ESE GATE PSUs

State Engg. Exams

MADE EASY WORKBOOK 2024

Detailed Explanations of Try Yourself *Questions*

Civil Engineering

Engineering Hydrology

MADE EASY PUBLICATIONS

1 Precipitation and General Aspects of Hydrology

Detailed Explanation of Try Yourself Questions

T1 : Solution

The calculations are tabulated below:

Isohyetal Interval (cm)	Average value of precipitation (cm)	Inter-Isohyetal area (km^2)	Fraction of total area col. 3/640	Weighted p(cm) col. 2 × col. 4
(1)	(2)	(3)	(4)	(5)
14-12	13	90	0.1406	1.8278
12-10	11	140	0.2187	2.4062
10-8	9	125	0.1953	1.7578
8-6	7	140	0.2187	1.5312
6-4	5	85	0.1328	0.6641
4-2	3	40	0.0625	0.1875
2-0	1	20	0.0312	0.0312
		$\Sigma A = 640$		$\Sigma p = 8.40625$

Thus average depth of precipitation over the basin is 8.40625 cm.

T2 : Solution

As the normal rainfall values vary more than 10%, the normal ratio method is adopted.

$$P_D = \frac{92.01}{3} \times \left(\frac{91.11}{80.97} + \frac{72.23}{67.59} + \frac{79.89}{76.28} \right) = 99.40 \text{ cm}$$

T3 : Solution

The scale of map is 1 : 50,000. It means that 1 cm on the map represents 50,000 cm on the ground.

$\therefore$ 1 cm on map = 50,000 cm on ground

= 500 m on the ground

= 0.5 km on the ground

$\therefore$ 1 cm^2 on the map = 0.5×0.5 km^2 on the ground

= 0.25 km^2 on the ground

The calculations are tabulated below:

Map Area (cm^2)	Ground Area(km^2) col.1 $\times$ 0.25	Fraction total area col.2/25	Rainfall (cm)	Weighted P (cm) col.3 $\times$ col.4
1	2	3	4	5
25	6.25	0.25	125	31.25
30	7.5	0.30	175	52.50
30	7.5	0.30	225	67.50
10	2.5	0.10	275	27.50
5	1.25	0.05	325	16.25
Total	25	1.00		195

Mean depth of the rainfall = 195 cm

Volume of rainfall = $1.95 \times 100 \times (50000)^2 \times 10^{-4}\ m^3 = 48.75\ Mm^3$

Average annual discharge at the outlet is given by

$$Q = \frac{\text{Run off coefficient} \times \text{Volume of rainfall}}{365 \times 24 \times 60 \times 60}$$

$$\Rightarrow \quad Q = \frac{0.3 \times 487500}{31536000} \times 100 = 0.464\ m^3/s$$

T4 : Solution

The equivalent uniform depth is given by

$$P = \frac{\left\{60 \times \left(\frac{75+90}{2}\right) + 275\left(\frac{90+100}{2}\right) + 260\left(\frac{100+125}{2}\right) + 150\left(\frac{125+140}{2}\right) + 380\left(\frac{140+150}{2}\right) + 215\left(\frac{150+165}{2}\right) + 120\left(\frac{165+180}{2}\right)\right\}}{(60+275+260+150+380+215+120)}$$

$$= \frac{189862.5}{1460} = 130.04$$

If the coefficient of runoff, $c = 0.4$ then.

