



**King Saud University**  
**College of Engineering**  
**Civil Engineering Department**

**CE 499**

**Design of Flood Control Gravity Dam in Wadi Banban ,Riyadh**

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**Abstract**

In this project a gravity dam was designed across wadi Banban north of Riyadh city. The dam was designed for flood control where a total of more than one million cubic meter of water were detained. In order to assure an effective flood control, five outlets were provided. These outlets allow stored water to be discharged within one day keeping the dam empty for the next flood and smoothing the flood hydrograph over a longer duration. Designed dam was checked for stability condition and found safe for the loading combination used. An Ogee spillway was designed to pass a 100 year flood without causing any damage to the structure. Also the spillway was provided with a USBR type II stilling base to dissipate excess energy.

***Chapter (1)***  
***Introduction***

## 1.1 Importance of Flood Control

Water is one of God's most important blessings. It has a fundamental role in the continuation of all forms of life, not to mention its huge roles in the development of societies, which is why it is important to study water and water sources as well as their properties. One of the most important sources of water is through precipitation, in the form of "rainfall". Rainfall intensity has many degrees. If too intense, it may cause floods.

Flooding is the overflow of water that submerges land. Floods, if not handled correctly, can be very deadly. They can cause harm to people, structures, trees, crops and food supplies. Also, floods are a major cause of the spreading of diseases such as cholera and Typhoid.

In the flood that reached Jeddah/Saudi Arabia in 2009, the number of deaths reached more than 120 deaths, and the number of damaged houses was estimated to be more than 6000 housing units and damaged more than 5000 cars. In Jeddah 2010, the number of deaths amounted to 10 deaths, 6 missing persons, more than 25,000 damaged buildings and contaminated more than 60% of the water tanks. But in the Riyadh/Saudi Arabia in 2010, there was less damage than Jeddah, where the number of deaths amounted to 5 and 155 wounded. So, control of flood is an important task that needs to be considered through different practices, one of which is to build a flood control gravity dam.

## 1.2 Study objectives

This project aims to design a flood control gravity dam in Wadi Banban in Riyadh to reduce any possible flood damages.

### **Some of the objectives of this project are:**

- 1-gather hydraulic and topographic data for the site
- 2- do the hydrologic analysis required for the design
- 3-design the gravity dam in the Wadi
- 4-design the dam spillway

### **1.3 Project Outline**

Once the project is complete, it will contain a final report of five chapters done by a team of three students. The first chapter is an introduction to the report. Chapter two contains hydrologic and hydraulic background of the project. Chapter three includes hydrologic study. Chapter four presents design of the gravity dam and spillway, and finally, chapter 5 includes conclusion and recommendations.

***Chapter (2)***  
***Hydrologic and Hydraulic***  
***Background***

## 2.1 Hydrologic Background

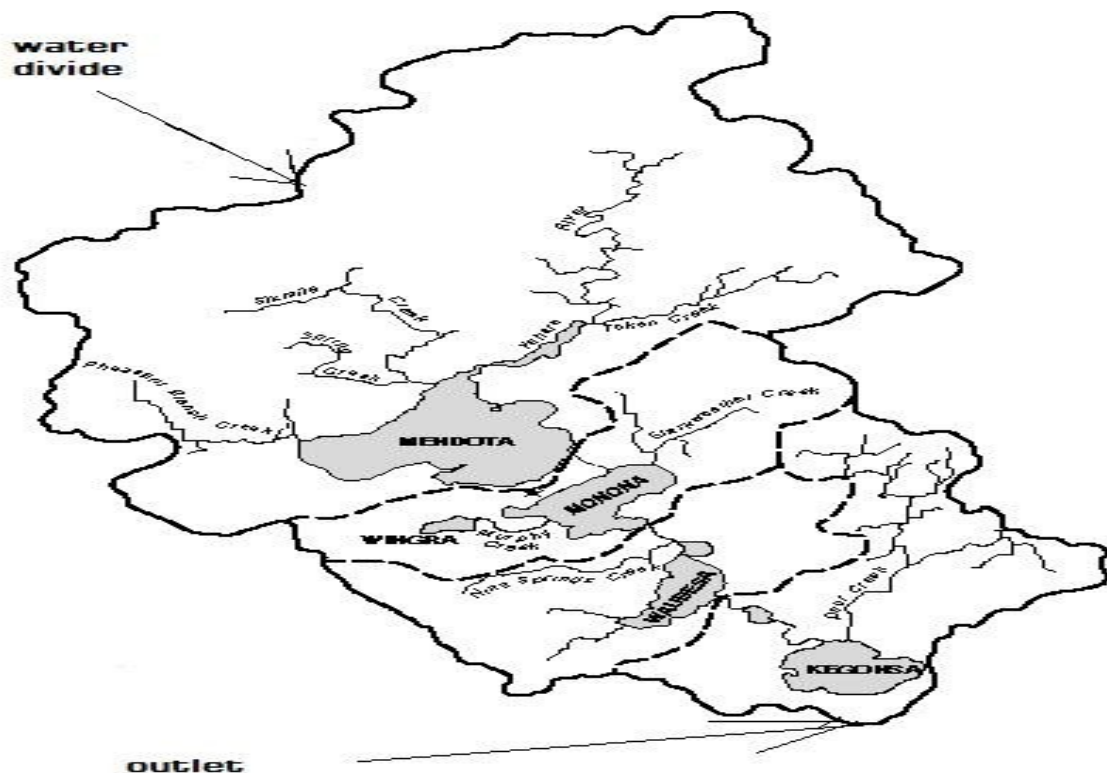
### 2.1.1 Catchment area

- **Introduction**

A catchment area is a hydrological unit each drop of precipitation that falls into a catchment area eventually ends up in the same river or stream if it doesn't evaporate. However, it can take a very long time. Catchment areas are separated from each other by water divide.

Catchments are divided into sub catchments. Shape of catchment area influences the rainfall-runoff relation for the area.

Catchment boundaries are located by using the contour lines on a topographic map. Catchment area known as watershed drainage area is an important subject in hydrological analysis required prior to any hydraulics structure design. Figure 2.1 shows a typical view of a catchment.



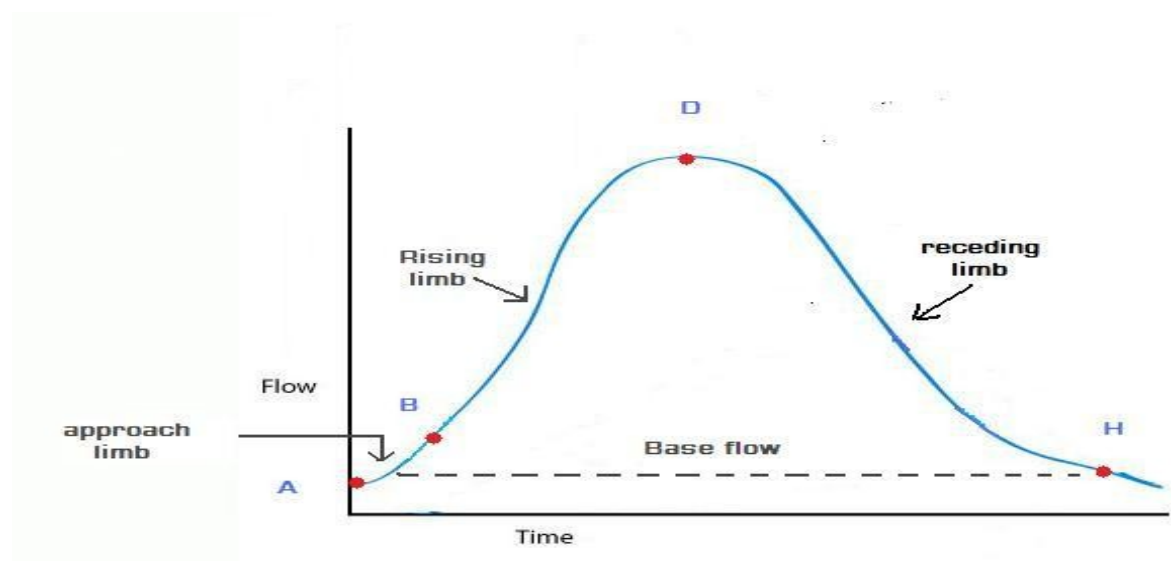
**Figure.2.1** Typical view of a catchment (Nancy D. Gordon, *Stream hydrology*, 2004)

## 2.1.2 Hydrograph

The objective of many hydrological designs and analysis problems is to determine the surface runoff from a watershed due to a practical storm. This process is commonly referred to as rainfall-runoff analysis.

Hydrograph is a graph of the flow (discharge) in a stream over a period of time. Hydrograph can also refer to a graph showing the volume of water reaching a particular outfall. Hydrographs are commonly used in the design of sewerage, more specifically, the design of surface water sewerage systems and combined sewers.

A typical storm runoff hydrograph is shown in Fig.2.2. It consists of three distinct components, the approach limb AB, the rising limb BD, and the receding limb DH. Main factors affecting hydrograph are given in Table 2.1



**Figure 2.2** Typical Runoff Hydrograph

Physical factors	Climatic factors
1. Basin characteristics: <ol style="list-style-type: none"> <li>Shape.</li> <li>Size.</li> <li>Slope.</li> </ol>	1. Storm characteristics : precipitation Intensity duration, magnitude and movement of storm.
2. Infiltration characteristics: <ol style="list-style-type: none"> <li>Land use cover.</li> <li>Soil type and geological conditions</li> </ol>	2. Initial loss.
3. Channel characteristics: cross section roughness and storage capacity.	3. Evapotranspiration.

**Table 2.1** Factors Affecting Flood Hydrograph (Task committee on Hydrology Handbook, *Hydrology Handbook*, 1996)

- **Unit hydrograph**

The Unit Hydrograph (UH) of a catchment is defined as the direct runoff hydrograph resulting from a unit volume of excess rainfall of constant intensity and uniformly distributed over the drainage area.

### 2.1.3 Synthetic Unit Hydrograph (SUH)

This method is used for deriving UHs for ungaged basins. It is based on theoretical or empirical formulas relating hydrograph peak flow and timing to basin characteristics. One of the popular SUH methods is Snyder's method. Snyder method was developed using the analysis of a large number of hydrographs from different catchment regions.

The basic procedure of Snyder's method for estimating runoff can be summarized as follows:

- Time of peak in hours " $t_p$ "

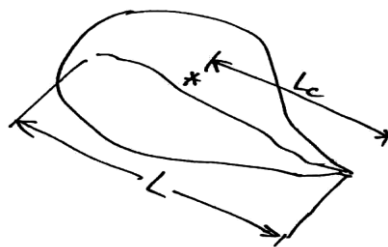
$$t_p = c_t(LL_c)^{0.3}$$

Where:

$L$  = distance from station to catchment boundary measured along the main stream channel, in miles.

$L_c$  = distance from gauging station to centroid of catchment area measured along the main stream channel to the nearest point, in miles

$c_t$  = a coefficient depending on units and drainage basin characteristics (1.8 – 2.2)



- Unit hydrograph duration  $t_r$ , hr

$$t_r = \frac{t_p}{5.5}$$

- Peak discharge " $Q_p$ " in cfs

$$Q_p = \frac{c_p A}{t_p} \times 640$$

Where:

A= catchment area in (mi<sup>2</sup>)

$c_p$ = coefficient depending on units and drainage basin characteristics (0.4 – 0.8)

- Time base in days

$$T = 3 + \frac{t_p}{8}$$

- for small catchment

$$T = 4t_p$$

- For any duration,  $\overline{t_p}$

$$\overline{t_p} = t_p + \frac{t - t_r}{4}$$

- To construct Snyder's unit hydrograph, W<sub>50</sub>, W<sub>75</sub> are also considered as recommended by US corps of engineers. (Vijay P. Singh, Elementary Hydrology , 1992)

W<sub>50</sub>=width of unit hydrograph at 50 % of peak discharge (in hours).

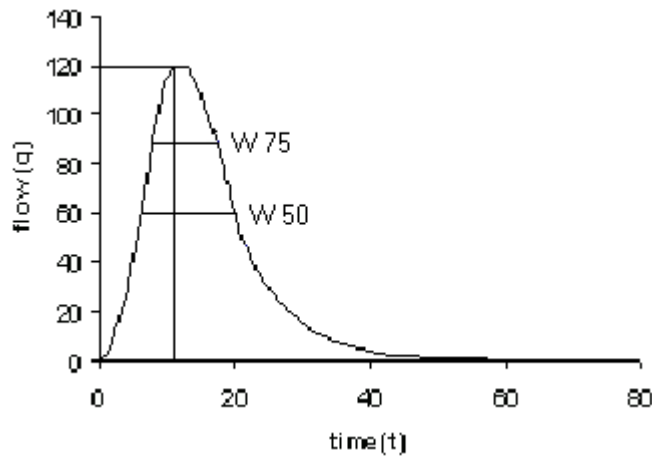
$$W_{50} = \frac{830}{q_p^{1.1}}$$

W<sub>75</sub>=width of unit hydrograph at 75 %of peak discharge (in hours).

$$W_{75} = \frac{470}{q_p^{1.1}}$$

Where  $q_p = \frac{Q_p}{A}$

Fig.2.3 shows a typical UH



**Figure 2.3** Snyder's Unit Hydrograph (J. A. Ramírez, *Water Resources, Hydrologic and Environmental Sciences*, 2005)

### 2.1.4 Time of Concentration

Time of concentration is the longest travel time it takes for a particle of water to reach an outlet point in a watershed

Kirpich suggested the following equation

$$t_c = 0.0078L^{0.77}S^{-0.385} \text{ (Ven Te Chow, David Maidment, Larry Mays, } \textit{Applied Hydrology}, 1988)$$

Where:

$t_c$  = time of concentration, minutes

L = length from head water to outlet, ft

S = average watershed slope, ft/ft.

This time of concentration is the time needed to be considered in rainfall-runoff calculation.

