.	
3	400	650				
4	800	1200				
5	1000	1800				
6	900					
7	700					
8	550					
9	400					
10	300					
11	250					
12	200					
13	150					
14	120					
15	100					

Time (hr)	Inflow (m^3/s)	I_1+I_2	[3Q] $2S_1/\Delta t-Q_1$	[5Q] $2S_2/\Delta t+Q_2$	Q_2	I-Q
0	100	100	-	-	100	0
1	150	250	300.0	550.0	110.0	40.0
2	250	400	330.0	730.0	146.0	104.0
3	400	650	438.0	1088.0	217.6	182.4
4	800	1200	652.8	1852.8	370.6	429.4
5	1000*	1800	1111.7	2911.7	582.3	417.7
6	900	1900	1747.0	3647.0	729.4	170.6
7	700	1600	2188.2	3788.2	757.6*	-57.6 → bunu almıyoruz
8	550	1250	2272.9	3522.9	704.6	
9	400	950	2113.8	3063.8	612.8	
10	300	700	1838.3	2538.3	507.7	
11	250	550	1523.0	2073.0	414.6	
12	200	450	1243.8	1693.8	338.8	
13	150	350	1016.3	1366.3	273.3	
14	120	270	819.8	1089.8	218.0	
15	100	220	653.9	873.9	174.8	
					Σ	1344.1

ΣQ = gain in storage



170.6 burada meselâ Sonra da -57. ya geç do su 170.6'yı alıyoru.

a) $Q_{max} = 757.6\ m^3/s$

$t_p = 7$ hours. (time to peak)

b) Attenuation: $1000 - 757.6 = 242.4\ m^3/s$

1000 → in 5 hours

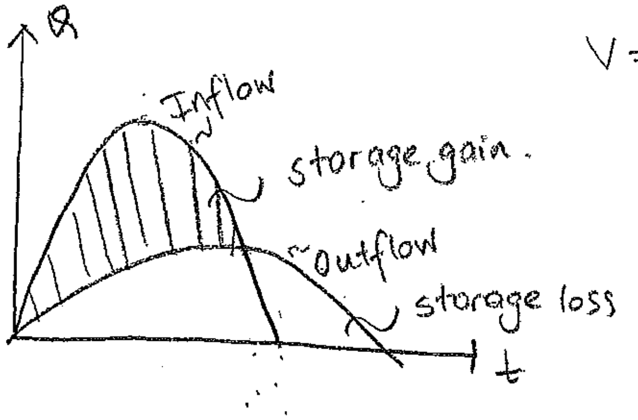
757.6 → in 7 hours time to peak

Translation: $7-5 = 2$ hours.

c) $\underbrace{7200 * Q_{max}}_{\text{Maximum storage}} = 757.6 * 7200 = 54.55 * 10^5\ m^3$

(su burada zaten vardı, maximum store ettiğim miktar)

d)



$V = \Sigma q \cdot \Delta t$

$= 1344.1 * 3600$

$= 48.39 * 10^5\ m^3$

↓

gain in storage

(15 saatlik zamandaki gain sadece)

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Recitation 6

PROBLEM: Monthly inflow volumes to a reservoir are given in the table below. Assuming 100% regulation policy with a repeating 12-month cycle and using mass curve analysis, determine the following:

a) Monthly withdrawal (demand).
b) Critical months.
c) Reservoir capacity.
d) Reservoir content at each month.

Month	Supply (10^6 m3)	Demand	S – D	Reservoir Content		Reservoir Condition
1	10	10	0	10+0=10		No change
2	16	10	6	10+6=16		
3	24	10	14	16+14=30		FILLING
4	13	10	3	30+3=33		
5	3	10	-7*	33-7=26	Second step.	
6	2	10	-8*	26-8=18		
7	3	10	-7*	18-7=11		EMPTYING
8	4	10	-6*	11-6=5		
9	5	10	-5*	0		
10	11	10	1	1	First step	
11	13	10	3	1+3=4		FILLING
12	16	10	6	4+6=10		

+

Σ120

a) $\Sigma S = \Sigma D$ (100% R.P.)

$\Sigma S = 120 * 10^6 m^3 = \Sigma D$

$D_{monthly} = \frac{120*10^6}{12} = 10*10^6$

b) 5, 6, 7, 8, 9th months.

c) Reservoir capacity

$|-7| + |-8| + |-7| + |-6| + |-5| = 33*10^6 m^3$

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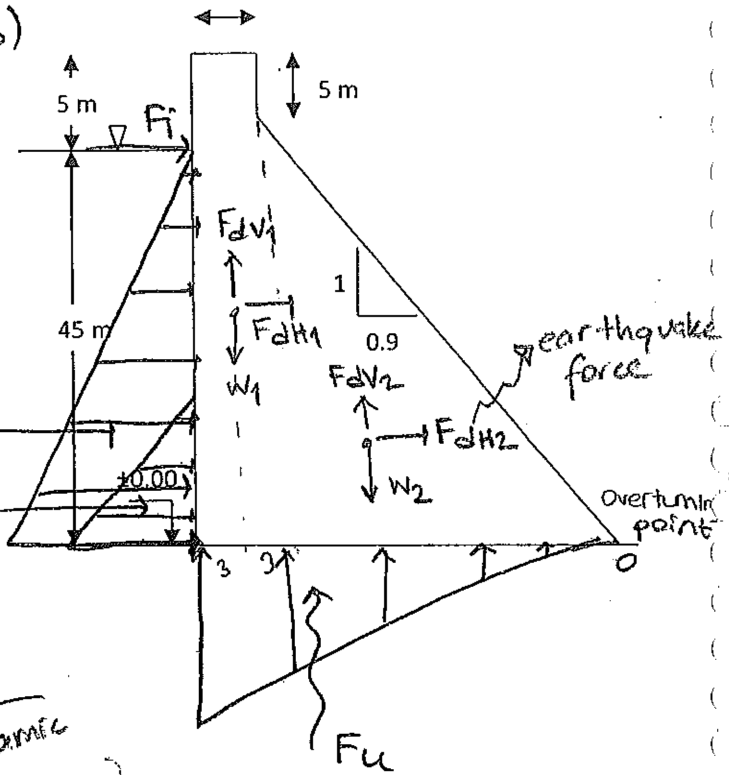
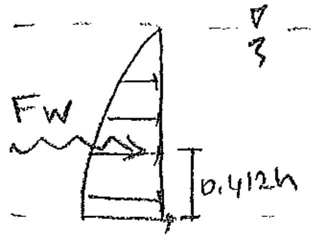
Recitation 7

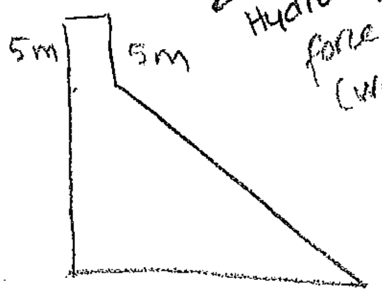
PROBLEM: A 50 m-high (from thalweg) concrete gravity dam will be designed. Carry out stability analysis under extreme loading. Assume that the whole dam acts as a monolithic structure. The dam is located in the 4th earthquake zone.

- Normal operating level: **45 m**
- Shear strength at the base level: **5 MPa** (τ_s)
- Compressive strength of concrete and foundation material: **30 MPa** and **25 MPa**, respectively
- Coefficient of friction at the base level: **0.75**
- Specific weights of concrete and water: **24 kN/m³** and **10 kN/m³**, respectively.
- Submerged specific weight of sediment accumulated in the reservoir: **11 kN/m³** with **θ=32°**, **h_s=5 m**
- Earthquake coefficients: **k_h=0.1, k_v=0.05**
- Ice force: **100 kN/m**
- There is no tailwater.
- Drainage system reduces the uplift pressure by 40%.



