Chapter 6
Hydrograph

6.1 Hydrograph:

Hydrograph: It is a graph between the rate of flow (m^3 /s) & Time (h or day or month or year). Discharge is measured in a stream / river. Discharge is generated due to rainfall of different depth & different duration in the watershed / catchment / drainage area / basin of the stream

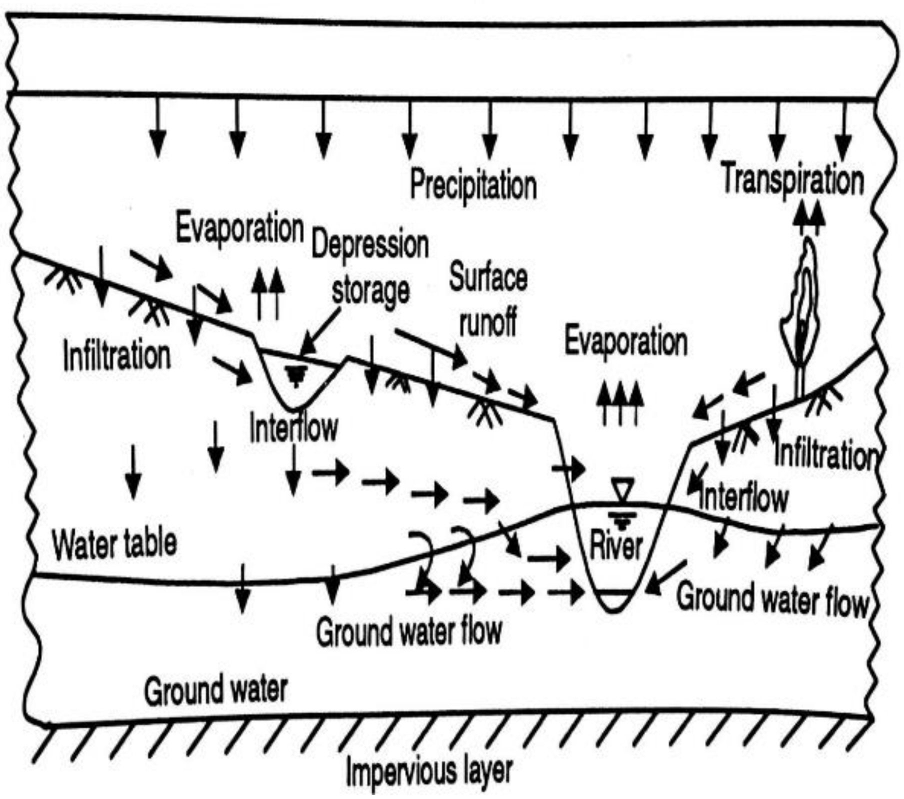


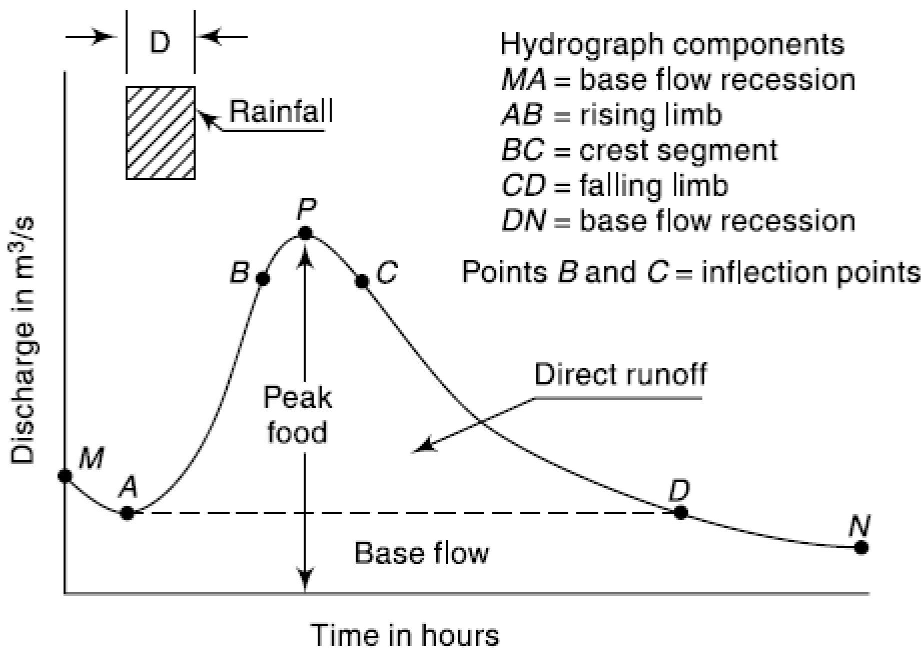
Figure 1 shows how runoff in different forms occurs after precipitation.

A part of precipitation goes to atmosphere by **evaporation** and **transpiration**.

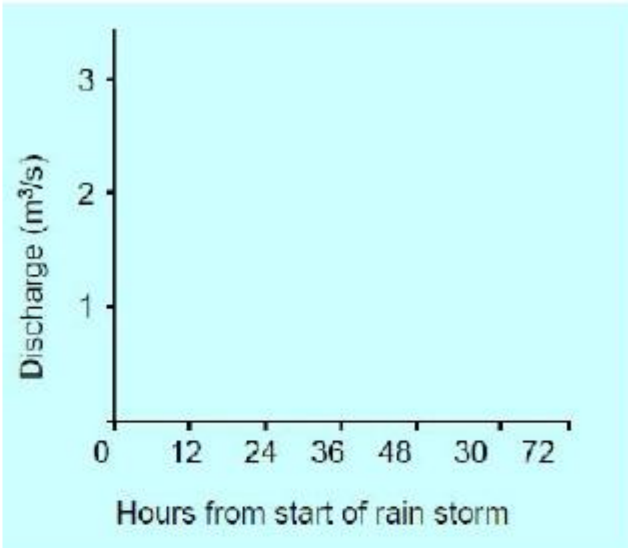
- The remaining part goes to the stream or river of the catchments as:
  1. Surface water flow or overland flow
  2. Interflow or sub surface flow
  3. Groundwater flow
- The runoff is defined as a part of precipitation, which is **not evapotranspirated.**
- Two type of runoff: **surface** and **subsurface**
- **Surface runoff** is a major component of water cycle.

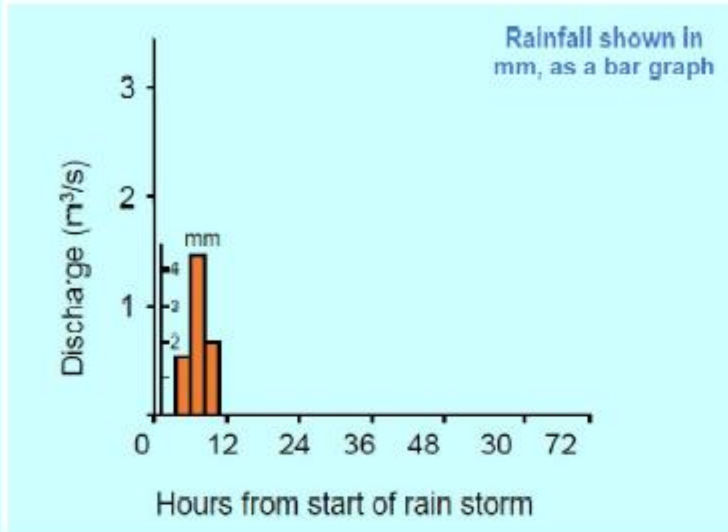
- Theoretically, surface runoff is the net amount of rainfall after subtracted by evapotranspiration and infiltration.
- In reality, surface runoff is equivalent to **river or stream flow** (Q in m^3/s, or ft^3/s) of the catchment.

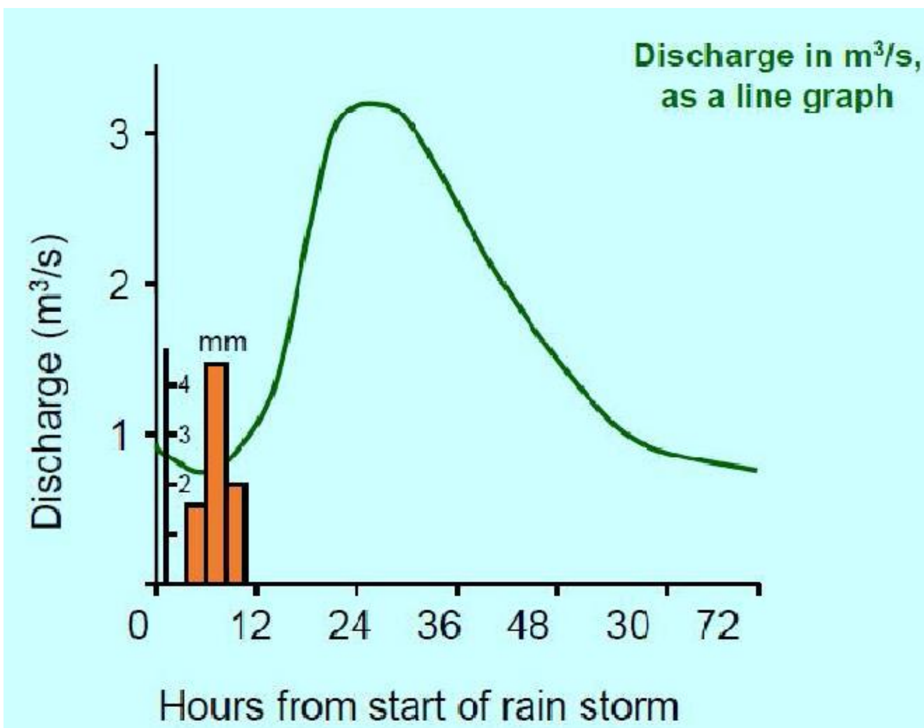
6.2 Natural Hydrograph

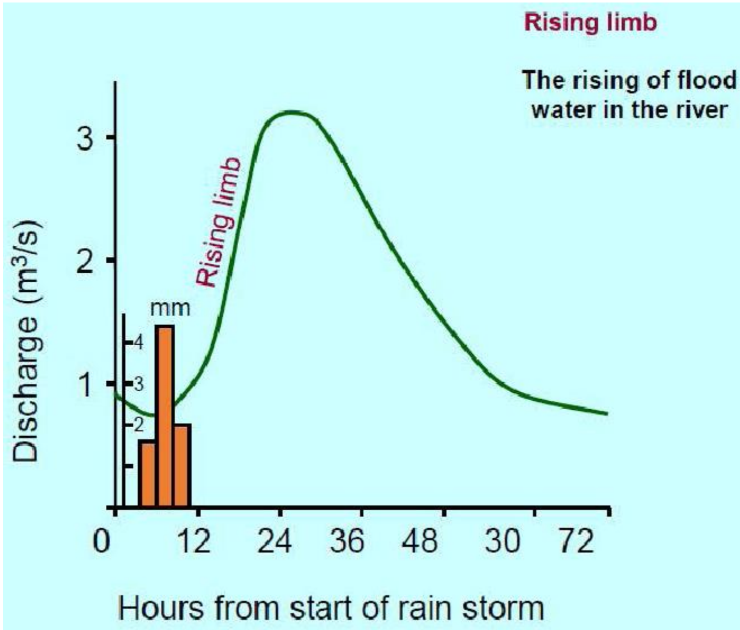


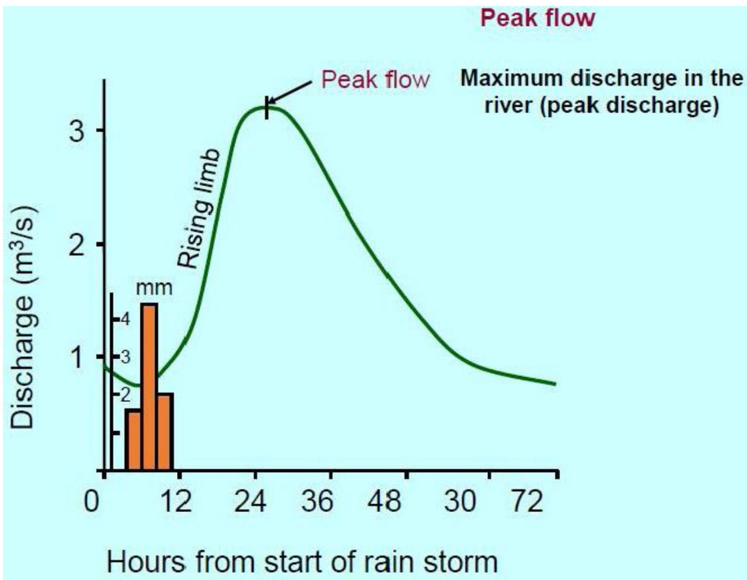
An observed Q -t relationship of a catchment due to rainfall event.
•A rainfall event produces a single hydrograph.
•A natural hydrograph has important characteristics;
•Base flow recession
•Rising limb
•Falling limb
•Peak flow
•Inflection points

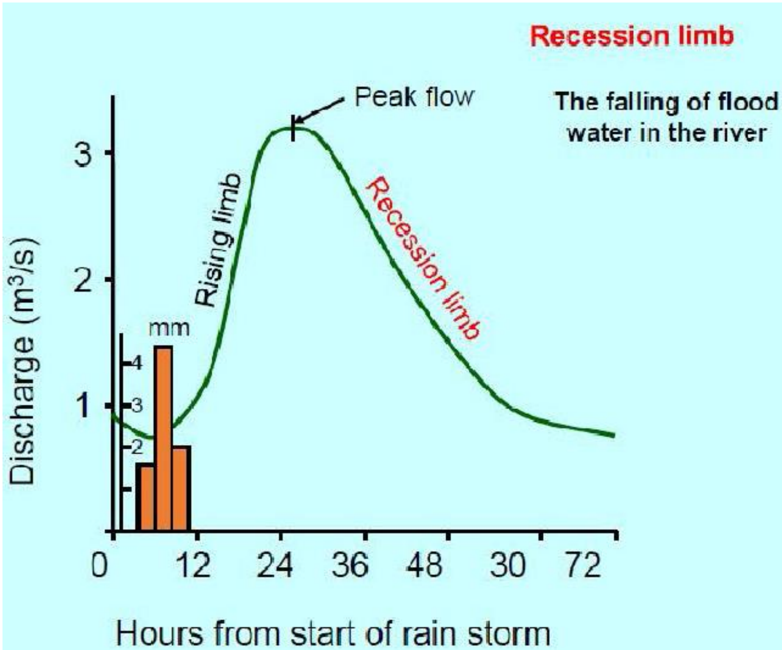

Discharge (m3/s)
3
2
1
0
12
24
36
48
30
72
Hours from start of rain storm

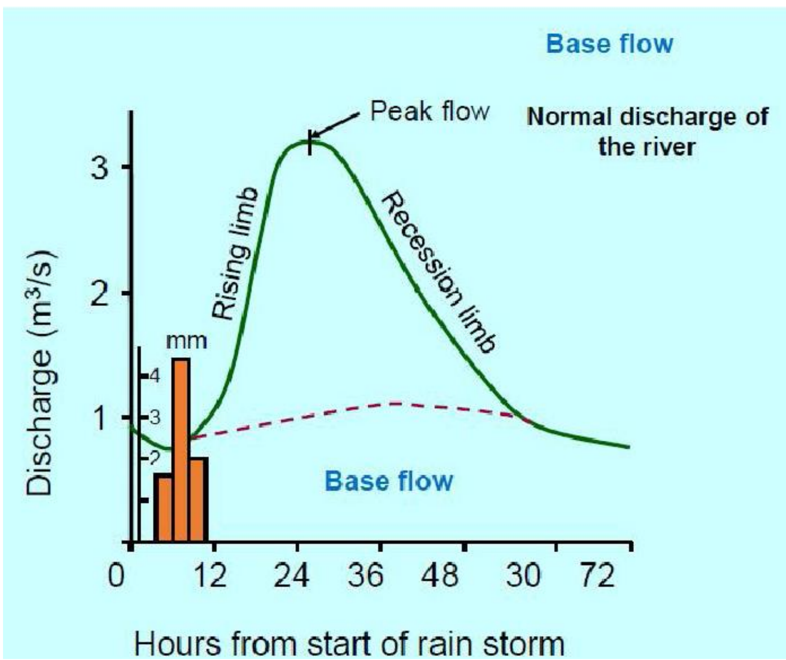

Rainfall shown in
mm, as a bar graph
Discharge (m3/s)
3
2
1
mm
0
12
24
36
48
30
72
Hours from start of rain storm

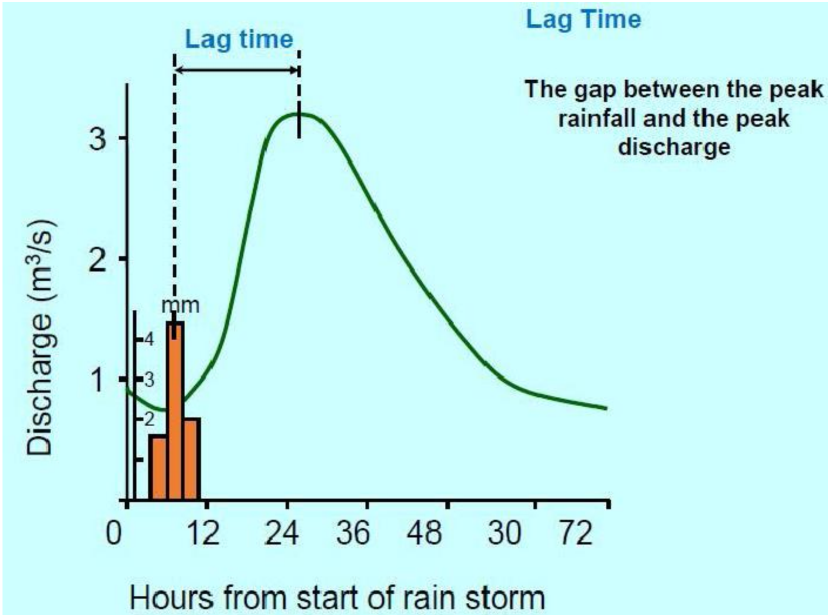

Discharge in m3/s,
as a line graph
Discharge (m3/s)
3
2
1
mm
4
3
2
0
12
24
36
48
30
72
Hours from start of rain storm


Rising limb
The rising of flood water in the river
Rising limb
Discharge (m3/s)
mm
Hours from start of rain storm
0 12 24 36 48 30 72


Peak flow
Maximum discharge in the river (peak discharge)
Peak flow
Rising limb
Discharge (m3/s)
mm
Hours from start of rain storm
0 12 24 36 48 30 72


Recession limb
Peak flow
The falling of flood water in the river
Rising limb
Recession limb
mm
Discharge (m³/s)
3
2
1
4
3
2
0
12
24
36
48
30
72
Hours from start of rain storm


Base flow
Peak flow
Normal discharge of the river
Rising limb
Recession limb
mm
Discharge (m³/s)
Base flow
3
2
1
4
3
2
0
12
24
36
48
30
72
Hours from start of rain storm



The storage of water in the basin exists as (i) surface storage, which includes both surface detention and channel storage, (ii) interflow storage, and (iii) groundwater storage, i.e. base-flow storage. Barnes (1940) showed that the recession of a storage can be expressed as

$$Q_t = Q_0 K_r^t \qquad (1)$$

in which Q_t is the discharge at a time t and Q_0 is the discharge at $t = 0$; K_r is a recession constant of value less than unity. Equation (1) can also be expressed in an alternative form of the exponential decay as

$$Q_t = Q_0 e^{-at} \qquad (1a)$$

where $a = -\ln K_r$.