Depth of flow = $cP = 0.4 \times 130.04 = 52.02$ mm

Volume of runoff = $52.02 \times$ Area of the catchment

$= 52.02 \times (60 + 275 + 260 + 150 + 380 + 215 + 120) \times 10^6$

$= 75949.2 \times 10^6\ mm^3$

$= 75.95\ m^3$

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2 Evaporation, Transpiration, Evapotranspiration and Stream Flow Measurement

Detailed Explanation of Try Yourself Questions

T1 : Solution

Let us use subscripts 1, 2 and 3 for U/S section, D/S section and middle section

Now, hydraulic mean depth, $R_1 = \frac{A_1}{P_1} = \frac{108.6}{65.3} = 1.663$ m

$$R_2 = \frac{A_2}{P_2} = \frac{99.80}{59.40} = 1.680 \text{ m}$$

$$R_3 = \frac{A_3}{P_3} = \frac{103.1}{60.7} = 1.699 \text{ m}$$

Conveyance, $K_1 = \frac{1}{n} A_1 R_1^{2/3} = \frac{1}{0.029} \times (108.6) \times (1.663)^{2/3} = 5256.46$

$$K_2 = \frac{1}{n} A_2 R_2^{2/3} = \frac{1}{0.029} \times (99.80) \times (1.680)^{2/3} = 4863.39$$

$$K_3 = \frac{1}{n} A_3 R_3^{2/3} = \frac{1}{0.029} \times (103.10) \times (1.699)^{2/3} = 5062.01$$

Average conveyance is given by

$$K_{avg} = (K_1 \times K_2 \times K_3)^{1/3}$$
$$= (5256.46 \times 4863.39 \times 5062.01)^{1/3} = 5058.14 \text{ m}$$

(i) 1st iteration

Assuming, $v_1 = v_2$

$$h_f = (h_1 - h_2) + \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right) - K\left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right)$$

Friction loss, $h_f = (h_1 - h_2) = 316.80 - 316.55 = 0.25$ m

Now, $Q = K_{avg}\sqrt{\frac{h_f}{L}} = 5058.14\sqrt{\frac{0.25}{250}} = 159.952 \text{ m}^3/\text{sec}$

$$v_1 = \frac{Q}{A_1} = \frac{159.952}{108.6} = 1.473 \text{ m/s}$$

$$v_2 = \frac{Q}{A_2} = \frac{159.952}{99.8} = 1.603 \text{ m/s}$$

(ii) 2nd iteration

Take, $K = 0.1$ for gradual contraction

$$h_f = (h_1 - h_2) + \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right) - K\left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right)$$

$$= (316.8 - 316.55) + \left(\frac{1.47^2}{2\times 9.81} - \frac{1.602^2}{2\times 9.81}\right) - 0.1\left(\frac{1.47^2}{2\times 9.81} - \frac{1.602^2}{2\times 9.81}\right)$$

$$= 0.25 - 0.021 + 0.0021$$

$$= 0.23166 \text{ m}$$

$$Q = K_{avg}\sqrt{\frac{h_f}{L}} = 5057.40 \times \sqrt{\frac{0.228}{250}} = 153.973 \text{ m}^3\text{/sec}$$

$$v_1 = \frac{Q}{A_1} = \frac{153.96}{108.6} = 1.42 \text{ m/s}$$

$$v_2 = \frac{Q}{A_2} = \frac{153.96}{99.8} = 1.543 \text{ m/s}$$

(iii) 3rd iteration

$$h_f = 0.25 + \left(\frac{1.406^2}{2\times 9.81} - \frac{1.53^2}{2\times 9.81}\right) - 0.1\left(\frac{1.406^2}{2\times 9.81} - \frac{1.53^2}{2\times 9.81}\right)$$

$$= 0.25 - 0.0186 + 0.00186 = 0.2333 \text{ m}$$

$$Q = K_{avg}\sqrt{\frac{h_f}{L}} = 5057.40\sqrt{\frac{0.234}{250}} = 154.512 \text{ m}^3\text{/sec}$$

$$v_1 = \frac{Q}{A_1} = \frac{154.73}{108.6} = 1.423 \text{ m/s}$$

$$v_2 = \frac{Q}{A_2} = \frac{154.73}{99.8} = 1.548 \text{ m/s}$$

(iv) 4th iteration

$$h_f = 0.25 + \left(\frac{1.425^2}{2\times 9.81} - \frac{1.55^2}{2\times 9.81}\right) - 0.1\left(\frac{1.425^2}{2\times 9.81} - \frac{1.55^2}{2\times 9.81}\right) = 0.233 \text{ m}$$

$$Q = K_{avg}\sqrt{\frac{h_f}{L}} = 5057.40\sqrt{\frac{0.233}{250}}$$

$$= 154.42 \text{ m}^3\text{/sec}$$

As the value of *Q* found from 3rd iteration and 4th iteration are close each other so iteration process ends here