ALSO Hydrodynamic force (waves etc...)





FORMULAE

$$K_a = \frac{1-\sin\theta}{1+\sin\theta}$$

$$F_w = 0{,}726\, c\, k\, \gamma_w h_u^2$$

$$c = 0{,}7\left(1-\frac{\theta'}{90}\right)$$

(Bizde "0" burası)

Safety Criteria for extreme loading:

$FS_s > 1.0$, $FS_o > 1.2$ $\quad FS_{ss} > 1.0$ shear and sliding.

$\sigma_{max,concrete} \leq \sigma_c$

$\sigma_{max,foundation} \leq \sigma_f / 1.3$

After completing the tables:

$\Sigma M_o = 466117{,}02$ kN.m/m $\rightarrow FS_o = \dfrac{903690{,}00}{466117{,}02}$

$\Sigma M_r = 903690{,}00$ kN.m/m $FS_o = 1{,}93$ ✓ OK.

$\Sigma V = 21339$ kN/m

$\Sigma H = 1420335$ kN/m

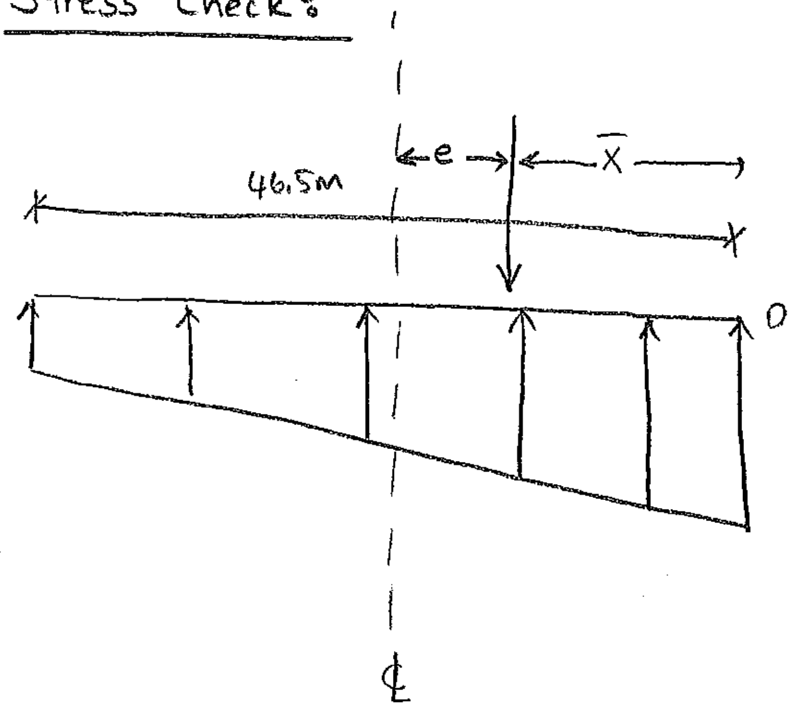
$\rightarrow FS_s = \dfrac{f\,\Sigma V}{\Sigma H} = \dfrac{0{,}75 \times 21339}{1420335} = 1{,}126 > 1{,}0$ ✓ OK

$\rightarrow \underbrace{FS_{ss}}_{\text{shear-end sliding}} = \dfrac{f\,\Sigma V + r\,\overbrace{A}^{\text{dam area}}\,\tau_s}{\Sigma H} = \dfrac{0{,}75 * 21339 + 1{,}0 * 46{,}5 * 5000}{14203{,}35}$

$= 17{,}49 > 1{,}0$ ✓ OK

$r = 1{,}0$ for extreme loading-case [r- changes from loading type]

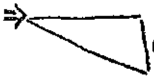
Stress Check:

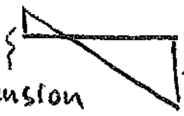


Point of application of the resultant force with respect to toe:

$\bar{X} = \dfrac{\Sigma M_r - \Sigma M_o}{\Sigma V} = 20{,}506$m

$e = B/2 - \bar{X} = 2{,}744$m

$e = B/6 \Rightarrow$  ~compression

$e > B/6 \Rightarrow$  ~compression

tension

$e < B/6 \Rightarrow$  ~compression

$M = \Sigma Ve = 21339 * 2{,}744 = 58554{,}216$ kN.m/m

$\sigma_{max,min} = \dfrac{\Sigma V}{A} \mp \dfrac{Mc}{I}$

$c = \dfrac{B}{2} = 23{,}25$m

$I = \dfrac{B^3}{12} = 8378{,}72\ m^4$/per m

$\sigma_{max,min} = \dfrac{21339}{46{,}5} + \dfrac{58554{,}216 * 23{,}25}{8378{,}72}$

$\sigma_{max} = 623{,}13$ kN/m²

$\sigma_{min} = 294{,}67$ kN/m²

$\sigma_{min} > 0$ ✓ O.K.

$\sigma_{max} < \sigma_c$ ✓ O.K. $\sigma_{max} < \sigma_{c}/(4.3)$ ✓ O.K.

Forces (kN/m)	Moment Arm about "O" (m)	Moment (kNm/m)
$W_1 = \gamma_c h b_1 = 24*50*6 = 7200 \downarrow$	$6*0.5+40.5=43.5$	313200
$W_2 = \gamma_c h_2 b_2 / 2 = 24*45*40.5*0.5 = 21870 \downarrow$	$40.5*(2/3)=27$	590490
$F_h = ½ (h_u)(\gamma_w h_u) = 0.5*45*10*45 = 10125 \longrightarrow$	$45*(1/3)=15$	151875
$F_u = \phi/2\ \gamma_w h_u B = 0.6*0.5*10*45*46.5 = 6277.5 \uparrow$ (drainage'i %40'la azalttık. 0.6'yla çarpıyoruz)	$46.5*(2/3)=31$	194602.5
$F_i = 100 \longrightarrow$	45	4500
$F_s = ½ \gamma_s h_s^2 K_a = 0.5*11*5^2*0.31 = 42.248 \longrightarrow$	$5*(1/3)=1.667$	70.41
$F_w = \mathbf{0.726}\ C k \gamma_w h_u^2 = 0.726*0.7*0.1*10*45^2 = 1029.105 \longrightarrow$	$45*0.412$	19079.61
$F_{dh1} = k_h W_1 = 0.1*7200 = 720 \longrightarrow$	$50*0.5=25$	18000
$F_{dh2} = k_h W_2 = 0.1*21870 = 2187 \longrightarrow$	$45*(1/3)=15$	32805
$F_{dv1} = k_v W_1 = 0.05*7200 = 360 \uparrow$	43.5	15660
$F_{dv2} = k_v W_2 = 0.05*21870 = 1093.5 \uparrow$	27	29524.5

$$K_a = \frac{1-\sin\theta}{1+\sin\theta} = \frac{1-\sin 32}{1+\sin 32} = 0.31$$

$$C = 0.7\left(1-\frac{\theta'}{90}\right) = 0.7$$

Safety Criteria for Extreme Loading;

$FS_c > 1.0$ $\quad FS_o \geq 1.2$ $\quad F.S_{ss} > 1.0$ $\quad \sigma_{max\,concrete} \leq \sigma_c$

