Addis Ababa University
Addis Ababa Institute of Technology
School of Graduate Studies
School of Civil and Environmental Engineering

Spillway Design Discharge Estimation: Its Limitation and Proposed Solution (Case study Kesem Dam)

A thesis Submitted to the School of Graduate Studies in Partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Major Hydraulic Engineering)

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DECLARATION

I, hereby declare that this thesis is my original work and has not been presented for a degree in any other university, and that all sources of materials have been duly acknowledged.

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List of Abbreviations and Acronyms

ASTER	Advanced Spaceborne Thermal Emission and Reflection Radiometer
A	Wetted Area
A	Flow Cross sectional Area
B	Water Surface Width
c	Resistance Coefficient
C	Discharge Coefficient
CP	Control Point
DEM	Digital Elevation Model
FRL	Full Reservoir Level
GIS	Geographical Information System
g	Gravitational Acceleration
GDEM	GLOBAL Digital Elevation Model
GVF	Gradually Varied Flow
1D	One Dimensional
2D	Two Dimensional
H	Design Head above Spillway Crest Length
H_a	Depth Difference Between Upstream and Downstream Water Level
IDF	Inflow Design Flood
IS	Indian Standard
I	Inflow
K_P	Pier Contraction Coefficient
K_a	Abutment Contraction Coefficient
L	Effective Crest Length
L'	Actual Crest Length
LHS	Left Hand Side
m	Meter
m^3	Cubic Meter
MWL	Maximum Water Level
MCM	Million Cubic Meter
n	Manning Roughness Coefficient
N	Number of Piers
O	Outflow
PMF	Probable Maximum Flood
R	Hydraulic Radius
RWS	Reservoir Water Surface
RHS	Right Hand Side
Sec	Second
S	Bottom Slope
SCL	Spillway Crest Level
S	Storage
Δt	Time Interval
USGS	United States Geological Survey
UTM	Universal Transvers Mercator
u	Mean Flow Velocity
V	Mean Cross sectional Velocity
WWDSE	Water Works Deign and Supervision Enterprise

Abstract

Spillway design discharge and stage are determined by the routing method chosen, and they are crucial in determining the dam crest level. Because a dam is such a large structure, its height has a substantial impact on its cost estimation, the necessity of optimum spillway capacity and design discharge estimation need worth consideration. The commonly used Modified Pul's routing method, which is widely accepted as a standard method, presupposes that there is a horizontal water surface level above the FRL which controls by the downstream structure. However, upstream boundaries such as the inlet cross sectional geometry, river bed slope, and surface roughness regulate the flow, therefore upstream control prevails at the entrance, and downstream structure controls the water surface level at the exit. As a result, there is a variation in elevation between the upstream and downstream edges, resulting in a sloped reservoir water surface. When we substitute the horizontal water surface of the usual Modified Pul's Technique assumption with this sloped water surface, the reservoir adds an extra storage that the common method does not account for, and this extra storage reduces the out-flow discharge. This research modify the common Modified Pul’s Method which tries to handle and consider this extra storage which ignored in the common method using a MATLAB computer program by taking Kesem Dam as a case study and develop a standalone application. The result shows that there is a reduction in the out-flow discharge and design head through the new modified method.

Keywords: Modified Pul’s routing; Spillway design discharge; Upstream Control; Downstream Control; Horizontal Water Surface

1. CHAPTER ONE: - Introduction

1.1. Background

Spillway is one of the most important structural components of a dam. Spillways are provided for storage, multipurpose, and detention dams to release excess water or flood water that cannot be contained in the designated storage space, as well as diversion dams to bypass flows that have been turned (redirected) into the diversion system. Normally, excess storage is pulled from the reservoir's top and transferred back to the river or a natural drainage channel by waterways or hydraulic structures (such as spillways, outlet works, canals, pipelines, and so on). (Reclamation, no date). The design of spillways requires a comprehensive knowledge of civil, structural, and hydraulic engineering, specifically with dams and hydraulic structures. Safety considerations consistent with economy, hydrology, site conditions, type of dam, purpose of dam and operating conditions are factors affecting the design of a spillway (IS: 11223-1985). Spillways are hydraulically sized to safely pass floods equal to or less than the IDF. The IDF will be equal to or less than the current critical Probable Maximum Flood (PMF)(No, Spillway and Considerations, 2014).

Spillway capacity is the capability of the spillway, as determine by its dimensions, crest level and hydraulic characteristics in disposing off water at any specific level (Reclamation, no date). The determination of adequate spillway design discharge and surcharge is of paramount importance since upon it depends on the maximum flood level attained and the consequent safety of the dam itself as also pattern of flooding upstream and downstream of the reservoir. The other main guideline in the design of spillway is Inflow design flood for the safety of the dam. It is the flood for which, when used with standard specifications of other factors, the performance of the dam should be safe against overtopping, structural failure and the spillway and its energy dissipation arrangements, if provided for a lower flood, should function reasonably well (IS: 11223-1985).