$\therefore \qquad Q_{eq} = 154.42 \text{ m}^3/\text{s}$

T2 : Solution

The characteristic equation is given as

$$V(\text{m/s}) = 0.65\,N + 0.03$$

where N is the number of revolutions per second.

Assuming depth at a distance of 27 m from one bank is zero.

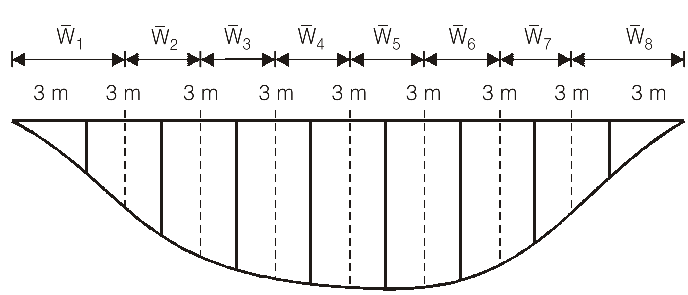
The total discharge is calculated by method of mid sections.

For the first and last section average width, $\bar{W} = \dfrac{\left(W_1 + \dfrac{W_2}{2}\right)^2}{2W_1} = \dfrac{\left(3 + \dfrac{3}{2}\right)^2}{2 \times 3} = 3.375 \text{ m}$

For the rest of segments, $\bar{W} = \left(\dfrac{3}{2} + \dfrac{3}{2}\right) = 3 \text{ m}$

Distance from one bank (m)	Depth(m) y	Average width (m) $\bar{W}$	V	Segmental discharge $\Delta Q = y \times V \times \bar{W}$
3.0	0.4	3.375	0.16	0.216
6.0	0.8	3.0	0.28	0.672
9.0	1.2	3.0	0.485	1.746
12.0	2.0	3.0	0.843	5.058
15.0	3.0	3.0	1.655	14.895
18.0	2.5	3.0	2.63	19.725
21.0	2.2	3.0	2.14	14.124
24.0	1.0	3.375	0.48	1.62
27.0	0	–	–	$\Sigma\Delta Q_i = 58.056 \text{ m}^3/\text{s}$

$\therefore \qquad$ Total discharge, $Q = 58.056 \text{ m}^3/\text{sec}$



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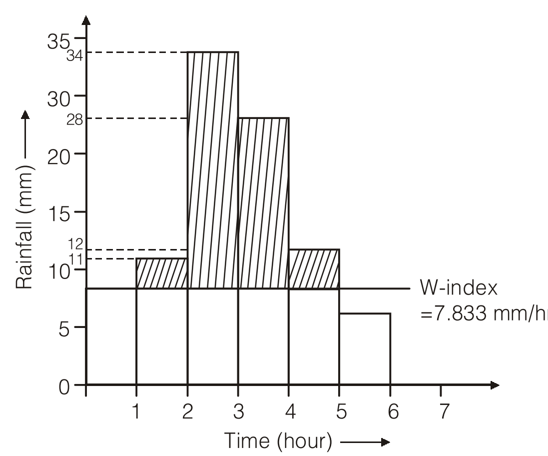
3 Infiltration, Runoff and Hydrographs

Detailed Explanation of Try Yourself Questions

T1 : Solution

Time	Rainfall (mm)
0	0
1	6.0
2	11.0
3	34.0
4	28.0
5	12.0
6	6.0
7	0.0



Total precipitation,

$$P = 97 \text{ mm}$$

Total runoff,

$$Q = \frac{25000}{50 \times 10^4} = 0.05 \text{ m} = 50 \text{ mm}$$

$\therefore$ $$\text{W-index} = \frac{P - Q}{t} = \frac{97 - 50}{6} = 7.833 \text{ mm/hr}$$