$\sigma_{max\,found.} \leq \sigma_F / 1.3$

$\Sigma M_o = 466117.02\ kNm/m$ $\qquad \Sigma V = 21339\ kN/m$

$\Sigma M_r = 903690.00\ kNm/m$ $\qquad \Sigma H = 14203.35\ kN/m$

$$FS_o = \frac{903690.00}{466117.02} = 1.93 > 1.2 \quad \text{O.K.} \checkmark$$

$$FS_s = \frac{f\,\Sigma V}{\Sigma H} = \frac{0.75 \times 21339}{14203.35} = 1.126 > 1.0 \quad \text{O.K.} \checkmark$$

$$FS_{ss} = \frac{f\,\Sigma V + \tau A Z_s}{\Sigma H} = \frac{0.75 \times 21339 + 1.0 \times 46.5 \times 5000}{14203.35} = 17.496 > 1.0$$

O.K. ✓

Point of application of the resultant force with respect to toe;

$$\bar{x} = \frac{\Sigma M_r - \Sigma M_o}{\Sigma V} = 20.506m \qquad e = B/2 - \bar{x} = 2.744m$$

$B/6 = 7.75$

$B/6 > e$

Compression

$$M = \Sigma V \cdot e = 21339 \times 2.744 = 58554.216\ kNm/m$$

Forces (kN/m)	Moment Arm about "O" (m)	Moment (kNm/m)
$W_1 = \gamma_c h b_1 = 24 \times 50 \times 6 = 7200$ (↓)	$6 \times 0.5 + 40.5 = 43.5$	313200
$W_2 = \gamma_c h_2 b_2 / 2 = 24 \times 45 \times 40.5 \times 0.5 = 21870$ (↓)	$40.5 \times (2/3) = 27$	590490
$F_h = ½ (h_u)(\gamma_w h_u) = 0.5 \times 45 \times 10 \times 45 = 10125$ (→)	$45 \times (1/3) = 15$	151875
$F_u = \phi/2\, \gamma_w h_u B = 0.6 \times 0.5 \times 10 \times 45 \times 46.5 = 6277.5$ (↑)	$46.5\,(2/3) = 31$	194602.5
$F_i = 100$ (→)	45	4500
$F_s = ½ \gamma_s h_s^2 K_a = 0.5 \times 11 \times 5^2 \times 0.31 = 42.248$ (→)	$5 \times (1/3)$ 1.667	70.41
$F_w = 0.726\, C k \gamma_w h_u^2 = 0.726 \times 0.7 \times 0.1 \times 10 \times 45^2 = 1029.105$ (→)	45×0.412 18.54	19079.61
$F_{dh1} = k_h W_1 = 0.1 \times 7200 = 720$ (→)	50×0.5 25	18000
$F_{dh2} = k_h W_2 = 0.1 \times 21870 = 2187$ (→)	$45 \times (1/3)$ 15	32805
$F_{dv1} = k_v W_1 = 0.05 \times 7200 = 360$ (↑)	43.5	15660
$F_{dv2} = k_v W_2 = 0.05 \times 21870$ 1093.50 (↑)	27	29524.5

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Recitation 8

PROBLEM: A 200 m high dam will be constructed in a valley. There are two possible sites, 1 km apart from each other, having desirable geological formation, i.e. axis 1 and axis 2 shown in the Figure 1. The following possible alternatives will be constructed in this design.

1. Design of a concrete gravity dam using usual loading at axis 1 or axis 2. In the design take: t_c=10 m, H*=10 m, m=0, n=1, f=0.75, τ_s=5 MPa, σ_c=30 MPa, ϕ=0.6, σ_f=60 MPa, γ_c=24 kN/m^3, γ_w=10 kN/m^3, normal operating level=175 m, no tailwater, ignore silt and ice forces.

2. Design of an arch dam at axis 1 and axis 2, separately, using the simplified arch-rib analysis. In the design, consider: t_c=6 m, γ_w=10 kN/m^3, σ_{all}=6000 kN/m^2, and θ_a=133°

Compare the results according to the cross-sectional details of the designs and discuss your findings.

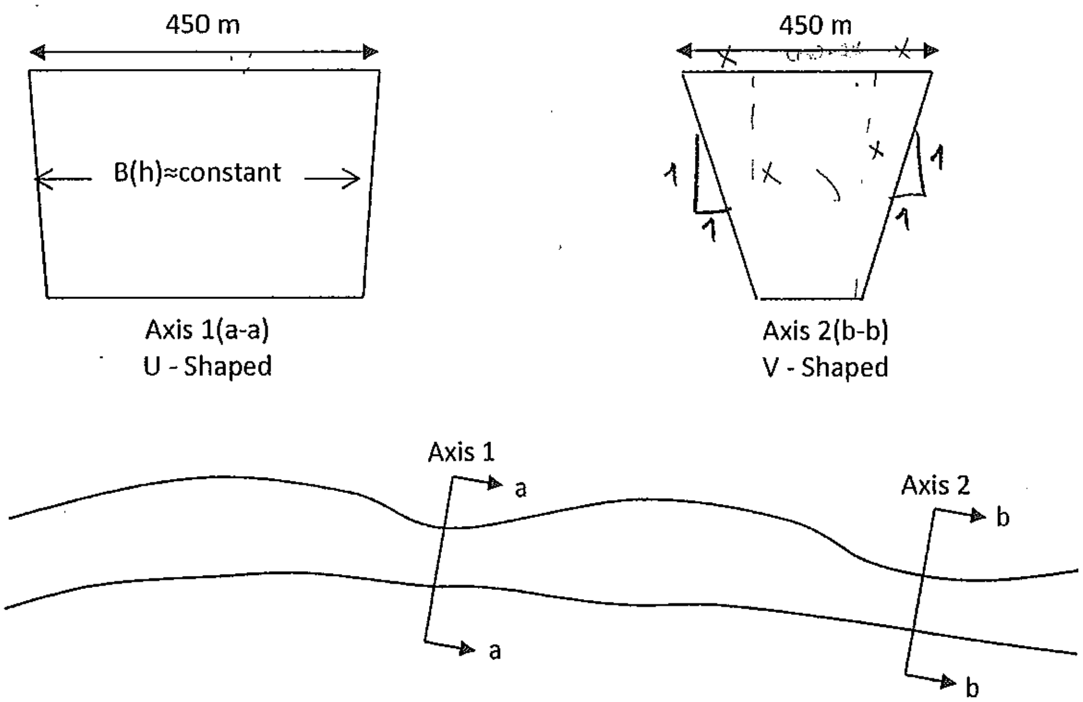
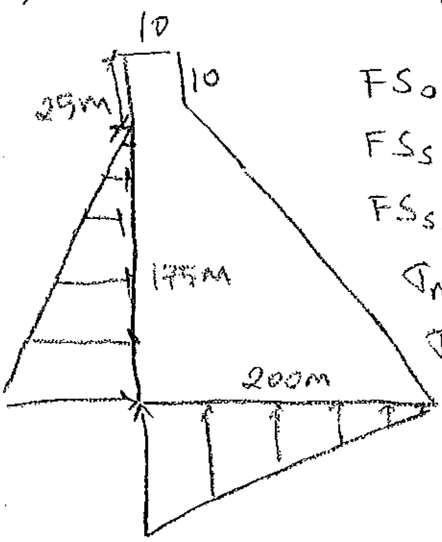


Figure 1. Cross sections of proposed dam locations

1)