Spillways in earth dams may constitute more than 30 percent of the total dam construction costs. Hence, the importance of optimum spillway capacity estimation need worth consideration. In most design of hydraulic structures it is not practical from economic considerations to provide for the safety of the structure. It should be noted that because of the flood routing process in the reservoir, peak outflow varies for different spillway design parameters. One of the major problems in the field of water resources is the prediction of stages, discharges, and other flow characteristics of floods in natural streams and rivers. The determination of such unsteady flow phenomena is referred to as "flood routing". Flood routing is the technique of determining the flood hydrograph at a section of a river by utilizing the data of flood flow at one or more upstream sections. The hydrologic analysis of problems such as flood forecasting, flood protection, reservoir design and spillway design invariably include flood routing. The common method that most dams in Ethiopia designs through a reservoir / hydrologic routing method is called Modified Pul's which based on a finite difference approximation of the continuity equation coupled with an empirical representation of the momentum equation and utilizes the simple concept that storage is a function of outflow. Typically determination of a unique storage-outflow relationship is a key factor in the application of the Modified Pul's method. The method was developed by U.G. Pul's and accepted as a standard is called also the "storage indication method" which is superior to graphical methods. Correct computation of the outflow hydrograph rests on the assumption that the storage depends primarily, so the estimation of capacity of the reservoir needs special consideration. Modified Pul's also known as Level pool routing which is a procedure for calculating the outflow hydrograph from a reservoir with a horizontal water surface assumption. This horizontal surface controls by the downstream and extends to the upstream as it is, but practically and logically thinks at the upstream the discharge filling the storage depends on geometric characteristics, bottom roughness, inflow discharge and bed slope. In general in flood routing a full unsteady flow routing (1D or 2D) will be more accurate. This method can capture the water surface slope through the pool as the inflowing hydrograph arrives, as well as the change in water surface slope that occurs in a long distance reservoirs. This is the main limitation of the common Modified Pul's Method. Reservoirs with long narrow pools will exhibit greater water surface slope upstream of the dam than reservoirs that are wide and short. Therefore, the most accurate modeling technique to capture pool elevations and outflows of long narrow reservoirs is full dynamic wave (unsteady flow) routing. For wide and short reservoirs, level pool routing may be appropriate. Even if this papers study area Kesem

Reservoir is long reservoir it designed by the common Modified Pul's Method because of its simplicity so this paper tries to improve the limitation of the method and modify the method by considering the storage occurred by the water surface slope because using the fully dynamic routing models are time-consuming and data intensive, modifying the level pool method reduces the limitation of the common method and estimate the stage and outflow hydrograph simply. The level pool routing procedures are generally accepted in reservoir and storage design.

The maximum design discharge of the spillway is determined by studying the flood hydrograph of some past years and the maximum flood discharge may be computed which is to be disposed of completely through the spillways. The water level in the reservoir should never be allowed to rise above the maximum pool level and should remain in normal pool level, so the volume of water collected between maximum pool level and minimum pool level computed which indicates the discharge capacity of spillways. The maximum flood discharge may also be computed from other investigation like rainfall records, flood routing, empirical flood discharge formula, etc. From the above factors the highest flood routing discharge is ascertained to fix the design discharge of spillways. The natural climates are beyond the grip of human being so an allowance of about 25% should be given to the computed highest flood discharge which is to be disposed of. The size of spillways is designed according to the design discharge. When we compute the spillway design discharge even over the peak inflow design flood which is recorded at instantaneous time it takes much more time to fill the total determined volume of water of the spillway capacity. For example in Kesem Dam the maximum inflow design flood is 9237.7m3/sec the spillway capacity surcharge storage) is about 320MCM which takes about 9.6hr is needed to fill the wall volume in the surcharge storage (WWDSE 2005). Which means the spillway design discharge of the dam is much higher related to the incoming peak inflow design flood and it's logically impossible to fill computed volume at an instant time. This shows there is an over estimated design head which may exaggerate the dam height. The design of our dams are much safe but not efficient. As mentioned earlier spillway takes 30% of the total dam cost and the height of the dam which depends on the spillway height, design head and freeboard also a significant impact on the dam cost estimation. Most Ethiopian dam spillways have this design limitation and investigations needed in this area to understand the case and estimate an optimum spillway design discharge. The flood routing process of the dam which determining the flood hydrograph needs investigation to consider the dynamics of the water or flood wave movement as a function of both time and space in the estimation of spillway design discharge is one of the solution, but also investigating and modifying the usual or common Modified Pul's Method is the simplest option and proposed solution in this research to design the safe and efficient out-flow discharge.