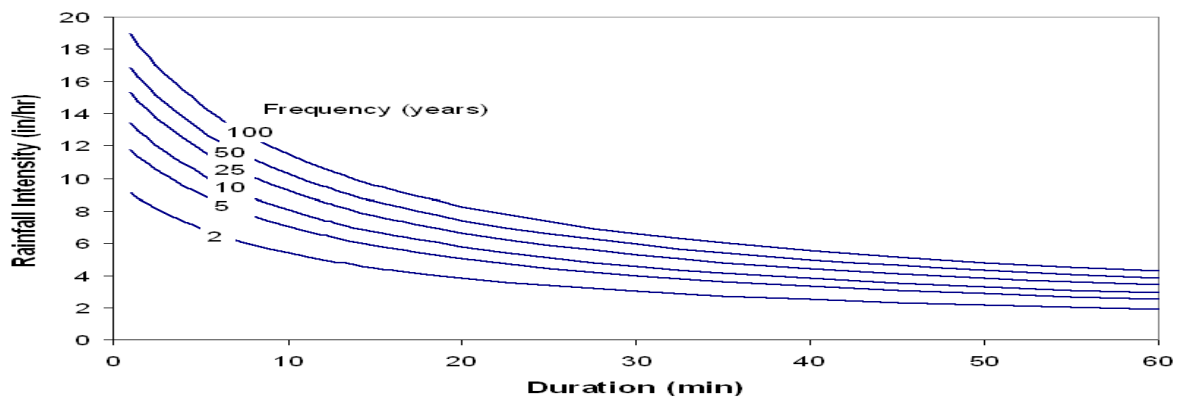
### 2.1.5 Depth-Duration-Frequency (DDF) or Intensity- Duration-Frequency (IDF curve)

Relationships which utilize recorded events in order to predict future exceedance probabilities and thus quantify risk and maximize design efficiencies are key concepts in the design of hydraulic structures. Probability distributions are used to find these relations.

The characteristics that must be identified in either assessing an actual storm or developing a design storm are:

1. Duration : the length of time over which a precipitation event occurs.
2. Depth : the amount of precipitation that occurs over the storm duration.
3. Frequency : the frequency of occurrence of events that have the same depth and duration.
4. Intensity : intensity of rainfall that is depth over duration.

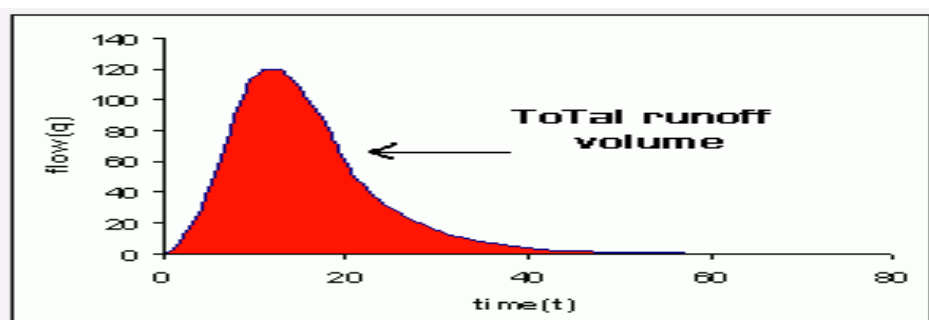
These relations can be presented either by graph or tables. Figure 2.4 presents a typical IDF curve.



**Figure 2.4** Typical Intensity, duration, frequency curves (IDF) (Allen P. Davis, Richard H. McCuen, *Storm water management for smart growth*, 2005)

### 2.1.6 Storage Volume

Water volume that needs to be stored behind the dam can be evaluated at given frequency (return period) using the  $t_c$ -SUH and the IDF relations. This volume is then used to find dam height from depth-volume curve given for the area. Fig 2.5 shows total runoff volume derived from hydrograph. Runoff coefficient is then used to estimate runoff from total rainfall volume. Fig 2.6 shows typical volume height relation.



**Figure 2.5** Total runoff volume

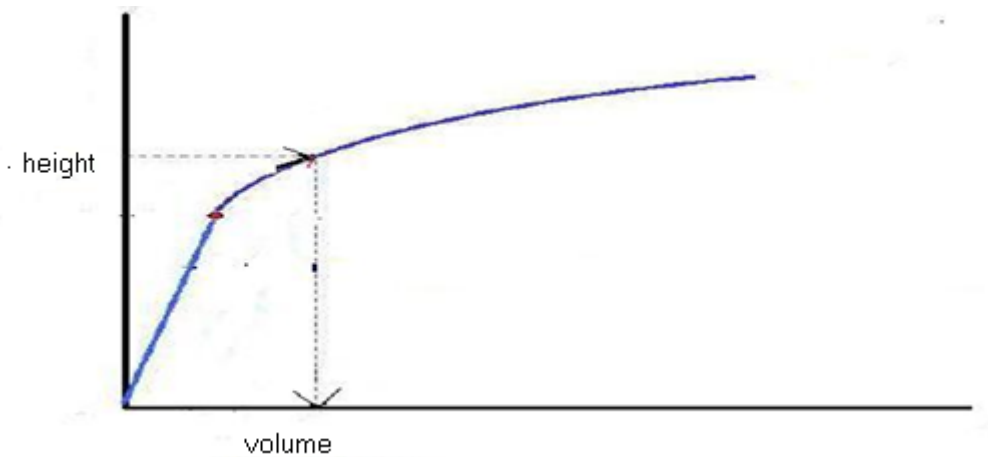


Figure 2.6 Volume and height relations

## 2.2 Hydraulic Background

### 2.2.1 Dams

A dam is a barrier constructed across a waterway to hold back or control the flow of water built for the following purposes

- 1-flood control
- 2-irrigation
- 3-water supply
- 4-power generation (electric power)

- **Gravity dam:** the type of dam that depends on its weight for stability.

- **Characteristics**

- 1-weight of dam resists opposing forces
- 2-made of concrete or masonry
- 3- Usually has a triangular cross-section
- 4-strong rock foundation

- **Advantages of gravity dam**

- 1-high durability
- 2-simple design

- **Disadvantages of gravity dams**

- 1-expensive
- 2-require large amount of materials and construction  
(About Dams. The British Dam Society, 2008)

## 2.2.2 Forces acting on a dam

There are three types of forces acting on a gravity dam:-

- 1-Primary forces: are those of major importance, they are present in every dam.
- 2-Secondary forces: importance limited to certain types of dams depending on several factors such as location of the dam and nature of the area.
- 3-Exceptional forces: has a low probability of occurrence.

### 1-Primary loads

- **Self-weight loads**

The most important element in the stability of concrete dams. It is accounted for in the terms of resultant  $P_m$ , which is concentrated at the centroid of the cross-sectional area of the dam " $A_p$ "

$$P_m = \gamma_c \times \text{vol. (kN/unit length)} \quad (2.1)$$

$\gamma_c$ : unit weight of concrete=23.5kN/m<sup>3</sup>

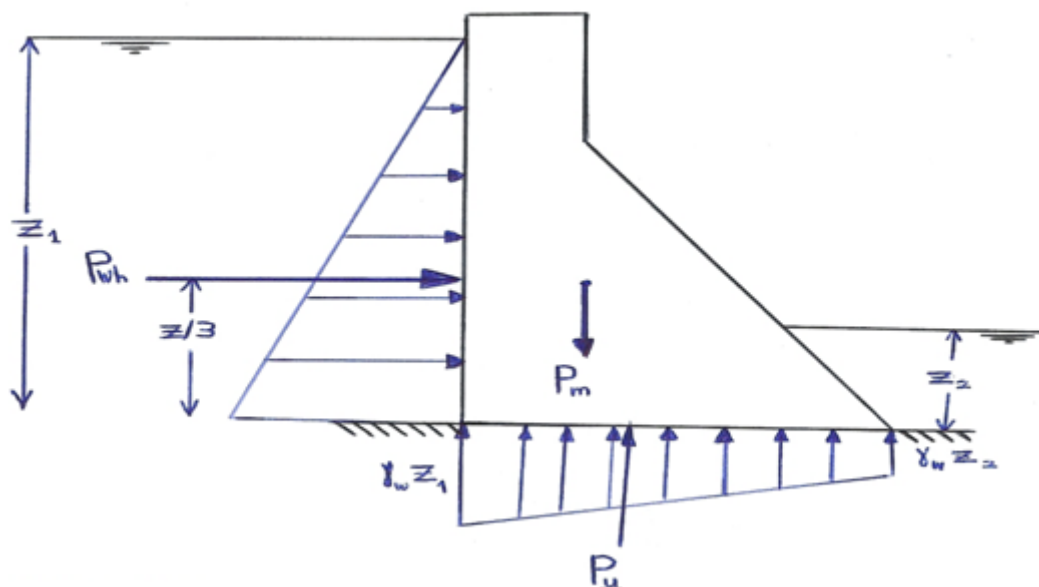
vol.: volume of concrete for one unit length

- **Water loads**

The external hydrostatic horizontal pressure " $P_{wh}$ " acts at height  $z/3$  and is determined as

$$P_{wh} = \gamma_w z^2 / 2 \quad (\text{kN/unit length}) \quad (2.2)$$

**z: height of the dam**



**Figure2.7** showing where forces act on a dam

- **Seepage and uplift loads**

Developed in concrete dams and its foundation as a result of water penetration along discontinuities (joint planes, cracks, etc) and also by seepage within the pore structures of rock and concrete. It is a negative force directed upwards and it can be calculated as follows

$$P_u = \gamma_w(z_1+z_2)/2 \times (B \times \text{unit width}) \quad (2.3)$$

$z_1, z_2$ : water depth at upstream and the downstream of the dam respectively  
 $B$ : the width of the base of the dam

**With pressure relief drains**

In modern dams internal uplift is controlled by the provision of vertical relief drains close behind the upstream face (i.e.  $0.1 Z_1$ ). Formed drains rise the full height of dam from an inspection gallery located as low as practicable in relation to the tail water level. The effective head at the line of the drain,  $Z_d$ , is

$$Z_d = Z_2 + K_D(Z_1 - Z_2) \text{ unit length} \quad (2.4)$$

$K_D$  is an empirical coefficient and a function of relief drain geometry assumed to be 0.33

Modern drains are typically of 200 mm diameter at 3 m distance to centers

## 2- Secondary loads

- **Wave loads**

The transient hydrodynamic thrust generated by wave action against the face of the dam,  $P_{\text{wave}}$ , is considered only in exceptional cases.

$$P_{\text{wave}} = 2 \gamma_w H_s$$

$H_s$ : significant wave height

$P_{\text{wave}}$  acts at  $3/8 H_s$

$$H_s = 0.032 \sqrt{(V \cdot F)} + 0.763 - 0.271 \sqrt[4]{F} \quad \text{for } F < 32 \text{ km}$$

$$H_s = 0.032 \sqrt{(V \cdot F)} \quad \text{for } F < 32 \text{ km}$$

$V$ : wind velocity (km/hr)

$F$ : fetch length (km)

- **Sediment load**

Rivers and valleys carry large amounts of sediments, mainly silts, the accumulate against the face of the dam and generates a resultant horizontal force  $P_s$  that can be calculated as follows

$$P_s = [(1 - \sin\Phi) / (+\sin\Phi)] (\gamma'_s / 2) Z_s^2 \text{ (kN /unit length )} \quad (2.6)$$

$\gamma_s$ : sediment saturation unit weight= 18-20 kN/m<sup>2</sup>

$\Phi$ : angle of shearing resistance of sediments= 30°

$Z_s$ : accumulated depth from the base of the wadi

- **Ice load**

In very high and very cold places, ice is formed on the surface of the lake. Pressure exerted on the dam is a complex function of ice thickness, scale and rate of temperature rise resulting in expansion

For ice thickness > 0.6m  $P_{ice} = 145 \text{ kN/m}^2$

For ice thickness < 0.4m ice load may be neglected

- **Wind load**

Effects the area exposed to the wind. In gravity dam, it may be neglected, and in very high dams, it may be taken as 1-1.5 kN/m<sup>2</sup>

- **Thermal and dam foundation interaction effects.**

### 3- Exceptional loads

- **Seismic (earthquake)**

The intensity of a shock is expressed by acceleration coefficients representing the ratio of peak seismic ground acceleration to gravitational acceleration.  $\alpha_h$  horizontal depends on the damage level and  $\alpha_v$  vertical.  $\alpha_h = (1.5-2.0) \alpha_v$  Both the mass of dam and water are affected by seismic loads

The horizontal force

$$P_{emh} = \pm \alpha_h p_m$$

The vertical force

$$P_{emv} = \pm \alpha_v p_m$$

$P_{emh}$  and  $P_{emv}$  act at the centroid of the dam section

- **Water reaction (hydrodynamic forces)**

$$P_{ewh} = 0.66 C_e \alpha_h Z_1 \gamma_w (Z_1 Z_{max})^{1/2}$$

Acts at elevation  $0.4Z_1$

$C_e$ : dimensionless pressure factor

## 2.2.3 Gravity dam stability analysis

### 1-Overturning or Rotational stability

A simplistic factor of safety with respect to overturning,  $FO$ , can be expressed in terms of the moments operating about the downstream toe of any horizontal plane.

$$FO = \Sigma M_{+ve} / \Sigma M_{-ve}$$

$\Sigma M_{+ve}$ : summation of all resisting moments

$\Sigma M_{-ve}$ : summation of all overturning moments

$FO > 1.25$  regarded as acceptable

$FO \geq 1.5$  desirable

### 2-Sliding stability:

Sliding stability is a function of loading pattern and of the resistance to translational displacement which can be mobilized on any plane. It is conventionally expressed in terms of a factor of safety or stability factor against sliding,  $SF$ , estimated as follows:

(1). sliding factor,  $SF$

$$SF = \Sigma H / \Sigma V$$

$\Sigma H$ : summation of horizontal loads

$\Sigma V$ : summation of vertical loads

Where  $SF$  is less than the friction coefficient (0.65-0.75) for dams that are purely friction resistant

For more efficient design, we must consider shear forces with friction by calculating what is known as shear friction factor,  $SFF$

$$SFF = (\mu \Sigma V) / \Sigma H$$

### 3-Stresses at the upstream and downstream ends

Normal stresses in case of a full dam

$$\sigma_{zu} = \sum V/B(1-6e/B)$$

$$\sigma_{zd} = \sum V/B(1+6e/B)$$

$\sigma_{zu}$ : stress at the upstream of the dam

$\sigma_{zd}$ : stress at the downstream of the dam

e: eccentricity of the resultant load

B: width of the dam

In the case of full dam, we change the signs in the two previous equations

## 2.2.4 Spillway

### Introduction

Spillways are some important auxiliary works of dams and reservoirs, the main function of which is to dispose of surplus flood water which cannot be stored safely in the reservoir.