$FS_o = 2.18 > 2.0$ ✓

$FS_s = 1.84 > 1.5$ ✓

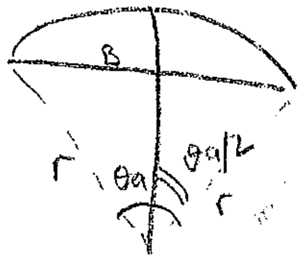
$FS_{ss} = 4.00 > 3.00$ ✓

$\sigma_{max} = 2432.96 < \sigma_f/4$ ✓

$\sigma_{min} = 1329.04 > 0$ ✓

2) Axis 1

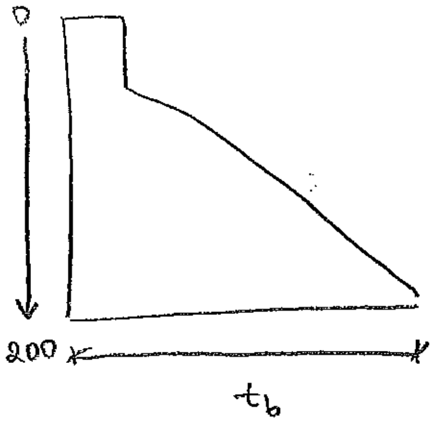
$B = 450m$, $r = \dfrac{B}{2\sin\frac{\theta_a}{2}}$



$\dfrac{B}{2} = r \cdot \sin\left(\dfrac{\theta_a}{2}\right)$ $r = \dfrac{B}{2\sin\frac{\theta_a}{2}}$

$$r = \frac{450}{2\sin\left(\frac{133}{2}\right)} = 245{,}35\,m \qquad t = \frac{\gamma \cdot h \cdot r}{\sigma_{all}} = \frac{10 * h * 245{,}35}{6000}$$

$$t = 0.409h$$



$$t_b = t(200) = 0.409 * 200 = 81.8\ m$$

$$t_{Hc} = t(H_c) = 6 = 0.409 * H_c$$

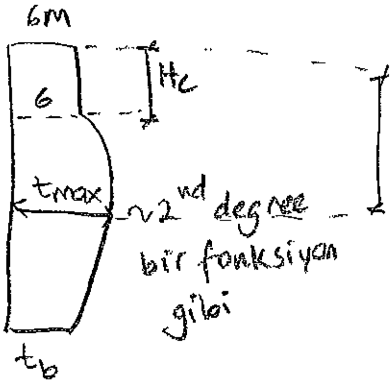
$$H_c = 14{,}7\,m$$

Axis 2

$$B(h) = 450 - 2h,$$

$$r(h) = \frac{450-2h}{2\sin\left(\frac{133}{2}\right)} = 245{,}35 - 1{,}09(h)$$

$$t(h) = \frac{\gamma \cdot h \cdot [245{,}35 - 1{,}09h]}{\underbrace{6000}_{\sigma_{all}}} = 0.409h - 0.0018h^2$$



- $t_b = t(200) = 9{,}8\,m$.
- $t_{Hc} \rightarrow t(H_c) = 6m \rightarrow 0{,}409H_c - 0{,}0018H_c^2 = 6$

$$H_c = 15{,}76\,m.$$

at t_{max}

$$\frac{dt}{dh} = 0$$

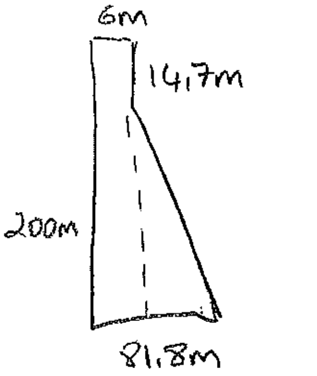
$$0{,}409 - 2 * 0{,}0018h = 0$$

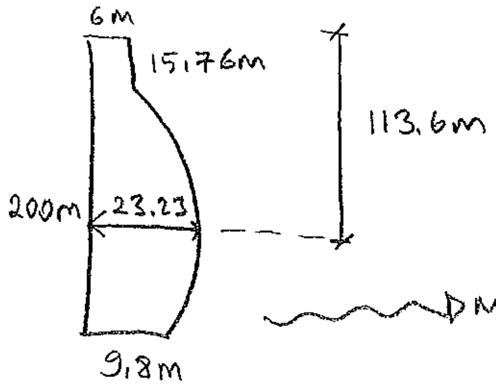
$$\rightarrow h = 113{,}61\,m = H_{max}$$

$$t_{max} = t(113{,}61) = 0{,}409(113{,}61) - 0{,}0018(113{,}61)^2$$

$$= 23{,}23\,m$$

Axis 1 (Arch Dam)





⟶ More economical!

$A_2 \cong 8223\,m^2$

$$A_3 = \int_{15,76}^{200} (0{,}409h - 0{,}0018h^2)\,dh + (6 * 15{,}76)$$

$$A_3 = 3495\,m^2$$

[Arch dam olması için gerekli koşullar nelerdir?]

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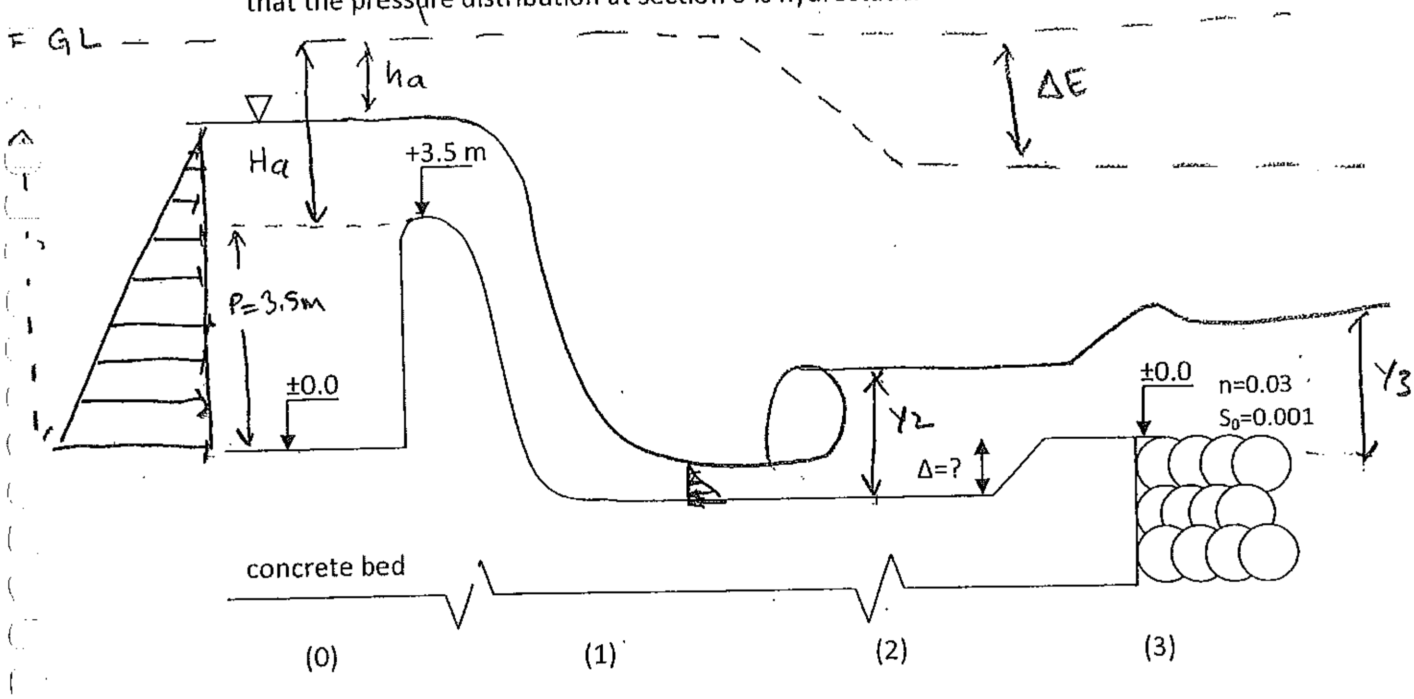
Recitation 9