The recession constant K_r can be considered to be made up of three components to account for the three types of storages as

$$K_r = K_{rs} \cdot K_{ri} \cdot K_{rb}$$

where K_{rs} = recession constant for surface storage, K_{ri} = recession constant for interflow and K_{rb} = recession constant for base flow. Typically the values of these recession constants, when time t is in days, are

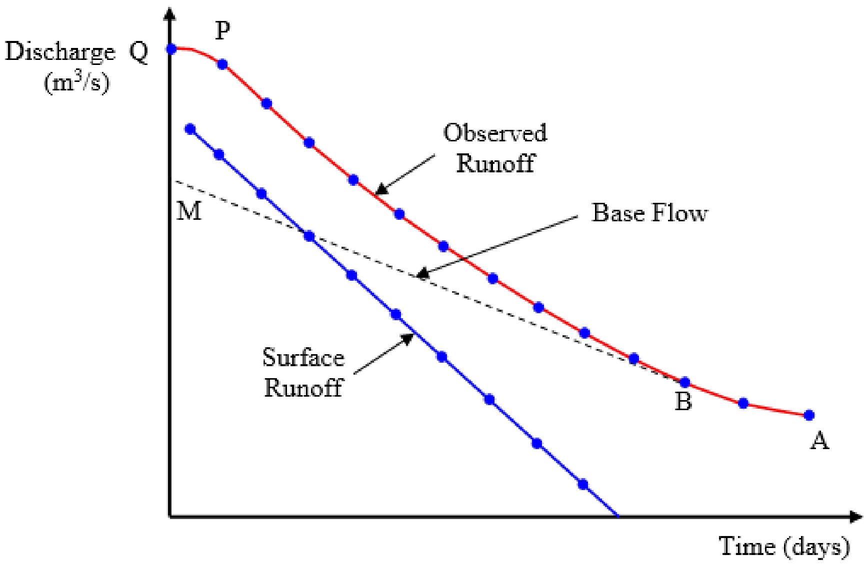
$$K_{rs} = 0.05 \text{ to } 0.20 \qquad K_{ri} = 0.50 \text{ to } 0.85 \qquad K_{rb} = 0.85 \text{ to } 0.99$$

When the interflow is not significant, K_{ri} can be assumed to be unity.
If suffixes 1 and 2 denote the conditions at two time instances t_1 and t_2,

EXAMPLE .1 *The recession limb of a flood hydrograph is given below. The time is indicated from the arrival of peak. Assuming the interflow component to be negligible, estimate the base flow and surface flow recession coefficients. Also, estimate the storage at the end of day-3.*

Time from peak (day)	Discharge (m^3/s)	Time from Peak (day)	Discharge (m^3/s)
0.0	90	3.5	5.0
0.5	66	4.0	3.8
1.0	34	4.5	3.0
1.5	20	5.0	2.6
2.0	13	5.5	2.2
2.5	9.0	6.0	1.8
3.0	6.7	6.5	1.6
		7.0	1.5

$Q_t / Q_o = K_{rb}^{\,t}$ $\qquad\qquad$ $\log K_{rb} = \frac{1}{t} \log (Q_t / Q_o)$



$Q_o = 6.6\ m^3/s.$, $t = 2$ days , $Q_t = 4\ m^3/s.$

$\log K_{rb} = \frac{1}{2} \log (4 / 6.6) \Longrightarrow K_{rb} = 0.78$

$Q_o = 26\ m^3/s.$, $t = 2$ days , $Q_t = 2.25\ m^3/s.$

$\log K_{rs} = \frac{1}{2} \log (2.25 / 26) \Longrightarrow K_{rs} = 0.29$

$K_r = 0.29 * 0.78 * 1 = 0.226$

6.3 Factors Affecting Runoff Hydrograph

1. Basin Characteristics:

a) Shape
b) Size
c) Slope
d) Nature of the valley
e) Elevation
f) Drainage density

2. Infiltration Characteristics:

a) Land use and cover
b) Soil type and geological conditions
c) Lakes, swamps and other storage

3. Channel Characteristics:

a) Cross-section
b) Roughness
c) Storage capacity

4. Storm Characteristics:

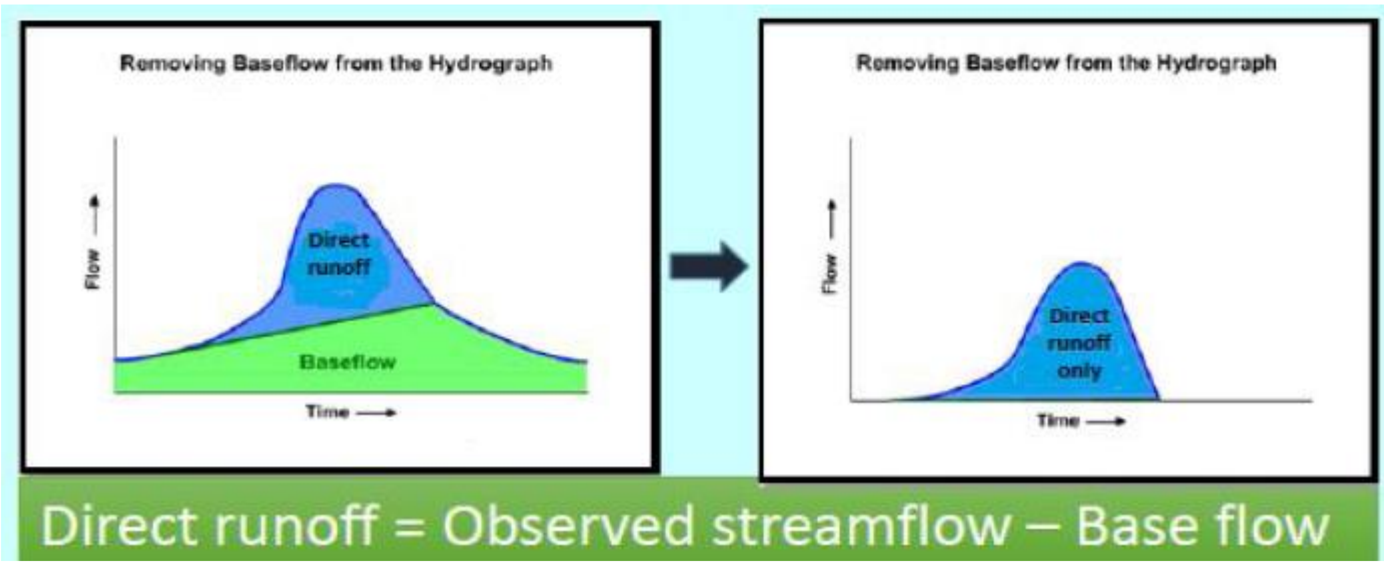
a) Precipitation
b) Intensity
c) Duration
d) Magnitude
e) Movement of storm

5. Initial loss

6. Evaporation and **Transpiration**

6.4 Base Flow Separation

✓Natural hydrograph consists of two main components; **runoff component and base flow component.**
✓The **direct runoff** is obtained by separating the base flow from the natural hydrograph.

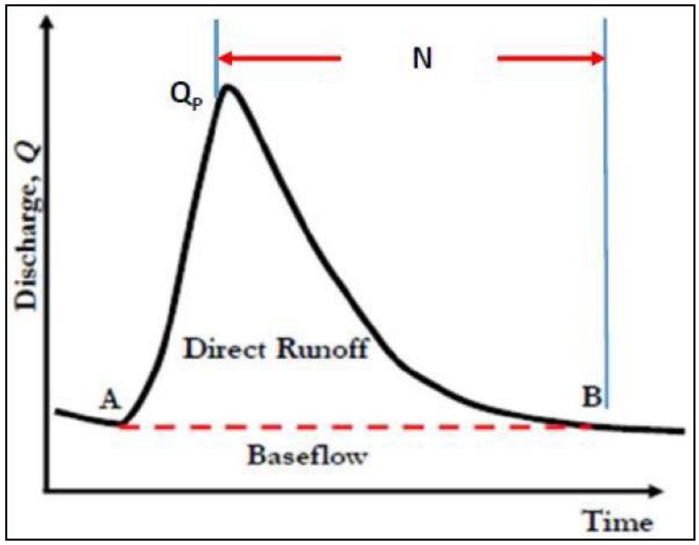


Notice
✓The base flow is the initial flow of the river before the rain comes.
✓It is produced from previous season (rainfall) and also considered to be mostly from the ground water contribution.

Three Techniques for base flow separation:
A. Straight-line method
B. Fixed base length method
C. Variable slope method

Method 1: A. Straight-line method

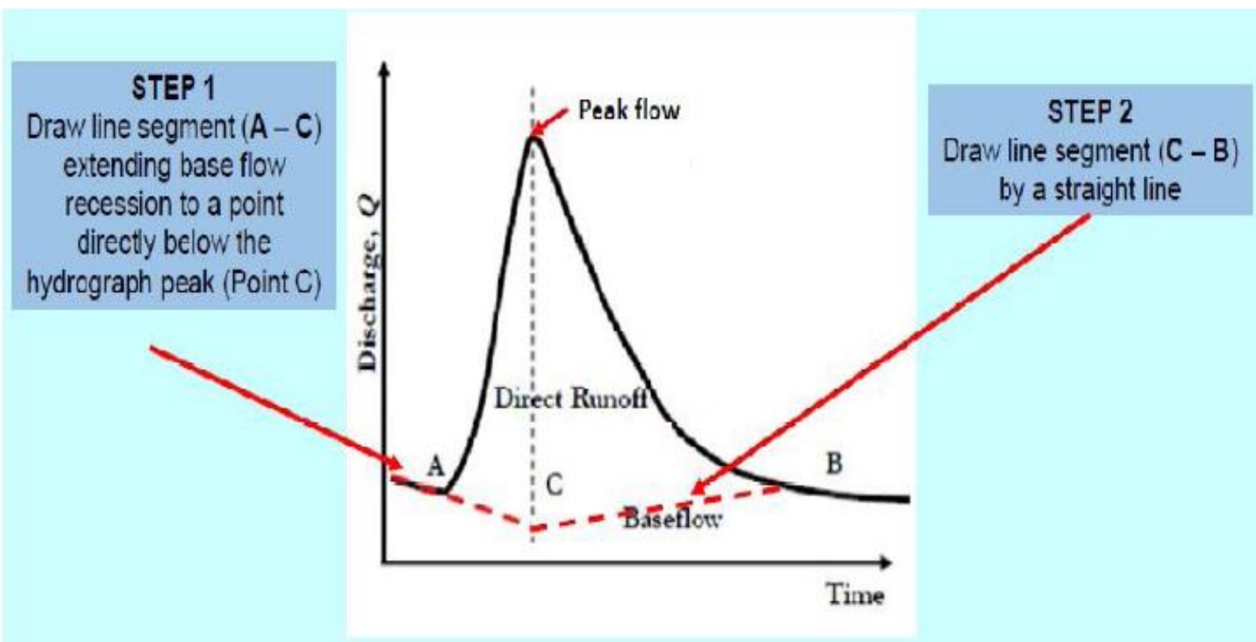
- Point **A** represents the beginning of the direct runoff.
- Point **B** represents the end of the direct runoff.



$$\mathbf{N = 0.83\ A^{0.2}}$$

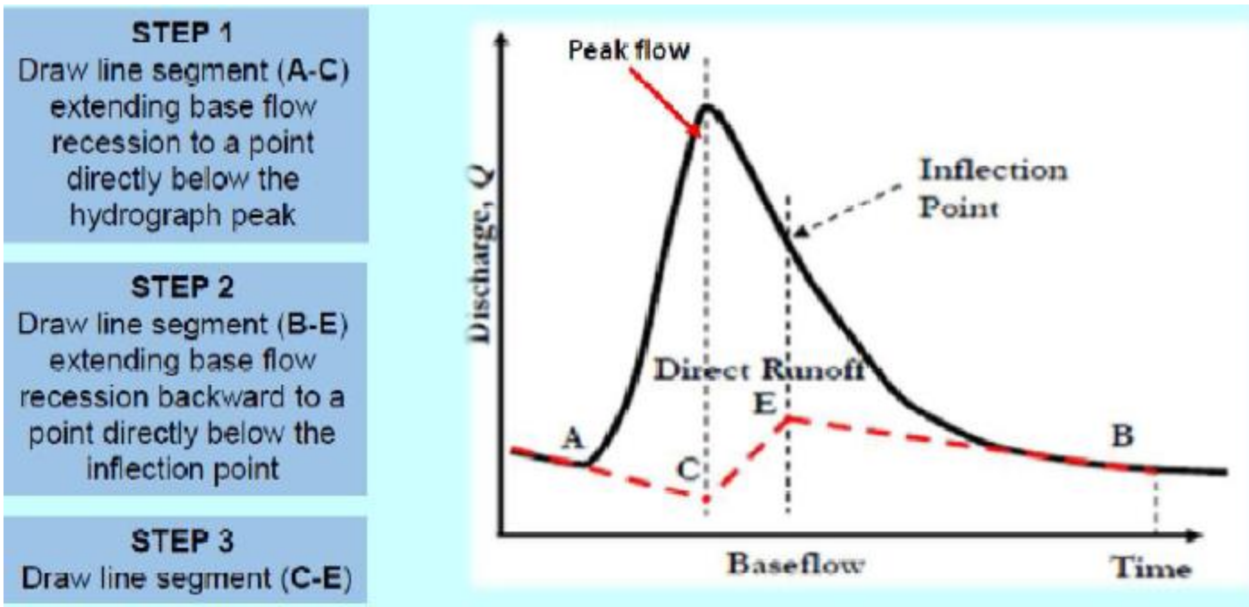
Where;
A = catchment area (km^2)
N = time intervals (days) from the peak to the point B.

Method 2: Fixed base length method



Segment **A-C** and **C-B** separate the base flow and direct runoff.

Method 3: Variable slope method



- The surface runoff hydrograph obtained after the base-flow separation is also known as *Direct Runoff Hydrograph (DRH).*

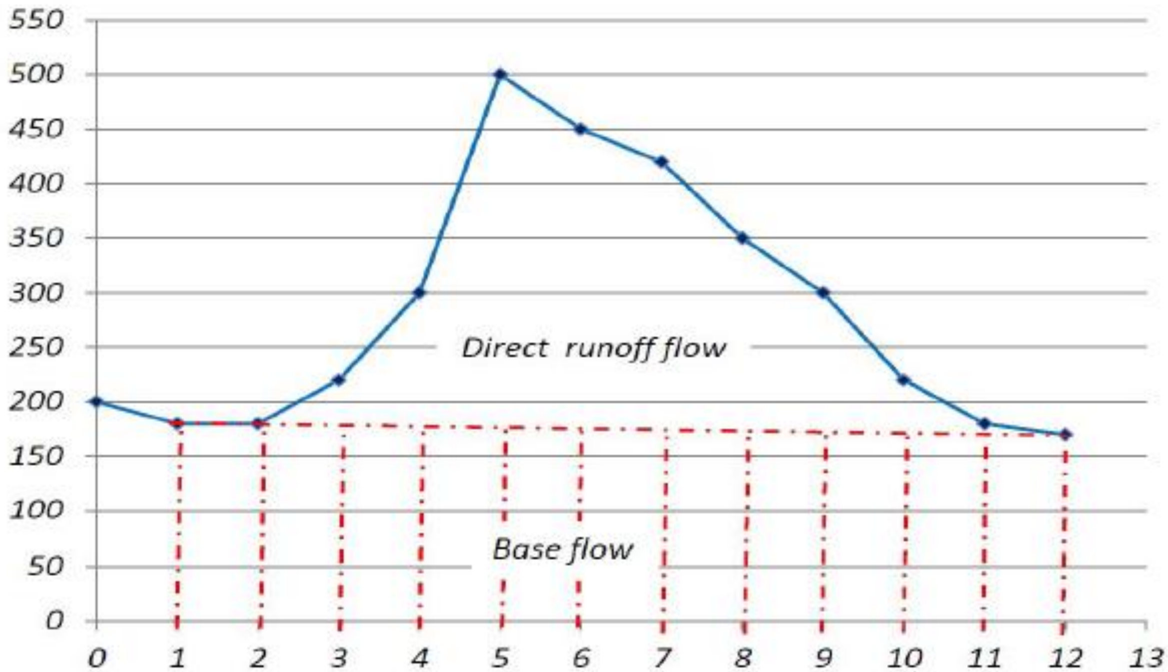
Example

Given below are ordinates of hydrograph. Separate the base flow from direct runoff flow by using the:-
1- Straight line method.
2- Fixed base length method.
3- Variable slopes method. (using basin area 8000 km^2)

Time (day)	0	1	2	3	4	5	6	7	8	9	10	11	12
Flow(m^3/sec)	200	180	180	220	300	500	450	420	350	300	220	180	160

Solution

1- Straight line method.