1.2. Statement of the Problem

Flow routing is used in the design of reservoirs and their outflow structures, such as spillways and low-level outlet pipes. It's a crucial phrase for calculating spillway design discharge. The storage indication or Modified Pul's approach, which is based on storage outflow relation, is the most often used routing method in our country for spillway design discharge estimation. For reservoirs with a horizontal water surface, the approach has a unique (invariable) relationship between storage and outflow holds. The downstream outlet structure controls the presumed horizontal surface above the spill-way crest level (SCL), implying that the surface rise in the downstream extends upstream with zero slopes or at the same elevation. The method adopt simpler one-dimensional models based on the level pool reservoir (or uniform storage) hypothesis, but in fact, the flow is controlled at the entrance by the discharge filling the storage with its geometric characteristics, river bed slope, and bottom roughness. In reservoirs having a pool that is wide and deep compared with their length in their flow direction the first flow reaches to the outlet with in short period of time and the outflow begins at the same time inflow begins so, presumably, the inflow at the head of the reservoir passes instantaneously through the reservoir regardless of its short length. But when this situation seldom exists on larger reservoirs the first flow takes some time to reach to the outlet. Reservoirs with long narrow pools will exhibit greater water surface slope at the upstream of the dam when inflow hydrograph arrives, so in large reservoirs the Modified Pul's Method has some unrealistic approach. This limitation of the common Modified Pul's Method highly affects the reservoir capacity and add extra temporarily floodplain storage which allows a part of the flood volume to be temporarily stored but not considered in this routing method. This neglected extra storage is high in the long-distance reservoirs which can reduce the outflow discharge. As such the common Modified Pul's method usually exaggerates the design flood discharge estimated which in turn make the design flood level and hence the dam height to be larger. In most cases to safely accommodate flood events the preferred option will be a combination of providing reservoir flood storage and spillway discharge capacity so if the extra storage created by the water surface slope created from reservoir inflow towards the spillway location is considered, such larger design flood would have been reduced. Thus, this research will try to modify the concept of Modified Pul's Method so that the additional storage created by the water surface gradient (from upstream entrance towards the spillway) is carefully handled and thus the design discharge is properly estimated. This research will take Kesem Reservoir as a case study site and discuss the possible improvement on the estimated design discharge.

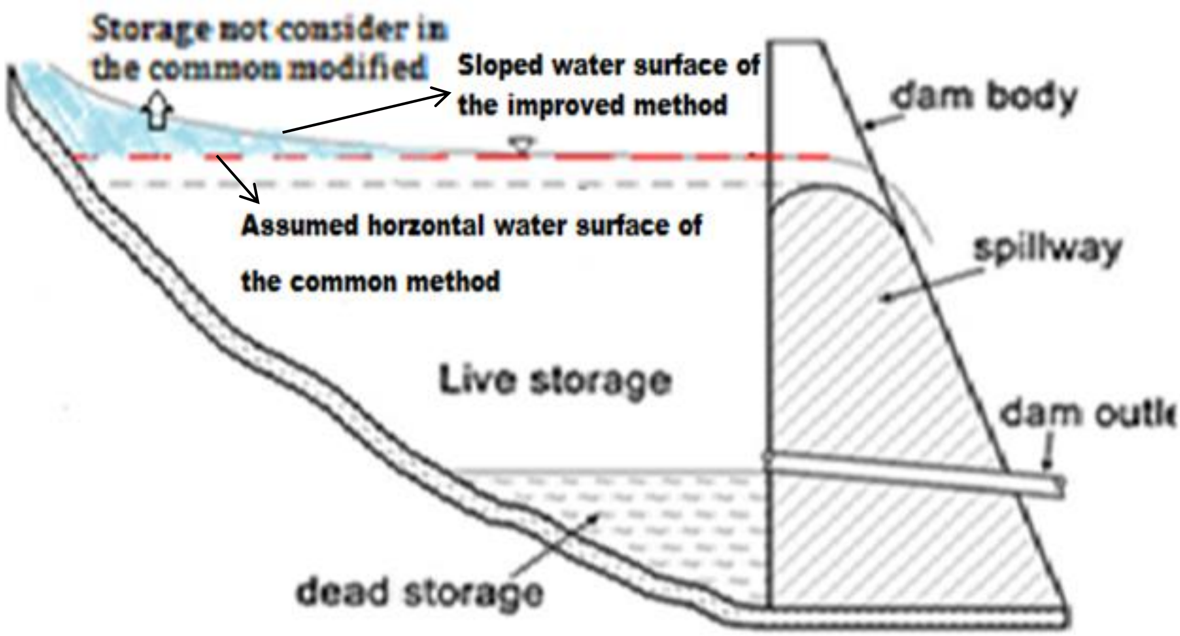


Figure 1.1 The water flows along a reservoir and over a spillway

1.3. Research Questions

- What are the limitations in spillway design discharge estimation?
- What are the proposed solutions for the spillway design discharge estimation limitations?
- Can we create a standalone application which modifies the common Modified Pul's routing method for spillway design discharge determination?