PROBLEM: An uncontrolled overflow spillway shown in the figure below will be designed to evacuate $Q_0 = 370\ m^3/s$. The stilling basin, approach channel and tailwater section will be of rectangular cross-section, 60 m wide. Ignore the headlosses over the spillway face and at a possible end sill.

a) Determine the height of lowering the base elevation of the apron, Δ, at the toe of the spillway.
b) Determine the type and characteristic dimensions of the stilling basin to be designed.
c) Determine the percent energy loss through the hydraulic jump (between sections 1 & 2).
d) Determine the horizontal component of the resultant force acting on the spillway. Assume that the pressure distribution at section 0 is hydrostatic.



$$y_2 = \frac{y_1}{2}\left(\sqrt{1+8F_{r1}^{\ 2}}-1\right) \qquad \Delta E = \frac{(y_2-y_1)^3}{4y_1y_2} \qquad Q_0 = C_0 L H_0^{3/2}$$

(C_0 verildiğinde iteration yapmana gerek yok!)

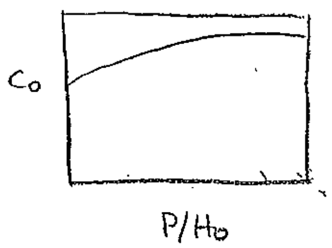
C_0, H_0: Bilinmiyor.

$Q_0 = 370\ m^3/s$

$L = 60\ m$

$P = 3.5\ m$

Assumed H_0	P/H_0	C_0	Q_0
1.75	2	2.171	301.51
2	1.75	2.167	367.7
2.008	1.743	2.166	370.0



page 118 in the book

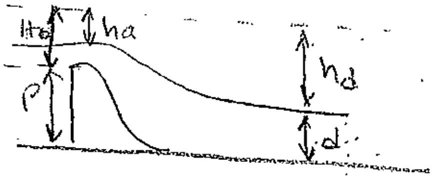
- at (0):

$E_0 = 2.008 + 3.5 = 5.508\ m$

- at (3):

$$Q = \frac{A}{n} R^{2/3}\sqrt{S_0} \longrightarrow 370 = \frac{(60y_3)}{0.03}\left(\frac{60y_3}{60+2y_3}\right)^{2/3}\sqrt{0.001} \longrightarrow y_3 = 2.998\ m$$

- Check for apron effect & Submergence effect



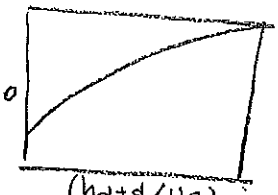
$\rightarrow \frac{h_d + d}{H_o} = \frac{2.51 + 2.998}{2.008}$

$= 2.743$

$\downarrow$

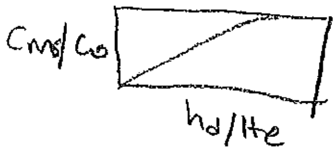
$C_{ma}/C_o = 1$

(no apron effect)



apron effect

$h_d = E_o - Y_3$



$\rightarrow \frac{h_d}{H_o} = \frac{2.51}{2.008} = 1.25 \rightarrow C_{MS}/C_o = 1$

(no submergence effect)

Submergence effect.

a)

$\Delta E = E_o - E_3$

$u_3 = q/y \quad (q = Q/B) = 370/60 = 60.160 \, m^3/s/m$

$E_3 = Y_3 + \frac{u_3^2}{2g} = 2.998 + \frac{(60.160/2.998)^2}{2 * 9.81} = 6.167 M$

$\Delta E = 2.294 m$

$\Delta E = \frac{(Y_2 - Y_1)^3}{4 Y_1 Y_2} \qquad Y_2 = \frac{Y_1}{2}\left(\sqrt{1 + 8Fr_1^2} - 1\right)$

$Y_1 = 0.616 m \qquad Y_2 = 3.255 m.$

$u_2 = q/Y_2 = 1.895 M/s$

$E_2 = Y_2 + \frac{u_2^2}{2g} = 3.438 M$

$E_3 + \Delta = E_2$ (apron elevation)

$\Delta = E_2 - E_3 = 3.438 - 3.214 = 0.224 M.$

d) (continued)

$F_{h2} = \frac{1}{2} \gamma Y_1^2 = \frac{1}{2} * 9.81 * 0.616^2$

$= 1.861 kN/M$

Momentum eqn btw (0) & (1)

$F_{h1} - F_R - F_{h2} = \rho Q \Delta u$

$60 * 145.317 - F_R - 60 * 1.861$

$= 1 * 370 * (10.011 - 1.133)$

$F_R = 5322.5 kN \; (\longleftarrow)$

- Tabular Solution: (page 151, table 4.6)

After finding ΔE, determine y_c.

$Y_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{6.167^2}{9.81}} = 1.571 m$

$\Delta E / Y_c = 1.46 \qquad Y_2/Y_1 = 5.286 \qquad Y_1/Y_c = 0.392$

$Y_1 = 0.616 M$

$Y_2 = 3.255 m$

b) page 133, Figure 4.29

$u_1 = q/Y_1 = 6.167/0.616 = 10.01 m/s$

$Fr_1 = u_1/\sqrt{gy} = 10.01/\sqrt{9.81 * 0.616} = 4.07$

Type IV basin

$L_{IV} = 6.1 * Y_2 = 6.1 * 3.255 = 19.856$

c) $E_1 = Y_1 + \frac{u_1^2}{2g} = 0.616 + \frac{(6.167/0.616)^2}{2 * 9.81} = 5.724.$

$E_2 = 3.438 \qquad \% \text{ energy loss} = \frac{E_1 - E_2}{E_1} * 100 = 40\%$

d) $E_o = Y_o + \frac{u_o^2}{2g} = Y_o + \frac{(q/Y_o)^2}{2g}$

$= 5.508 M$

$\rightarrow Y_o = 5.443 M$

$u_o = 1.133 m/s$

$F_{h1} = \frac{1}{2} \gamma Y_o^2 = \frac{1}{2} * 9.81 * 5.443 = 145.3$ kN/m

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Recitation 11

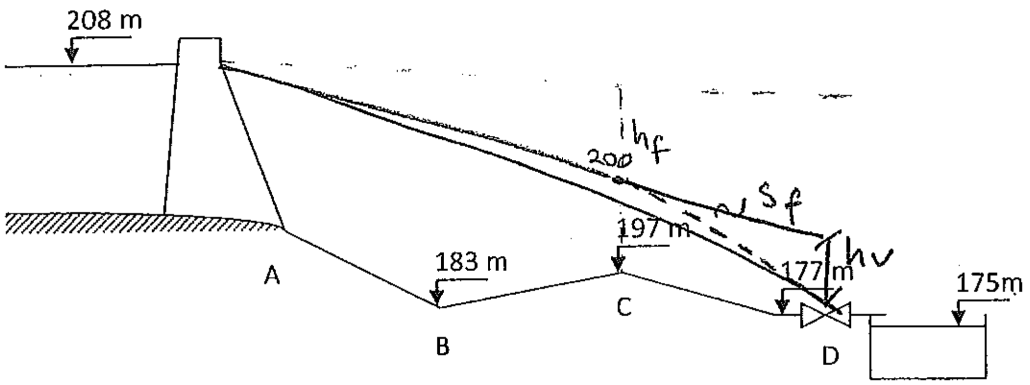
PROBLEM: Determine the pipe sizes of the following transmission line shown below which transmits Q=110 lt/s. Assume that f=0.02 and L=2000 m for all pipes and commercial pipes are available in 10 cm increment of diameter. Calculate also the head loss at the valve D.