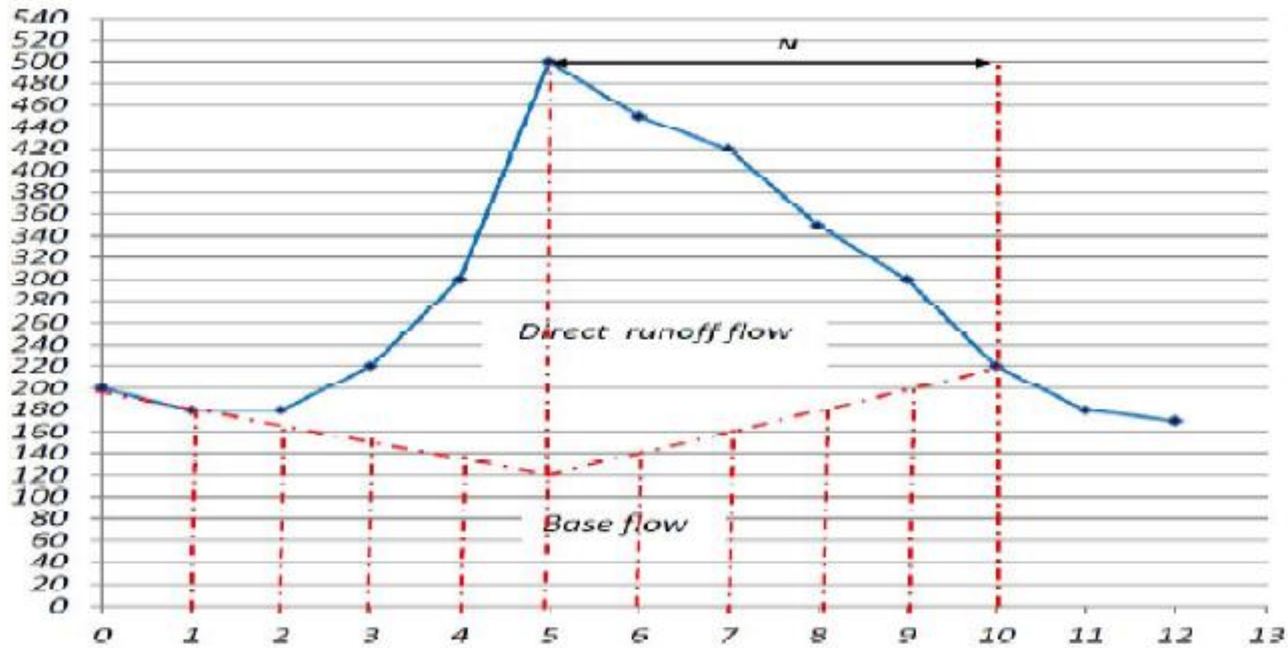


Time (day)	0	1	2	3	4	5	6	7	8	9	10	11	12
Flow (m^3/sec)	200	180	180	220	300	500	450	420	350	300	220	180	160
Base flow (m^3/sec)	-	180	180	178	176	174	172	170	168	166	164	162	160
Direct flow (m^3/sec)	-	0	0	42	124	326	278	250	182	134	56	18	0

2- Fixed base length method

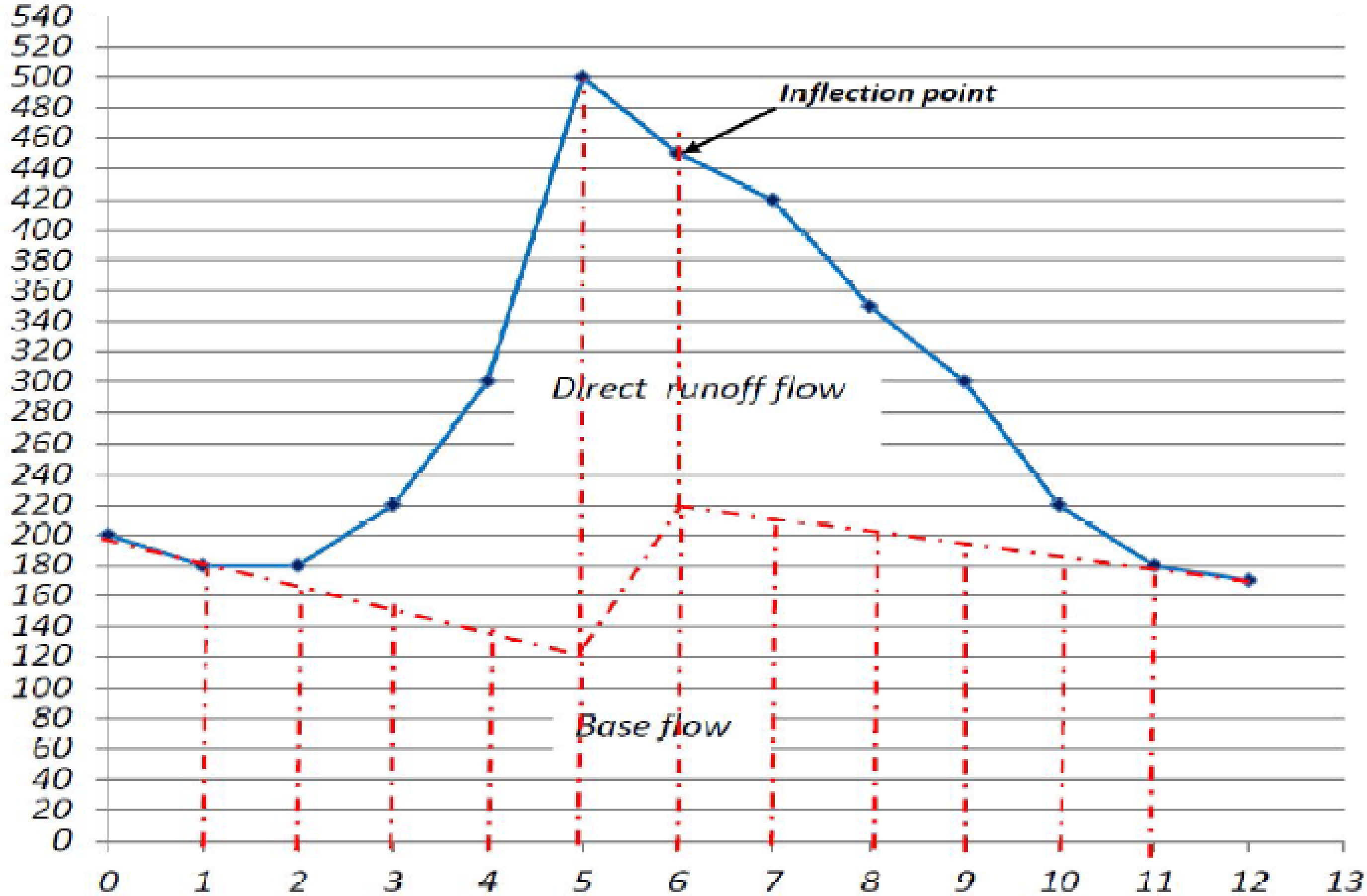
$$N = 0.83\ A^{0.2}$$

$$= 0.83\ (8000)^{0.2} = 5 \text{ day}$$



Time (day)	0	1	2	3	4	5	6	7	8	9	10	11	12
Flow (m^3/sec)	200	180	180	220	300	500	450	420	350	300	220	180	160
Base flow (m^3/sec)	–	180	165	150	135	120	140	160	180	200	220	–	–
Direct flow (m^3/sec)	–	0	15	70	165	380	310	260	170	100	0	–	–

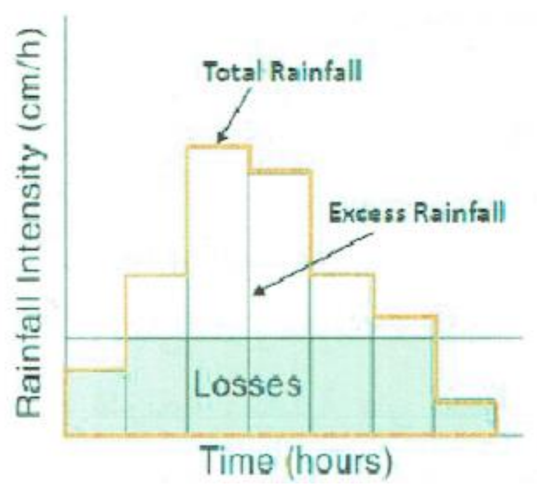
3- Variable slopes method.



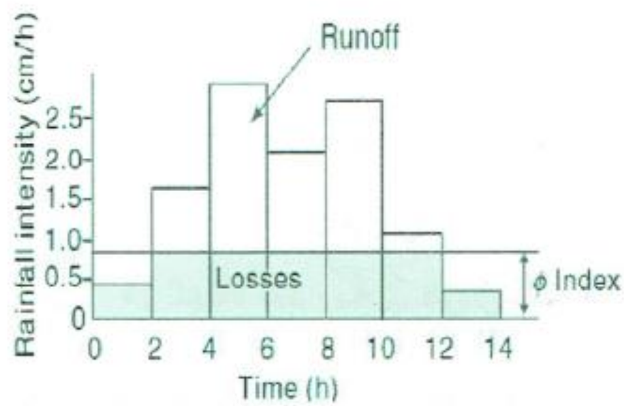
Time (day)	0	1	2	3	4	5	6	7	8	9	10	11	12
Flow (m^3/sec)	200	180	180	220	300	500	450	420	350	300	220	180	160
Base flow (m^3/sec)	-	180	165	150	135	120	220	210	200	195	185	180	-
Direct flow (m^3/sec)	-	0	15	70	165	380	230	210	150	105	35	0	-

6.5 Effective Rainfall

- ✓ The ***Effective Rainfall Hyetograph*** **(ERH)** is obtained by subtracting the initial loss and infiltration losses from the hyetograph of a storm.
- ✓ It is also known as ***Hyetograph of Excess Rainfall***
- ✓ Effective rainfall definition;
  - •Not retained on land surface
  - •Not infiltrated into soil
- ✓ Both DRO and ERH represent the same total quantity but in different units.
- ✓ ERH is usually in cm/hr plotted against time.



- **Effective Rainfall = Excess Rainfall = Total Rainfall – (Initial Losses + Infiltration)**
- **Total losses =**(Initial Losses + Infiltration)
- **Total losses** =Water absorbed by infiltration an evaporation.
- Parameters of infiltration equations can be determined by ϕ -index.



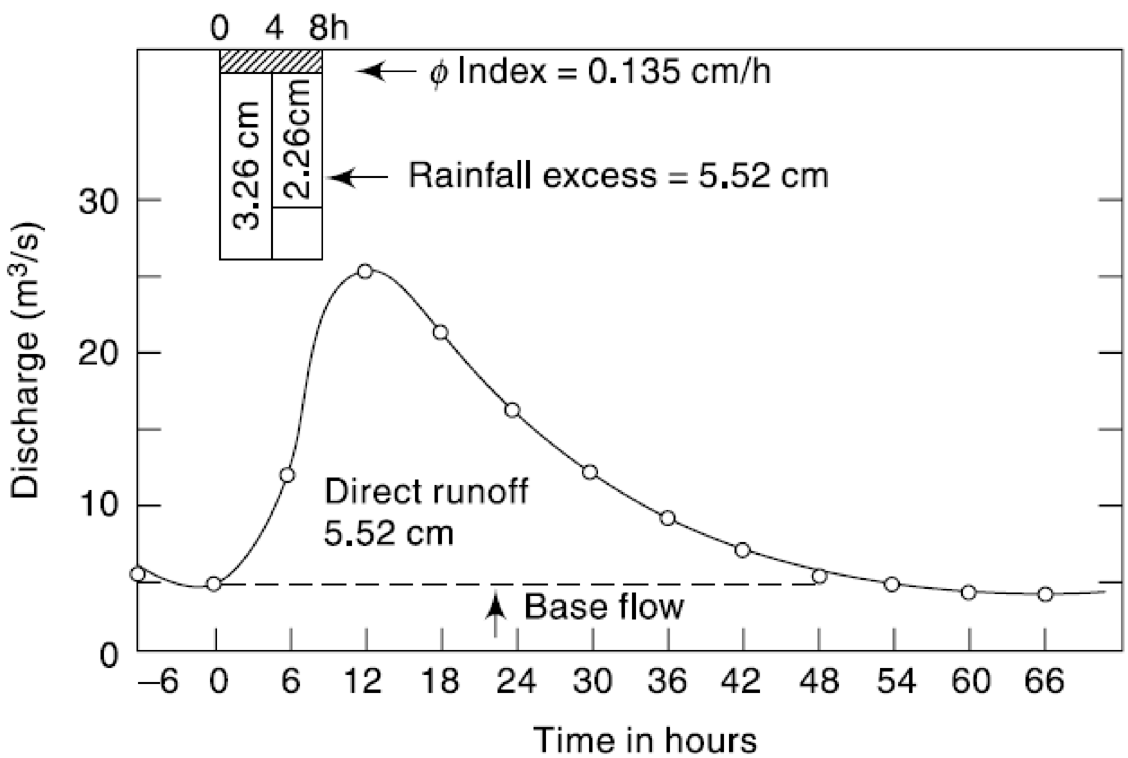
Φ- INDEX

- Constant rate of abstraction yielding effective rainfall hyetograph with depth equal to depth of direct runoff.
- **Total rainfall (P)−ϕ.t = Depth of direct runoff = Effective Rainfall**

EXAMPLE *Rainfall of magnitude 3.8 cm and 2.8 cm occurring on two consecutive 4-h durations on a catchment of area 27 km² produced the following hydrograph of flow at the outlet of the catchment. Estimate the rainfall excess and ϕ index.*

Time from start of rainfall (h)	–6	0	6	12	18	24	30	36	42	48	54	60	66
Observed flow (m^3/s)	6	5	13	26	21	16	12	9	7	5	5	4.5	4.5

Solution:



N = 0.83 (27) $^{0.2}$ = 1.6 day = 38.5 hr.
Time of N = 48 – 12 = 36 hr.

V. DRH = 6*60*60[8+21+16+11+7+4+2] = $1.4904*10^6$ m^3

Depth of Runoff = Runoff vol./ Area = $1.4904*10^6$ / $27*10^6$ = 5.52 cm. (excess rainfall)

Total Rainfall = 2.8 + 3.8 = 6.6 cm.

Time of Duration = 8 hr.

Φ index = (6.6 – 5.52) / 8 = 0.135 cm/hr.

6.6. Unit Hydrograph

- Is the (DRH) hydrograph resulting from 1 in or 1 cm of excess rainfall generated uniformly over the drainage area at a constant rate for an effective duration.

1 cm or 1in, D= Duration, UH= unit hydrograph.

ordinate of DRH = ordinate of UH * ER

6.7. Unit Hydrograph Assumptions

- The excess rainfall has a constant intensity within the effective duration
- The excess rainfall is uniformly distributed throughout the whole drainage area
- The base time of the DRH (the duration of direct runoff) resulting from an excess rainfall of given duration is constant
- The ordinates of all DRHs of a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph
- For a given watershed, the hydrograph resulting from a given excess rainfall reflects the unchanging characteristics of the watershed (Constant: hydrograph shape,time to peak, recession time)

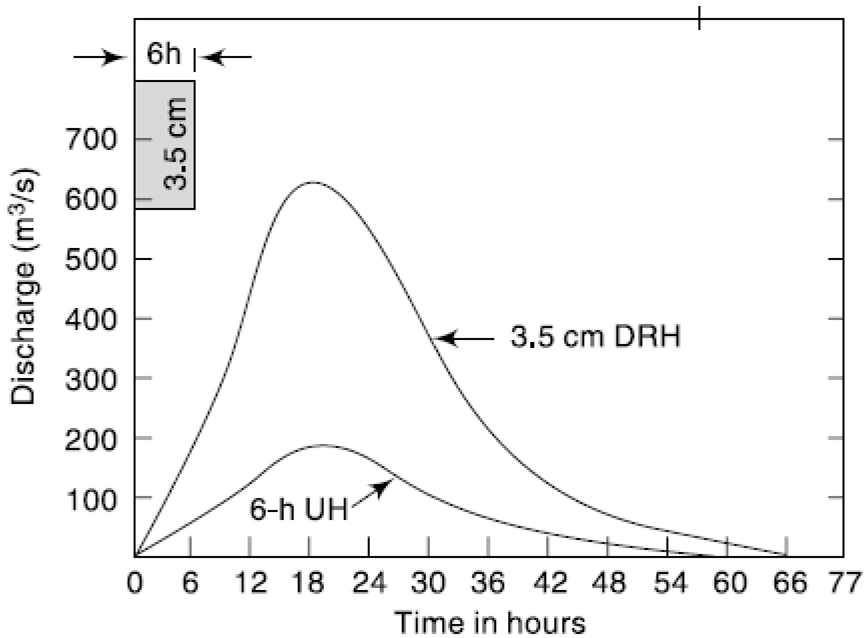
EXAMPLE *Given below are the ordinates of a 6-h unit hydrograph for a catchment. Calculate the ordinates of the DRH due to a rainfall excess of 3.5 cm occurring in 6 h.*

Time (h)	0	3	6	9	12	15	18	24	30	36	42	48	54	60	69
UH ordinate (m^3/s)	0	25	50	85	125	160	185	160	110	60	36	25	16	8	0

SOLUTION: The desired ordinates of the DRH are obtained by multiplying the ordinates of the unit hydrograph by a factor of 3.5 as in Table . The resulting DRH as also the unit hydrograph are shown in Fig. . Note that the time base of DRH is not changed and remains the same as that of the unit hydrograph. The intervals of coordinates of the unit hydrograph (shown in column 1) are not in any way related to the duration of the rainfall excess and can be any convenient value.

Calculation of DRH due to 3.5 ER

Time (h)	Ordinate of 6-h unit hydrograph (m^3/s)	Ordinate of 3.5 cm DRH (m^3/s)
1	**2**	**3**
0	0	0
3	25	87.5
6	50	175.0
9	85	297.5
12	125	437.5
15	160	560.0
18	185	647.5
24	160	560.0
30	110	385.0
36	60	210.0
42	36	126.0
48	25	87.5
54	16	56.0
60	8	28.0
69	0	0



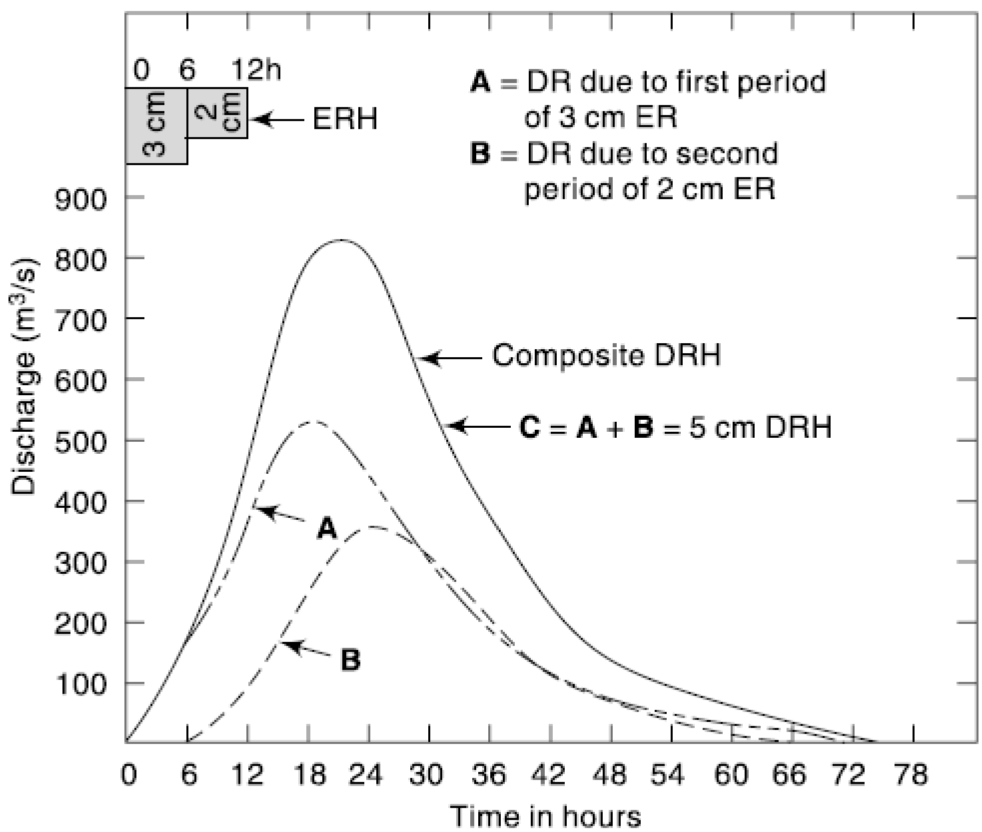
(3.5 cm DRH derived from 6-h unit hydrograph)

EXAMPLE *Two storms each of 6-h duration and having rainfall excess values of 3.0 and 2.0 cm respectively occur successively. The 2-cm ER rain follows the 3-cm rain. The 6-h unit hydrograph for the catchment is the same as given in Example ↑. Calculate the resulting DRH.*

Calculation of DRH by method of Superposition

Time (h)	Ordinate of 6-h UH (m^3/s)	Ordinate of 3-cm DRH (col. 2) × 3	Ordinate of 2-cm DRH (col. 2 lagged by 6 h) × 2	Ordinate of 5-cm DRH (col. 3 + col. 4) (m^3/s)
1	2	3	4	5
0	0	0	0	0
3	25	75	0	75
6	50	150	0	150
9	85	255	50	305
12	125	375	100	475
15	160	480	170	650
18	185	555	250	805

(21)	(172.5)	(517.5)	(320)	(837.5)
24	160	480	370	850
30	110	330	320	650
36	60	180	220	400
42	36	108	120	228
48	25	75	72	147
54	16	48	50	98
60	8	24	32	56
(66)	(2.7)	(8.1)	(16)	(24.1)
69	0	0	(10.6)	(10.6)
75	0	0	0	0



6.8. Derivation of Unit Hydrographs

EXAMPLE *Following are the ordinates of a storm hydrograph of a river draining a catchment area of 423 km² due to a 6-h isolated storm. Derive the ordinates of a 6-h unit hydrograph for the catchment*