1.4. Objective of the Research

1.4.1. General Objective

The main objective of this research is to assess the limitation of spillway design discharge estimation and its proposed solution.

1.4.2. Specific Objectives

- To assess the limitations of the common Modified Pul's routing method
- To propose alternative routing solution for spillway design discharge estimation
- To create a standalone application for spillway size/capacity estimation

1.5. Relevance of the Research

The goal of this research was to alter the common Modified Pul's approach to estimate the accurate out-flow discharge of a spillway. The findings of this research can be used to calculate the design head above the spillway crest level, which is utilized to calculate dam height. This is significant in dam design because, as we all know, a dam is a massive and huge structure, and even a small change in dam height may save a lot of money. The research aims to take into account the storage that is often ignored in the Modified Pul's approach, which reduces the out-flow discharge.

1.6. Scope and Limitation of the Research

The research focuses on the spillway design flood estimation by taking Kesem Dam as a study area which is one of the largest dams in Ethiopia through improving the common Modified Pul's routing method. Aiming at the objective, this research did not take into consideration the inflow design flood evaluation and it has no contradiction with the previously estimated inflow design flood takes it directly from the design report for this research. Moreover, it was beyond the scope of this research to identify the exact water surface slope exerted by the upstream control condition it needs other experiment and investigation using gradually varied flow assumption. The research also did not take into account the areal change and effect of the irregular shaped reservoir on storage determination which occurred due to the rise in elevation at the upstream side.

1.7. Outline of the Research

The thesis is organized into five chapters from introduction to the conclusion and recommendation. The first and the second chapters' deals with introduction, statement of the problem, relevance of the research, scope of the research, the objective of the research, limitation of the research and the literature review. The third and fourth chapters illustrates the method, material and procedure, the description of the research area, data used in the research and its analysis, the result of the analysis and evaluates the result. The last chapter of the paper concentrates on the conclusion and recommendation of the thesis which concludes the research and recommends the possible solutions.

2. CHAPTER TWO: - Literature Review

2.1. Spillway Design Approach

2.1.1. Spillway Function

Spillway are provided for storage and detention dams to release surplus or flood water which cannot be contained in the allotted storage space and at diversion dams to by-pass flows exceeding those which are turned into the diversion dam. Spillways are classified according to their feature either as it pertains to the discharge carrier or some other component. It may be gated or ungated. The primary function of spillway is to release surplus waters from the reservoir in order to prevent overtopping and possible failure of the dam. In some case, dam get overflow of water which may damage the whole design and structure if we do not discharge excess water from it. The water discharged over the spillway of a dam attains a very high velocity due to its static head, which is generally much higher than the safe non-eroding velocity in the downstream. This high velocity flow may cause serious scour and erosion of river bed downstream. To dissipate this excessive energy and to establish safe flow conditions in the downstream of a dam spillway, energy dissipaters are used as remedial devices (Reclamation, no date).

2.1.2. Dam Classification

The dams may be classified according to size by using the hydraulic head (from normal or annual average flood level on the downstream to the maximum water level) and the gross storage behind the dam as given below. The type of the dam influences the design flood and spillway. Overall size classification for the dam would be greater of the following two parameters (IS:11223-1985).

Table 2.1 Classification of Dam

Classification	Gross Storage	Hydraulic Head
Small	Between 0.5 and 10 million m3	Between 7.5 m and 12 m
Intermediate	Between 10 and 60 million m3	Between 12 m and 30 m
Large	Greater than 60 million m3	Greater than 30 m

The inflow design flood for safety of the dam would be as follows:

Table 2.2 IDF for Safety of Dam

Size as determined above	Inflow design flood for safety of dam
Small	Generally 50 years return period flood should be adopted for design of surplus sing arrangement. Where dam breach may cause loss of human lives or great damage to property etc., the inflow design flood may be adopted as per IS 11223: 1985
Intermediate	Standard project flood (SPF)
Large	Probable maximum flood (PMF)