$$h_f = \frac{8fL}{g\pi^2 D^5} Q^2 \qquad \left(h_f = f \frac{L}{D} \frac{u^2}{2g} \right)$$

Constant reservoir level: 208 m

Operational Requirements: $0.5 \text{ m/s} \le u \le 2.0 \text{ m/s}$

$3 \text{ m} \le P/\gamma \le 80 \text{ m}$



- Determination of the minimum friction slope: $\left(\frac{h_f}{L} = \text{friction slope} \right)$

$$S_{AC} = \frac{208-200}{4000} = 0.002 \ ✓$$

$$S_{AD} = \frac{208-177}{6000} = 0.005167 \ ✗$$

$S_{min} = S_{AC}$

- Determination of the diameter D_{AC}:

$$D = \left(\frac{8fQ^2}{g\pi^2 S_f} \right)^{1/5} \rightarrow D_{AC} = \left(\frac{8 \times 0.02 \times 0.11^2}{9.81 \pi^2 \, 0.002} \right)^{1/5} = 0.4 \text{ M} = 400 \text{ MM}$$

(mesela D=0,38 olsaydı, D=0,40'a yuvarlanmamız gerekirdi)

- chosen diameter: $D_{AC} = 0.4 \text{ M}$

$$u_{AC} = \frac{Q}{A_{AC}} = \frac{0.11}{\pi (0.4)^2/4} = 0.88 \text{ m/s. O.K.}$$

(Eğer D=0,4 gibi tam bir değer çıkmamış olsaydı, 200'den büyük bir head olacaktı.

Buradan hesaplarız: $H_C = H_A - h_{fAC}$ → yeni hız

$4000 \times 0{,}88^2$

- The friction slope of segment C-D:

$$S_{fCD} = \frac{200-177}{2000} = 0.0115$$

$$D_{CD} = \left(\frac{8 * 0.02 * 0.11^2}{9.81\, \pi^2\, 0.0115}\right)^{1/5} = 0.28m = 28cm = 280MM$$

chosen $D_{CD} = 0.3m = 30cm$. (10 CM increments in pipe diameters)

$$u_{CD} = \frac{Q}{A_{CD}} = \frac{0.11}{\pi \frac{0.3^2}{4}} = 1.56 m/s \quad O.K.$$

[Diameter'i daha büyük seçtiğimiz için vanada headloss oluşacak]

$$H_C = h_{fCD} + h_v + Z_D$$

$$200 = 0.02 \frac{2000}{0.3} \frac{1.56^2}{2*9.81} + h_v + 177m$$

$$h_v = 6.25m.$$

$$K_a = \frac{1-\sin\theta}{1+\sin\theta} = \frac{1-\sin 32}{1+\sin 32} = 0.31$$

$$C = 0.7\left(1-\frac{\theta'}{90}\right) = 0.7$$

Safety Criteria for Extreme Loading;

$FS_c > 1.0$ $\quad FS_o \geq 1.2$ $\quad F.S_{ss} > 1.0$ $\quad \sigma_{max\,concrete} \leq \sigma_c$

$\sigma_{max\,fund.} \leq \sigma_F / 1.3$

$\Sigma M_o = 466117.02$ kNm/m $\qquad \Sigma V = 21339$ kN/m

$\Sigma M_r = 903690.00$ kNm/m $\qquad \Sigma H = 14203.35$ kN/m

$$FS_o = \frac{903690.00}{466117.02} = 1.93 > 1.2 \quad \text{OK} \checkmark$$

$$FS_s = \frac{f\,\Sigma V}{\Sigma H} = \frac{0.75 \times 21339}{14203.35} = 1.126 > 1.0 \quad \text{O.K.} \checkmark$$

$$FS_{ss} = \frac{f\,\Sigma V + r A \tau_s}{\Sigma H} = \frac{0.75 \times 21339 + 1.0 \times 46.5 \times 5000}{14203.35} = 17.496 > 1.0$$

O.K. ✓

Point of application of the resultant force with respect to toe;

$$\bar{x} = \frac{\Sigma M_r - \Sigma M_o}{\Sigma V} = 20.506\text{m}$$

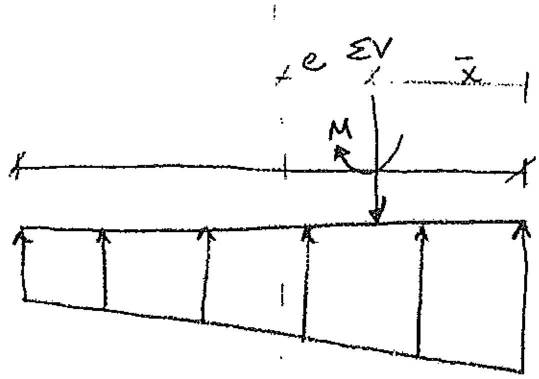
$e = B/2 - \bar{x} = 2.744$m

$B/6 = 7.75$

$B/6 > e$

Compression

$M = \Sigma V \cdot e = 21339 \times 2.744 = 58554.216$ kNm/m



Base pressure distribution

$$\sigma_{max,min} = \frac{\Sigma V}{A} \pm \frac{Mc}{I}$$

$c = B/2 = 23.25 m$

$I = \frac{B^3}{12} = 8378.72\ m^3$

$$\sigma_{max} = \frac{21\,339}{46.5} + \frac{58\,554.22 \times 23.25}{8378.72} = 623.13\ kN/m^2$$

$\sigma_{max} < \sigma_c$ O.K. ✓

$\sigma_{min} = 294.67\ kN/m^2$

$\sigma_{max} < \sigma_f / 1.3$ O.K. ✓

$\sigma_{min} > 0$ O.K. ✓

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Fall 2012-2013

Recitation 12

PROBLEM: The layout of a separate sewer system is shown in Figure 1. Design the storm and sanitary sewers between manholes 6 and 8. By investigating the topographical characteristics of the city, the flow directions in sewers are estimated as shown in Figure 1. A typical cross-section of a trench is shown in Figure 2. The design criteria for both storm and sanitary sewer systems are given below:

- Manning's roughness coefficient, N_{full} is 0.016 for all pipes and is variable.
- Maximum allowable flow velocity, u_{max} is 4 m/s.
- Minimum allowable full flow velocity, u_{min} is 0.6 m/s.
- Street slopes are 0.01

For the storm sewer design, apply the rational method and use the rainfall intensity-duration-frequency curves given in Figure 3. Consider the following data for the storm sewer system:

- Inlet times for all areas are 10 minutes.
- The flow time between two successive manholes is 2 minutes.
- T_r =25 years.
- Runoff coefficient, C for all areas are 0.7.
- Pipe sizes are available for every 50 mm increments of diameter. Take D_{min} as ϕ300.