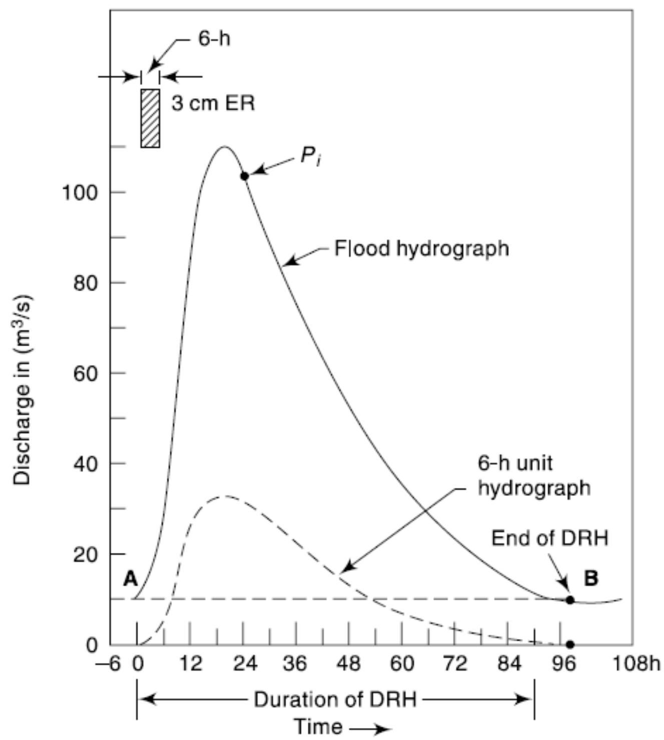
Time from start of storm (h)	–6	0	6	12	18	24	30	36	42	48
Discharge (m^3/s)	10	10	30	87.5	115.5	102.5	85.0	71.0	59.0	47.5
Time from start of storm (h)	54	60	66	72	78	84	90	96	102	
Discharge (m^3/s)	39.0	31.5	26.0	21.5	17.5	15.0	12.5	12.0	12.0	

A = beginning of DRH $t = 0$
B = end of DRH $t = 90$ h
P_m = peak $t = 20$ h

Hence

$N = (90 - 20) = 70$ h $= 2.91$ days

$N = 0.83\,(423)^{0.2} = 2.78$ days



Derivation of Unit Hydrograph from a flood Hydrograph

Time from beginning of storm (h)	Ordinate of flood hydro-graph (m^3/s)	Base Flow (m^3/s)	Ordinate of DRH (m^3/s)	Ordinate of 6-h unit hydro-graph (Col. 4)/3
1	2	3	4	5
–6	10.0	10.0	0	0
0	10.0	10.0	0	0
6	30.0	10.0	20.0	6.7
12	87.5	10.5	77.0	25.7
18	111.5	10.5	101.0	33.7
24	102.5	10.5	101.0	33.7
30	85.0	11.0	74.0	24.7
36	71.0	11.0	60.0	20.0
42	59.0	11.0	48.0	16.0
48	47.5	11.5	36.0	12.0
54	39.0	11.5	27.5	9.2
60	31.5	11.5	20.0	
66	26.0	12.0	14.0	
72	21.5	12.0	9.5	
78	17.5	12.0	5.5	
84	15.0	12.5	2.5	
90	12.5	12.5	0	
96	12.0	12.0	0	
102	12.0	12.0	0	

$$\text{Volume of DRH} = 60 \times 60 \times 6 \times (\text{sum of DRH ordinates})$$
$$= 60 \times 60 \times 6 \times 587 = 12.68 \text{ Mm}^3$$
$$\text{Drainage area} = 423 \text{ km}^2 = 423 \text{ Mm}^2$$
$$\text{Runoff depth} = \text{ER depth} = \frac{12.68}{423} = 0.03 \text{ m} = 3 \text{ cm.}$$

The ordinates of DRH (col. 4) are divided by 3 to obtain the ordinates of the 6-h unit hydrograph

EXAMPLE *(a) The peak of flood hydrograph due to a 3-h duration isolated storm in a catchment is 270 m³/s. The total depth of rainfall is 5.9 cm. Assuming an average infiltration loss of 0.3 cm/h and a constant base flow of 20 m³/s, estimate the peak of the 3-h unit hydrograph (UH) of this catchment.*

SOLUTION:

Duration of rainfall excess = 3 h — Loss @ 0.3 cm/h for 3 h = 0.9 cm

Total depth of rainfall = 5.9 cm — Rainfall excess = 5.9–0.9 = 5.0 cm

Peak flow:

Peak of flood hydrograph = 270 m^3/s — Peak of DRH = 250 m^3/s

Base flow = 20 m^3/s

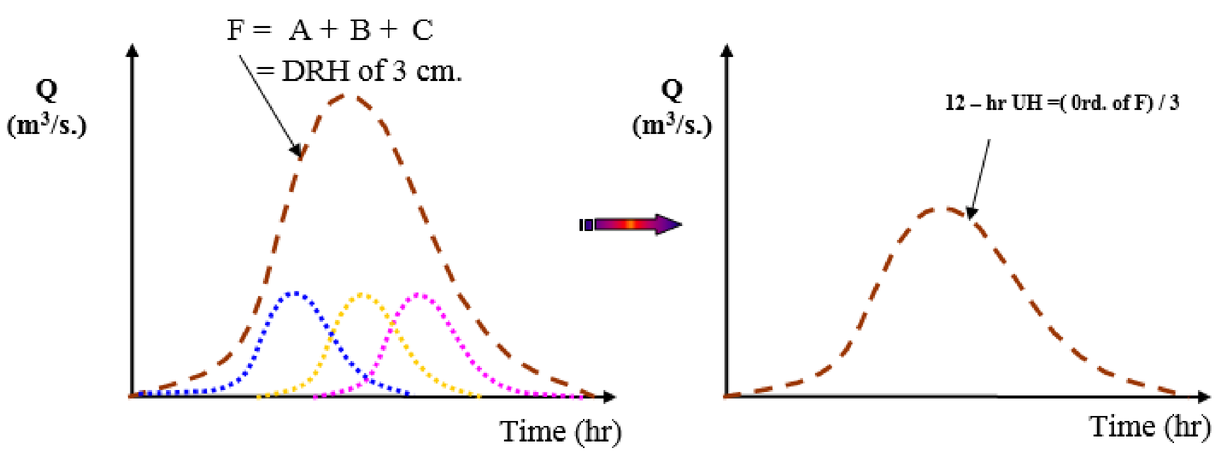
$$\text{Peak of 3-h unit hydrograph} = \frac{\text{peak of DRH}}{\text{rainfall excess}} = \frac{250}{5.0} = 50\ m^3/s$$

6.9. Unit Hydrograph for Different Duration

Two methods are available for developing unit hydrograph of different durations

- Method of superposition
- The *S*-curve

METHOD OF SUPERPOSITION



EXAMPLE *Given the ordinates of a 4-h unit hydrograph as below derive the ordinates of a 12-h unit hydrograph for the same catchment.*

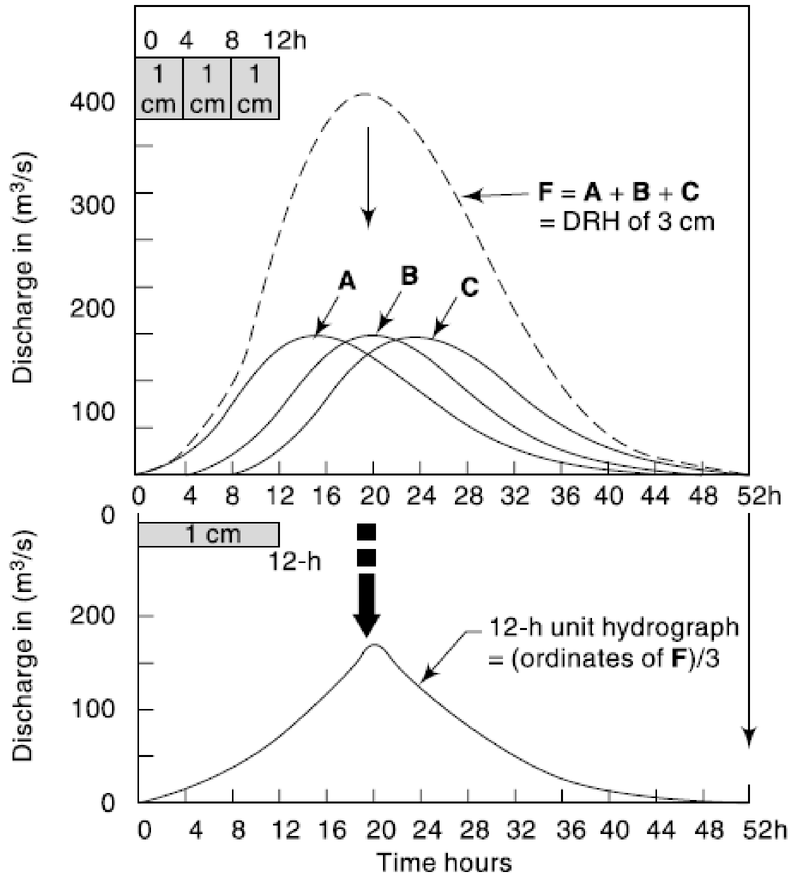
Time (h)	0	4	8	12	16	20	24	28	32	36	40	44
Ordinate of 4-h UH	0	20	80	130	150	130	90	52	27	15	5	0

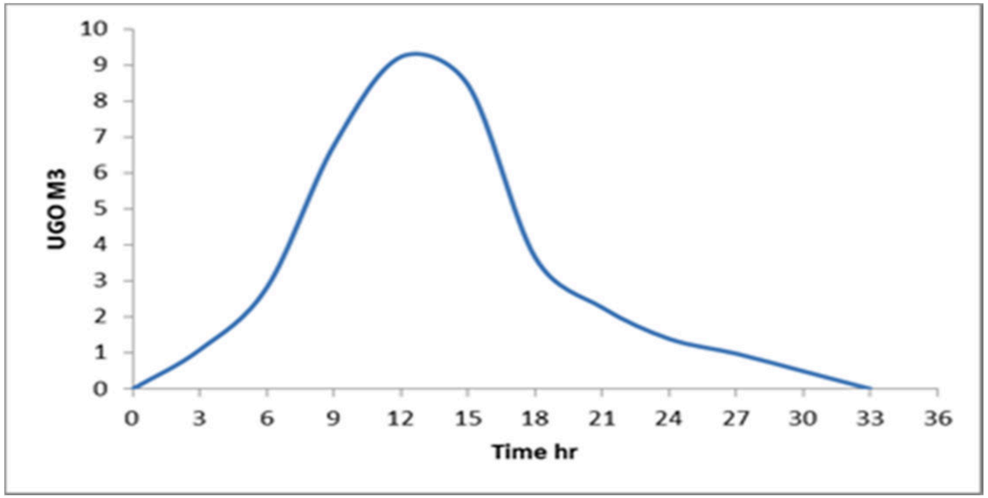
Solution:

Column 3 = ordinates of 4-h UH lagged by 4-h
Column 4 = ordinates of 4-h UH lagged by 8-h
Column 5 = ordinates of DRH representing 3 cm ER in 12-h
Column 6 = ordinates of 12-h UH = (Column 5)/3

Calculation of a 12-h Unit Hydrograph from a 4-H Unit Hydrograph

Time (h)	Ordinates of 4-h UH (m^3/s)			DRH of 3 cm in 12-h (m^3/s) (Col. 2+3+4)	Ordinate of 12-h UH (m^3/s) (Col. 5)/3
	A	*B* Lagged by 4-h	*C* Lagged by 8-h		
1	2	3	4	5	6
0	0	—	—	0	0
4	20	0	—	20	6.7
8	80	20	0	100	33.3
12	130	80	20	230	76.7
16	150	130	80	360	120.0
20	130	150	130	410	136.7
24	90	130	150	370	123.3
28	52	90	130	272	90.7
32	27	52	90	169	56.3
36	15	27	52	94	31.3
40	5	15	27	47	15.7
44	0	5	15	20	6.7
48		0	5	5	1.7
52			0	0	0


0 4 8 12h
1 cm
1 cm
1 cm
Discharge in (m3/s)
400
300
200
100
F = A + B + C
= DRH of 3 cm
A
B
C
0 4 8 12 16 20 24 28 32 36 40 44 48 52h
0
1 cm
12-h
200
100
0
12-h unit hydrograph
= (ordinates of F)/3
Time hours



- <u>Unit Hydrograph for Different Duration:</u>
- There are several ways derivation standard hydrograph which sustainability nD - an hour from a standard water scheme sustainability D - an hour, and most important of these ways:
- Super Position Method .
- S - Curve Method .

- Super Position Method .
- If the availability of a standard hydrograph who sustainability D - an hour and was required is a derivative standard water scheme nD for - hour, where n is an integer,
- Example (8) / information given is the Y-coordinates record hydrograph sustainability 4 - hour, derived the y-coordinate is 12 - hour water hydrograph record.

Time hr	0	4	8	12	16	20	24	28	32	36	40	44	48	52
Y-coordinates record hydrograph	0	20	80	130	150	130	90	52	27	15	5	0	-	-

- Solution : make table

12 –UH إحداثيات ساعة (العمود 5 ÷ 3)	الأعمدة 4+3+2	C يزحف بـ 8 ساعة	B يزحف بـ 4 ساعة	A	Time hr
6	**5**	**4**	**3**	**2**	**1**
0	0	–	–	0	0
6.7	20	–	0	20	4
33.3	100	0	20	80	8
76.7	230	20	80	130	12
120	360	80	130	150	16
136.7	410	130	150	130	20
123.3	370	150	130	90	24
90.7	272	130	90	52	28
56.3	169	90	52	27	32
31.3	94	52	27	15	36
15.7	47	27	15	5	40
6.7	20	15	5	0	44
1.7	5	5	0	–	48
0	0	0	–	–	52

- S - Curve Method :
- This method is used if desired derivative record hydrograph sustainability mD where m non an integer.
- Ex / Prepared solution of the previous example S curved way.

العمود 6 ÷ (4/12)	عمود 4 – عمود 5	منحني S متخلف بـ 12 ساعة	إحداثيات منحنيS (3+2)	منحني S	إحداثيات UH-4 hr	الوقت (ساعة)
7	6	5	4	3	2	1
0	0	–	0	0	0	0
6.7	20	–	20	0	20	4
33.3	100	–	100	20	80	8
76.7	230	0	230	100	130	12
120	360	20	380	230	150	16
136.7	410	100	510	380	130	20
123.3	370	230	600	510	90	24
90.7	272	380	652	600	52	28
56.3	169	510	679	652	27	32
31.3	94	600	694	679	15	36
15.7	47	652	699	694	5	40
6.7	20	679	699	699	0	44
1.7	5	694	699	699	–	48
0	0	699	699	–	–	52

- Ex/ Vertical coordinates of hydrograph 4 - hour shown below. Use these coordinates and derived hydrograph coordinates sustainability hydrograph 2 - hour for the same basin by S - Curve Method .

العمود 6 ÷ (4/2) (UH – 2hr)	عمود 4 – عمود 5	منحني S متخلف بـ 2 ساعة	إحداثيات منحنيS (3+2)	منحني S	إحداثيات UH-4 hr	الوقت (ساعة)
7	6	5	4	3	2	1
0	0	–	0	–	0	0
16	8	0	8	–	8	2
24	12	8	20	0	12	4
62	31	20	51	8	31	6
98	49	51	100	20	49	8
122	61	100	161	51	61	10
138	69	161	230	100	69	12
154	77	230	307	161	77	14
146	73	307	380	230	73	16
138	69	380	449	307	119	18
122	61	449	510	380	61	20
102	51	510	561	449	51	22
78	39	561	600	510	39	24
62	31	600	631	561	31	26
42	21	631	652	600	21	28
34	17	652	669	631	17	30
20	10	669	679	652	10	32
20	10	679	689	669	10	34
10	5	689	694	679	0	36
10	5	694	699	689	0	38

- <u>Distribution percentages:</u>
- The distribution shows the percentages of total unit hydrograph, which occur during successive uniform time increments.
- The procedure of deriving the distribution graph is first to separate the base flow from the total runoff .
- Example / Analysis of the DRO for a one day unit storm over a basin for the following data: 18,96,120,82,47,25,12,and 2m^3. Determine the distribution graph percentages.

Day since beginning of direct runoff	*DRO on mid-day (cumec)*	*Percentage of ΣDRO*	*Remarks*
1	*2*	*3*	*4*
1	18	4.5	$= \frac{18}{402} \times 100$
2	96	24	
3	120	30	
4	82	20	
5	47	12	
6	25	6	
7	12	3	
8	2	0.5	
8 equal time (*i.e.*, a day) intervals	Σ *DRO* = 402	Total = 100.0	

- Example /
- Basin area of 200 hectares, the rainfall in three consecutive days and the depths of the rain was the 7.5, 2 and 5 cm, respectively. the index rate Φ 2.5 cm / day, the distribution percentage of surface runoff per day rainstorm that one day is 5,15, 40,25, 10,5. determine the hydrograph of runoff ?

runoff		(cm) Distribution of ER			distribution %percentages	ER (cm)	(cm/day) Φ	Rainfall (cm)	time (day)
m^3 / s	cm	2.5	0	5					
0.0579*	0.25			0.25	5	5	2.5	7.5	1− 0
0.1736	0.75		0	0.75	15	0	2.5	2	2− 1
0.4919	2.125	0.125	0	2	40	2.5	2.5	5	3 – 2
0.3762	1.625	0.375	0	1.25	25				4 – 3
0.3472	1.5	1	0	0.5	10				5 – 4
0.2025	0.875	0.625	0	0.25	5				6 – 5
0.0579	0.25	0.25	0	0	0				7 – 6
0.0289	0.125	0.125							8 – 7
0	0	0							9 − 8

- السيح في يوم واحد = (60*60*24 / 10^4*200) = 23.148 م^2 / ثا
- 0.25 /100*(23.148) =0.0579 m^3/s

- Example
- The 3-hr unit hydrograph ordinates for a basin are given below. There was a storm, which commenced on July 15 at (6-hr), which was followed by another storm on July 16 at (12-hr). the amount of rainfall on July 15 was 5.75 cm for first (3-hr) and 3.75 cm for next (3-hr) , and on July 16, 4.45 cm for all (12-hr). Assuming an average loss of 0.25 cm/hr and 0.15 cm/hr for the two storms, respectively, and a constant base flow of 10 m^3. Determine the stream flow hydrograph and peak flow .

Time (hr):	*0*	*3*	*6*	*9*	*12*	*15*	*18*	*21*	*24*	*27*
UGO (cumec):	*0*	*1.5*	*4.5*	*8.6*	*12.0*	*9.4*	*4.6*	*2.3*	*0.8*	*0*

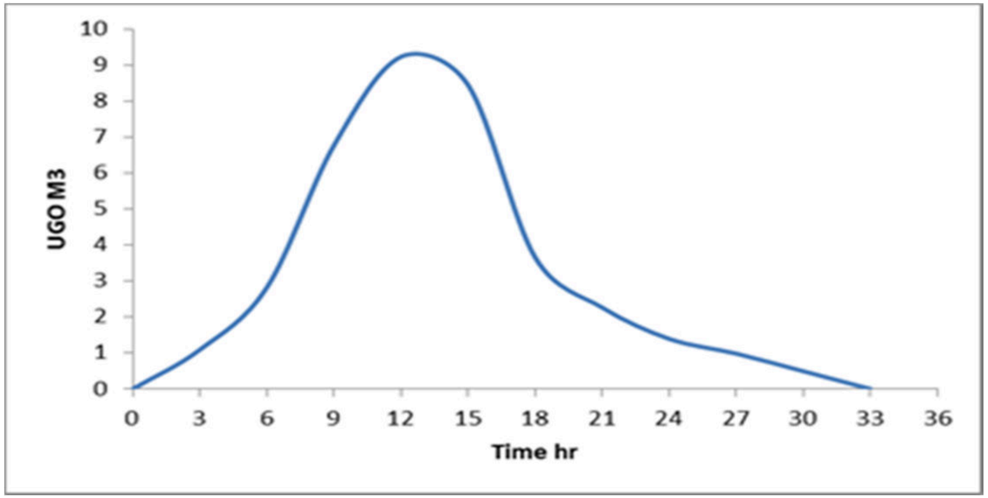
- Solution
- Since the duration of the UG is 3 hr, the 6-hr storm can be considered as 2-unit storm .
- A net rain of 5.75 – 0.25 × 3 = 5 cm in the first 3-hr period and a net rain of 3.75 – 0.25 × 3 = 3 cm in the next 3-hr period.
- The unit hydrograph ordinates are multiplied by the net rain of each period by 3 hr. Similarly,
- Another unit storm by 12 hr (next day) produces a net rain of 4.45 – 0.15 × 3 = 4 cm which is multiplied by the UGO .
- Table show the rainfall excesses due to the three storms are added up to get the total direct surface discharge ordinates.