2.1.3. Factors Affecting Spillway Design

There are some factors which affect spillway design. Many failures of dams have resulted from improperly designed spillway or spillways of inadequate capacity. Structures with proper design of adequate capacity may be found to be only moderately higher in cost than a structure of inadequate capacity. The spillway design depends on Inflow discharge, its frequency, and shape of hydrograph, height of dam, capacity curve, geological and other site condition, important topographical features which are: Steepness of terrain, amount of excavation and possibility of its use as embankment material, the possibility of scour, stability of slopes, safe bearing capacity and permeability of soils. The type of dam has also another effect which influences the design flood and spillway design. Earth and rock fill dams have to be provided with ample spillway capacity (IS:11223-1985). Earth and rock fill dams are likely to be destroyed if overtopped; whereas, concrete dams may be able to withstand moderate overtopping. Usually, the increase in cost is not directly proportional to the increase in capacity. The cost of a spillway having ample capacity is often only moderately higher than the cost of a spillway that is too small. In addition to providing sufficient capacity, the spillway must be hydraulically and structurally adequate and must be located so that spillway discharges do not erode or undermine the downstream toe of the dam. The spillway's bounding surfaces must be erosion resistant to withstand the high scouring velocities created by the drop from the reservoir surface to the tail water level. Usually, a device is required to dissipate the energy of the water at the bottom of the drop.

The frequency of spillway use should be determined by the runoff characteristics of the drainage basin, which includes the nature of its development. Ordinary river flows are usually stored in the reservoir, diverted through head works, or released through outlets; the spillway is not required to function. However, spillway flows do occur during floods or periods of sustained high runoff when the capacities of the other facilities are exceeded. Where large reservoir storage is provided or large outlet or diversion capacity is available, the spillway will be used infrequently. But at diversion dams where storage space is limited and diversions are relatively small compared with normal river flows, the spillway will be used almost constantly(Reclamation, no date).

Dams impounding large reservoirs on principal rivers with high runoff potential should unquestionably be considered to be in the high-hazard category. For such developments, conservative design criteria should be selected because failure could involve the loss of life or damages of disastrous proportions. Conversely, small dams built on isolated streams in rural areas where failure would neither jeopardize human life nor create damages beyond the sponsor's financial capabilities may be considered to be in a low-hazard category. For such developments, design criteria may be established on a much less conservative basis. There have been numerous instances, however, where the failure of a small dam with small storage capacity has resulted in the loss of life and heavy property damage. "Most small dams require a reasonable conservatism in design, primarily because a failure must not present a serious hazard to human life(Reclamation, no date).

2.2. Spillway Design Discharge

Spillway capacity allows efficient passage of flood flows without any significant increase in flood pond elevation which prevents dangerous overtopping of dam embankments or flood damage to upstream property. Estimating design discharge is based on either analytical methods or physical models. Analytical methods will typically be used for all levels of design (appraisal, feasibility, and final design levels), while physical models are typically limited to final design levels. Furthermore, physical models are usually employed for typical designs involving unusual topography, geometry, and/or discharges or velocities that exceed experience levels. Once design discharge has been determined, it is usually presented in the form of drawings (discharge curves) and/or tables with discharges related to RWS elevations which are elevation capacity curve. Determining the hydraulic control(s) is a key in the estimation of discharge capacity for the full range of spillway operation (i.e. full range of RWSs that would invoke spillway releases) (No, Spillway and Considerations, 2014).

2.2.1. Crest Control (Uncontrolled or Free Flow)

Spillway crest is the highest elevation of the floor of the spillway along a centerline profile through the spillway. A controlled spillway is one which provided with the gates over the crest to regulate the rate of flow over the crest. Crest control occurs when there is a free (water) surface and subcritical flow conditions exist upstream of the control structure (such as an ogee crest structure), then pass through a critical state (Reclamation 2009). (i.e. when the Froude number is equal to unity or when the specific energy is at a minimum for a given discharge) at the control structure to a supercritical flow condition downstream of the control structure. The governing equation for crest control is the weir equation (No, Spillway and Considerations, 2014)

Weir equation

$$Q = CLH^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \ldots (1)$$

For control structures with piers which used to support bridges over wide spillways or needed to partition spillway bays to accommodate gates, stop logs, or bulkheads and abutments that cause side contractions of flow, the effective crest length (L) is less than the actual crest length (L'). The effective crest length (L) can be determined by the following equation: (No, Spillway and Considerations, 2014)

Effective crest length

$$L = L' - 2(Nk_P + k_a)H_o \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where N is the number of piers, K_p pier contraction coefficient, and K_a abutment contraction coefficient

2.2.2. Orifice Control (Controlled Flow)

Orifice flow control is where an orifice plate helps to control the flow of water into a sewer or reduce the flowing pressure downstream of the orifice plate. The use of a fixed restriction orifice can be beneficial and economical by reducing the demands on other flow system components. A constriction of the wetted area (such as a partially opened gate) between the upstream reservoir and downstream conveyance features (such as a chute or a conduit that is free flowing, not pressurized) creates a pressure and velocity change. The governing equation is derived from the Bernoulli and continuity equations (No, Spillway and Considerations, 2014).