$\because$ $$\phi\text{-index} \geq \text{W-index}$$

$$\phi\text{-Index} = \frac{\text{(Total infiltration + infiltration during period when no excess rain occurs)}}{\text{Period of excess rainfall}}$$

$$= \frac{47 - 6 - 6}{4} = 8.75 \text{ mm/hr}$$

T2 : Solution

1 Time	2 UH 1 m³/sec	3 S-curve addition	4 S-curve ordinate	5 Lagged by 3hr	3hr UH Col.4-col.5 (4/3) m³/sec
0	0	–	0	0	0
1	10	–	10	0	13.3
2	60	–	60	0	80
3	120	–	120	0	160
4	170	0	170	10	213.33
5	200	10	210	60	200
6	180	60	240	120	160
7	150	120	270	170	133.33
8	124	170	294	210	112
9	104	210	314	240	98.67
10	88	240	328	270	77.33
11	73	270	343	294	65.33
12	59	294	353	314	52
13	48	314	362	328	45.33
14	36	328	364	343	28
15	28	343	371	353	24
16	20	353	373	362	19.67
17	10	362	375	364	17.33
18	8	364	375	371	5.33
19	3	371	375	373	2.67
20	0	373	375	375	0
21	0	375	–	–	0
22	–	375	–	–	0
23	–	375	–	–	–
24	–	375	–	–	–

In 3 hr

Rainfall = $3 \times 6 = 18$ mm = 1.8 cm

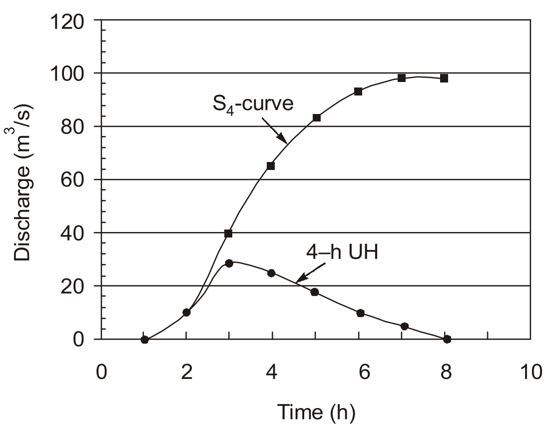
Peak discharge = $213.33 \times 1.8 = 384$ m³/sec

T3 : Solution

Computations are shown in table. In this table col. 2 shows the ordinates of the 4-h unit hydrograph, col. 3 gives the *S*-curve additions and col. 4 gives the ordinates of the *S*-curve. The sequence of entry in col. 3 is shown by arrows. Values of entries in col. 4 is obtained by using eq. i.e., by summing up of entries in col. 2 and col. 4 along each row.

Time in hours	Ordinate of 4-h UH	S-curve addition (m³/s)	S_4-curve ordinate (m³/s). (col. 2 + col. 3)
1	2	3	4
0	0		0
4	10	0	10
8	30	10	40
12	25	40	65
16	18	65	83
20	10	83	93
24	5	93	98
28	0	98	98

- At t = 4 hours, Ordinates of 4h UH = 10 m^3/s
  S-curve addition = ordinate of 4h UH@ {t = (4 – 4) ≠ 0 hours} = 0
  Hence S-curve ordinate = 10 + 0 = 10 m^3/s
- At t = 8 hours; Ordinates of 4h UH = 30 m^3/s
  S-curve addition = ordinate of 4h UH @ {t = (8 – 4) = 4 hours} = 10 m^3/s
  Hence S-curve ordinate = 30 + 10 = 40 m^3/s
- At t = 12 hours; Ordinate of 4h UH = 25 m^3/s
  S-curve addition = ordinate of 4h UH@ {t = (12 – 4) = 8 hours} = 40 m^3/s
  Hence S-curve ordinate = 25 + 40 = 65 m^3/s
  This calculation is repeated for all time intervals till t = base width of UH = 28 hours.
  Plots of the 4h UH and the derived S-curve are shown in figure.