Therefore, spillway is called very often the safety valve or surplusing work of dams and reservoir. Spillway is essential in every dam-storage project. If the flood water overtops the earthen dams, eventually the dam fails. Once the dam fails, catastrophic flood of extreme magnitude occurs causing damage to property, dwelling houses, and roads, lines of communication, crop fields, and even human and wild life. To save such catastrophe, spillway is the only device in each and every dam-reservoir project.

Depending upon the type of dam, material of construction, topography at dam and reservoir site different types of spillways are to be selected.

### Types of Spillway

Nearly all spillways fall into one of the following or are made up of a combination of them.

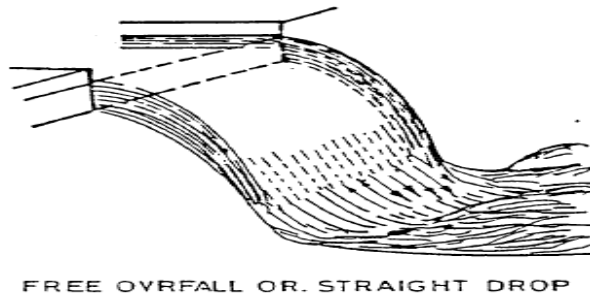
Overflow spillway type is far the most common and is adapted to masonry dams which usually have sufficient crest length to provide the required capacity and where foundation can be protected from the scour of overflowing water.

Trough and chute spillways are commonly used in narrow canyons. Siphon spillway is provided to maintain automatically a constant required water level. Thus different types are suitable in different types of dams under different condition.

### 1. Straight drop spillway

This is simply type of spillway which is constructed in the form of low height weir having downstream face either vertical or nearly vertical.

Water drops freely from the crest and forms a pulsating fluctuating jet. Occasionally, the crest is extended in the form of an overhanging lip (similar to that provided in notch falls) to direct the small discharge away from the face of the overfall. However, after some time the falling jet will form a deep plunge pool.



**Figure 2.8** straight Drop spillway

### 2. Ogee spillway

This is the most common type of spillway provided on gravity dams. The profile of the spillway is ogee or 's' shaped. The overflowing water is guided smoothly over the crest and profile of the spillway so that the overflow water does not break contact with the spillway surface.

The spillway can also be used in arch and buttress dam. Such spillway can easily be used on valleys where the width of the river is sufficiently more to provide sufficient crest length, and the river bed can be protected from scouring at moderate cost.

### 3. chute spillway

A chute spillway is the one which passes the surplus discharge through a steep sloped open channel, called chute or trough, placed either along a dam abutment or through a saddle. Generally, this type of spillway is provided on earth or rock fill dam and is isolated from the main dam. Its crest is kept normal to its center line. It consists of a discharge channel to the river in an excavated trench which is usually paved with concrete in whole or in part.

### 4. side channel spillway

A side channel spillway is one in which the flow, after passing over a weir or ogee crest, is carried away by a channel running essentially parallel to the crest. Discharge

characteristics of a side channel spillway are similar to those of an ordinary overflow spillway and are dependent on the selected profile of the weir crest.

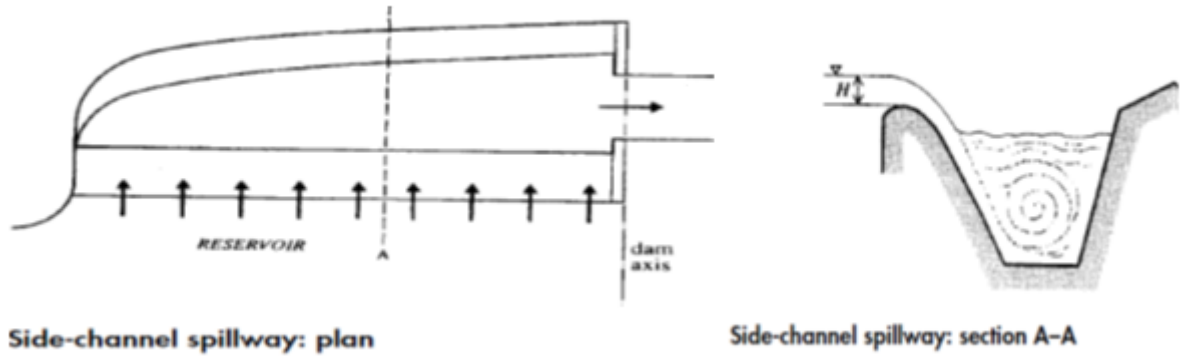


Figure 2.9 side channel spillway (section and plan)

### 5. siphon spillway

Siphon spillway is the one which utilizes the siphonic action to discharge the surplus water. generally, asiphon spillway consists of a closed conduit system formed in the shape of an inverted U,

positioned so that the inside of the bend of the upper passageway is at normal resevoir level.

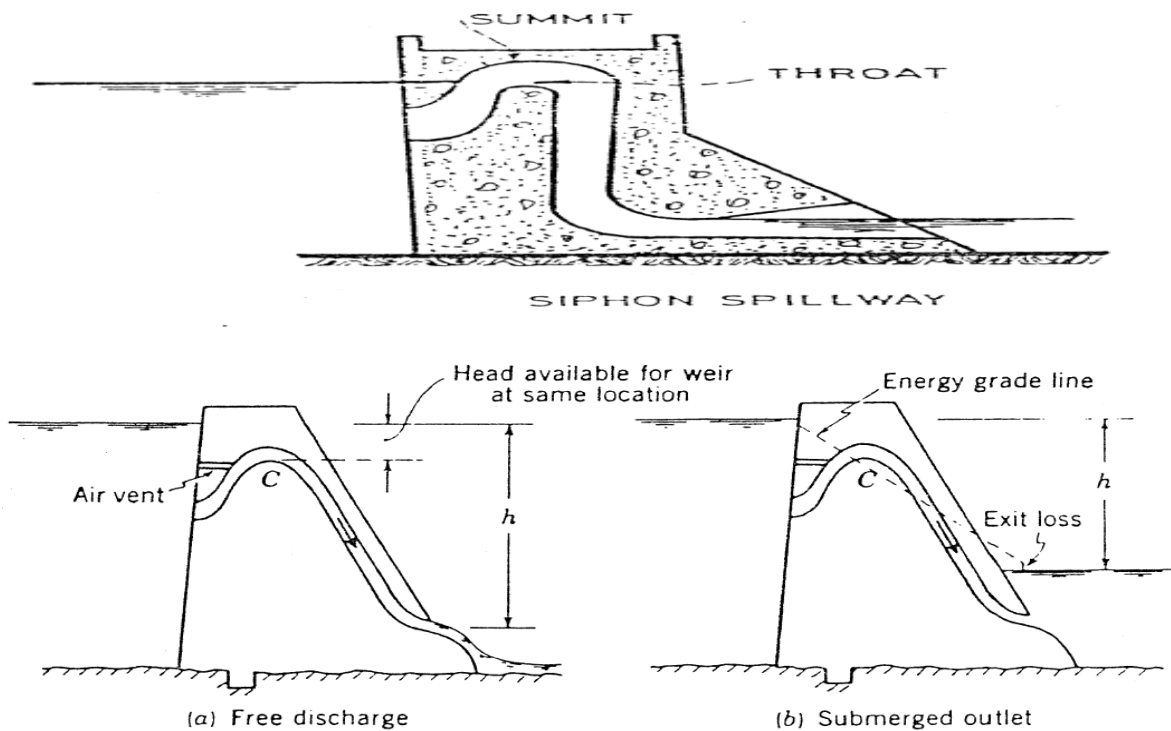


Figure 2.10 cross section of siphon spillway

## 6. Shaft spillway

Drop inlet spillway, shaft spillway or morning glory spillway is the one which has horizontal positioned lip through which water enters and then drops through a vertical or sloping shaft and then to a horizontal conduit which convey the water past the dam.

## Spillway Profile

### (a) Downstream profile

The US Army Corps of Engineers developed several standard spillway shapes at Waterways Experimental Station (WES), these profiles called WES.

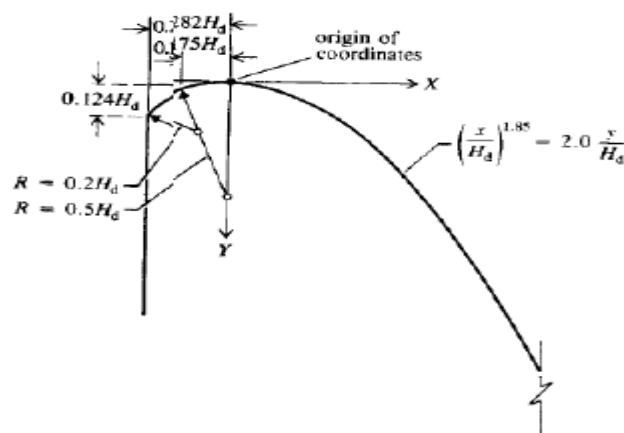


Fig 2.11 Standard spillway crest

The standard ogee spillway crest shape can be represented by

$$Xn = K H_d^{n-1} Y$$

Where:

X and Y : are coordinates of the crest profile with the origin at the highest point of the crest.

$H_d$  : is the design head excluding the velocity head.

K and n : are parameters depending on the slope of the u/s face.

U/S face slope	K	n
Vertical	2	1.85
3V:1H	1.936	1.836
3V:2H	1.939	1.810
3V:3H	1.873	1.776

## b. Upstream profile

US Army recommended the following equation (origin at the crest):

$$Y = 0.724 \frac{(X+0.27H_d)^{1.85}}{H_d^{0.85}} + 0.126 H_d - 0.4315 H_d^{0.375} (X + 0.27 H_d)^{0.625}$$

The curve extended to  $x= 0.27 H_d$  u/s and  $y= 0.126 H_d$  downward from the crest point.

## 2.2.5 Energy Dissipation

Water flowing over spillway is associated with high velocity at the toe of the structure. Such high velocity causes severe erosion to the stream bed and banks below the dam. Hence, the dam and its appurtenant works must be designed so that harmful erosion is minimized.

This can be achieved by the so called energy dissipaters and the bed protection downstream. Several methods are available for the dissipation of excess energy. These include;-

- (i) Hydraulic jump stilling basins
- (ii) Bucket type energy dissipaters
- (iii) Jet stilling basins.

### 2.2.5.1 Hydraulic Jump Stilling Basins

A stilling basin is the most common form of energy dissipater converting the supercritical flow from the spillway into a subcritical flow compatible with the downstream flow regime. The best method of achieving this transition is through a simple submerged jump formed in a rectangular cross section stilling basin.

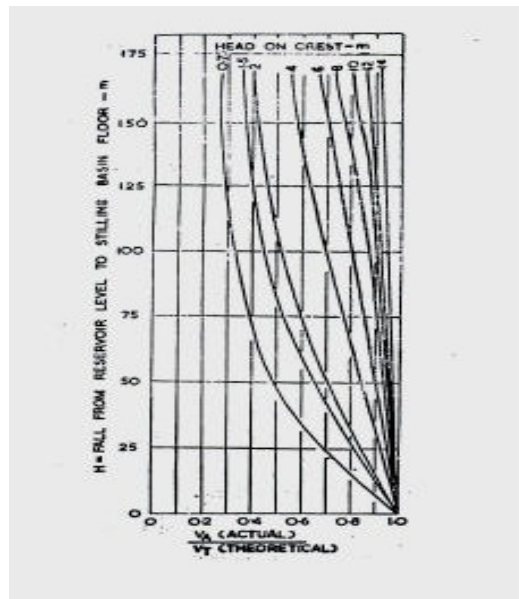


Fig.2.12 Relation of actual and theoretical velocity

**One type of hydraulic jumps is USBR**

**USBR basins** The United States Bureau of Reclamation (USBR) basins were developed based on model studies and evaluation of existing basins.

**a) Basin I and IV** Basin I is designed to control the oscillating jump which is formed in the stilling basin when  $F_1$  lies between 2.5 to 4.5 inclusive.

**b) USBR II and III** For  $F_1 > 4.5$  and approach velocity less than 20 m/sec use basin II.

And if  $F_1 > 4.5$  and approach velocity greater than 20 m/sec use basin III.