For the sanitary sewer system, the following data are given:

- $Q_{average}$ = 165.625 lt/s
- 70% of the average daily demand returns to the sanitary sewer system.
- Groundwater infiltration is 0.4 lt/s/ha.
- Rainfall contribution is 0.5 lt/s/ha.
- Minimum depth of flow is 2 cm.
- Pipe sizes are available for every 50 mm increments of diameter. Take D_{min} as ϕ200.

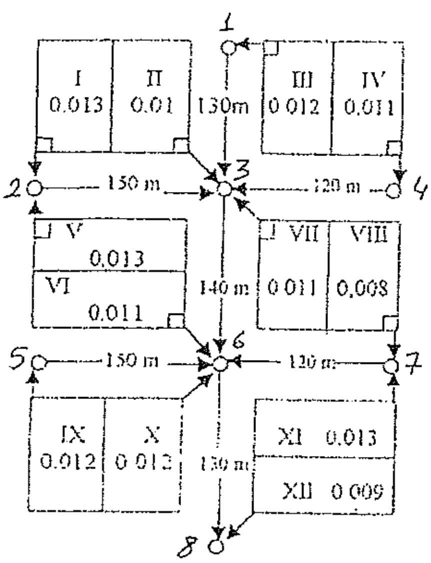

Figure 1 Layout of the system (areas in km²)

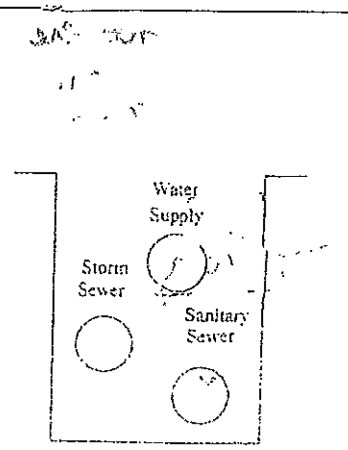

Figure 2 Bedding of pipes in the trench

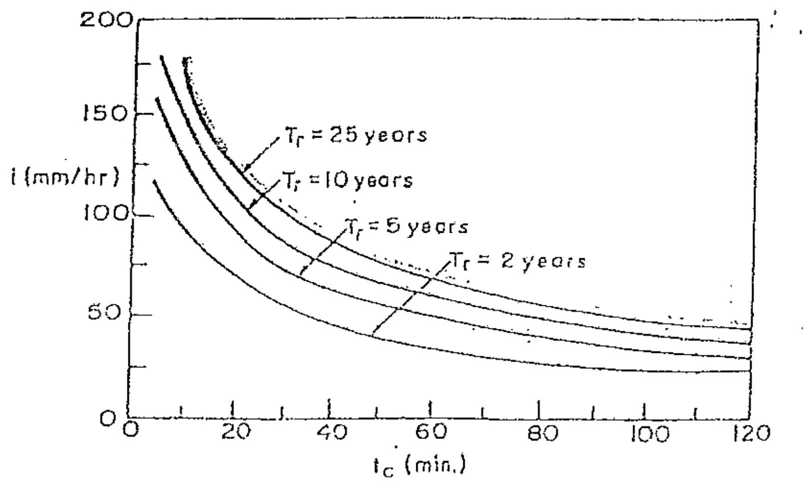

Figure 3 Rainfall intensity-duration-frequency curves

$Q_{av} = 165{,}625$ lt/s (verilen)

$Q_{ave} = 165{,}625 * 0{,}7 = 115{,}9375$ lt/s

$Q_{des} = PF_{max} * Q_{av} + GW_I$ + Rainfall contribution

$Q_{low} = PF_{dry} * Q_{av} + GW_I$

(page 379, Yanmaz)

$Q_{des} = 2{,}6 * 115{,}9375 + 5{,}04 + 6{,}3 = 312{,}781$ lt/s

$Q_{low} = 1{,}6 * 115{,}9375 + 5{,}04 = 190{,}5$ lt/s

- Minimum Slope (page 380, Yanmaz)

$S_{0,6} = 0.09\,\% < 0.01$ ✓

- Manning Equation

$0{,}31278 = \frac{0{,}312}{0.016} D^{8/3} \sqrt{0.01} \Rightarrow D = 0{,}503$ m

select D = 550 mm

$Q_{full} = \frac{0{,}312}{0.016} (0{,}55)^{8/3} \sqrt{0.01} = 0{,}396\ m^3/s$

$u_{full} = 1{,}67$ m/s.

(from table)

$\frac{Q_{des}}{Q_{full}} = 0{,}79 \rightarrow \frac{d}{D} = 0{,}75$

$\frac{u_{design}}{u_{full}} = 0{,}98 \rightarrow u_{design} = 1{,}64$ m/s OK ✓

$\frac{Q_{low}}{Q_{full}} = \frac{190{,}5}{396} = 0{,}48$

$\rightarrow \frac{d}{D} = 0{,}54 > 0{,}5$

$< 0{,}75$

OK ✓.

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Spring 2009-2010

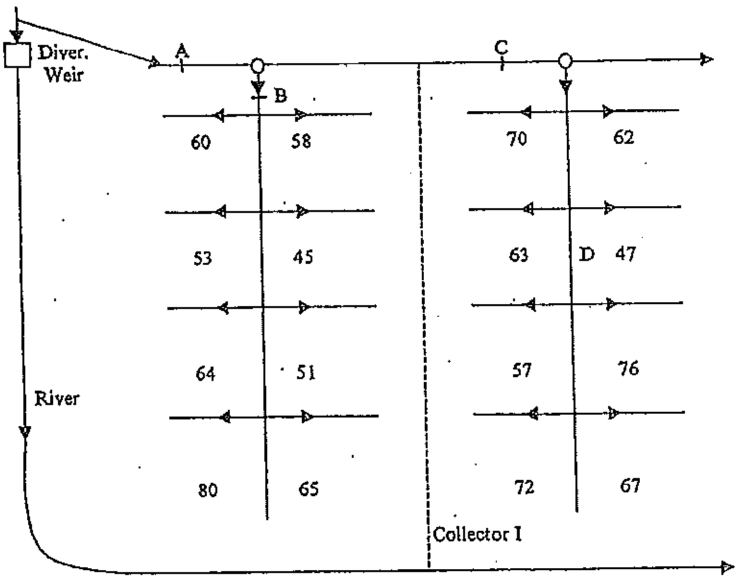
Recitation 13

PROBLEM 1: Preliminary layout of a classical irrigation network is shown in figure. Various types of crops are planned for cultivation. The monthly weighted average values of the crops are shown in the table below. The overall irrigation efficiency is 60%.

a) Determine the capacities of irrigation canals at the points indicated on the figure using demand method.

b) Determine the section dimensions of the trapezoidal irrigation channels at points A & C having side slopes of 1V:1.5H and bottom widths of b=1.5 m & 1.0 m (at A & C respectively). Compute the average flow velocities in channels. Take S_0=0.0006 and n=0.015.

Month	Weighted Average CIR (mm/month)
June	154
July	193
August	186
September	73



*All units are in hectares.

PROBLEM 2: Compute the spacing between two successive drains under steady state case by using Donnan's equation for the following data. The root zone depth is 1.5 m. Drains are located at 1.8 m below the ground surface. Take D=6.7 m, q=1.2 mm/day, K=0.5 m/day, and r_0=6.25 cm.