Time (hr)	UGO*	DRO due to rainfall excess I	II	III	Total DRO	BFO	TRO
		UGO × 5 cm	UGO × 3 cm	UGO × 4 cm	(3) + (4) + (5)	(constant)	(6) + (7)
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
0	0	0	—	—	0	10	10.0
3	1.5	7.5	0	—	7.5	10	17.5
6	4.5	22.5	4.5	—	27.0	10	37.0
9	8.6	43.0	13.5	—	56.5	10	66.5
12	12.0	60.0	25.8	0	85.8	10	95.8
15	9.4	47.0	36.0	6	89.0	10	99.0 — Peak flood
18	4.6	23.0	28.2	18	69.2	10	79.2
21	2.3	11.5	13.8	34.4	59.7	10	69.7
24	0.8	4.0	6.9	48	58.9	10	68.9
27	0	0	2.4	37.6	40.0	10	50.0
30			0	18.4	18.4	10	28.4
33				9.2	9.2	10	19.2
36				3.2	3.2	10	13.2
39				0	0	10	10.0

- Example /Storm rainfalls of 3.2, 8.2 and 5.2 cm occur during three successive hours over an area of 45 km^2. The storm loss rate is 1.2 cm/hr. The distribution percentages of successive
- hours are 5, 20, 40, 20, 10 and 5. Determine the stream flows for successive hours assuming a constant base flow of 10 m3.
- Solution

Time (hr)	Distribution percentages	Rainfall excess (cm)	DRO due to rainfall excess (cm)			Total DRO		BF (cumec)	Stream flow (cumec)
		P-loss = P_{net}	2	7	4	(cm)	(cumec)		
1	5	3.2 – 1.2 = 2	0.10	—	—	0.10	12.5*	10	22.5 ←
2	20	8.2 – 1.2 = 7	0.40	0.35	—	0.75	93.75	10	103.75
3	40	5.3 – 1.2 = 4	0.80	1.40	0.20	2.40	300	10	310
4	20		0.40	2.80	0.80	4.00	500	10	510 ←
5	10		0.20	1.40	1.60	3.20	400	10	410
6	5		0.10	0.70	0.80	1.60	200	10	210
7	—		—	0.35	0.40	0.75	93.75	10	103.75
8	—		—	—	0.20	0.20	25	10	35
Total	100	13	2.00	7.00	4.00	13.00	1625		

$$* \frac{0.10}{100} \frac{(45 \times 10^6)}{1 \times 60 \times 60} = 12.5 \text{ cumec.}$$



- <u>Unit Hydrograph for Different Duration:</u>
- There are several ways derivation standard hydrograph which sustainability nD - an hour from a standard water scheme sustainability D - an hour, and most important of these ways:
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- If the availability of a standard hydrograph who sustainability D - an hour and was required is a derivative standard water scheme nD for - hour, where n is an integer,
- Example (8) / information given is the Y-coordinates record hydrograph sustainability 4 - hour, derived the y-coordinate is 12 - hour water hydrograph record.

Time hr	0	4	8	12	16	20	24	28	32	36	40	44	48	52
Y-coordinates record hydrograph	0	20	80	130	150	130	90	52	27	15	5	0	-	-

- Solution : make table

إحداثيات UH– 12 ساعة (العمود 5 ÷ 3)	الأعمدة 4+3+2	C يزحف بـ 8 ساعة	B يزحف بـ 4 ساعة	A	Time hr
6	5	4	3	2	1
0	0	–	–	0	0
6.7	20	–	0	20	4
33.3	100	0	20	80	8
76.7	230	20	80	130	12
120	360	80	130	150	16
136.7	410	130	150	130	20
123.3	370	150	130	90	24
90.7	272	130	90	52	28
56.3	169	90	52	27	32
31.3	94	52	27	15	36
15.7	47	27	15	5	40
6.7	20	15	5	0	44
1.7	5	5	0	–	48
0	0	0	–	–	52

- S - Curve Method :
- This method is used if desired derivative record hydrograph sustainability mD where m non an integer.
- Ex / Prepared solution of the previous example S curved way.

العمود 6 ÷ (4/12)	عمود 4 – عمود 5	منحني S متخلف بـ 12 ساعة	إحداثيات منحنيS (3+2)	منحني S	إحداثيات UH-4 hr	الوقت (ساعة)
7	6	5	4	3	2	1
0	0	–	0	0	0	0
6.7	20	–	20	0	20	4
33.3	100	–	100	20	80	8
76.7	230	0	230	100	130	12
120	360	20	380	230	150	16
136.7	410	100	510	380	130	20
123.3	370	230	600	510	90	24
90.7	272	380	652	600	52	28
56.3	169	510	679	652	27	32
31.3	94	600	694	679	15	36
15.7	47	652	699	694	5	40
6.7	20	679	699	699	0	44
1.7	5	694	699	699	–	48
0	0	699	699	–	–	52

- Ex/ Vertical coordinates of hydrograph 4 - hour shown below. Use these coordinates and derived hydrograph coordinates sustainability hydrograph 2 - hour for the same basin by S - Curve Method .

العمود 6 ÷ (4/2) (UH – 2hr)	عمود 4 – عمود 5	منحنى S متخلف بـ 2 ساعة	إحداثيات منحنيS (3+2)	منحنى S	إحداثيات UH-4 hr	الوقت (ساعة)
7	6	5	4	3	2	1
0	0	–	0	–	0	0
16	8	0	8	–	8	2
24	12	8	20	0	12	4
62	31	20	51	8	31	6
98	49	51	100	20	49	8
122	61	100	161	51	61	10
138	69	161	230	100	69	12
154	77	230	307	161	77	14
146	73	307	380	230	73	16
138	69	380	449	307	119	18
122	61	449	510	380	61	20
102	51	510	561	449	51	22
78	39	561	600	510	39	24
62	31	600	631	561	31	26
42	21	631	652	600	21	28
34	17	652	669	631	17	30
20	10	669	679	652	10	32
20	10	679	689	669	10	34
10	5	689	694	679	0	36
10	5	694	699	689	0	38

- <u>Distribution percentages:</u>
- The distribution shows the percentages of total unit hydrograph, which occur during successive uniform time increments.
- The procedure of deriving the distribution graph is first to separate the base flow from the total runoff .
- Example / Analysis of the DRO for a one day unit storm over a basin for the following data: 18,96,120,82,47,25,12,and 2m^3. Determine the distribution graph percentages.

Day since beginning of direct runoff	*DRO on mid-day (cumec)*	*Percentage of ΣDRO*	*Remarks*
1	*2*	*3*	*4*
1	18	4.5	$= \frac{18}{402} \times 100$
2	96	24	
3	120	30	
4	82	20	
5	47	12	
6	25	6	
7	12	3	
8	2	0.5	
8 equal time (*i.e.*, a day) intervals	Σ *DRO* = 402	Total = 100.0	

- Example /
- Basin area of 200 hectares, the rainfall in three consecutive days and the depths of the rain was the 7.5, 2 and 5 cm, respectively. the index rate Φ 2.5 cm / day, the distribution percentage of surface runoff per day rainstorm that one day is 5,15, 40,25, 10,5. determine the hydrograph of runoff ?

runoff		(cm) Distribution of ER			distribution %percentages	ER (cm)	(cm/day) Φ	Rainfall (cm)	time (day)
m^3 / s	cm	2.5	0	5					
0.0579*	0.25			0.25	5	5	2.5	7.5	1– 0
0.1736	0.75		0	0.75	15	0	2.5	2	2– 1
0.4919	2.125	0.125	0	2	40	2.5	2.5	5	3 – 2
0.3762	1.625	0.375	0	1.25	25				4 – 3
0.3472	1.5	1	0	0.5	10				5 – 4
0.2025	0.875	0.625	0	0.25	5				6 – 5
0.0579	0.25	0.25	0	0	0				7 – 6
0.0289	0.125	0.125							8 – 7
0	0	0							9 – 8

- السيح في يوم واحد = (60*60*24 / 10^4*200) = 23.148 م^2 / ثا
- 0.25 /100*(23.148) =0.0579 m^3/s

- Example
- The 3-hr unit hydrograph ordinates for a basin are given below. There was a storm, which commenced on July 15 at (6-hr), which was followed by another storm on July 16 at (12-hr). the amount of rainfall on July 15 was 5.75 cm for first (3-hr) and 3.75 cm for next (3-hr) , and on July 16, 4.45 cm for all (12-hr). Assuming an average loss of 0.25 cm/hr and 0.15 cm/hr for the two storms, respectively, and a constant base flow of 10 m^3. Determine the stream flow hydrograph and peak flow .

Time (hr):	*0*	*3*	*6*	*9*	*12*	*15*	*18*	*21*	*24*	*27*
UGO (cumec):	*0*	*1.5*	*4.5*	*8.6*	*12.0*	*9.4*	*4.6*	*2.3*	*0.8*	*0*

- Solution
- Since the duration of the UG is 3 hr, the 6-hr storm can be considered as 2-unit storm .
- A net rain of 5.75 – 0.25 × 3 = 5 cm in the first 3-hr period and a net rain of 3.75 – 0.25 × 3 = 3 cm in the next 3-hr period.
- The unit hydrograph ordinates are multiplied by the net rain of each period by 3 hr. Similarly,
- Another unit storm by 12 hr (next day) produces a net rain of 4.45 – 0.15 × 3 = 4 cm which is multiplied by the UGO .
- Table show the rainfall excesses due to the three storms are added up to get the total direct surface discharge ordinates.

Time (hr)	*UGO**	*DRO due to rainfall excess* *I* *UGO × 5 cm*	*II* *UGO × 3 cm*	*III* *UGO × 4 cm*	*Total DRO (3) + (4) + (5)*	*BFO (constant)*	*TRO (6) + (7)*	
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	
0	0	0	—	—	0	10	10.0	
3	1.5	7.5	0	—	7.5	10	17.5	
6	4.5	22.5	4.5	—	27.0	10	37.0	
9	8.6	43.0	13.5	—	56.5	10	66.5	
12	12.0	60.0	25.8	0	85.8	10	95.8	
15	9.4	47.0	36.0	6	89.0	10	99.0	— Peak flood
18	4.6	23.0	28.2	18	69.2	10	79.2	
21	2.3	11.5	13.8	34.4	59.7	10	69.7	
24	0.8	4.0	6.9	48	58.9	10	68.9	
27	0	0	2.4	37.6	40.0	10	50.0	
30			0	18.4	18.4	10	28.4	
33				9.2	9.2	10	19.2	
36				3.2	3.2	10	13.2	
39				0	0	10	10.0	

- Example /Storm rainfalls of 3.2, 8.2 and 5.2 cm occur during three successive hours over an area of 45 km^2. The storm loss rate is 1.2 cm/hr. The distribution percentages of successive
- hours are 5, 20, 40, 20, 10 and 5. Determine the stream flows for successive hours assuming a constant base flow of 10 m3.
- Solution

Time (hr)	*Distribution percentages*	*Rainfall excess (cm)*	*DRO due to rainfall excess (cm)*			*Total DRO*		*BF (cumec)*	*Stream flow (cumec)*
		P-loss = P_{net}	*2*	*7*	*4*	*(cm)*	*(cumec)*		
1	5	3.2 – 1.2 = 2	0.10	—	—	0.10	12.5*	10	22.5 ←
2	20	8.2 – 1.2 = 7	0.40	0.35	—	0.75	93.75	10	103.75
3	40	5.3 – 1.2 = 4	0.80	1.40	0.20	2.40	300	10	310
4	20		0.40	2.80	0.80	4.00	500	10	510 ←
5	10		0.20	1.40	1.60	3.20	400	10	410
6	5		0.10	0.70	0.80	1.60	200	10	210
7	—		—	0.35	0.40	0.75	93.75	10	103.75
8	—		—	—	0.20	0.20	25	10	35
Total	100	13	2.00	7.00	4.00	13.00	1625		

$$* \frac{0.10}{100} \frac{(45 \times 10^6)}{1 \times 60 \times 60} = 12.5 \text{ cumec.}$$

- Example /Flood frequency studies are made for the 30-year flood data (from 1939-1968) of lower Tapi river as shown in Table the area is 62000km^2 determine the percentage probability by Weibull andGumbel's methods also three Coefficients of flood

year	1939	1940	1941	1942	1943	1944	1945	1946	1947	1948	1949	1950	1951	1952	1953
Annual flood x 1000 m3	42.45	37.3	29.3	24.2	22.62	21.24	20.86	19.65	18.7	18.3	14.57	14	12.88	12.45	11.43
year	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968
Annual flood x 1000 m3	10.34	9.72	9.68	8.5	8.44	7.65	7.27	7.22	6.48	6.23	6.09	5.81	4.82	4.39	3.68

- Solution :
- a – Weibull method :

Year	Annual flood x 1000 m3	No. of time (m)	T= (n+1)/m	percentage probability P= 1/T *100%
1939	42.45	1	31.00	3.23
1940	37.3	2	15.50	6.45
1941	29.3	3	10.33	9.68
1942	24.2	4	7.75	12.90
1943	22.62	5	6.20	16.13
1944	21.24	6	5.17	19.35
1945	20.86	7	4.43	22.58
1946	19.65	8	3.88	25.81
1947	18.7	9	3.44	29.03
1948	18.3	10	3.10	32.26
1949	14.57	11	2.82	35.48
1950	14	12	2.58	38.71
1951	12.88	13	2.38	41.94
1952	12.45	14	2.21	45.16
1953	11.43	15	2.07	48.39
1954	10.34	16	1.94	51.61
1955	9.72	17	1.82	54.84
1956	9.68	18	1.72	58.06
1957	8.5	19	1.63	61.29
1958	8.44	20	1.55	64.52
1959	7.65	21	1.48	67.74
1960	7.27	22	1.41	70.97
1961	7.22	23	1.35	74.19
1962	6.48	24	1.29	77.42
1963	6.23	25	1.24	80.65
1964	6.09	26	1.19	83.87
1965	5.81	27	1.15	87.10
1966	4.82	28	1.11	90.32
1967	4.39	29	1.07	93.55
1968	3.68	30	1.03	96.77
		n= 30		

- (b) Gumbel's method:

tcm = 1000 cumec

(*i*) Mean flood, $\bar{x} = \frac{\Sigma x}{n} = \frac{426.27}{30}$ = 14.21 tcm or 14210 cumec

(*iii*) Standard deviation,

$$\sigma = \sqrt{\frac{\Sigma(x-\bar{x})^2}{n-1}} = \sqrt{\frac{2724.26}{30-1}} = 9.7$$

(*iv*) Coefficient of variation,

$$C_v = \frac{\sigma}{\bar{x}} = \frac{9.7}{14.21} = 0.68$$

(*v*) Coefficient of skew,

$$C_s = \frac{\Sigma(x-\bar{x})^3}{(n-1)\,\sigma^3} = \frac{34319}{(30-1)\,(9.7)^3} = 1.3, \ (C_s = 2C_v)$$

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$$C_f = \frac{\bar{x}}{A^{0.8}/2.14} = \frac{14210}{(62000)^{0.8}/2.14} = 4.46$$

Gumbel's method

$x-\bar{x}$	$(x-\bar{x})^2$	$^*(x-\bar{x})^3$	Reduced variate[+] $y = \frac{(x-\bar{x}) + 0.45\,\sigma}{0.7797\,\sigma}$	Recurrence interval, yr $T = \frac{1}{1-e^{-e^{-y}}}$	Per cent probability $P = \frac{1}{T} \times 100\,\%$
28.24	798	22600	4.31	74	1.35
23.09	537	12400	3.63	38	2.63
15.09	227	3430	2.57	13	7.7
9.99	99.9	1000	1.90	7	14.3
8.41	70.7	595	1.69	5.5	18.2
7.03	49.4	347	1.51	5	20.0
6.65	44.2	294	1.46	4.8	20.8
5.44	29.6	161	1.30	3.7	27.0
4.49	20.2	91	1.17	3.3	30.3
4.09	16.7	68	1.12	3.1	32.2
0.36	0.13	0.05	0.63		
−0.21	0.04	−0.01	0.56	2.3	43.5
−1.33	1.77	−2.36	0.40		
−1.76	3.10	−8.45	0.34	2.0	50.0
−2.78	7.72	−21.50	0.21		
−3.87	15.0	−58.20	0.06		
−4.49	20.2	−90.80	−0.02		
−4.53	20.6	−93.5	−0.023		
−5.71	32.6	−186.0	−0.18		
−5.77	33.2	−192.0	−0.19		
−6.56	43.1	−283.0	−0.29		
−6.94	48.1	−334.0	−0.34		
−6.99	48.9	−342.0	−0.35		
−7.73	59.8	−463.0	−0.44		
−7.98	63.7	−508.0	−0.48	1.25	80
−8.12	66.0	−536.0	−0.50		
−8.40	70.6	−593.0	−0.55		
−9.39	88.2	−828.0	−0.67		
−9.82	96.3	−945.0	−0.72		
−10.53	112.2	−1183.0	−0.82		

$\Sigma(x-\bar{x})^2 = 2724.26$

$34319 = (x-\bar{x})^3$ (algebraic sum) $= (40986 - 6667)$

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- Flood routing is the process of determining the storage reservoir stage. عمليات حساب الاحتياطي من الخزين في الحوض المائي
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- Routing reservoir :
- Studying effect of the flood inside the tank, the knowledge of the characteristics of size - attributed to the tank
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$$I = O + \Delta S$$

- where I = inflow rate
- O = outflow rate
- ΔS = incremental storage, at any instant.
- Taking a small interval of time, t (called the routing period and designating the initial and final conditions by subscripts 1 and 2 between the interval, Eq. may be written as :

$$\left(\frac{I_1 + I_2}{2}\right)t - \left(\frac{O_1 + O_2}{2}\right)t = S_2 - S_1$$

- Muskingum's Method for Routing طريقة ماسكنجام في الإستتباع

$$Q_2 = C_o I_2 + C_1 I_1 + C_2 Q_1$$

$$C_o = \frac{-kx + 0.5\Delta t}{k - kx + 0.5\Delta t}$$

$$C_1 = \frac{kx + 0.5\Delta t}{k - kx + 0.5\Delta t}$$

$$C_2 = \frac{k - kx - 0.5\Delta t}{k - kx + 0.5\Delta t}$$

$$C_o + C_1 + C_2 = 1$$

k : ثابت فترة الخزن

x : معامل موزون

- Example / routing the river of k= 12 hr. and x= 0.2 if the outflow is 10 m^3/s and the table show the inflow with the time of flood ?

54	48	42	36	30	24	18	12	6	0	Time hr
15	20	27	35	45	55	60	50	20	10	(m^3/s) In flow

- Solution :
- From the table the first $\Delta t = 6$ hr.