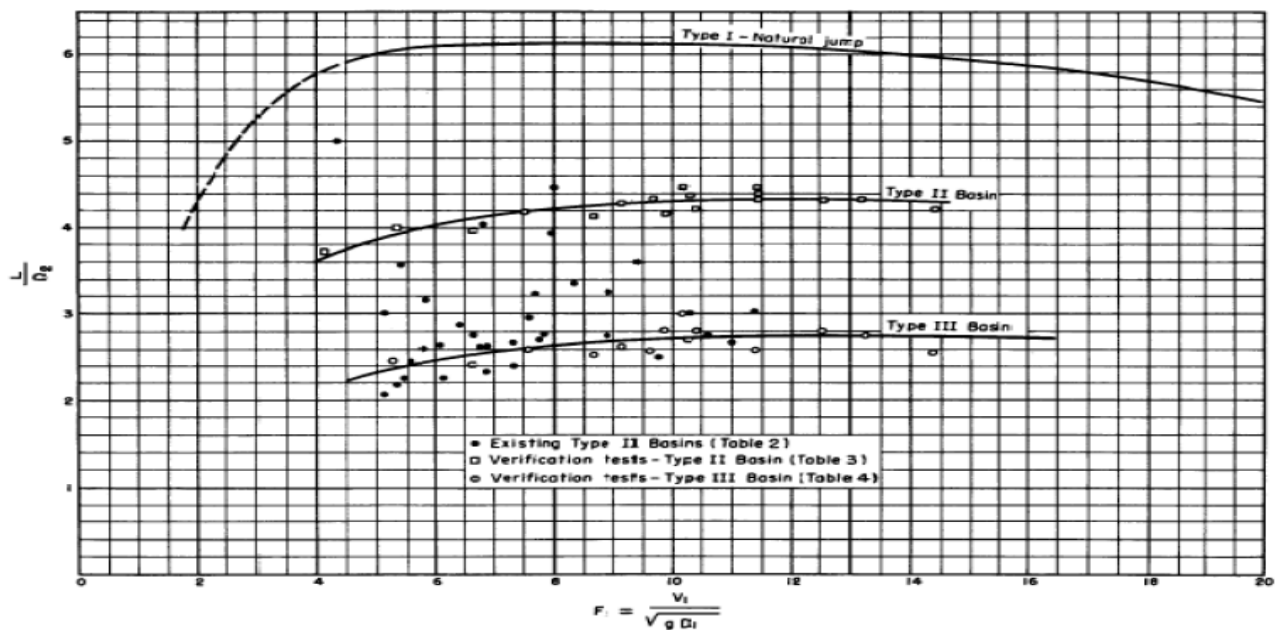


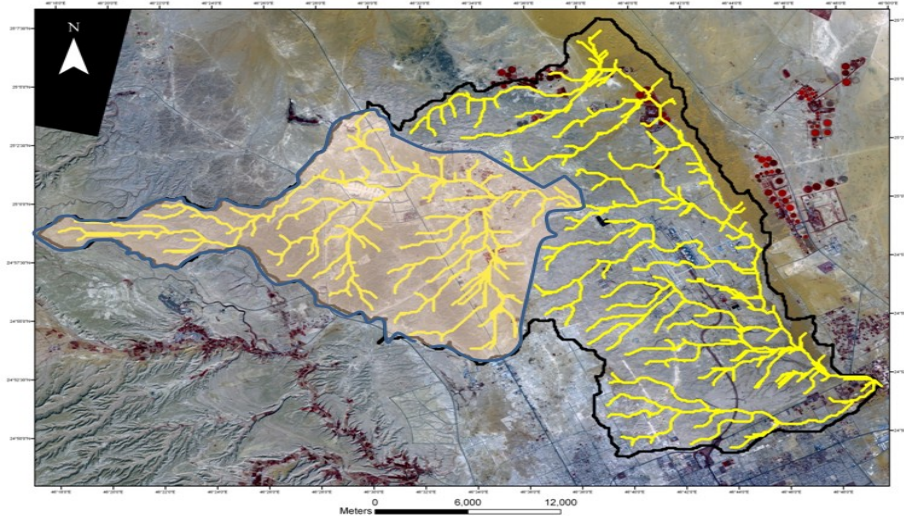
Fig.2.13 length of jump on horizontal floor

***Chapter (3)***

***Hydrologic study***

### 3.1 Site Description

The site of the catchment is located at Banban wadi north of Riyadh city, just a short distance from King Khalid international airport. It is really a sub-catchment for wadi Solay with an area of 308.88 km<sup>2</sup> (119.259mile<sup>2</sup>). The site has an average annual rainfall of 120 mm. Slope of area has an average value of 0.0175. Fig 3.1 shows the map of the site.



**Figure 3.1:** map of the catchment area (1cm: 4800 m)

The length of the catchment is equal to 31.93 Km and the width of the catchment is 39.3Km. The total length of all streams is equal to 250 Km and the distance from station to water divide measured along the main stream is  $L=41.79$  Km = 25.967 mile.

Catchment area parameters were estimated as follows:

- Drainage density =  $\frac{\text{Total length of all the streams}}{\text{Area of catchment}} = \frac{250}{308.88} = 0.809 \text{ km}^{-1}$
- Form factor =  $\frac{\text{watershed area}}{(\text{watershed length})^2} = \frac{119.259}{(19.840)^2} = 0.303 < 1$
- Shape factor =  $\frac{(\text{watershed length})^2}{\text{watershed area}} = \frac{(19.840)^2}{119.259} = 3.301 > 1$
- Elongation ratio =  $\frac{1.128 A^{0.5}}{L} = \frac{1.128 \times 119.259^{0.5}}{19.840} = 0.621 < 1$
- Circularity ratio =  $\frac{12.57A}{P_f^2} = \frac{12.57(119.259)}{94.697^2} = 0.167 \leq 1$
- Compactness coefficient =  $\frac{0.282(P_f)}{A^{0.5}} = \frac{0.282(94.697)}{119.259^{0.5}} = 2.445 > 1$

### 3.2 Synthetic Unit Hydrograph Derivation

To calculate  $L_c$ , the centroid of the catchment needs to be determined. So the catchment was divided into subareas rectangular and triangular in shapes ( $A_1, A_2, A_3 \dots$  etc) as shown in figure 3.2

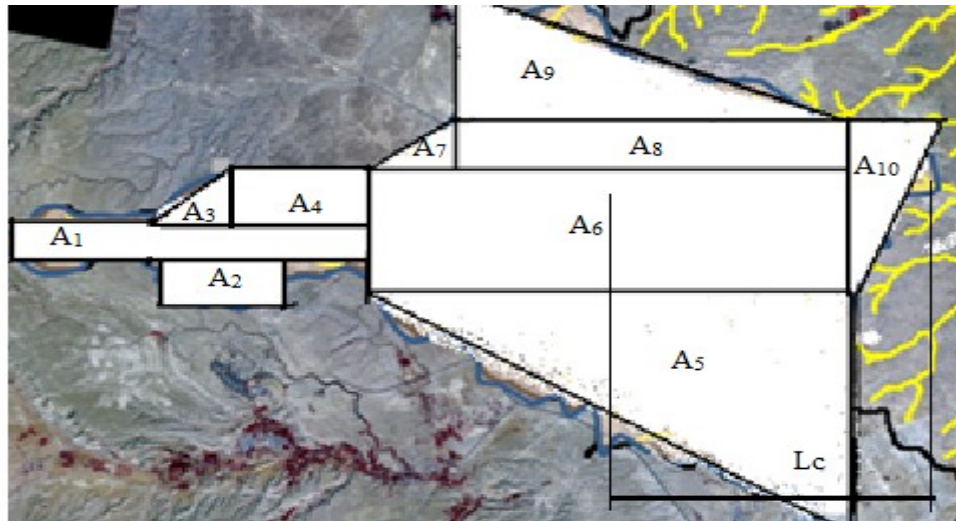


Figure 3.2: Catchment subareas

- Sum of regular shapes areas

$$A_s = A_1 + A_2 + A_3 + A_4 + A_5 + A_6 + A_7 + A_8 + A_9 + A_{10}$$

$$A_s = 9.1 + 1.875 + 0.21 + 2.55 + 20.16 + 32.87 + 0.93 + 8.875 + 12.78 + 5 = 94.32 \text{ cm}^2.$$

**1 cm: 1875 m**

$$A_s = 331.59 \text{ Km}^2$$

An error is calculated to verify the result

$$\text{Error} = \left| \frac{308.88 - 331.59}{308.88} \times 100 \right| = 7.35 \% < 10\% \text{ O.K}$$

$$L_c = \frac{\sum C_i A_i}{\sum A_i}$$

$$= \frac{14.25 \times 12.9 + 0.45 \times 14 + 9.1 \times 13.8 + 0.21 \times 14 + 2.55 \times 12.1 + 20.16 \times 4.1 + 32.87 \times 6.1 + 0.93 \times 9.4 + 8.88 \times 5.3 + 12.78 \times 6.45 + 5 \times 1.2}{94.32}$$

$$= 6.48 \text{ cm from outlet}$$

**And since 1 cm : 1875 m**

$$L_c = 14.25 \text{ km} = 8.8545 \text{ mile}$$

- Applying Snyder's method to the catchment of Wadi Banban, taking in consideration the following:

Catchment Area (mi <sup>2</sup> )	Slope (%)	Main length (mi)	(L <sub>c</sub> ) (mi)
119.259	0.0175	25.967	8.8535

- Time to peak**

$$t_p = c_t(LL_c)^{0.3}$$

$$L=25.97 \text{ mile} \quad L_c = 8.8535 \text{ mile} \quad c_t = \frac{1.8+2.2}{2} = 2$$

$$t_p = 2(25.97 \times 8.8545)^{0.3} = 10.222 \text{ hrs}$$

- Unit hydrograph duration  $t_r$**

$$t_r = \frac{t_p}{5.5} = \frac{10.222}{5.5} = 1.859 \text{ hrs}$$

- Peak discharge " $Q_p$ "**

$$Q_p = \frac{c_p A}{t_p} \times 640$$

$$A=119.259 \text{ mile}^2 \quad t_p = 10.222 \text{ hrs} \quad c_p = \frac{0.4+0.8}{2} = 0.6$$

$$Q_p = \frac{0.6 \times 119.259}{10.222} \times 640 = 4480.088 \text{ cfs}$$

- Time base**

$$T=4t_p = 4 \times \left(\frac{10.222}{24}\right) = 1.704 \text{ days}$$

Here, it is required to construct  $t_c$ -UH

- Time of concentration**

$$t_c = 0.0078L^{0.77}S^{-0.385} \text{ (Kirpich equation)}$$

$$L=31.93 \text{ Km} = 104757.2178 \text{ ft} \quad S = 0.0175$$

$$t_c = 0.0078(104757.2178)^{0.77}(0.0175)^{-0.385} = 271.686 \text{ min}$$

- Adjusted peak time and peak discharge**

$$t_c = 271.686 \text{ min} = 4.5281 \text{ hrs}$$

$$\bar{t}_p = t_p + \frac{t_c - t_r}{4} = 10.222 + \frac{4.5281 - 1.859}{4} = 10.889 \text{ hrs}$$

- **Peak discharge " $Q_p$ "**

$$Q_p = \frac{c_p A}{t_p} \times 640 = \frac{0.6 \times 119.259}{10.889} \times 640 = 4205.66 \text{ cfs}$$

- **Time base**

$$T = 4 \bar{t}_p = 4 \times \frac{10.889}{24} = 1.8148 \text{ days}$$

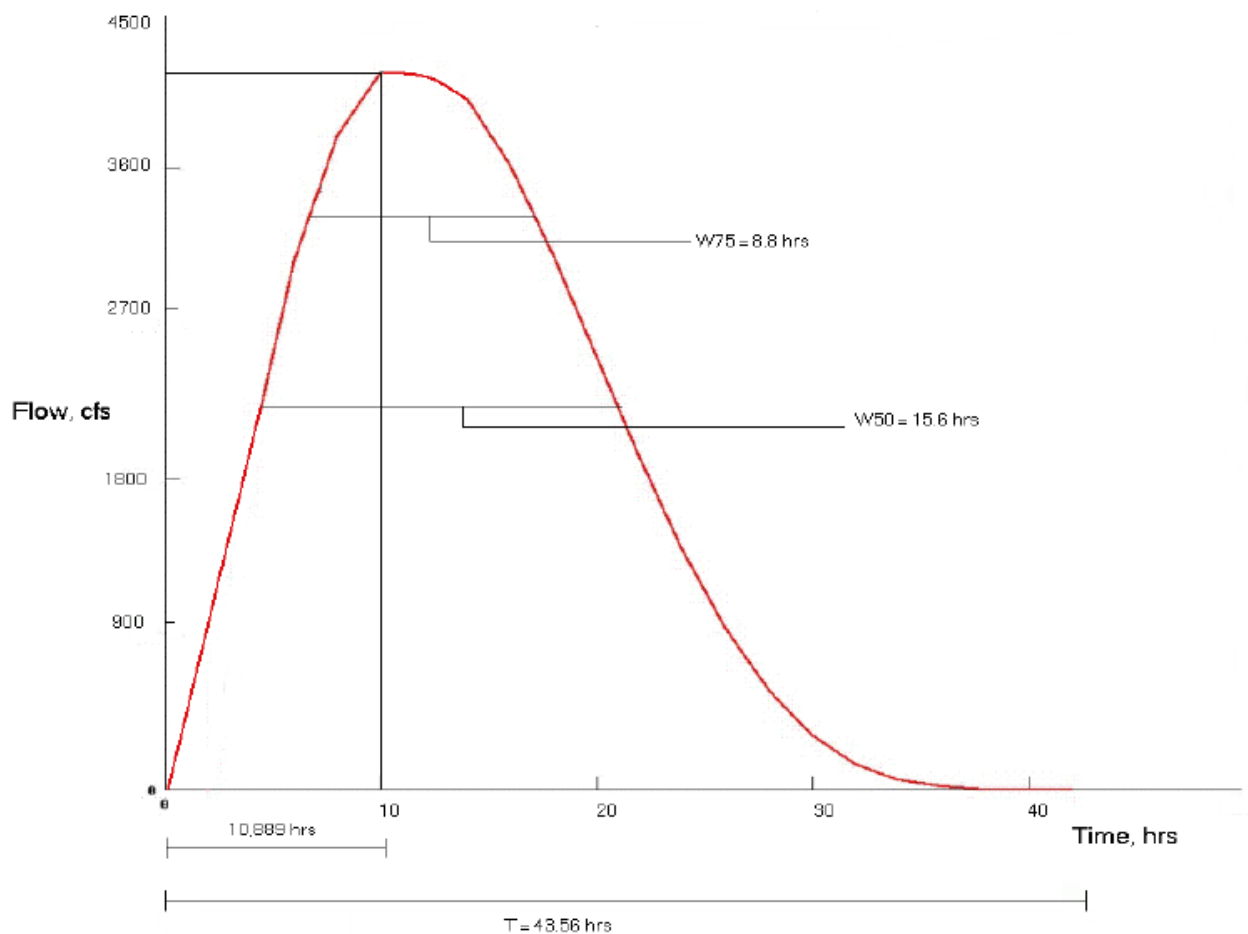
- **W50 and W75 calculations**

$$q_p = \frac{Q_p}{A} = \frac{4205.66}{119.259} = 35.265 \frac{\text{cfs}}{\text{mile}^2}$$

$$W_{50} = \frac{830}{q_p^{1.1}} = \frac{830}{35.265^{1.1}} = 16.48 \text{ hrs}$$

$$W_{75} = \frac{470}{q_p^{1.1}} = \frac{470}{35.265^{1.1}} = 9.332 \text{ hrs}$$

Figure 3.3 shows developed 4.5281 hr-UH



**Fig 3.3** tc-SUH (4.5281 hr UH)

### 3.2 storage volume

From **unit hydrograph** shown in Fig3.3 ( $t_c - \text{SUH}$ ) and IDF relation, the hydrograph can be developed to calculate total runoff volume. This volume is then used to find height of dam. IDF curves for Riyadh is given in figure 3.4