Orifice equation

$$Q = CA\sqrt{2gH_a} \quad \ldots\ldots (3)$$

2.2.3. Pipe Control (Pressurized Flow)

Pipe control (pressurized flow) exists when rather than a free (water) surface, the water is confined in a closed system (such as a conduit or tunnel) between the upstream reservoir and downstream river channel, creating pressure and velocity change. The governing equation is derived from the Bernoulli and the continuity equations (No, Spillway and Considerations, 2014).

Form of Bernoulli equation

$$Q_2 = Q_1\sqrt{\frac{H_2}{H_2}} \rightarrow Q_2 = K_2\sqrt{H_2} \quad \ldots\ldots (4)$$

2.3. Existing Spillway Capacity

The primary source for existing spillways of a given dam for current spillway discharge capacity information is the standard operating procedure. The discharge capacities found in the standard operating procedure represent existing operating conditions and will typically provide adequate information unless operational and/or physical changes are being considered, such as raising the normal or flood-induced maximum RWS, or modifying or replacing features of the existing spillway. In these cases, the existing discharge capacity should be reevaluated and (if needed) re estimated. Another source for existing spillway discharge capacities information is physical (hydraulic) model study reports, which are available for many reclamation facilities. Also, actual flow measurements from river gages, flow meters, or other measuring devices can be used to verify existing discharge curves or to develop discharge curves. For existing spillways discharge capacity information may not always be available. For this case, estimates will be developed using either analytical methods or physical models. The hydraulic control(s) for the existing spillway is determined so that the discharge capacities can be estimated. Finally, it is stressed that when evaluating existing spillway discharge capacity, attention should be given to the possibility of a hydraulic control shift if hydraulic heads greater than the maximum design head could occur (No, Spillway and Considerations, 2014)

2.4. Elevation Area Capacity Curves

For planning and operation purposes reservoir elevation area capacity curves are important. Topographic survey of the reservoir area should form the basis for obtaining these curves, which are respectively the plots of elevation of the reservoir versus surface area and elevation of the reservoir versus volume. The relation between stage and storage can be estimated using topographical maps. For preliminary studies, in case suitable topographic map with contours, say at intervals less than 2.5 m is not available, stream profile and valley cross sections taken at suitable intervals may form the basis for computing the volume. Aerial survey may also be adopted when facilities are available (Reclamation 2009). The original reservoir elevation-area-capacity curve at a dam site can be prepared from the available topographical maps. The incremental volume between any two contour elevations and live capacity of reservoir are calculated using the formula,

$$\Delta V_i = \frac{\Delta h\left(A_i + A_{I+1} + \sqrt{A_i A_{i+1}}\right)}{3} \quad \ldots \ldots (5)$$

$$V_i = \sum_{k=}^{i} \Delta V_k \quad \ldots \ldots (6)$$

$$V_i = \sum_{I=1}^{N-1} \Delta V_i \quad \ldots \ldots (7)$$

where

ΔV_i = Volume between contour elevations i and $i+1$,

Δh = Contour interval,

A_i = Area at contour elevation i,

A_{i+1} = Area at contour elevation 1+i,

Ya = Live capacity of reservoir,

N = Number of contour elevations.