- $C_o = \frac{-kx + 0.5\Delta t}{k - kx + 0.5\Delta t}$ = (-12 *0.2 +0.5 *6)/ (12-12*0.2+0.5*6) =0.048
- $I_2 = 20$ $C_o I_2 = 20*0.048 = 0.96$

- $I_1 = 10$ $C_1 I_1 = 4.29$
- $Q_1 = 10$ $C_2 Q_1 = 5.23$

- $Q_2 = C_o I_2 + C_1 I_1 + C_2 Q_1$
- $Q_2 = 0.96+5.23+4.29= 10.48\ m^3/s$

Q	0.523 Q_1	0.429 I_1	0.048 I_2	I (m^3/s)	Time hr
10	5.23	4.29	0.96	10	0
10.48	5.48	8.58	2.4	20	6
16.46	8.61	21.45	2.88	50	12
32.49	17.23	25.74	2.64	60	18
45.61	23.85	23.6	2.16	55	24
49.61	25.95	19.3	1.68	45	30
46.93	24.55	15.02	1.3	35	36
40.87	21.38	11.58	0.96	27	42
33.92	17.74	8.58	0.72	20	48
27.04				15	54

- Example /Flood frequency studies are made for the 30-year flood data (from 1939-1968) of lower Tapi river as shown in Table the area is 62000km^2 determine the percentage probability by Weibull andGumbel's methods also three Coefficients of flood

year	1939	1940	1941	1942	1943	1944	1945	1946	1947	1948	1949	1950	1951	1952	1953
Annual flood x 1000 m3	42.45	37.3	29.3	24.2	22.62	21.24	20.86	19.65	18.7	18.3	14.57	14	12.88	12.45	11.43
year	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968
Annual flood x 1000 m3	10.34	9.72	9.68	8.5	8.44	7.65	7.27	7.22	6.48	6.23	6.09	5.81	4.82	4.39	3.68

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1957	8.5	19	1.63	61.29
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1963	6.23	25	1.24	80.65
1964	6.09	26	1.19	83.87
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$$Q_2 = C_o\, I_2 + C_1\, I_1 + C_2\, Q_1$$

$$C_o = \frac{-kx + 0.5\Delta t}{k - kx + 0.5\Delta t}$$

$$C_1 = \frac{kx + 0.5\Delta t}{k - kx + 0.5\Delta t}$$

$$C_2 = \frac{k - kx - 0.5\Delta t}{k - kx + 0.5\Delta t}$$

$$C_o + C_1 + C_2 = 1$$

k : ثابت فترة الخزن

x : معامل موزون

- Example / routing the river of k= 12 hr. and x= 0.2 if the outflow is 10 m^3/s and the table show the inflow with the time of flood ?

54	48	42	36	30	24	18	12	6	0	Time hr
15	20	27	35	45	55	60	50	20	10	(m^3/s) In flow

- Solution :
- From the table the first Δt = 6 hr.

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- $I_2 = 20$ $C_o I_2 = 20*0.048 = 0.96$

- $I_1 = 10$ $C_1 I_1 = 4.29$
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- $Q_2 = C_o I_2 + C_1 I_1 + C_2 Q_1$
- $Q_2 = 0.96+5.23+4.29= 10.48\ m^3/s$

Q	0.523 Q_1	0.429 I_1	0.048 I_2	I (m^3/s)	Time hr
10	5.23	4.29	0.96	10	0
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49.61	25.95	19.3	1.68	45	30
46.93	24.55	15.02	1.3	35	36
40.87	21.38	11.58	0.96	27	42
33.92	17.74	8.58	0.72	20	48
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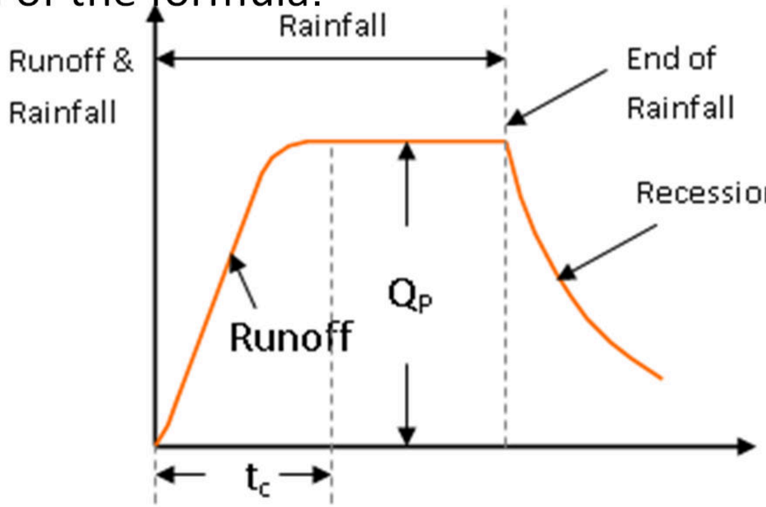
Chapter six
FLOODS

- A flood is an unusual high stage of a river due to runoff from rainfall and/or melting of snow in quantities too great to be confined in the normal water surface elevations of the river or stream,
- The maximum flood that any structure can safely pass is called the “ safe design flood “
- and this occur after consideration of economic and hydrologic actors.

- <u>ESTIMATION OF PEAK FLOOD:</u>
- The maximum flood discharge (peak flood) in a river may be determined by the following methods:-
- (i) Rational method. الطريقة العقلانية
- (ii) Empirical formulae and curves.(التجريبية) الطريقة الوضعية
- (iii) Concentration time method.طريقة وقت التركيز
- (iv) Overland flow hydrograph .
- (v) Physical indications of past.
- (vi) Unit hydrograph . طريقة الهيدروغراف
- (vii) Flood frequency studies. دراسات تردد الفيضان
-

- Rational Method :
- The rational method is based on the application of the formula:



- after (t_c), the runoff will be steady at the peak value (Q_P):
- $Q = C\ i\ A \qquad t \geq t\ c$
- C = Runoff / Rainfall , A : Area of basin
- i : intensity of Rainfall
-

- When using field units can be written above equation as follows:

$$Q_P = \frac{1}{3.6} C \, (i_{tcp}) \, A$$

Q_P : (m^3/s) تصريف الذروة

C : معامل السيح

i_{tcp} : P وإحتمالية t_c لإستدامة (ملم / ساعة) متوسط شدة المطر

A : (كم^2) مساحة التصريف

- 2- Time of Concentration (t_c) :
- Kirpich Equation : معادلة كيربج
- $tc = 0.01947 \, L^{0.77} \, S^{-0.385}$
- tc : (min) فترة التركيز
- L : (m) أقصى مسافة يقطعها الماء
- S = Δ H / L : إنحدار الجابية
- Δ H : فرق المنسوب من أبعد نقطة في الجابية إلى المخرج
-

- Example (1)
- Area of 0.85 km^2 a runoff coefficient of 0.3, If you know that a decline in basin 0.006 and the maximum distance traveled by water equal to 950 m and was rainfall during the reference period of 25 years are as in the following table Calculate the peak discharge rate (Q_p) for the design of origin at the port of this region for a period of 25 years back.

60	40	30	20	10	5	(min) Time
62	57	50	40	26	17	(mm) Rainfall drop

$$t_c = 0.01947 * (950)^{0.77} * (0.006)^{-0.385} = 27.4 \text{ min.}$$

أقصى عمق للمطر لإستدامة 27.4 دقيقة (ملم) :

$$\frac{50-10}{10} * 7.4 + 40 = 47.4 \text{ mm}$$

متوسط الشدة i_{tcp} (ملم / ساعة) :

$$i_{tcp} = \frac{47.4}{27.4} * 60 = 103.8 \text{ mm/hr.}$$

$$Q_p = \frac{0.3 * 103.8 * 0.85}{3.6} = 7.35 \text{ m}^3\text{/s.}$$

- Empirical Formulas الصيغ التجريبية
- Based on statistical Flood Peak – Area Relationships :
- $Q_P = f(A)$
- 1- Dickens Formula :
- $Q_P = C_D A^{3/4}$

Q_P : (m^3/s) التصريف الأقصى للفيضان

A : (كم^2) مساحة الجابية

- C_D : (30 – 6) ثابت ديكنز

- Ryves Formula :
- $Q_P = C_R A^{2/3}$

Q_P : (m^3/s) التصريف الأقصى للفيضان

A : (كم^2) مساحة الجابية

C_R : ثابت رايف

= 6.8 للمناطق التي تبعد بحدود (80) كم عن الساحل

= 8.5 للمناطق التي تبعد بحدود (80 – 160) كم عن الساحل

= 10.2 لبعض المناطق قرب الجبال

- Inglis Formula :
- $Q_P = \frac{124A}{\sqrt{A + 10.4}}$

Q_P : (m^3/s) التصريف الأقصى للفيضان

A : $(^2$كم$)$ مساحة الجابية

- Fuller's Formula :
- $Q_{TP} = C_f A^{0.8} \ (1 + 0.8 \log T)$

Q_{TP} : (m^3/s) التصريف الأقصى خلال 24 ساعة بتردد T سنة

C_f : $(1.88 - 0.18)$ ثابت فولر

- Bird – McWarn Formula :

$$Q_{MP} = \frac{3025A}{(278 + A)^{0.78}}$$

-

- Example (2) Calculate the maximum discharge flood using the position and the amount of basin area of 40.5 km^2 by ?
- Dickens Formula (CD = 6)
- Ryves Formula (CR = 6.8)
- Inglis Formula
- Bird – McWarn Formula
- Solution :
- $QP = 6 * (40.5)^{0.75} = 96.3$ m3/s
- $QP = 6.8 (40.5)^{2/3} = 80.2$ m3/s

$$Q_P = \frac{124 * 40.5}{\sqrt{40.5 + 10.4}} = 704 \ m^3/s$$

$$Q_{MP} = \frac{3025 * 40.5}{(278 + 40.5)^{0.78}} = 1367 \ m^3/s$$

- METHODS OF FLOOD CONTROL:
- The damages due to the floods can be minimized by the following flood control measures, singly or in combination.
- (i) confining the flow between high banks by constructing dam or flood walls.
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- The purpose of a flood control reservoir is to temporarily store a portion of the flood so that the flood peaks are flattened out
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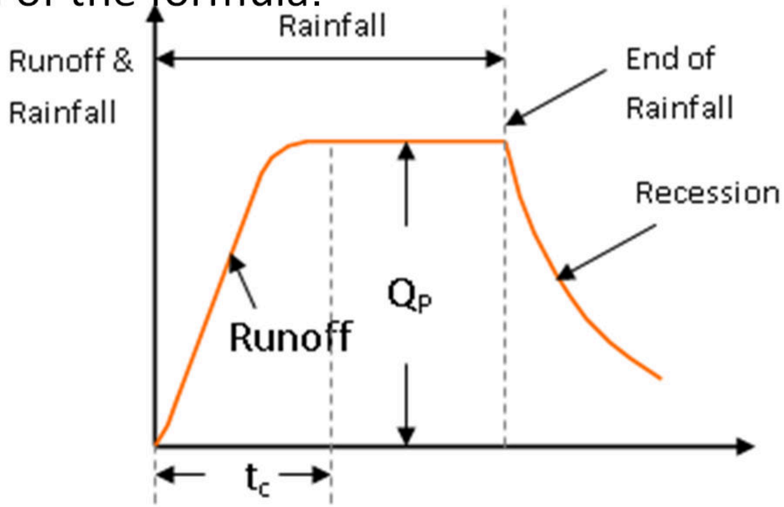
- In general, at least one-third of the total drainage area should come under one reservoir for effective flood control.

Chapter six
FLOODS

- <u>A flood </u>is an unusual high stage of a river due to runoff from rainfall and/or melting of snow in quantities too great to be confined in the normal water surface elevations of the river or stream,
- The maximum flood that any structure can safely pass is called the “ safe design flood “
- and this occur after consideration of economic and hydrologic actors.

- <u>ESTIMATION OF PEAK FLOOD:</u>
- The maximum flood discharge (peak flood) in a river may be determined by the following methods:-
- (i) Rational method. الطريقة العقلانية
- (ii) Empirical formulae and curves.(التجريبية) الطريقة الوضعية
- (iii) Concentration time method.طريقة وقت التركيز
- (iv) Overland flow hydrograph .
- (v) Physical indications of past.
- (vi) Unit hydrograph . طريقة الهيدروغراف
- (vii) Flood frequency studies. دراسات تردد الفيضان
-

- Rational Method :
- The rational method is based on the application of the formula:



- after (t_c), the runoff will be steady at the peak value (Q_P):
- $Q = C\ i\ A \qquad t \geq t\ c$
- C = Runoff / Rainfall , A : Area of basin
- i : intensity of Rainfall
-

- When using field units can be written above equation as follows:

$$Q_P = \frac{1}{3.6} C \, (i_{tcp}) \, A$$

Q_P : (m^3/s) تصريف الذروة

C : معامل السيح

i_{tcp} : P وإحتمالية t_c لإستدامة (ملم / ساعة) متوسط شدة المطر

A : ($كم^2$) مساحة التصريف

- 2- Time of Concentration (t_c) :
- Kirpich Equation : معادلة كيربج
- $tc = 0.01947 \, L^{0.77} \, S^{-0.385}$
- tc : (min) فترة التركيز
- L : (m) أقصى مسافة يقطعها الماء
- S = Δ H / L : إنحدار الجابية
- Δ H : فرق المنسوب من أبعد نقطة في الجابية إلى المخرج
-

- Example (1)
- Area of 0.85 km^2 a runoff coefficient of 0.3, If you know that a decline in basin 0.006 and the maximum distance traveled by water equal to 950 m and was rainfall during the reference period of 25 years are as in the following table Calculate the peak discharge rate (Q_p) for the design of origin at the port of this region for a period of 25 years back.

60	40	30	20	10	5	(min) Time
62	57	50	40	26	17	(mm) Rainfall drop

$$t_c = 0.01947 * (950)^{0.77} * (0.006)^{-0.385} = 27.4 \text{ min.}$$

أقصى عمق للمطر لإستدامة 27.4 دقيقة (ملم) :

$$\frac{50-10}{10} * 7.4 + 40 = 47.4 \text{ mm}$$

متوسط الشدة i_{tcp} (ملم / ساعة) :

$$i_{tcp} = \frac{47.4}{27.4} * 60 = 103.8 \text{ mm/hr.}$$

$$Q_p = \frac{0.3 * 103.8 * 0.85}{3.6} = 7.35 \text{ m}^3\text{/s.}$$

- Empirical Formulas الصيغ التجريبية
- Based on statistical Flood Peak – Area Relationships :
- $Q_P = f(A)$
- 1- Dickens Formula :
- $Q_P = C_D A^{3/4}$

Q_P : (m^3/s) التصريف الأقصى للفيضان

A : (كم^2) مساحة الجابية

- C_D : (30 – 6) ثابت ديكنز

- Ryves Formula :
- $Q_P = C_R A^{2/3}$

Q_P : (m^3/s) التصريف الأقصى للفيضان

A : (كم^2) مساحة الجابية

C_R : ثابت رايف

= 6.8 للمناطق التي تبعد بحدود (80) كم عن الساحل

= 8.5 للمناطق التي تبعد بحدود (80 – 160) كم عن الساحل

= 10.2 لبعض المناطق قرب الجبال

- Inglis Formula :
- $Q_P = \frac{124A}{\sqrt{A+10.4}}$

Q_P : (m^3/s) التصريف الأقصى للفيضان

A : $(^2$كم$)$ مساحة الجابية

- Fuller's Formula :
- $Q_{TP} = C_f A^{0.8} (1+ 0.8 \log T)$

Q_{TP} : (m^3/s) التصريف الأقصى خلال 24 ساعة بتردد T سنة

C_f : $(1.88 - 0.18)$ ثابت فولر

- Bird – McWarn Formula :

$$Q_{MP} = \frac{3025A}{(278+A)^{0.78}}$$

-

- Example (2) Calculate the maximum discharge flood using the position and the amount of basin area of 40.5 km^2 by ?
- Dickens Formula (CD = 6)
- Ryves Formula (CR = 6.8)
- Inglis Formula
- Bird – McWarn Formula
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- Flow of ground water except through coarse gravels and rock fills is laminar and the velocity of flow is given by Darcy’s law (1856), which states that ‘the velocity of flow in a porous medium is proportional to the hydraulic gradient ’

$$V = Ki,$$

$$i = \frac{\Delta h}{L}$$

$$Q = AV = AKi, \quad A = Wb,$$

$$Q = WbKi$$

where V = velocity of flow through the aquifer

K = coefficient of permeability of aquifer soil

i = hydraulic gradient

$= \frac{\Delta h}{L}$, Δh = head lost in a length of flow path L

A = cross-sectional area of the aquifer (= wb)

w = width of aquifer

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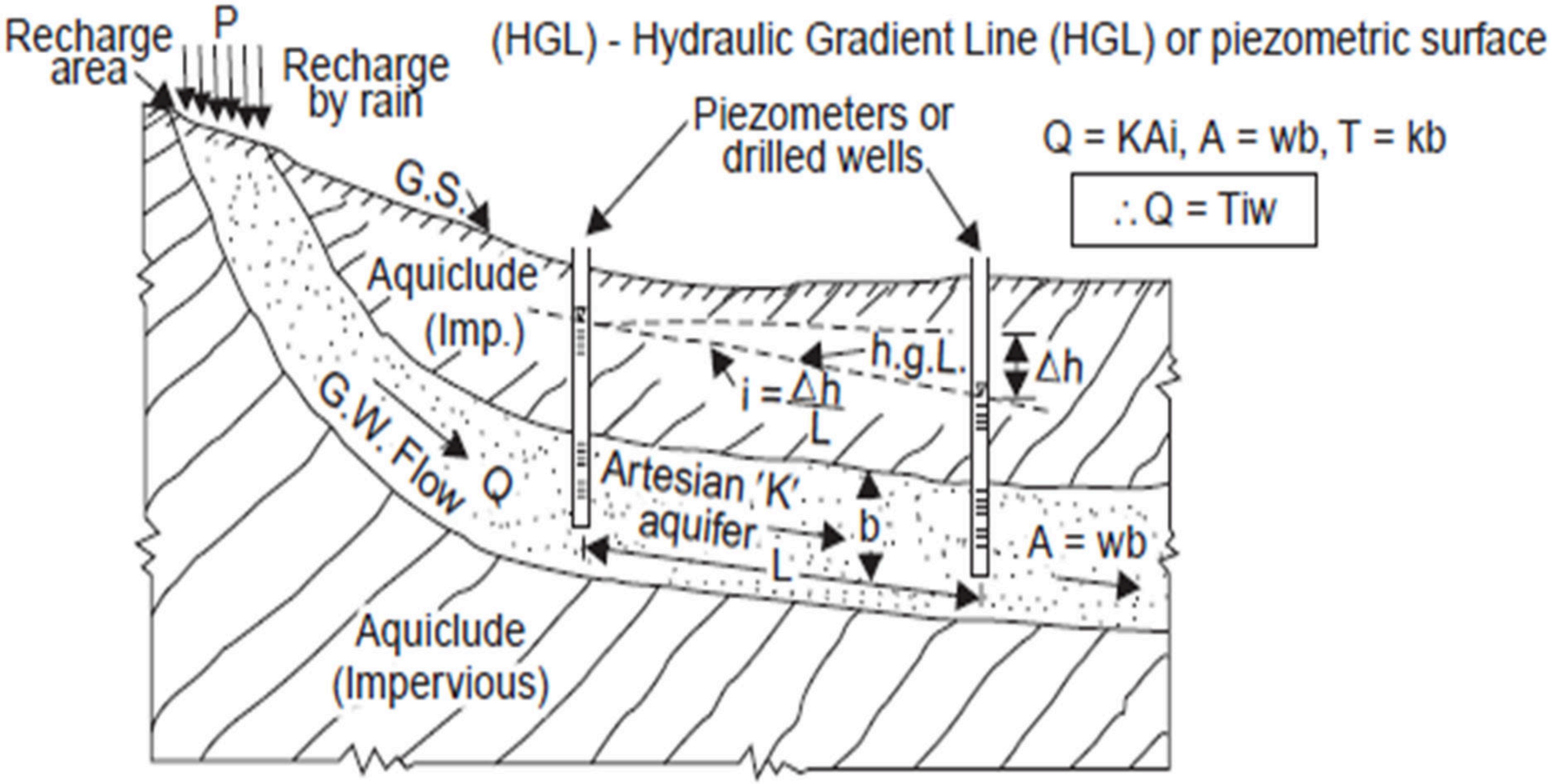