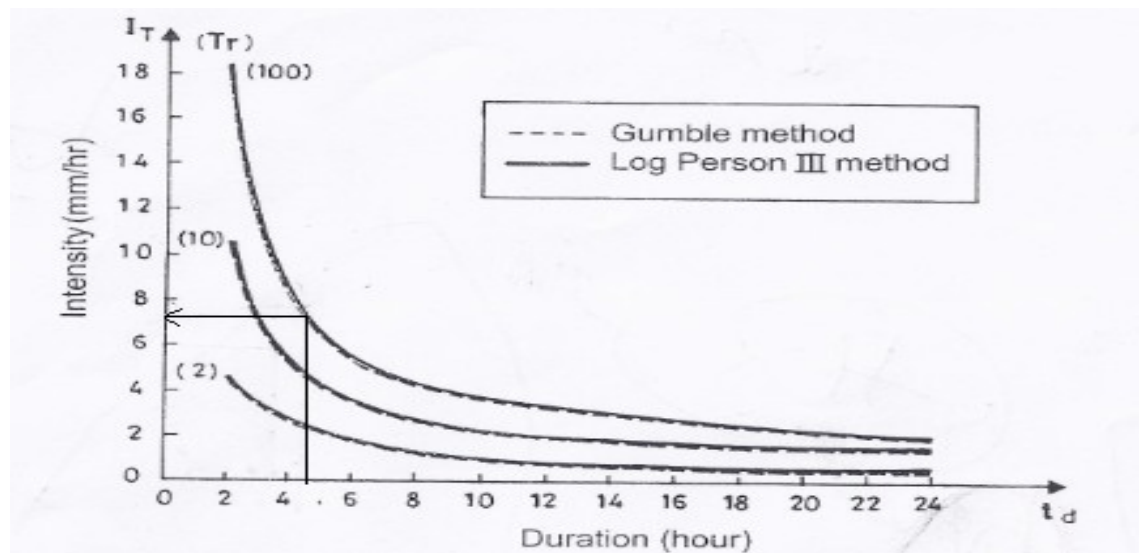


Fig 3.4 IDF curve

1- At **100 year** return period (frequency) :

From Fig 3.4, at duration **4.528 hrs** and **100 year** return period, Intensity (mm/hr) = 7.2 mm/hr

Rainfall Depth = Intensity  $\left(\frac{\text{mm}}{\text{hr}}\right) \times t_c \text{ (hr)} = 7.2 \times 4.528 = 32.60232 \text{ mm} = 1.283 \text{ inch}$

$Q_p = 1.283 \times 4205.66 = 5395.862 \text{ cfs.}$

Figure 3.5 shows total hydrograph, while figure 3.6 shows volume of rainfall (area under curve)

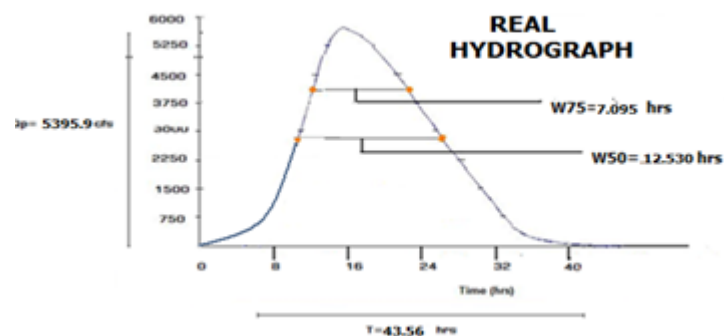


Fig 3.5 Hydrograph of the area

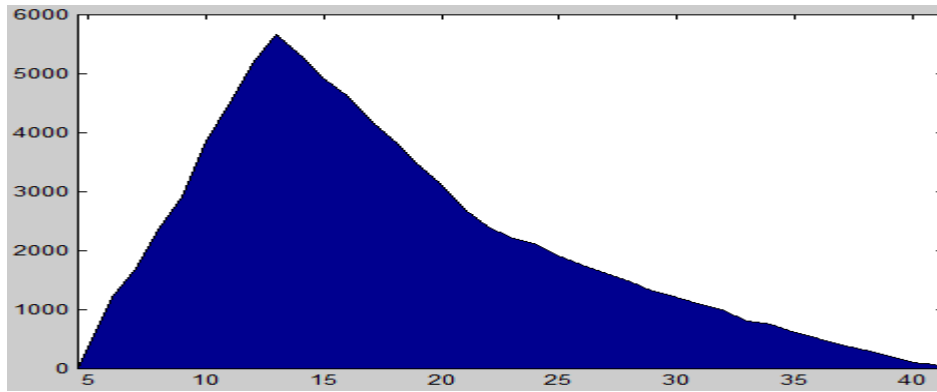


Fig 3.6 Volume under real hydrograph curve at 100 year period

- Total volume of Rainfall from hydrograph at **100 year** return period

$$= 91946 \frac{\text{ft}^3}{\text{sec}} \times 3600 \text{ sec} = 331005600 \text{ ft}^3 = 9268156.8 \text{ m}^3 \text{ " by using MATLAB program"}$$

For sandy clay soil, the runoff coefficient is ranging from 0.2 to 0.5 and for this case it is assumed that the runoff coefficient is the average value as 0.35.

$$\text{Runoff volume} = \text{Total volume of Rainfall} \times \text{Factor}$$

$$= 9268156.8 \times 0.35 = 3243854.88 \text{ m}^3$$

- At **50 year** return period:

The 50 year return period curve was not provided in the given IDF curves for Riyadh. Hence, empirical equation developed by Saleh AlHasson was used

$$I_t = \frac{153T^{0.35}}{t_c^{0.82}}$$

$$= \frac{153 \times 50^{0.35}}{271.686^{0.82}} = 6.0729 \frac{\text{mm}}{\text{hr}}$$

$$\text{Rainfall depth} = \text{Intensity} \left( \frac{\text{mm}}{\text{hr}} \right) \times t_c \text{ (hr)} = 6.0729 \times 4.528 = 27.498 \text{ mm} = 1.0826 \text{ inch}$$

$$Q_p = 1.0826 \times 4205.66 = 4553.048 \text{ cfs.inch}$$

- Total volume of Rainfall from hydrograph at **50 year** return period

$$= 75176.93 \frac{\text{ft}^3}{\text{sec}} \times 3600 \text{ sec} = 270636933.1 \text{ ft}^3 = 7663584.5 \text{ m}^3 \text{ " by using MATLAB program"}$$

$$\text{Runoff volume} = \text{Total volume of Rainfall} \times \text{Factor}$$

$$= 7663584.52 \times 0.35 = 2,682,254.58 \text{ m}^3$$

3- At **10 year** return period:

From Fig 3.5, at duration **4.528** hrs and **10** year period, Intensity (mm/hr) =4.7 mm/hr

$$\text{Rainfall depth} = \text{Intensity} \left( \frac{\text{mm}}{\text{hr}} \right) \times t_c \text{ (hr)} = 4.7 \times 4.528 = 21.282 \text{ mm} = 0.838 \text{ inch}$$

$$Q_p = 0.838 \times 4205.66 = 3524.343 \text{ cfs.inch}$$

- Total volume of Rainfall from hydrograph at **10 year** period

$$= 61371.92 \frac{\text{ft}^3}{\text{sec}} \times 3600 \text{ ses} = 220938914.4 \text{ ft}^3 = 6256293.4 \text{ m}^3 \text{ " by using$$

**MATLAB** program"

$$\text{Runoff volume} = \text{Total volume of Rainfall} \times \text{Factor}$$

$$= 6555293.35 \times 0.35 = 2294352.67 \text{ m}^3$$

4- At **2 year** return period :

From Fig 3.5, at duration **4.528** hrs and **2** year period, Intensity (mm/hr) =2.7 mm/hr

$$\text{Rainfall depth} = \text{Intensity} \left( \frac{\text{mm}}{\text{hr}} \right) \times t_c \text{ (hr)} = 2.7 \times 4.528 = 12.2256 \text{ mm} = 0.481 \text{ inch}$$

$$Q_p = 0.481 \times 4205.66 = 2024.280 \text{ cfs.inch}$$

Total volume of Rainfall from hydrograph at **2 year** period

$$= 36357 \frac{\text{ft}^3}{\text{sec}} \times 3600 \text{ ses} = 130885200 \text{ ft}^3 = 3706256.13 \text{ m}^3 \text{ " by using$$

**MATLAB** program"

$$\text{Runoff volume} = \text{Total volume of Rainfall} \times \text{Factor}$$

$$= 3706256.13 \times 0.35 = 1297189.646 \text{ m}^3$$

Return period (years)	Rainfall depth (mm)	Rainfall volume (m <sup>3</sup> )	Runoff coefficient	Runoff volume, m <sup>3</sup>
100	32.60232	9268156.8	0.35	3243854.88
50	27.498	7663584.52	0.35	2682254.58
10	21.282	6256293.4	0.35	2294352.67
2	12.2256	3706256.13	0.35	1297189.646

**Table 3.1** Rainfall quantities at different return period

### 3.4 Elevation -Volume Curves

Five cross sections were used in order to develop a relationship between area and depth of flow for each cross section by calculating the area at any given depth on the figure itself, which will be used to determine the volume at any given elevation by multiplying the area with the distance to the next cross section to find the total height of the dam until the 10 years return period storage volume is reached, noted that excavation was made on the upstream to increase the storage volume. Fig 3.7 presents original cross section & excavates cross section.

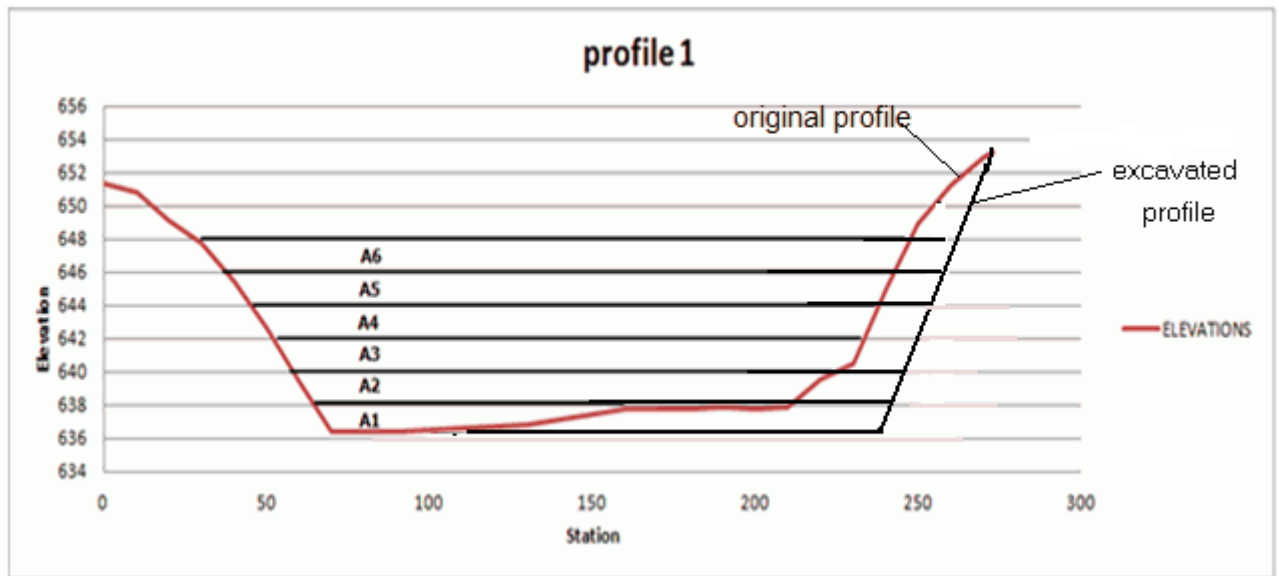
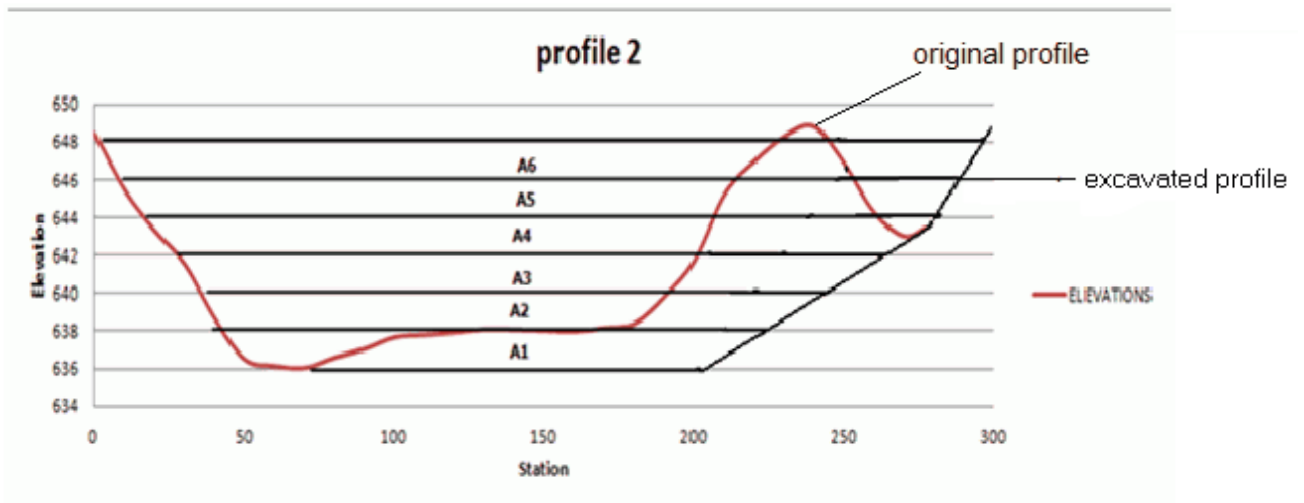