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4 Floods, Flood Routing and Flood Control

Detailed Explanation of Try Yourself Questions

T1 : Solution

Time (hr)	0	12	24	36	48
Inflow (m^3/s)	100	750	780	470	270

$$Q_{initial} = 100\ m^3/s$$
$$k = 18\ \text{hours}$$
$$x = 0.3$$
$$\Delta t = 12\ \text{hrs}$$

Using Muskingham equation

$$C_0 = \frac{-kx + 0.5\ \Delta t}{k(1-x) + 0.5\ \Delta t} = \frac{-18 \times 0.3 + 0.5 \times 12}{18(1-0.3) + 0.5 \times 12} = 0.0322$$

$$C_1 = \frac{kx + 0.5\ \Delta t}{k(1-x) + 0.5\ \Delta t}$$

$$= \frac{18 \times 0.3 + 0.5 \times 12}{18(1-0.3) + 0.5 \times 12} = 0.613$$

$$C_2 = \frac{k - kx - 0.5\ \Delta t}{k(1-x) + 0.5\ \Delta T} = \frac{18(1-0.3) - 0.5 \times 12}{18(1-0.3) + 0.5 \times 12} = 0.355$$

Now, $C_0 + C_1 + C_2 = 0.0322 + 0.613 + 0.355 = 1$

Initial flood discharge, $Q = 100\ m^3/sec$.

The outflow ordinates are worked out in the table using the general equation.

$$Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1}$$
$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$$

Time from start (h) (1)	Inflow (m^3/s) (2)	C_0I_2 $= 0.032I_2$ (3)	C_1I_1 $= 0.613I_1$ (4)	C_2Q_1 $= 0.355Q_1$ (5)	Q in cumecs (6)
0	100	–	–	–	100
12	750	0.032×750 = 24	0.613×100 = 61.3	0.355×100 = 35.5	120.8
24	780	24.96	459.75	42.88	527.59
36	470	15.04	478.14	187.29	680.47
48	270	8.64	288.11	241.566	538.31

The above routing shows that the peak which occurred at t = 24 h at upstream point of river reach, now occurs at t = 36 hr at downstream point i.e., at a lag of 12 hr.
The peak discharge also reduced from 780 m^3/s to 680.47 m^3/sec.

T2 : Solution

We know that $x_T = \bar{x} + Ks$

where $K = \dfrac{y_T - \bar{y}_n}{S_n}$

where $y_T = -\log_e \log_e \left(\dfrac{T}{T-1}\right)$

It may be noted that $\bar{y}_n$ and S_n remains same for one analysis.

Given data: $X_{50} = 20{,}600$ and $X_{100} = 22{,}150$

$X_{500} = ?$

$$y_{50} = -\log_e \log_e \left(\frac{50}{50-1}\right) = 3.90194$$

$$y_{100} = -\log_e \log_e \left(\frac{100}{100-1}\right) = 4.60015$$

$$y_{500} = -\log_e \log_e \left(\frac{500}{500-1}\right) = 6.21361$$

Now, we have

$$\bar{x} + \left(\frac{y_{50}}{S_n} - \frac{\bar{y}_n}{S_n}\right)\sigma = 20{,}600 \quad \ldots (i)$$

$$\bar{x} + \left(\frac{y_{100}}{S_n} - \frac{\bar{y}_n}{S_n}\right)\sigma = 22{,}150 \quad \ldots (ii)$$

Subtracting (i) from (ii), we get

$$\left(\frac{y_{100} - y_{50}}{S_n}\right)\sigma = 1550$$

$$\Rightarrow \quad (4.60015 - 3.90194)\frac{\sigma}{S_n} = 1550$$

$$\Rightarrow \quad \frac{\sigma}{S_n} = 2219.9625$$

Also, we have $$x_{500} = \bar{x} + \left(\frac{y_{500} - \bar{y}_n}{S_n}\right)\sigma \quad \text{... (iii)}$$