Recharge area
P
Recharge by rain
(HGL) - Hydraulic Gradient Line (HGL) or piezometric surface
Piezometers or drilled wells
Q = KAi, A = wb, T = kb
∴ Q = Tiw
G.S.
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Δh
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Q
Artesian 'K' aquifer
b
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- Wells Confined Flow ابار الجريان المحصور

يوضح الشكل أدناه بئراً يخترق تكويناً خازناً محصوراً سمكه B بافتراض أن للبئر تصريفاً ثابتاً مقداره Q ، فإذا كان الارتفاع H وكان عند بئر الضخ هو h_w ومنحني الهبوط فيه هو S_w :

$$Q = \frac{2\pi\ k\ T\ (h_2 - h_1)}{\ln\left(\frac{r_2}{r_1}\right)} \quad , \text{if} \quad S_1 = H - h_1 \ , \ S_2 = H - h_2 \ ,$$

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وعند حافة منطقة التأثير ($H = h_2$, $R = r_2$, $S = 0$) كما أن ($r_1 = r_w$, $h_1 = h_w$, $S_1 = S_w$) عند جدار البئر :

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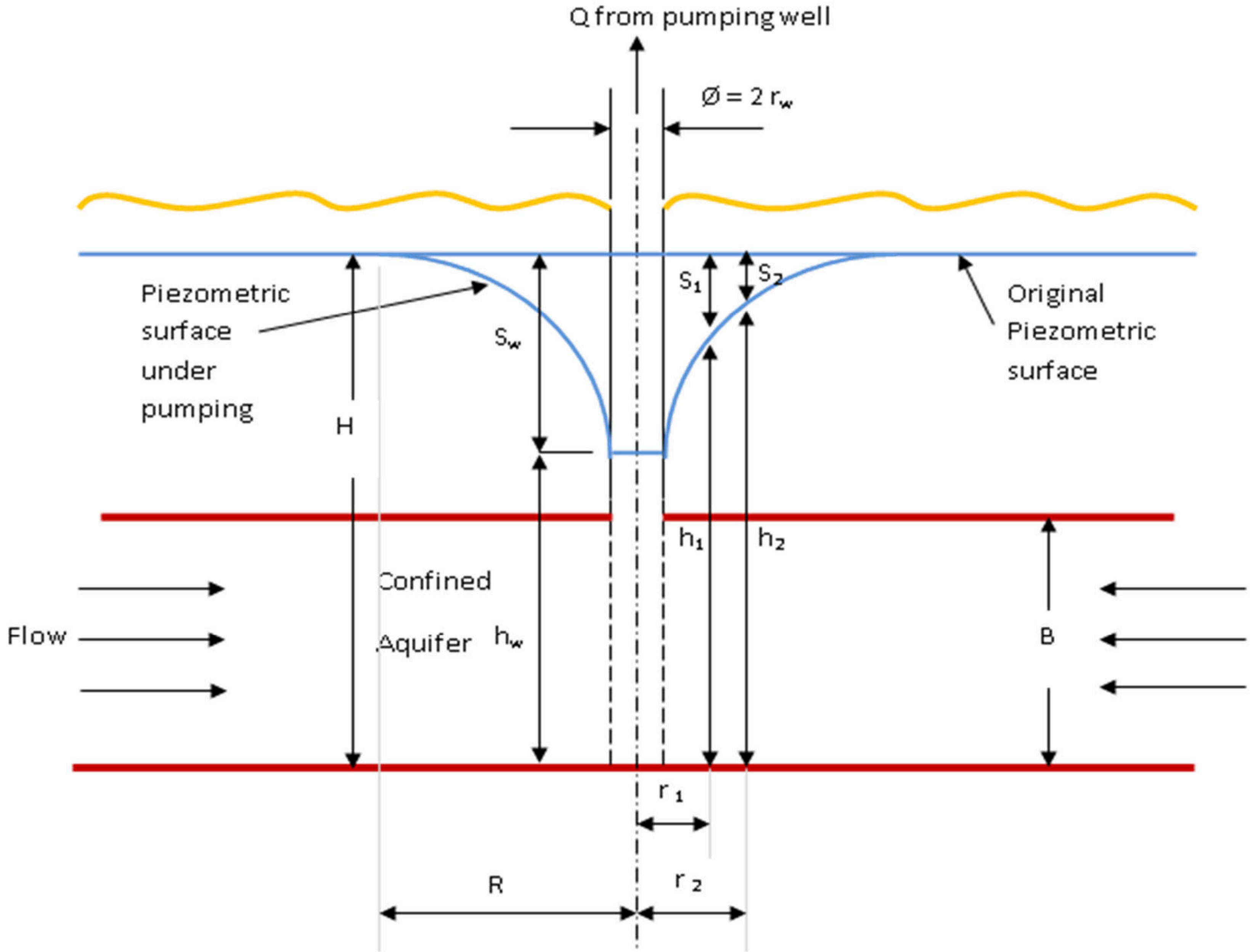

Q from pumping well
Ø = 2 r$_w$
Piezometric surface under pumping
S$_w$
S$_1$
S$_2$
Original Piezometric surface
H
h$_1$
h$_2$
Confined Aquifer
h$_w$
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- Example/
- Well of a 30 cm diameter penetrates the whole configuration limited permeability coefficient 45 m / day, if I learned that the thickness of the layer = 20 meters and be curved downward and radius of influence when pumping fixed the situation on 3 and 300 meters respectively, calculate the amount of discharge of the well?
- Solution

$R = 300$ m. , $r_w = 0.15$ m. , $S_w = 3$ m. , $B = 20$ m.

$k = 45 / (3600 * 24) = 5.208 * 10^{-4}$ m/s.

$T = 5.208 * 10^{-4} * 20 = 10.416*10^{-3}$ m^2/s.

$Q = 2 * \pi * 10.416*10^{-3} * 3 / \ln(300/0.15) = 0.02583$ m^3/s. = 1550 liter/min.

- Unconfined Flow (الجريان غير المحصور (الحر

$$Q = \frac{\pi k \ (h_2^2 - h_1^2)}{\ln (\frac{r_2}{r_1})}$$

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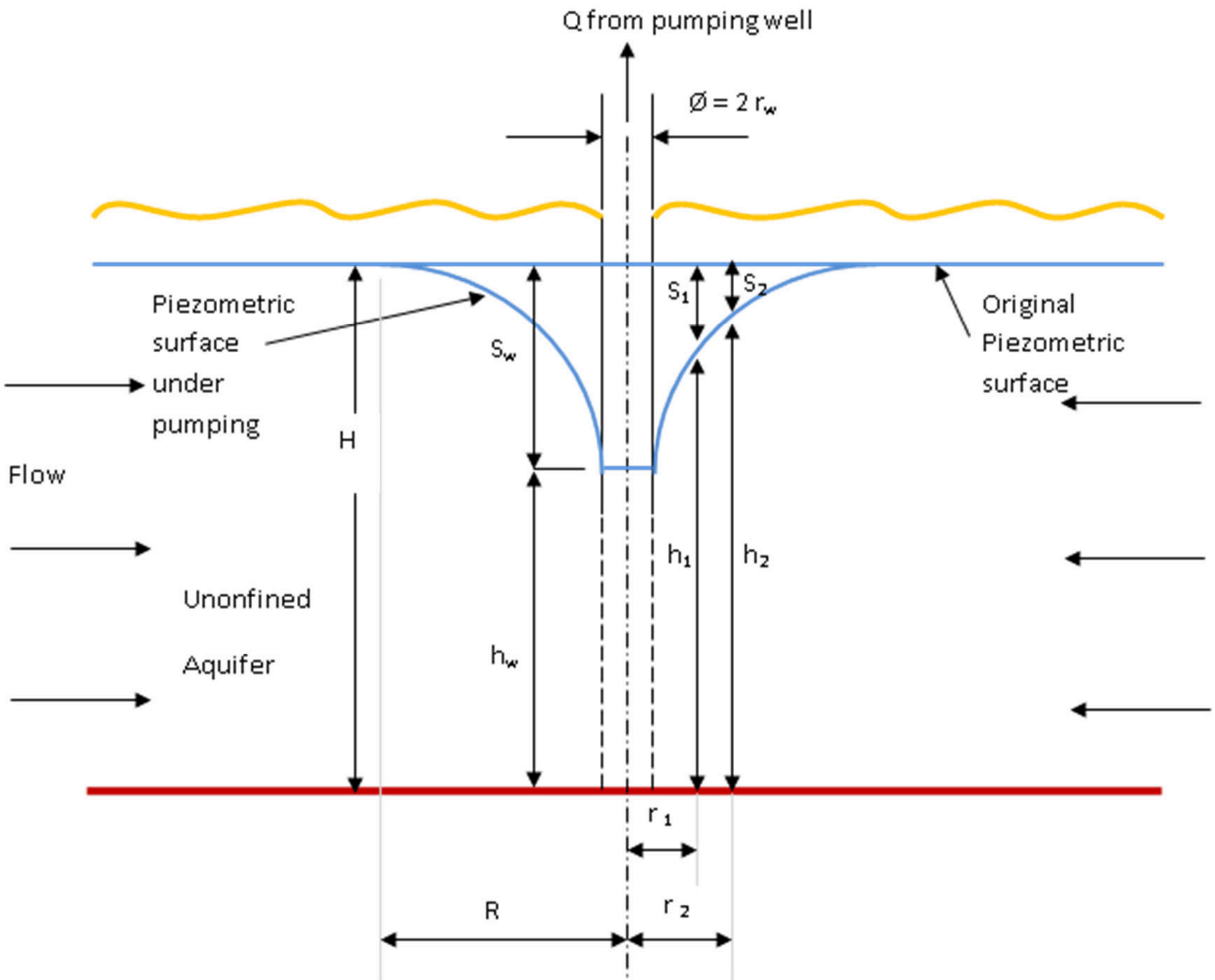

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surface
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S_w
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h_2
Unonfined
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- Example (3)
- wells 30 cm diameter penetrates the whole configuration free depth of 40 meters and long after pumping at a constant rate of 1 500 liters / minute, it appeared that curved downward in wells control for 25 and 75 meters from the well pumping are 3.5 and 2 meters respectively, calculate the coefficient configurable, how much curved landing at a well pumping

1. $Q = 1500*10^{-3} / 60 = 0.025\ m^3/s.$

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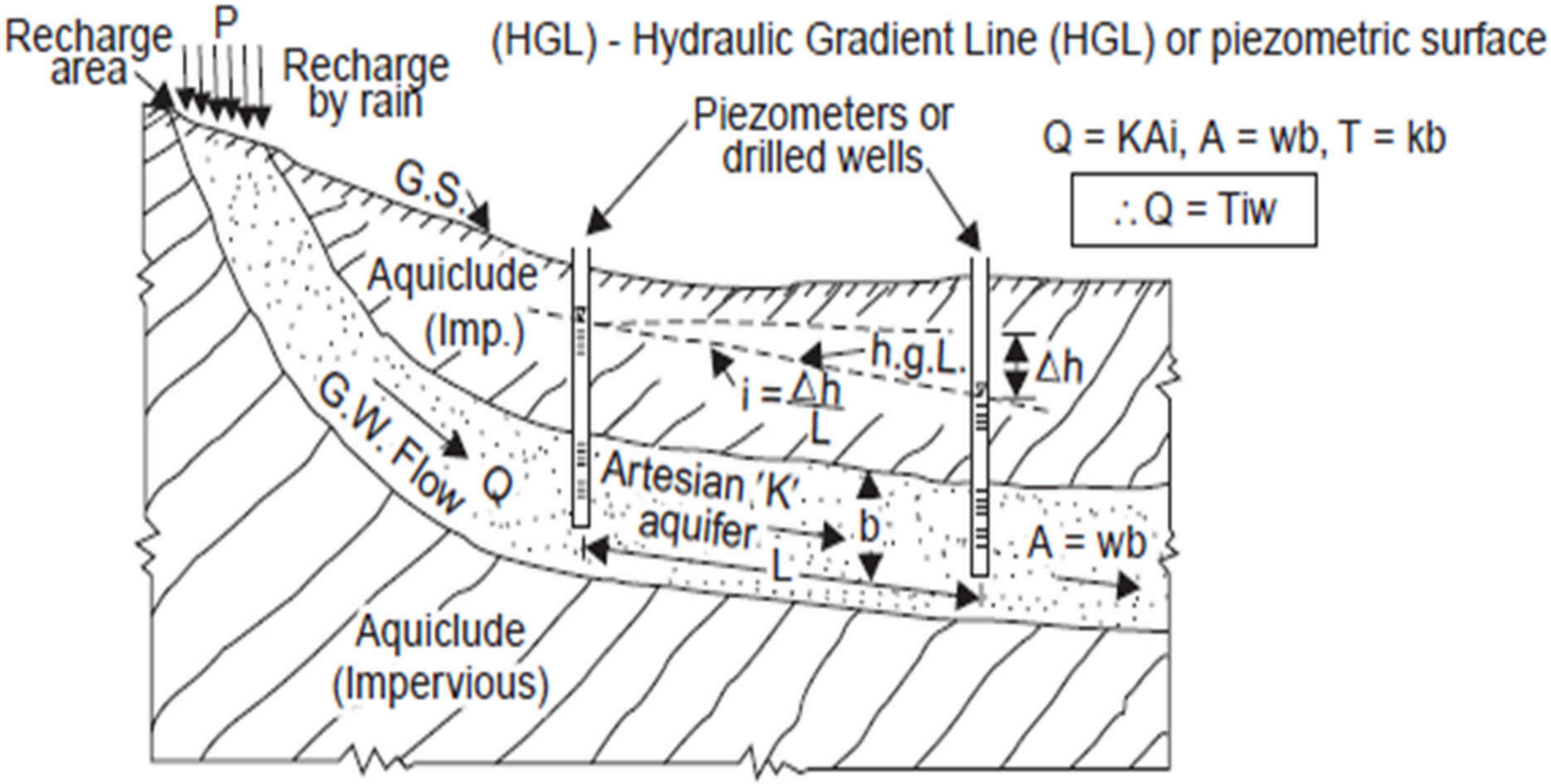
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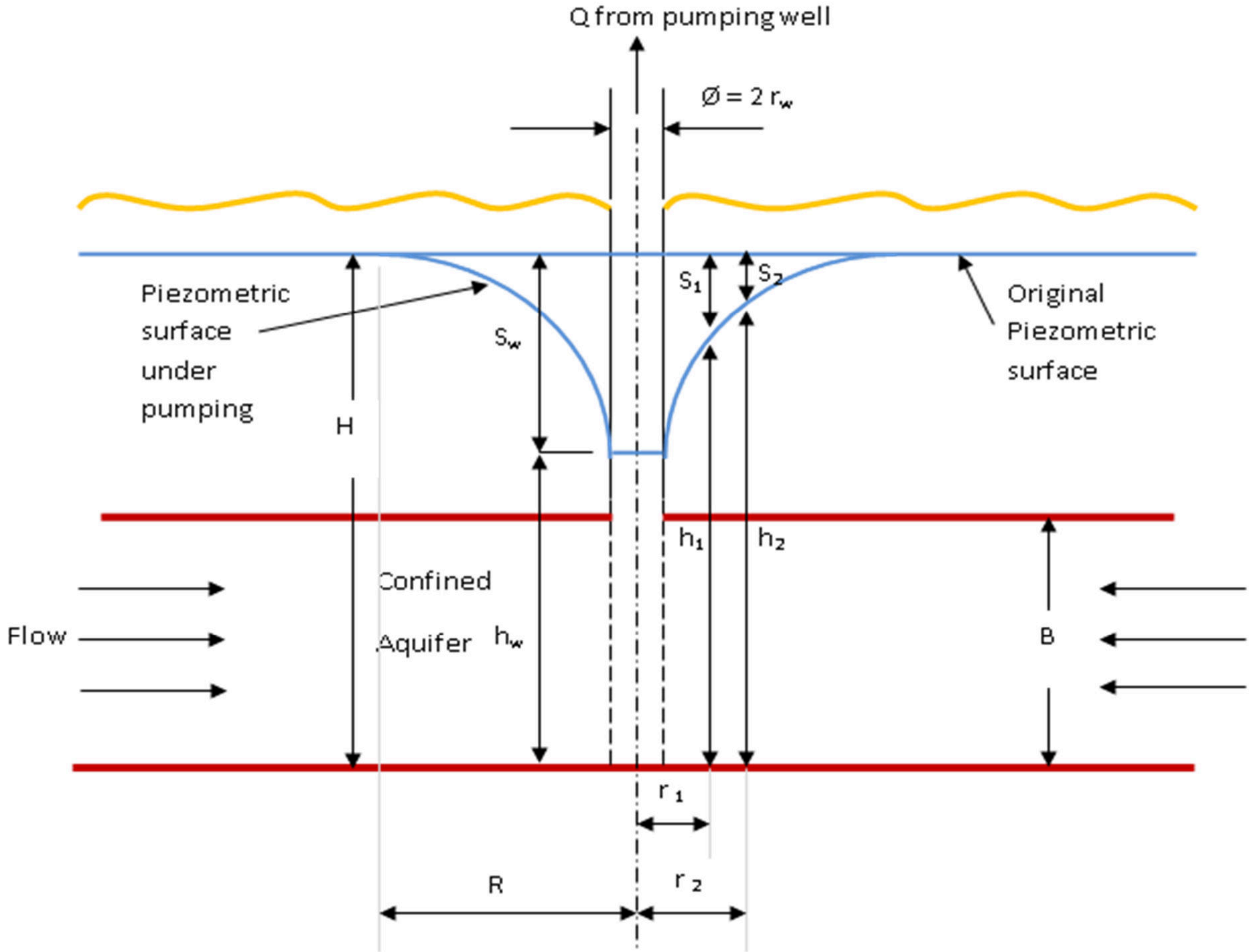

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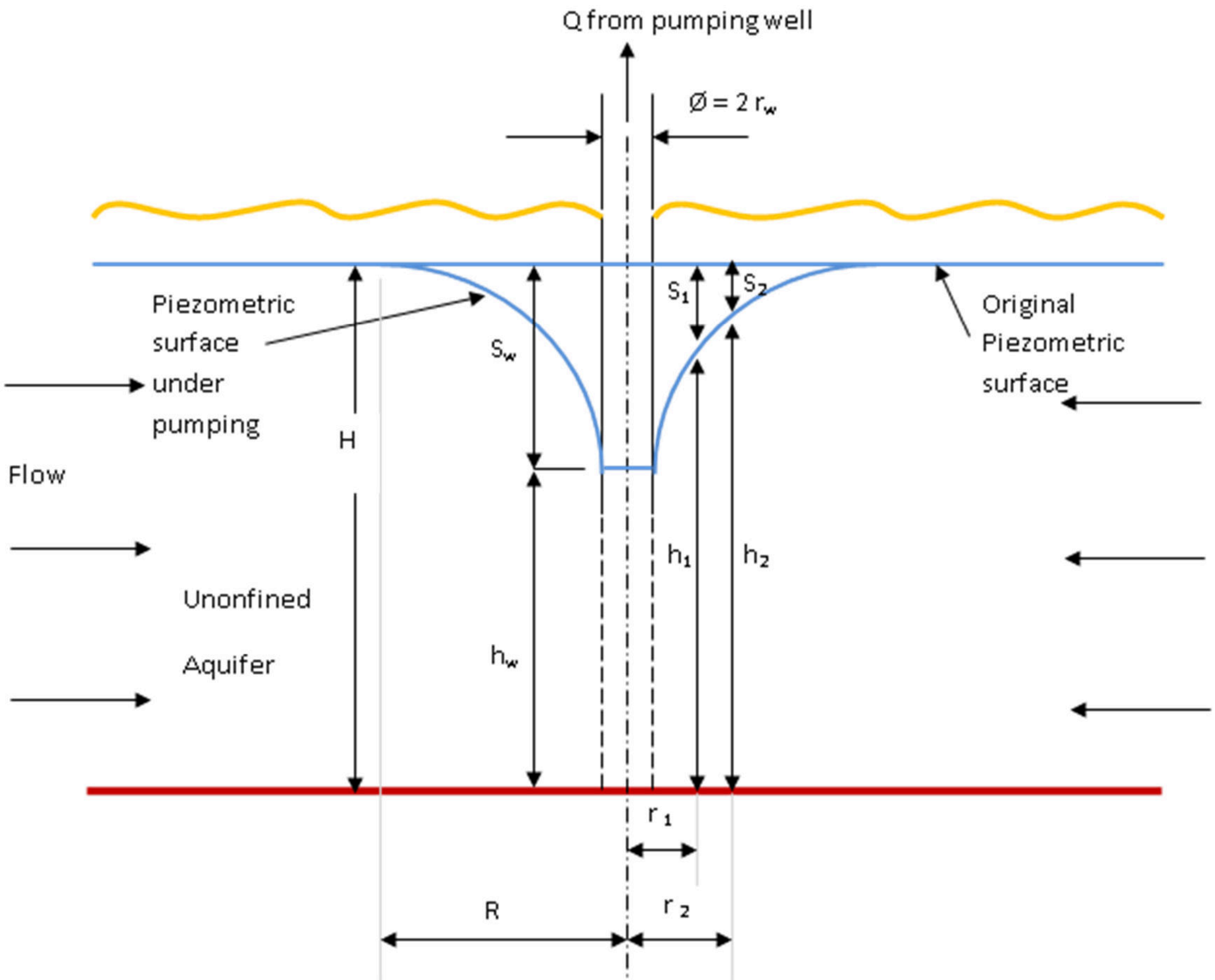

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$h_w = 28.49$ m.