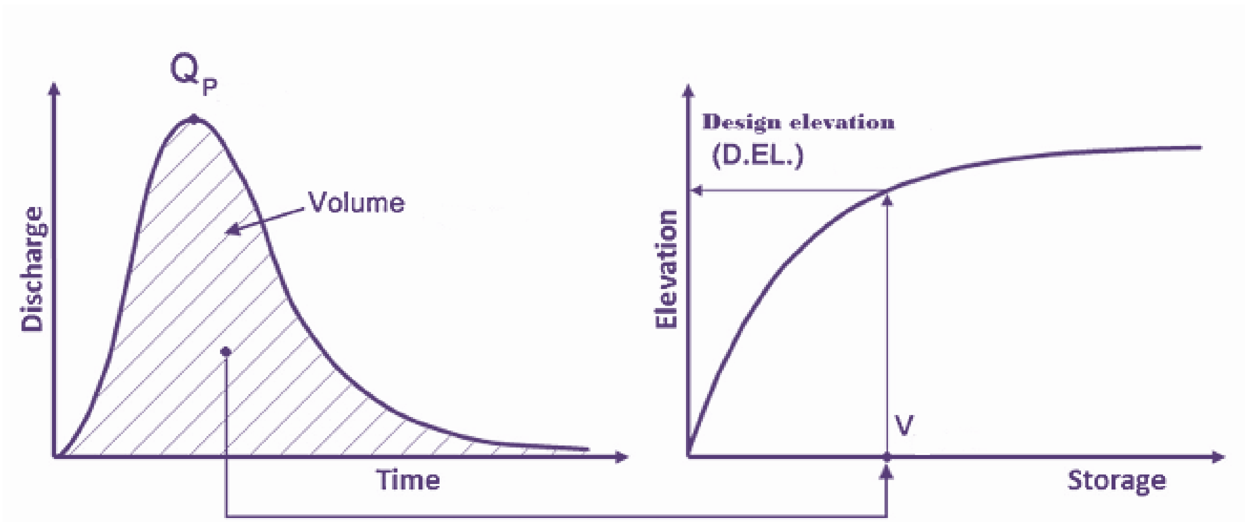


Figure 2.1 Estimation of the design elevation of the spillway based on the dam capacity (Samy, Jarbou & Amro 2018)

2.5. Reservoir Storage Zone and uses of Reservoir

A reservoir is an artificial lake where water is stored. It can also be formed from a natural lake whose outlet has been dammed to control the water level. The dam controls the amount of water that flows out of the reservoir. The role of water-storage reservoirs is to impound water during periods of higher flows, thus preventing flood disasters, and then permit gradual release of water during periods of lower flows. The capacity, volume or storage of a reservoir is usually divided into distinguishable area. It is nationally divided into three or four parts (Figure2.2) distinguished by corresponding levels. Dead or inactive storage refers to water in a reservoir that cannot be drained by gravity through a dam outlet works, spillway, or power plant intake and can only be pumped out.

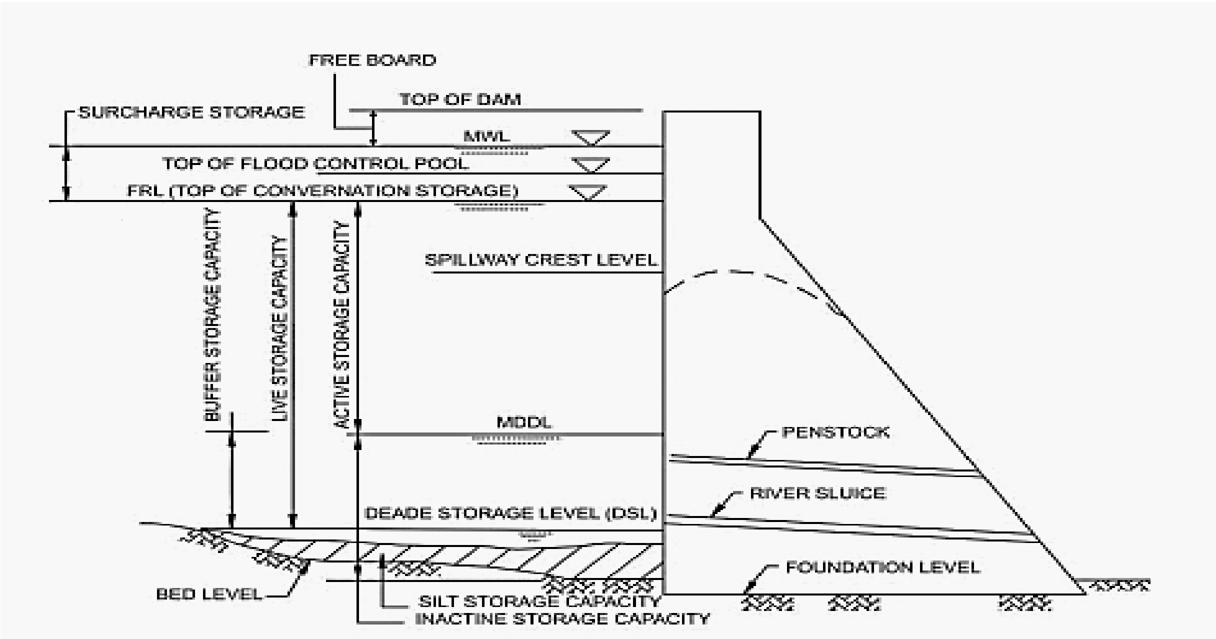


Figure 2.2 Schematic diagram showing storage zones (Version 2 CE IIT, Kharagpur)

2.6. Fixing of Flood and Surcharge Storage

In case of reservoirs having flood control as one of the purposes, separate flood control storage is to be set apart above the storage meant for power, irrigation and water supply. Flood control storage is meant for storing flood waters above a particular return period temporarily and to attenuate discharges up to that flood magnitude to minimize effects on downstream areas from flooding. Flood and surcharge storage between the full reservoir level (FRL), and maximum water level (MWL) attainable even with full surplus sing by the spillway takes care of high floods and moderates them (Reclamation 2009).