$CIR_{max} = 193$ mm/month (for July)

$TDR_{max} = CIR_{max} / e$

$TDR_{max} = 193 / 0.6 = 321.67$ mm/month

$$q_{max} \; (lt/s/ha) = \frac{TDR_{max} \; 10000}{(30-31) \times 86400} = \frac{321.67 \times 10000}{31 \times 86400} = 1.2 \; lt/s/ha$$

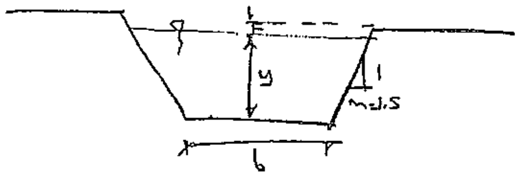
$$q_{max} = TDR \left(\frac{mm}{month}\right) \frac{1 \text{ month}}{(30\text{-}31) \times 86400 \text{ sec}} \times \frac{10000 \; m^2}{1 \; ha} \times \frac{1 m}{1000 \; mm} \times \frac{1000 \; lt}{1 m^3}$$

$Q = q \times F \times A$

page 475 table 10.8

Point	Area (ha)	q_{max}	F	Q (lt/s)
A	990		1.21	1437.5
B	476	1.2	1.31	748.3
C	514		1.30	801.8

b)



$S_o = 0.0006$

$n = 0.015$

$A = y(b + my)$

$P = b + 2y\sqrt{m^2+1}$

$f = 0.2(1+y)$ given

$$Q = \frac{1}{n} A \, R^{2/3} \, S_o^{1/2}$$

For point A

$b = 1.5$ m

$Q_A = 1.44 \; m^3/s$

$A = y(1.5 + 1.5y)$

$P = 1.5 + 2y\sqrt{1.5^2+1}$

$= 1.5 + 3.61y$

$$1.44 = \frac{y(1.5+1.5y)}{0.015} \left(\frac{y(1.5+1.5y)}{1.5+3.61y}\right)^{2/3} \sqrt{0.0006}$$

$y = 0.643$ m

$f = 0.2(1+0.643) = 0.33$ m

$u_A = \frac{Q_A}{A_A} = 0.91$ m/s

$0.5 \text{ m/s} < u_A < 2.5 \text{ m/s}$ ok

For point C

$b = 1.0$ m

$Q_c = 0.8 \; m^3/s$

$A = y(1 + 1.5y)$

$P = 1 + 2y\sqrt{1.5^2+1}$

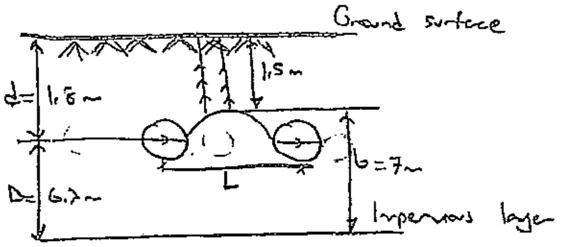
$= 1.0 + 3.61y$

$$0.8 = \frac{y(1.0+1.5y)}{0.015} \left(\frac{y(1.0+1.5y)}{1.0+3.61y}\right)^{2/3} \sqrt{0.0006} \qquad \boxed{y = 0.533 \text{ m}}$$

$f = 0.2(1+0.533) = 0.31$ m

$u_c = \frac{Q_c}{A_c} = 0.79$ m/s OK ✓

2)



$$L^2 = \frac{4K(b^2 - D^2)}{q}$$

$$L^2 = \frac{4 \times 0.5 \times (7^2 - 6.7^2)}{1.2 \times 10^{-3}}$$

$\boxed{L = 82.8 \text{ m}}$

dolu savak.

PROBLEM 2: The capacity of a spillway is 400 m³/s. Using the 11 years of data provided below, calculate the risk for the spillway, for an economical life of 50 years (Use normal distribution). Also find the probability of design flood occurring exactly twice in 10 years.

Year	Q_{peak} (m^3/s)
1997	273
1998	240
1999	279
2000	350
2001	293
2002	200
2003	305
2004	281
2005	294
2006	390
2007	296

$$\mu = \frac{\Sigma Q}{11} = \frac{3201}{11} = 291\ m^3/s$$

$$\sigma = \sqrt{\frac{\Sigma(Q-\mu)^2}{\underbrace{(n-1)}_{10}}} = \sqrt{2496{,}6} = 49{,}97\ m^3/s$$

$$Risk = 1 - q^n = 1 - (\underbrace{1-p}_{\text{gelme olasılığı}})^n$$

$$z = \frac{Q-\mu}{\sigma} = \frac{400-291}{49{,}97} = 2{,}18 \quad (\text{page 260 / Table 9,1})$$

$$p = 0{,}014629 \,/\, q = 0{,}985371$$

$$Risk = 1 - 0{,}985371^{50} = 0{,}521 = \boxed{52{,}1\ \%}$$

$10 - 8 = 2$

Probability of (n-k) occurances

(k) non occurances

$\downarrow$

8

hangi 2 yıl bilmiyoruz.

$$= \binom{n}{k} p^{(n-k)} q^k$$

$$\text{2 occurances} = \binom{10}{8} p^2 q^8 = \frac{10!}{8!\,2!} (0{,}014629)^2 \cdot (0{,}985371)^8 = 0{,}008589$$

$$= 0{,}86\%$$

②

PROB 2 :

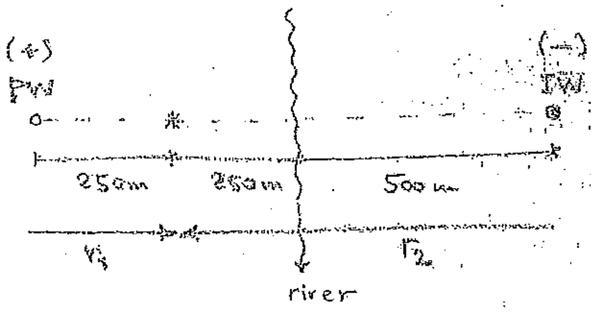
$b' = 10$ m

$K' = 3\times10^{-9}$ m/s

$S = 0.0001$

$Q = 0.1$ m^3/s (constant)

$b = 40$ m



permeability

a) SOIL TYPE : Medium Sand $\xrightarrow{\text{(Table 12.4, p. 338)}}$ $\boxed{K = 12 \text{ m/day}}$

$T = Kb$

$$T = \left(12 \frac{m}{day}\right)(40\ m)\left(\frac{1\ day}{86400\ s}\right) \Rightarrow \boxed{T = 0.0056\ m^2/s}$$

For aquitard: $B^2 = \frac{Tb'}{K'} \Rightarrow B = \sqrt{\frac{(0.0056)(10)}{3\times10^{-9}}} \Rightarrow \boxed{B = 4320.49\ m}$

$\frac{r_1}{B} = 0.0579$ $\frac{r_2}{B} = 0.1736$

$$u_1 = \frac{r_1^2 S}{4Tt} = \frac{(250)^2(0.0001)}{4(0.0056)(30)(3600)} = 2.5835\times10^{-3}$$

$$\xrightarrow[\text{Table 12.6}]{\text{p. 355}} \boxed{W\left(u_1, \frac{r_1}{B}\right) = 5.129 \quad (+)}$$

$$u_2 = \frac{r_2^2 S}{4Tt} = \frac{(750)^2(0.0001)}{4(0.0056)(30)(3600)} = 2.3251\times10^{-2}$$

$$\xrightarrow[\text{Table 12.6}]{\text{p. 355}} \boxed{W\left(u_2, \frac{r_2}{B}\right) = 2.955 \quad (-)}$$

$$S = \frac{0.1}{4\pi(0.0056)}\left[5.129 - 2.955\right] \Rightarrow \boxed{S = 3.0893\ m}$$

b) Drawdown in the Observation Well 2 will be zero, since it is on the other side of the boundary