Fig.3.7.1 Cross section 1 at x=0

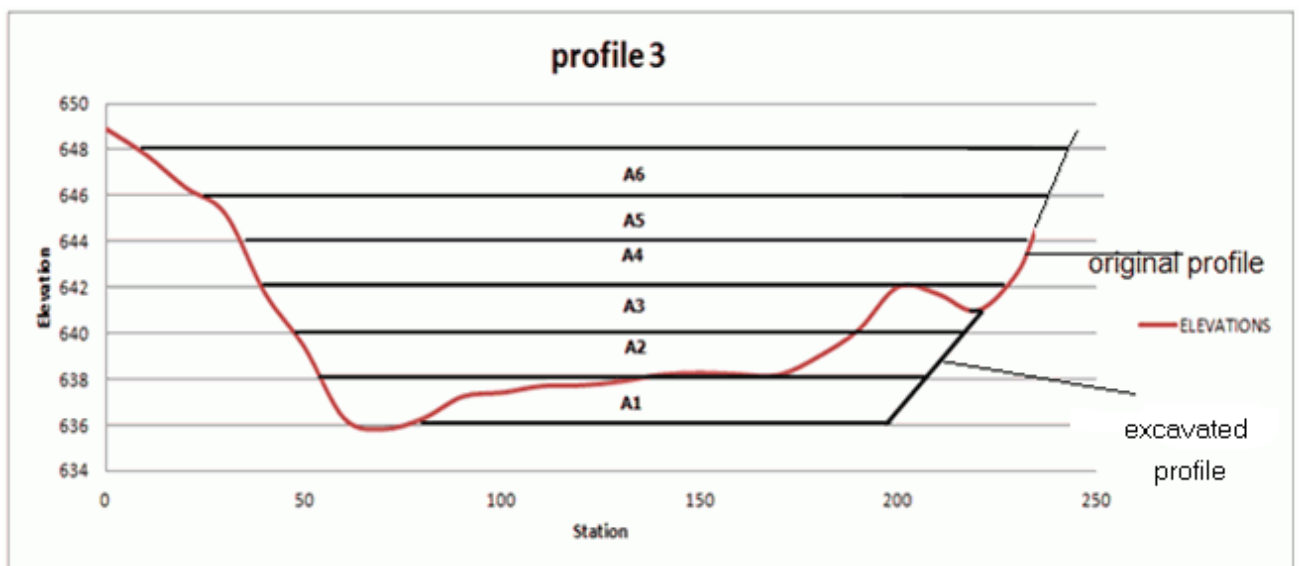
At Elevation = 638 m	—————→	Area 1 = 233.60 $m^2$
At Elevation = 640 m	—————→	Area 2 = 312.4 $m^2$
At Elevation = 642 m	—————→	Area 3 = 348 $m^2$
At Elevation = 644 m	—————→	Area 4 = 369.6 $m^2$
At Elevation = 646 m	—————→	Area 5 = 397 $m^2$
At Elevation = 648 m	—————→	Area 6 = 422.4 $m^2$

Where “x” is the section’s distance from the dam



**Fig.3.7.2** Cross section 2 at x=100 m

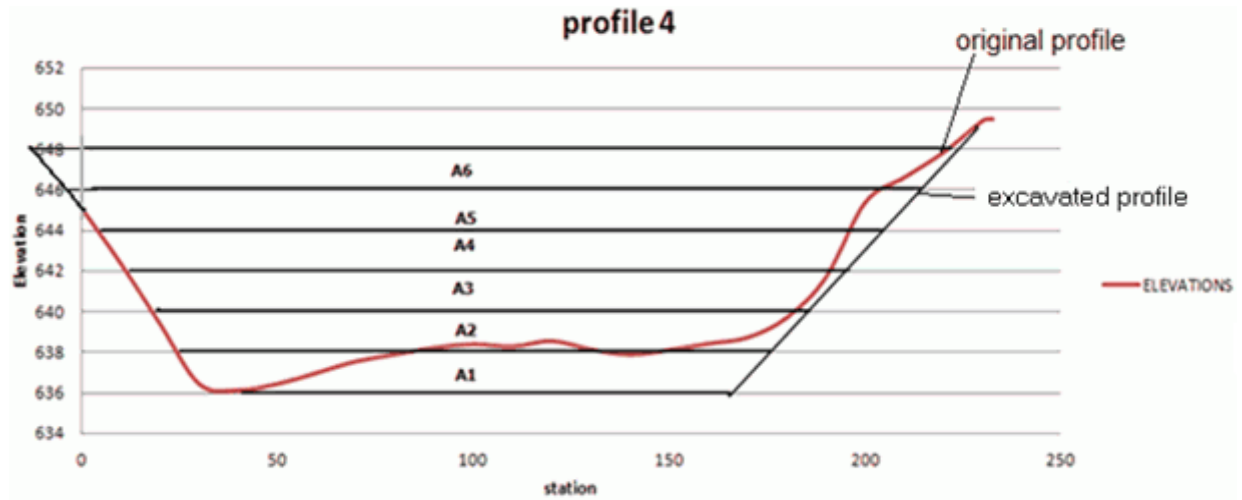
At Elevation = 638 m	—————→	Area 1 = 391.6 m <sup>2</sup>
At Elevation = 640 m	—————→	Area 2 = 429.4 m <sup>2</sup>
At Elevation = 642 m	—————→	Area 3 = 462 m <sup>2</sup>
At Elevation = 644 m	—————→	Area 4 = 495 m <sup>2</sup>
At Elevation = 646 m	—————→	Area 5 = 493 m <sup>2</sup>
At Elevation = 648 m	—————→	Area 6 = 488 m <sup>2</sup>



**Fig.3.7.3** Cross section 3 at x=200 m

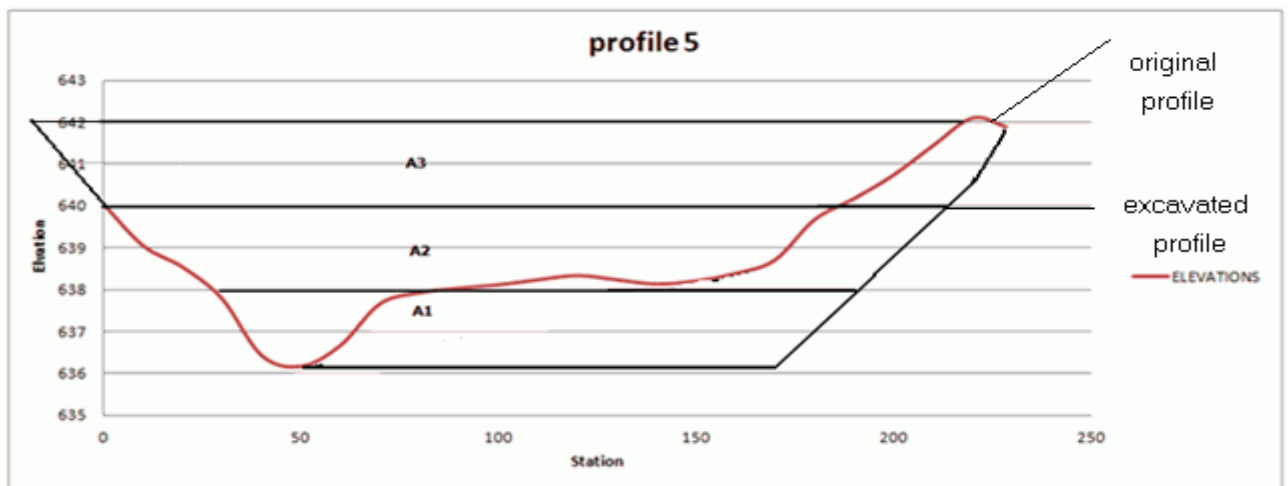
At Elevation = 638 m	—————→	Area 1 = 304 m <sup>2</sup>
At Elevation = 640 m	—————→	Area 2 = 335.5 m <sup>2</sup>

At Elevation = 642 m	—————→	Area 3 = 360 $m^2$
At Elevation = 644 m	—————→	Area 4 = 386 $m^2$
At Elevation = 646 m	—————→	Area 5 = 400 $m^2$
At Elevation = 648 m	—————→	Area 6 = 438 $m^2$



**Fig.3.7.4** Cross section 4 at x=300 m

At Elevation = 638 m	—————→	Area 1 = 318.5 $m^2$
At Elevation = 640 m	—————→	Area 2 = 344.4 $m^2$
At Elevation = 642 m	—————→	Area 3 = 370.7 $m^2$
At Elevation = 644 m	—————→	Area 4 = 400 $m^2$
At Elevation = 646 m	—————→	Area 5 = 424 $m^2$
At Elevation = 648 m	—————→	Area 6 = 443 $m^2$



**Fig.3.7.5** cross section 5 at x= 350 m

At Elevation = 638 m → Area 1 = 230.1 m<sup>2</sup>  
 At Elevation = 640 m → Area 2 = 339.5 m<sup>2</sup>  
 At Elevation = 642 m → Area 3 = 417 m<sup>2</sup>

Slope = slope of catchment = 0.0175

Now, a relationship between volume and height can be estimated. based on the area and height of each profile.

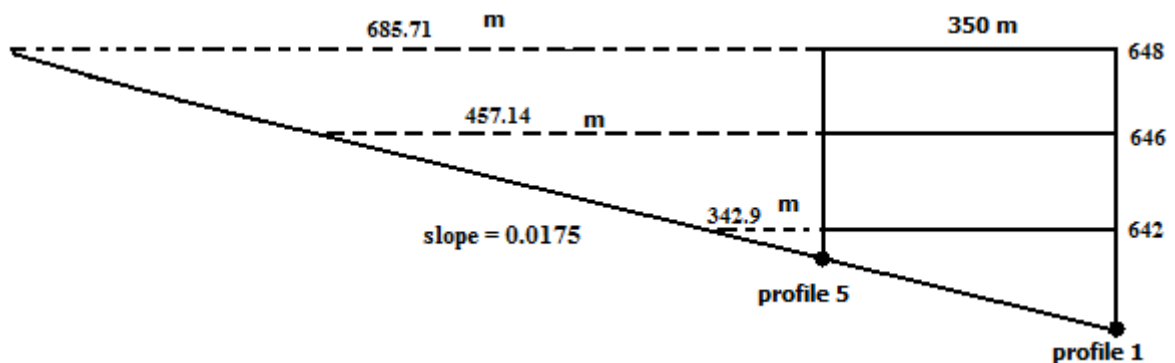
1. At Elevation = 638 m

$$\text{Volume} = \frac{A_1 + A_2}{2} \times \text{distance}(1 - 2) + \dots$$

$$V_1 = \frac{233.60 + 391.6}{2} \times 100 + \frac{391.6 + 304}{2} \times 100 + \frac{304 + 318.5}{2} \times 100 + \frac{318.5 + 230.1}{2} \times 50 +$$

$$\left(\frac{2}{0.0175}\right) \times 230.1 = 145857 \text{ m}^3$$

Figure 3.8 shows side view of the upstream profile



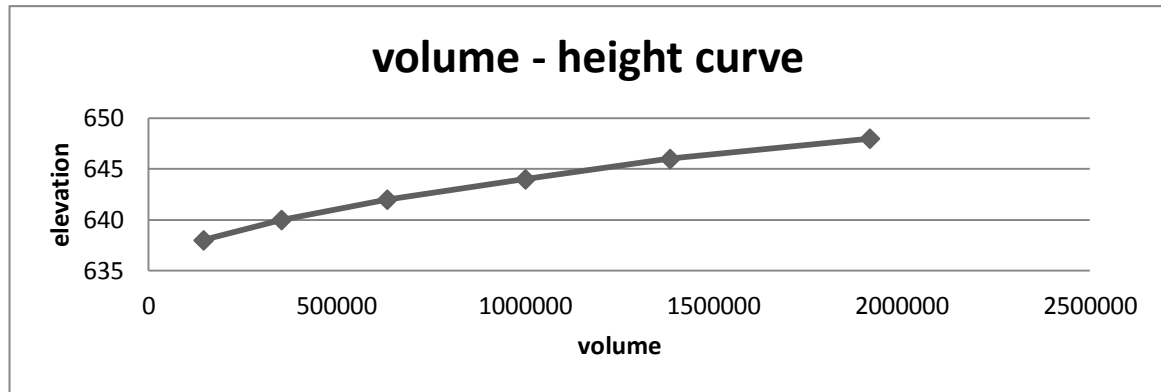
**Fig 3.8** Side view of the upstream profile

Each depth was subtracted by the slope to determine the rest of the longitudinal distances as shown above.

The same method is done to calculate the volume at the rest of the elevations. Table 3.2 shows volume at different depths and Fig 3.9 shows volume – elevation curve.

Elevation, m	Depth ,m	Volume , m <sup>3</sup>
638	2	145857
640	4	352906
642	6	633686
644	8	1000730
646	10	1385000
648	12	1915585

**Table 3.2** Volume at different depths



**Figure 3.9** Volume- elevation curve

Since the total runoff volume at 10 year period =  $2294352.67\text{m}^3$ , and the total volume of water that can be trapped at a 12 m high dam =  $1915585\text{m}^3$ , then the spillway will only have to discharge an amount of water equal to  $2294352.67\text{m}^3 - 1915585\text{m}^3 = 378768.3\text{m}^3$ .

# *Chapter (4)*

## *Hydraulic design*

## 4.1 Elementary Profile of a Gravity Dam

If only hydrostatic forces are considered, the elementary profile will be triangular in section (Fig. 4.1), having zero width at the water level; where water pressure is zero, and a maximum base width  $b$ , where the maximum water pressure acts.