Subtracting (ii) from (iii), we get

$$x_{500} - 22150 = (y_{500} - y_{100})\frac{\sigma}{S_n}$$

$$\Rightarrow \quad x_{500} = 22150 + (6.21361 - 4.60015) \times 2219.9625$$

$$\Rightarrow \quad x_{500} = 25731.82 \text{ m}^3/\text{sec}$$

T3 : Solution

Since k = 36 h and 2kx = 2 × 36 × 0.15 = 10.8 h, Δt should be such that $k > \Delta t > 2kx$ i.e. 36 h > Δt > 10.8 h

In the present case Δt = 12 hr is selected to suit the given inflow hydrograph ordinate interval.

We know that

$$O_n = C_o I_n + C_1 I_{n-1} + C_2 O_{n-1}$$

where $$C_o = \frac{-kx + 0.5\,\Delta t}{k - kx + 0.5\,\Delta t} = \frac{-36 \times 0.15 + 0.5 \times 12}{36 - 36 \times 0.15 + 0.5 \times 12} = 0.0164$$

$$C_1 = \frac{kx + 0.5\,\Delta t}{k - kx + 0.5\,\Delta t} = \frac{36 \times 0.15 + 0.5 \times 12}{36 - 36 \times 0.15 + 0.5 \times 12} = 0.3115$$

$$C_2 = \frac{k - kx - 0.5\,\Delta t}{k - kx + 0.5\,\Delta t} = \frac{36 - 36 \times 0.15 - 0.5 \times 12}{36 - 36 \times 0.15 + 0.5 \times 12} = 0.6721$$

For the first time interval, 0 to 12 h

$I_1 = 42$ $\quad C_1 I_1 = 0.3115 \times 42 = 13.08$

$I_2 = 45$ $\quad C_o I_2 = 0.0164 \times 45 = 0.74$

$O_1 = 42$ $\quad C_2 O_1 = 0.6721 \times 42 = 28.23$

$\therefore \quad O_2 = C_o I_2 + C_1 I_1 + C_2 O_1 = 0.74 + 13.08 + 28.23 = 42.05$

Peak of inflow hydrograph = 342 m^3/sec

Peak of outflow hydrograph = 231.13 m^3/sec

$\therefore$ Attenuation in peak flow discharge = 342 – 231.13 = 110.87 m^3/s.

The inflow hydrograph has a peak value at t = 48 h

The outflow hydrograph has a peak value at t = 84 h

$\therefore$ Time lag = 84 – 48 = 36 hrs.

Time(h)	$I(m^3/s)$	$0.0164\ I_2$	$0.3115\ I_1$	$0.6721\ O_1$	$O(m^3/s)$
0	42				42
12	45	0.74	13.08	28.23	42.05
24	88	1.44	14.02	28.26	43.72
36	272	4.46	27.41	29.38	61.25
48	342	5.61	84.73	41.17	131.51
60	288	4.72	106.53	88.39	199.64
72	240	3.94	89.71	134.18	227.83
84	198	3.25	74.76	153.12	231.13
96	162	2.66	61.68	155.34	219.68
108	133	2.18	50.46	147.65	200.29
120	110	1.80	41.43	134.61	177.84
132	90	1.48	34.26	119.53	155.27
144	79	1.30	28.03	104.36	133.69
156	68	1.12	24.61	89.85	115.58
168	61	1.00	21.18	77.68	99.86
180	56	0.92	19.00	67.12	87.04
192	54	0.89	17.44	58.50	76.83
204	51	0.84	16.82	51.64	69.3
216	48	0.79	15.89	46.58	63.26
228	45	0.74	14.95	42.52	58.21
240	42	0.69	14.02	39.12	53.83

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