$S_w = 40 – 28.49 = 11.51$ m

- Retarding Basins: احواض التاخير
- A retarding basin is provided with outlets like a large spillway and pipes with no control gates .
- The discharge capacity of a retarding basin when full should equal the safe discharging capacity of the channel downstream.
- The storage capacity of the basin should be equal to the volume of the design flood minus the volume of water released during the flood.
- After the peak of flood has passed, the inflow will gradually become equal to the outflow.

- <u>Construction of Levees :</u>
- The levees are constructed beyond the meander belt of a river.

حزام متعرج

The spacing and height of levees are determined by a series of trials .

A height is assumed and the discharge through the proper channel is computed.

The effects of levees on flood flow are:

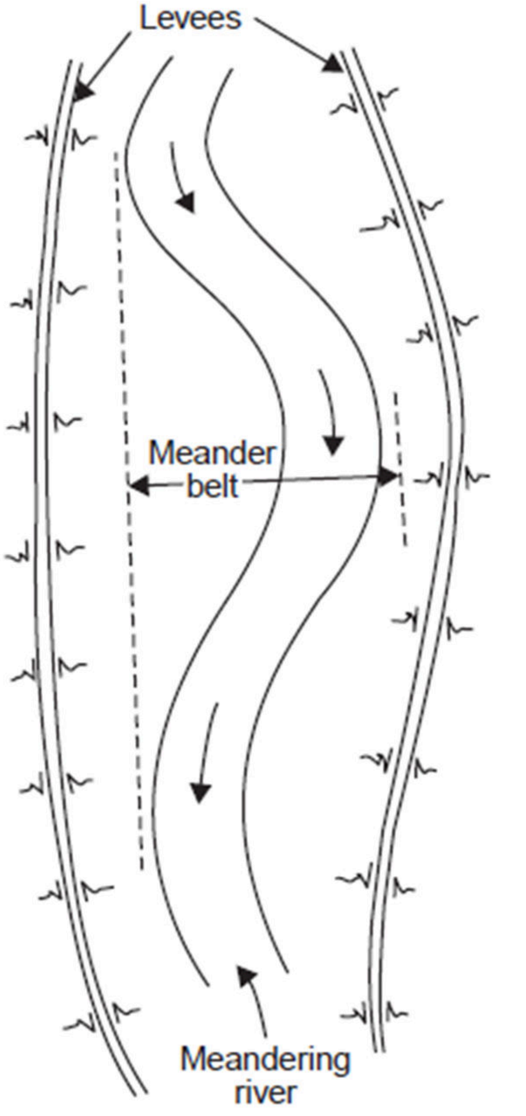
(i) increase in the rate of flood flow

(ii) increase in the flood water elevation

(iii) increase in the carrying capacity of the channel

(iv) increase in the scouring action

(v) decrease of surface slope of stream above the leveed section



(a) Levees along a meandering river

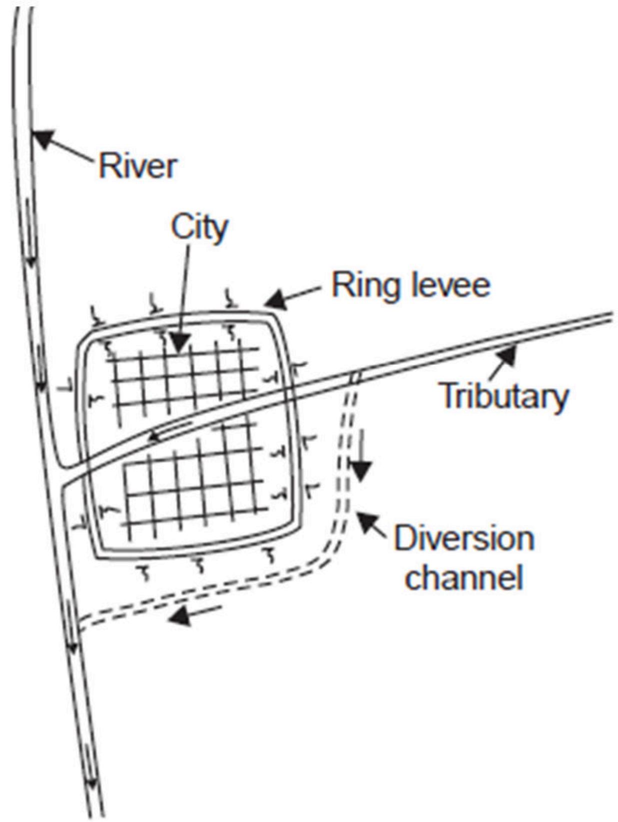
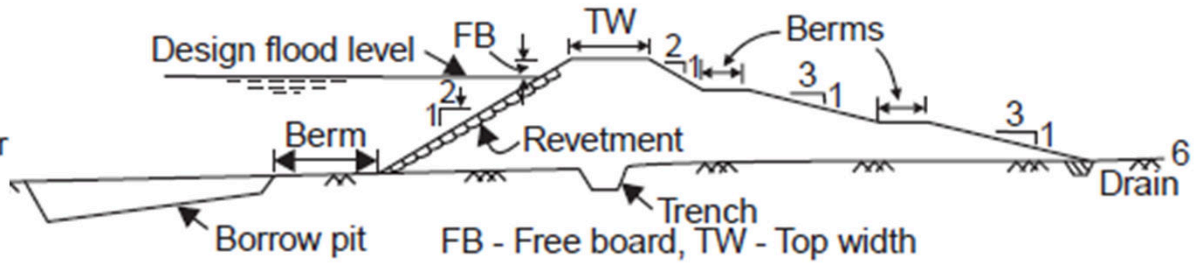


Fig. 8.9 Ring levee to protect a city



(b) Typical levee cross section

- <u>Channel Improvement :</u>
- Channel improvement increases the discharging capacity of the stream thereby decreasing the height and duration of the flood.
- Flood capacity can be increased either by increasing the cross-sectional area or by increasing the velocity along the river.
- The channel velocity (given by Manning's or Chezy's formulae) is affected by hydraulic mean radius, slope of river bed and roughness of the bed and sides.
- Roughness can be reduced by :
- (i) removing sand.
- (ii) removal of trees and other obstacles.
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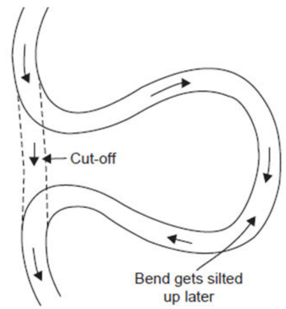
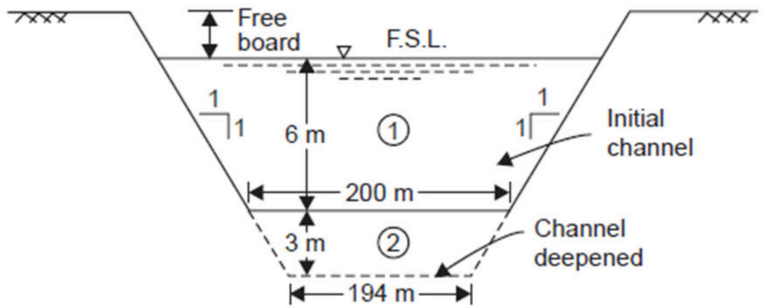


- Chezy’s formula, $V = C RS$
- Where
- V = velocity of flow in the channel
- R = hydraulic mean radius
- S = bed slope of the channel
- C = a constant, depending on the roughness of the bed and sides.
- Example / A channel has a bottom width of 200 m, depth 6 m and side slopes 1:1. If the depth is increased to 9 m by dredging, determine the percentage increase in velocity of flow in the channel. For the same increase in cross sectional area, if the channel is widened (instead of deepening), what is the percentage increase in the velocity of flow.

- Solution
- before deepening :
- the original area of cross section (A), wetted perimeter (P)
- A= ((200+212)/2)*6 = 1236 m^2
- P = 200+2 *(36+36)$^{0.5}$ = 217 m
- R= A/P = 1236/217 =5.7m
- After deepening from 6 m to 9 m.
- A= ((194+212)/2)*9 = 1827m^2
- P = 194+2 *(81+81)$^{0.5}$ = 219.4 m
- R= A/P = 1827/219.4 =8.33m



$$\text{Velocity increase by deepening} = \frac{\sqrt{8.33} - \sqrt{5.70}}{\sqrt{5.7}} \times 100 = \mathbf{21\%}$$

- (ii) For the same increase in the cross sectional area, widening the channel, Let the bottom width after widening be b′
- 1827= ((b′+ 12+ b′) /2)6
- 1827= (b′+ 12+ b′) 3
- 609= (2b′ +12)
- 597= 2b′
- b′ = 298.5 m
- P = 298.5+2 *(36+36)$^{0.5}$ = 315.42 m

$$R = \frac{A}{P} = \frac{1827}{315.42} = 5.8 \text{ m}$$

$$\text{Velocity increase on widening} = \frac{\sqrt{5.8} - \sqrt{5.7}}{\sqrt{5.7}} \times 100 = 0.84\%$$

- Thus, the velocity increase will be only 0.84% on widening as against 21% by deepening

- FLOOD CONTROL ECONOMICS
- The flood control costs include:
- (i) the construction of the structure to the required flood height.
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- <u>FLOOD FREQUENCY STUDIES :</u>
- Flood frequency dependent on statistical methods for analysis of frequency studies .
- 1- <u>Annual Flood Series</u> :
- The recurrence interval (T) is the average number of years during which a flood of given magnitude will be equaled or exceeded once and is computed by one of the following methods.
- California method : $T = \frac{n}{m}$
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- The probability of occurrence of a flood in any year : - $P = \frac{1}{T}$
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- Gumbel's theory :
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where e = base of natural logarithms

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$$y = \frac{1}{0.78\sigma}(x - \bar{x} + 0.45\,\sigma), \qquad \text{for } n > 50$$

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- three coefficients have to be calculated from observed flood data as :

$$(i) \text{ Coefficient of variation, } C_v = \frac{\sigma}{\bar{x}}$$

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$$(iii) \text{ Coefficient of flood, } C_f = \frac{\bar{x}}{A^{0.8}/2.14}$$

where A = area of the catchment in km^2

$$C_s \approx 2\,C_v$$

$$\text{Mean flood} = C_f \times \frac{A^{0.8}}{2.14}$$

- <u>Retarding Basins: </u> احواض التاخير
- A retarding basin is provided with outlets like a large spillway and pipes with no control gates .
- The discharge capacity of a retarding basin when full should equal the safe discharging capacity of the channel downstream.
- The storage capacity of the basin should be equal to the volume of the design flood minus the volume of water released during the flood.
- After the peak of flood has passed, the inflow will gradually become equal to the outflow.

- <u>Construction of Levees :</u>
- The levees are constructed beyond the meander belt of a river.

حزام متعرج

The spacing and height of levees are determined by a series of trials .

A height is assumed and the discharge through the proper channel is computed.

The effects of levees on flood flow are:

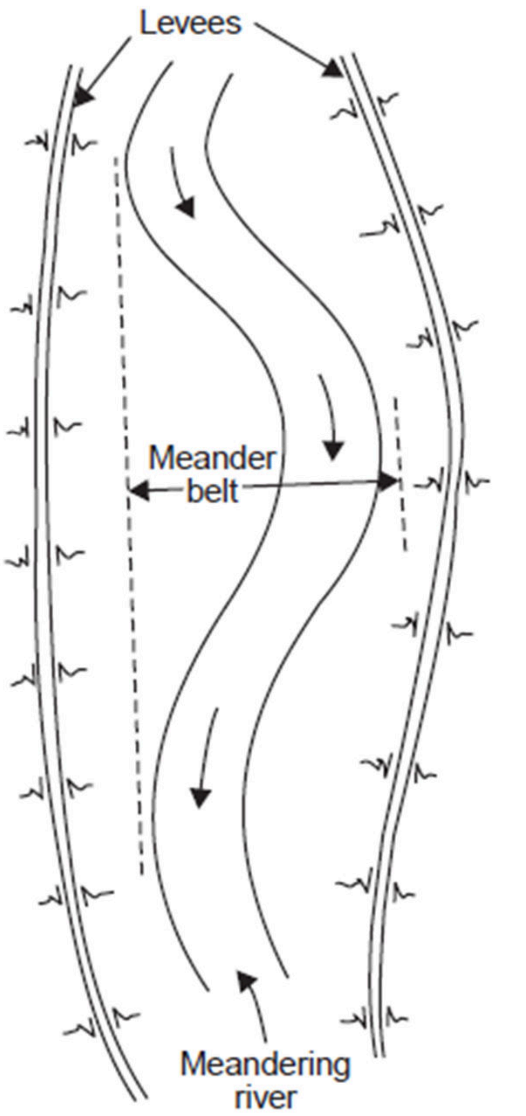
(i) increase in the rate of flood flow

(ii) increase in the flood water elevation

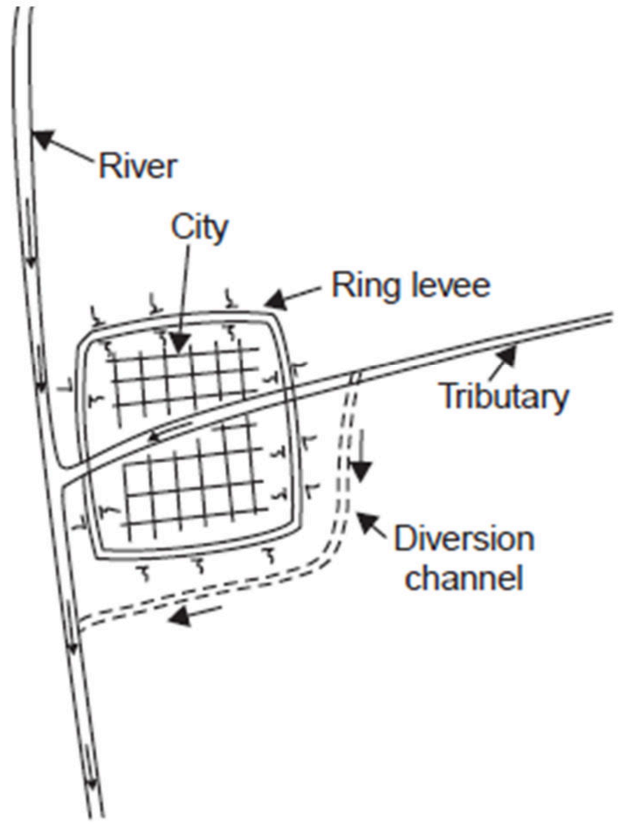
(iii) increase in the carrying capacity of the channel

(iv) increase in the scouring action

(v) decrease of surface slope of stream above the leveed section



(a) Levees along a meandering river



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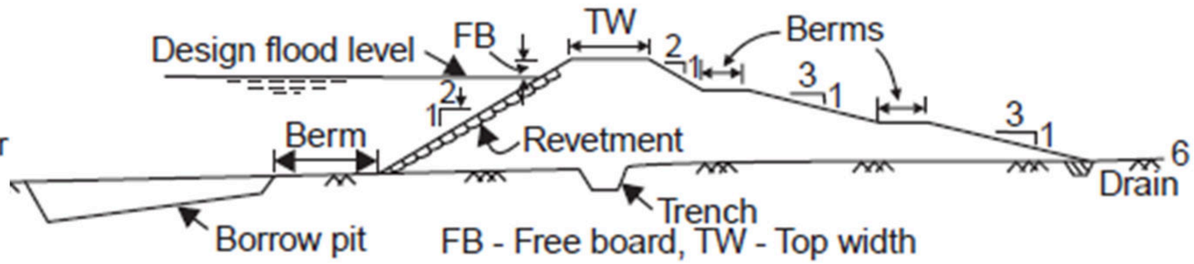

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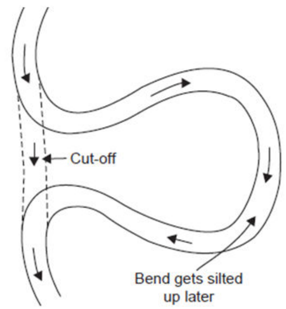

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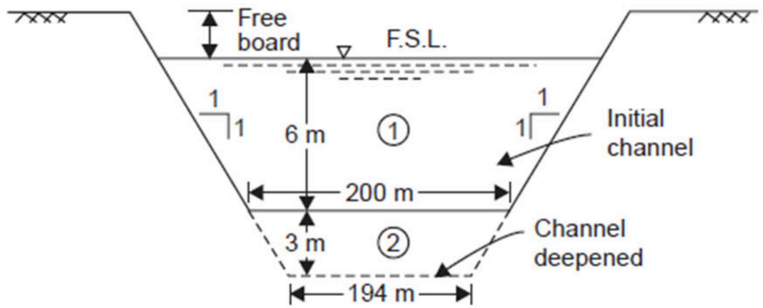

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