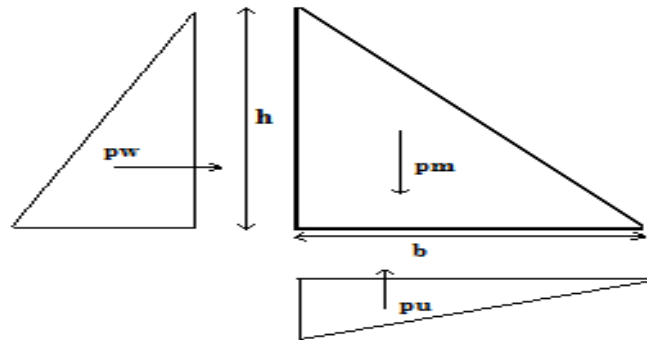


Fig 4.1 Gravity dam – elementary profile

### 4.1.1 Initial Base Width Required for Elementary Profile

For initial determination of the base width of the dam, it is assumed that safety against sliding is satisfied. Also it is assumed that rock foundation is about 5 m below the wadi bed level, and that the maximum water level above the spillway is 1.5 m. Hence for the above assumption the load acting on the preliminary dam cross section is:-

- Dam safety against sliding

For safety considerations  $F_{ss} = 0.75$

$$0.75 = \frac{\sum H}{\sum V}$$

The dam will have a foundation that is 5 m high, a spillway head of 1 m plus 0.5 m for safety. And since the height of water is 12 m, the total height of the dam will be as follows-

$$h = 5 + 1 + 0.5 + 12 = 18.5 \text{ m}$$

---


$$p_{m1} = 18.5 \times b \times 0.5 \times 24 = 222 \text{ b KN.m}^{-1}$$

$$\text{Point of action} = \frac{1}{3} \times b \text{ from heel}$$

$$p_u = 18.5 \times b \times 0.5 \times 9.81 = 90.74 \text{ b KN.m}^{-1}$$

$$\text{Point of action} = \frac{1}{3} \times b \text{ from heel}$$

$$p_{wh} = 18.5 \times 18.5 \times 0.5 \times 9.81 = 1678.74 \text{ KN.m}^{-1}$$

$$\text{Point of action} = \frac{1}{3} \times 18.5 = 6.167 \text{ m from wadi bed action}$$


---

$$\sum H = p_{wh} = 1678.74 \text{ KN.m}^{-1}$$

$$\sum V = p_{m1} - p_u = 222 \text{ b} - 90.74 \text{ b}$$

$$0.75(222 - 90.74) \text{ b} = 1678.74 \longrightarrow \text{b} = 17.05 \text{ m} \approx 17.1 \text{ m}$$

## 4.2 Modified Dam Cross Section Width and Free-Board

The initial dam cross section will be modified by adding a free board height and a top width. The top width of a gravity dam is generally fixed by the provision of a road-way and side walls. The economical top width of a dam is about 14% of the height of the dam. In this case, top width  
 $= 0.14 \times 18.5 = 2.60 \text{ m} \approx 3 \text{ m}$ .

The free board in the dam should be able to avoid overtopping of the dam during maximum flood coupled with waves. The economical free-board is approximately 10 % of the height of the dam.

The Free-Board in this case can be estimated as  $0.10 \times 18.5 = 1.85 \approx 2 \text{ m}$ . The modified dam cross section is shown in Fig. 4.2

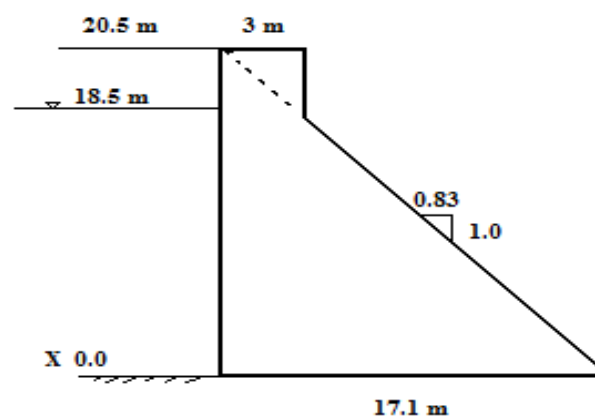


Fig.4.2 Gravity dam profile design

## 4.3 Stability Analysis of the Modified Dam Cross Section

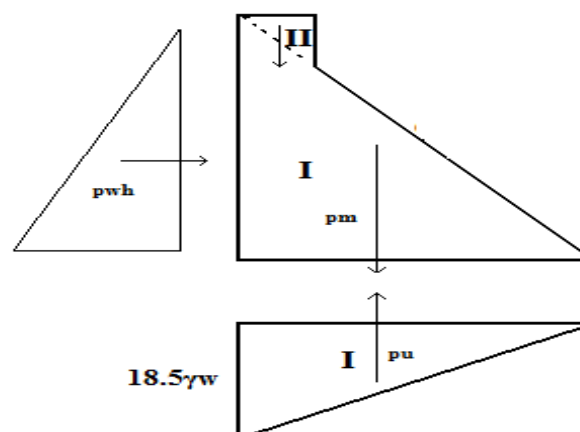


Fig.4.3 gravity dam load diagram

- **Self weight load**

The weight of the dam section can be found by dividing the section into simpler geometrical shapes like triangle, rectangle, etc. see Fig 4.3

The load of each part can be calculated by eq. (2.1)

---


$$p_{m1} = 17.1 \times 20.5 \times 0.5 \times 24 = 4206.6 \text{ KN.m}^{-1} \quad \text{Point of action} = \frac{1}{3} \times 17.1 = 5.7 \text{ m from heel}$$

$$p_{m2} = 3.51 \times 3 \times 0.5 \times 24 = 126.36 \text{ KN.m}^{-1} \quad \text{Point of action} = \frac{2}{3} \times 3 = 2 \text{ m from heel}$$


---

- **Uplift load**

The uplift intensity at the upstream heel end is  $18.5 \gamma_w$  and on the downstream tail water taken as zero (assuming there is no water on the downstream). The uplift pressure diagram (without drainage) is represented as a triangle. See fig.4.3

Uplift load can be calculated by eq. (2.3)

---


$$p_u = 17.1 \times 18.5 \times 0.5 \times 9.81 = 1551.697 \text{ KN.m}^{-1} \quad \text{Point of action} = \frac{1}{3} \times 17.1 = 5.7 \text{ m from heel}$$


---

- **Water load**

The water pressure acts horizontally on the upstream face and is given by eq (2.2).

---


$$p_{wh} = 18.5 \times 18.5 \times 0.5 \times 9.81 = 1678.74 \text{ KN.m}^{-1} \quad \text{Point of action} = \frac{1}{3} \times 18.5 = 6.167 \text{ m from river bed}$$


---

- **Silt load**

The silt load given by eq. (2.6)

$$p_s = k_a \times \gamma_s \times \frac{z_3^2}{2}$$

- $z_3 = z_1 = 18.5 \text{ m}$

- $\Phi$  angle of shearing resistance of sediment equal  $33^\circ$  based on (caly and sand)soil

$$k_a = \frac{1 - \sin \Phi}{1 + \sin \Phi} = \frac{1 - \sin 33}{1 + \sin 33} = 0.295$$

- $\gamma_s = \gamma_{sat} - \gamma_w$

$$\gamma_{sat} = \gamma_d (1 + W)$$

$$\gamma_d = \frac{G_s \times \gamma_w}{1 + e}$$

Where specific gravity " $G_s=2.80$  , void ratio " $e = 0.90$  and moisture content  $w = 20 \%$  based on (caly and sand)soil

$$\gamma_d = \frac{2.80 \times 9.81}{1 + 0.9} = 6.46 \text{ KN.m}^{-3}$$

$$\gamma_{\text{sat}} = \gamma_d(1 + W) = 6.46 \left(1 + \frac{20}{100}\right) = 7.75 \text{ KN.m}^{-3}$$

$$\gamma_s = \gamma_{\text{sat}} - \gamma_w = 7.75 - 9.81 = -2.06 \text{ KN.m}^{-3}$$

$$p_s = k_a \times \gamma_s \times \frac{z_3^2}{2}$$

$$p_s = 0.295 \times 7.54 \times \frac{18.5^2}{2} = 380.63 \text{ KN.m}^{-1} \text{ point of action } \frac{1}{3} \times 18.5 = 6.167 \text{ m}$$

- Earthquake forces were not considered because the dam is not being built in a seismically active region, wind loads are also neglected.

### 4.3.1 Checking the Stability of the Dam

- Overturning safety factor

$$F_0 = \frac{\sum M_{+ve}}{\sum M_{-ve}}$$

$$\sum M_{+ve} = p_{m1}(17.1 - 5.7) + p_{m2}(17.1 - 2) = 49863.28 \text{ KN.m.m}^{-1}$$

$$\sum M_{-ve} = p_{wh} \times 6.167 + p_s \times 6.167 + p_u(17.1 - 5.7) = 30389.48 \text{ KN.m.m}^{-1}$$

$$F_0 = 1.64 > 1.5 \text{ o.k.}$$

- Sliding safety factor

$$F_{ss} = \frac{\sum H}{\sum V}$$

$$\sum H = p_{wh} + p_s = 1678.74 + 380.63 = 2059.37 \text{ KN.m}^{-1}$$

$$\sum V = p_{m1} + p_{m2} - p_u = 4206.6 + 126.36 - 1551.697 = 2781.26 \text{ KN.m}^{-1}$$

$$F_{ss} = 0.740 < 0.75 \text{ o.k.}$$

- **Shear friction factor**

---


$$F_{SF} = \frac{C \times A_h - \sum V \tan \Phi}{\sum H} = \frac{1.5 \times 1000 \times 17.1 - 2781.26 \times \tan 0.8}{2059.37} = 12.4 > 4 \text{ o.k.}$$


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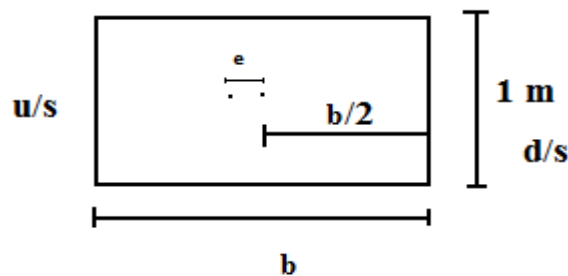
- **safe stresses**

The stresses in the dam should be within the specified limits for the body of the dam and

in the foundations. The stresses, on the upstream and downstream faces can be calculated by the formulae that are given below.

$$\sigma_{zu} = \frac{\sum V}{b} \left(1 - \frac{6 \times e}{b}\right), \quad \sigma_{zd} = \frac{\sum V}{b} \left(1 + \frac{6 \times e}{b}\right)$$

Moment  $M^*$  is taken about  $\frac{b}{2}$



- **Stress At foundation**

$$b = 17.1 \text{ m}$$

$$e = \frac{\sum M^*}{\sum V}$$

$$\sum V = p_{m1} + p_{m2} = 4206.6 + 126.36 = 4332.96 \text{ KN.m}^{-1}$$

$$\sum M^* = p_{m1} \left(\frac{17.1}{2} - 5.7\right) + p_{m2} \left(\frac{17.1}{2} - 2\right) - p_{wh} \times 6.167 - p_s \times 6.167 = 116.89 \text{ KN.m.m}^{-1}$$

$$e = 0.027 \text{ m} < \frac{T}{6} = 2.85 \text{ no tension on u/s O.K.}$$

$$\sigma_{zu} = \frac{\sum V}{b} \left(1 - \frac{6 \times e}{b}\right) = \frac{4332.96}{17.1} \left(1 - \frac{6 \times 0.027}{17.1}\right) = 250.989 \text{ KN.m}^{-2}$$

$$\sigma_{zd} = \frac{\sum V}{b} \left(1 + \frac{6 \times e}{b}\right) = \frac{4332.96}{17.1} \left(1 + \frac{6 \times 0.027}{17.1}\right) = 255.79 \text{ KN.m}^{-2}$$

$$\sigma_{zu} \text{ \& } \sigma_{zd} < 3 \text{ Mm.m}^{-2} \text{ O.K}$$

- **Stress At ground level**

$$b=12.93 \text{ m}$$

$$\sum V = 4332.96 \text{ KN. m}^{-1}$$

$$\sum M^* = 109.65 \text{ KN. m. m}^{-1}$$

$$e = 0.0253 \text{ m} < \frac{T}{6} = 2.155 \text{ no tension on u/s O.K}$$

$$\sigma_{zu} = 331.16 \text{ KN. m}^{-2}$$

$$\sigma_{zd} = 339.06 \text{ KN. m}^{-2}$$

$$\sigma_{zu} \text{ \& } \sigma_{zd} < 3 \text{ Mm. m}^{-2} \text{ O.K}$$

- **Stress At 3 m above ground level**

$$b = 10.43 \text{ m}$$

$$\sum V = 4332.96 \text{ KN. m}^{-1}$$

$$\sum M^* = 1347.62 \text{ KN. m. m}^{-1}$$

$$e = 0.31 \text{ m} < \frac{T}{6} = 1.733 \text{ no tension on u/s O.K}$$

$$\sigma_{zu} = 341.35 \text{ KN. m}^{-2}$$

$$\sigma_{zd} = 489.52 \text{ KN. m}^{-2}$$

$$\sigma_{zu} \text{ \& } \sigma_{zd} < 3 \text{ Mm. m}^{-2} \text{ O.K}$$

#### 4.4 Spillway Design

Spillway will be designed for 100 year return period flood and will be located in the middle of the dam with a clear water way of 70 m as an initial assumption. Discharge At 100 year period = 5,395.862 cfs. = 152.79 m<sup>3</sup>/sec, length of clear span = 70 m at 648 m elevation. Slope of d/s face is 1: 0.83, openings 10 m wide each and 6 rounded nose piers each 1 m thick.

Let us assume a 2.21 coefficient of discharge, hence

$$Q = c_w \times L \times H_d^{1.5}$$

$$152.79 = 2.21 \times 70 \times H_d^{1.5} \longrightarrow H_d = 0.991 \text{ m}$$

##### Checking $c_w$

$$\text{For vertical u/s } c_w = c_w \text{ max} = 2.21$$

Effect of approach flow depth,  $p/H = 17/0.992 = 17.14$ . As this is more than 4.0, there is no effect of approach depth discharge may be taken as 2.21. , and as  $p/H > 1.33$  there is no effect of approach velocity head

$$c_w = c_w \text{ max} = 2.21, H_d = 1 \text{ m}$$

**Checking discharge**

Rounded abutment,  $k_a = 0.1$ ,  $k_p = 0.01$ ,  $n = 6$ ,  $H_d = 1$  m

$$L = L - 2(nk_p + k_a)H_d = 70 - 2(6 \times 0.01 + 0.1)1 = 69.68 \text{ m}$$

$$Q = Q = c_w \times L \times H_d^{1.5} = 2.21 \times 69.68 \times 1^{1.5} = 153.99 \text{ m}^3/\text{sec} > 152.79 \text{ m}^3/\text{sec}$$

O.K.

∴ The crest profile will be designed for  $H_d = 1$  m

The total water way is  $70 + 6 \times 1 = 76$

**Ogee crest shapes**

- d/s profiles

The profile recommended by Waterways Experiment Station Vicksburg Mississippi U.S.A of U.S. Army is  $x^{1.85} = 2H_d^{0.85}y \longrightarrow y = 0.50 x^{1.85}$

**Tangent point coordinates**

The slope of the downstream is 0.83H: 1V. Differentiating equation of the downstream profile

$$\frac{dy}{dx} = 1.85 \times 0.50 \times x^{0.85} = \frac{1}{0.83}$$

$$X = 1.365 \text{ m}, y = 0.889 \text{ m}$$

The calculated coordinates for the downstream profile are:

<b>X(m)</b>	<b>0</b>	<b>0.3</b>	<b>0.6</b>	<b>0.9</b>	<b>1.2</b>	<b>1.365</b>	<b>14.80*</b>
<b>Y(m)</b>	0	0.054	0.194	0.411	0.701	0.889	17

$$*(17 - 0.889)0.83 + 1.365$$

- u/f profile

$$X = -0.27 H_d = -0.27 \text{ m}$$

$$Y = 0.126 H_d = 0.126 \text{ m}$$

The upstream profile is given by the equation:

$$y = \frac{0.724(x + 0.27H_d)}{H_d^{0.85}} + 0.126H_d - 0.4315H_d^{0.375} \times (x + 0.27H_d)^{0.625}$$

The coordinates of the profile are found out as follows

<b>X(m)</b>	0	-0.1	-0.2	-0.27	-0.27
<b>Y(m)</b>	0	0.107	0.0494	0.126	17

Max head =  $1.33H_d = 1.33 \text{ m}$  which will produce negative pressure  $-0.6H_d = 0.6 < 4.8 \text{ m}$

#### 4.4.1 Checking Stability of Spillway

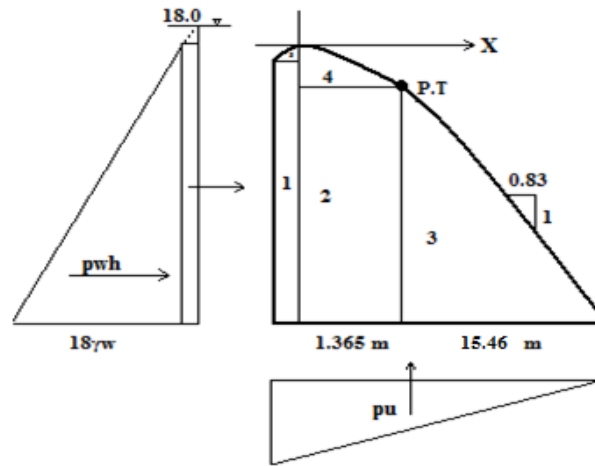


Fig 4.4 spillway shape

<b>Self weight load</b>			
Force calculation	L.A From toe	Moment @ Toe	L.A from base
$p_{m1} = 0.27 \times 16.874 \times 24 = 109.32 \text{ KN.m}^{-1}$	16.97	1855.16	8.437
$p_{m2} = 1.365 \times 16.11 \times 24 = 529.8 \text{ KN.m}^{-1}$	16.15	8523.32	8.055
$p_{m3} = 15.465 \times 16.11 \times 0.5 \times 24 = 2990.1 \text{ KN.m}^{-1}$	10.31	30823.81	5.37
$p_{m4} = 0.85 \times 24 = 20 \text{ KN.m}^{-1}$	16.38	327.47	16.41
$p_{m5} = 0.27 \times 0.126 \times 0.5 \times 24 = 0.43 \text{ KN.m}^{-1}$	16.92	6.91	16.916
<b>Water load</b>			
Force calculation	L.A From toe	Moment @ Toe	L.A from base
$p_{wh1} = 17 \times 1 \times 9.8 = 166.6 \text{ KN.m}^{-1}$	—————	1411	8.5
$p_{wh2} = 17 \times 17 \times 0.5 \times 9.8 = 1416 \text{ KN.m}^{-1}$	—————	8024	5.667
<b>Uplift load</b>			
Force calculation	L.A From toe	Moment @ Toe	L.A from base
$p_u = 18 \times (0.27 + 1.365 + 15.46) \times 0.5 \times 9.8$ $= 1508.22 \text{ KN.m}^{-1}$	11.4	17193.71	—————

#### Stability analysis for NLC

- **Overturning safety factor**

$$F_0 = \frac{\sum M_{+ve}}{\sum M_{-ve}}$$

$$\sum M_{+ve} = 40609.93 \text{ KN.m.m}^{-1}$$

$$\sum M_{-ve} = 26628.7 \text{ KN.m.m}^{-1}$$

---


$$F_0 = 1.525 > 1.5 \text{ o.k.}$$


---

- **Sliding safety factor**

$$F_{ss} = \frac{\sum H}{\sum V}$$

$$\sum H = p_{wh} = 1582 \text{ KN.m}^{-1}$$

$$\sum V = \sum p_m - p_u = 3597.3 - 1508.2 = 2089.3 \text{ Km.m}^{-1}$$

---


$$F_{ss} = 0.757 > 0.75 \text{ acceptable}$$


---

- **Shear friction factor**

---


$$F_{ss} = \frac{C \times A_h - \sum V \tan \Phi}{\sum H} = \frac{1.5 \times 1000 \times 17.1 - 2089.3 \times \tan 0.8}{1582}$$

$$= 16.20 > 4 \text{ o.k.}$$


---

- **safe stresses**

$$e = \frac{\sum M^*}{\sum V}$$

$$\sum V = 3597.3 \text{ KN.m}^{-1}$$

$$\sum M^* = 940.14 \text{ KN.m.m}^{-1}$$

$$e = 0.26 \text{ m} < \frac{b}{6} = 2.85 \text{ no tension on u/s O.K.}$$

$$\sigma_{zu} = \frac{\sum V}{b} \left(1 - \frac{6 \times e}{b}\right) = \frac{3597.3}{17.1} \left(1 - \frac{6 \times 0.026}{17.1}\right) = 208.45 \text{ Km.m}^{-2}$$

$$\sigma_{zd} = \frac{\sum V}{b} \left(1 + \frac{6 \times e}{b}\right) = \frac{3597.3}{17.1} \left(1 + \frac{6 \times 0.026}{17.1}\right) = 212.29 \text{ Km.m}^{-2}$$

$$\sigma_{zu} \text{ \& } \sigma_{zd} < 3 \text{ Mm.m}^{-2} \text{ O.K.}$$

#### 4.4 Energy Dissipation Below Spillway

- **Design Energy Devices Dissipation by using a hydraulic jump**

Length of spillway = 13 m, head above spillway = 1 m, discharge at 100 year = 152.79, clear length = 70 m

$$q = \frac{Q}{B} = \frac{152.79}{70} = 2.18 \text{ m}^3 / \text{ses} / \text{m}$$

$$V_T = \sqrt{2g(Z - H/2)} = \sqrt{2 \times 9.81(13 - 1/2)} = 15.66 \text{ m/sec}$$

$$\frac{V_A}{V_T} = 0.89 \text{ from fig 2.13}$$

$$V_A = 0.89 \times V_T = 13.93 \text{ m/sec} < 20 \frac{\text{m}}{\text{ses}}$$

$$y_1 = \frac{q}{V_A} = 0.157 \text{ m}$$

$$F_1 = \frac{V_A}{\sqrt{y_1 g}} = 11.22 > 4.5$$

$F_1 > 4.5$  &  $V_A < 20$  Use **USBR II**

$$\frac{y_2}{y_1} = 0.5 \left( \sqrt{(1 + 8F_1^2)} - 1 \right), y_2 = 2.4 \text{ m}$$

##### Basin characteristic

$$L_B = 4.4 \times y_2 = 10.56 \text{ m Say } 11 \text{ m from (fig.2.15)}$$

$$\text{End sill } h_E = 1.6 \times y_1 = 0.25 \text{ m, u/s slope } 2:1V$$

$$\text{Chut blocks } h_C = y_1 = 0.157 \text{ m, spacing} = 0.157 \text{ m}$$

219 chut blocks are use

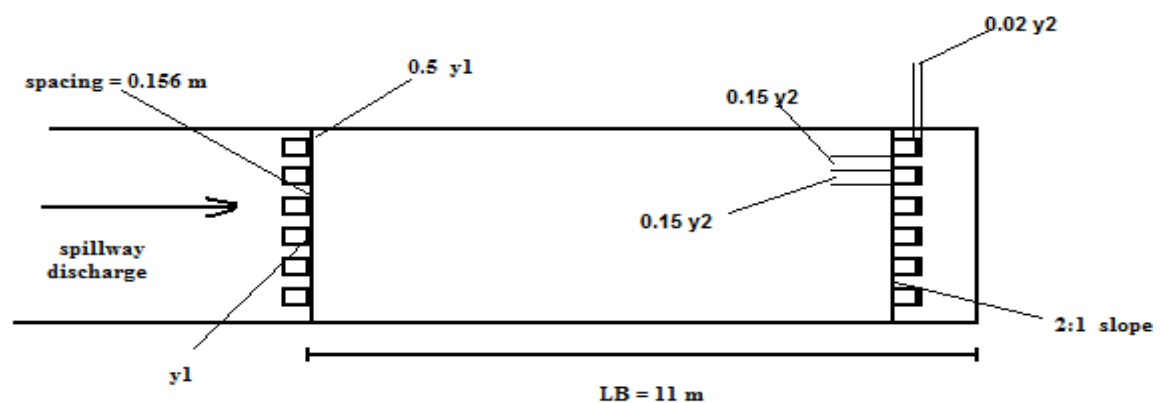
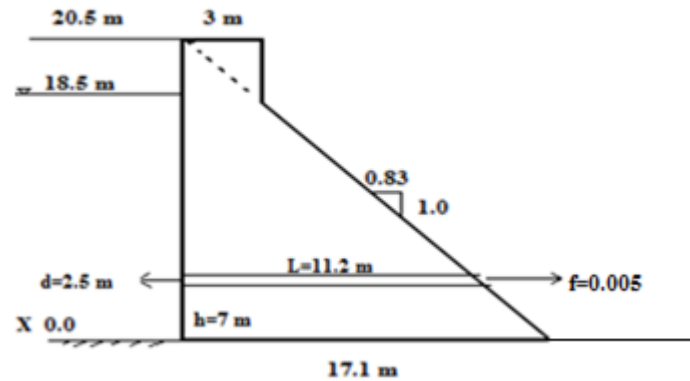


Fig 4.5 Hydraulic jump top view

## 4.5 Outlet Pipes

To distribute a flood resulting from rainfall by discharging it through the pipes over a one day period in order to make space for another flood.



**Figure 4.6** location of the pipe in the dam

$$A = \frac{\pi}{4} d^2 \longrightarrow A^2 = 24.09 \text{ m}^2$$

$$\Delta H = \frac{v^2}{2g} + H_L \text{ " Bernoulli equation "}$$

$$\Delta H = \frac{v^2}{2g} + \frac{.5v^2}{2g} + f \frac{L}{d} \frac{v^2}{2g}$$

$$12.25 = \frac{Q^2}{2gA} + \frac{.5Q^2}{2gA} + f \frac{L}{d} \frac{Q^2}{2gA}$$

$$f = a \left( 1 + .305 \frac{b}{R} \right)$$

Where a & b are empirical coefficients depend on the conduit type.

$$\text{Hydraulic radius} = \frac{A}{P}$$

A=area

P= wetted primete

$$R = \frac{4.9}{7.8} = 0.623$$

$$f = 0.005$$

$$10.25 = 0.81Q^2 + 0.4Q^2 + 0.18Q^2$$

$$Q = 2.71 \text{ m}^3/\text{s} = 234621.75 \text{ m}^3/\text{day} \text{ (for each pipe)}$$

Five pipes will be needed to increase the discharge process,

$$Q = 234621.75 \times 5 = 1173108.76 \text{ m}^3/\text{day}$$

## 4.6 Environmental Impacts of Dams

Dams are considered a major factor on how developed a country is. They serve many purposes as previously discussed. However, dams are not all pros, they have environmental impacts that need to be taken into account, which may include:-

- 1- The alteration of a wadi's flow, which affects downstream ecosystems and the landscape through which the river flows.
- 2- A dam also holds back sediments that would naturally replenish downstream ecosystems.
- 3- Large dams have led to the extinction of many fish and other aquatic species, the disappearance of birds in floodplains, huge losses of forest, wetland and farmland.
- 4- The spread of diseases, for dams are literally breeding grounds for mosquitoes, snails, and flies, the vectors that carry malaria, schistosomiasis and river blindness.
- 5- The emission of greenhouse gases (methane).
- 6- The displacement of human populations.

**All that is mentioned above are negative impacts that do not exist, or very minimum in our case study.**

## 4.7 Dam Operation Plan

This is a flood control dam, therefore it will not store water for long periods of time. Instead, it will distribute the water gathered from rainfall over one day periods by discharging water via spillway and outlet pipes.

As a flood control structure, dam operation plan will be as follows:

- 1- Dam should be empty before any flood events.
- 2- During the flood, outlets will be closed and only spillway is working.
- 3- After the storm is stopped and spillway stop discharging, outlets opened until dam is empty.

Hence by following the above operation plan, only part of the flood will reach the city during the peak of the flow while most of the flood will be detained behind the dam. Then the detained water will be released over a long period with a less effect on the city.

***Chapter (5)***  
***Conclusion***

## 5.1 Summary

A flood control gravity dam was designed across wadi Banban north of Riyadh city.

For a known catchment area, a unit hydrograph was established. Hydrographs for 2 to 100 years return periods were found using a predeveloped IDF curve. The dam designed for flood control with a total storage of more than one million cubic meter of water. In order to assure an effective flood control, five outlets were provided. These outlets allow stored water to be discharged within one day keeping the dam empty for the next flood and smoothing the flood hydrograph over a longer duration. Dam was checked for stability condition and found safe for the loading combination used. An Ogee spillway was designed to pass a 100 year flood without causing any damage to the structure. Also the spillway was provided with a USBR type II stilling base to dissipate excess energy.

## 5.2 Recommendations

The Designed structures cannot be considered as a final design as a number of information were assumed and need to be verified. So it is recommended to have more investigation regarding the soil type, geology of the catchment area and the land use so that a more accurate flood hydrographs may be obtained. Also, more information is needed for the wadi cross section for longer reaches and more detailed topographic maps to the dam site.

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