2.6.1. Flood Control Storage

Storage space is provided in the reservoir for storing flood water temporarily in order to reduce peak discharge of a specified return period flood and to minimize flooding of downstream areas for all floods (IS: 5477 (Part 1) : 1999) equal to or lower than the return period flood considered. In the case of reservoirs envisaging flood moderation as a purpose and having separate flood control storage, the flood storage is provided above the top of conservation pool.

2.6.2. Surcharge Storage

Surcharge storage is the storage between the full reservoir level (FRL) and the maximum water level (MWL) of a reservoir which may be attained with capacity exceeding the reservoir at FRL to start with. The spillway capacity has to be adequate to pass the inflow design flood making moderation possible with surcharge storage. The methods that are generally used for estimate of the Design Flood for computing the Flood Storage are broadly classified as under:

1. Application of a suitable factor of safety to maximum observed flood or maximum historical flood.
2. Empirical flood formulae.
3. Envelope curves.
4. Frequency analysis.
5. Rating method of derivation of design flood from storm studies and application of the Unit Hydrograph principle.

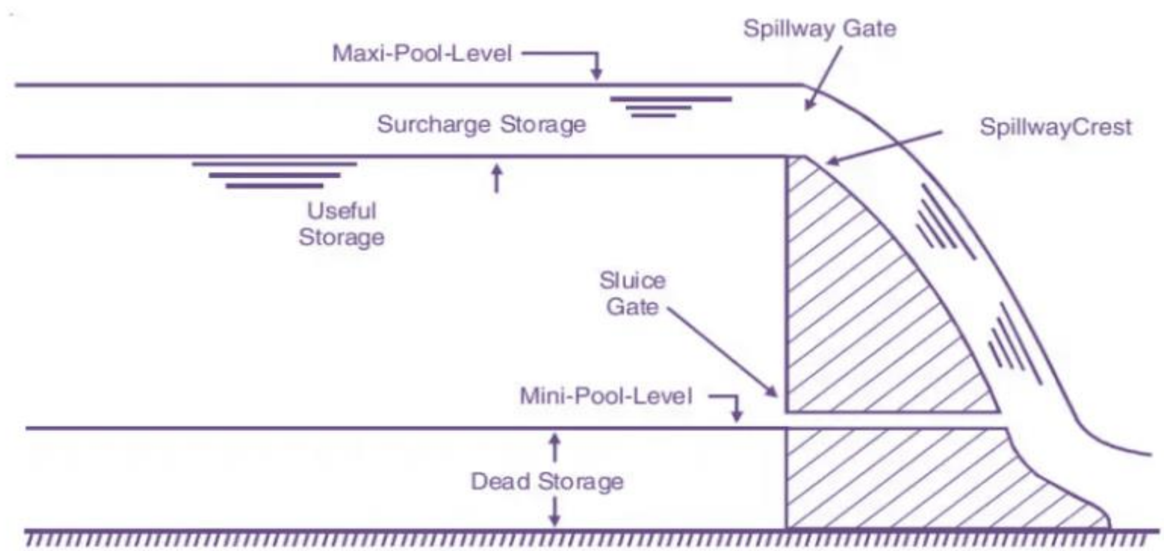


Figure 2.3 A typical dam reservoir and its storage components (Civilengineering Subjects tutorial 2020)

2.7. Reservoir Upstream Control

Flow that varies in depth and velocity along the channel is called non-uniform. Because of changes in channel section, slope, or roughness which causes the depths and average velocities of flow to vary from point to point along the channel, and the water surfaces will not be parallel to the streambed uniform flows rarely exists in either natural or man-made channels. Although moderate non-uniform flow actually exists in a generally uniform channel, it is usually treated as uniform flow in such cases. Uniform flow characteristics can readily be computed and the computed values are usually close enough to the actual for all practical purposes. We can assume that far upstream in the channel the flow is very nearly uniform. That sloping water surface upstream has to pass continuously into the horizontal water surface of the reservoir, where the water velocity is negligible. Question like what would the water-surface profile look like along a stream wise vertical cross section through the channel and the reservoir? Would it change very gradually, all the while sloping monotonically down toward the reservoir? Or would it continue unchanged all the way to the reservoir level, to meet the water surface in the reservoir by an abrupt change in water-surface slope? Are important to know how the water inters to the reservoir('CHAPTER 5 open-channel flow', 2006).

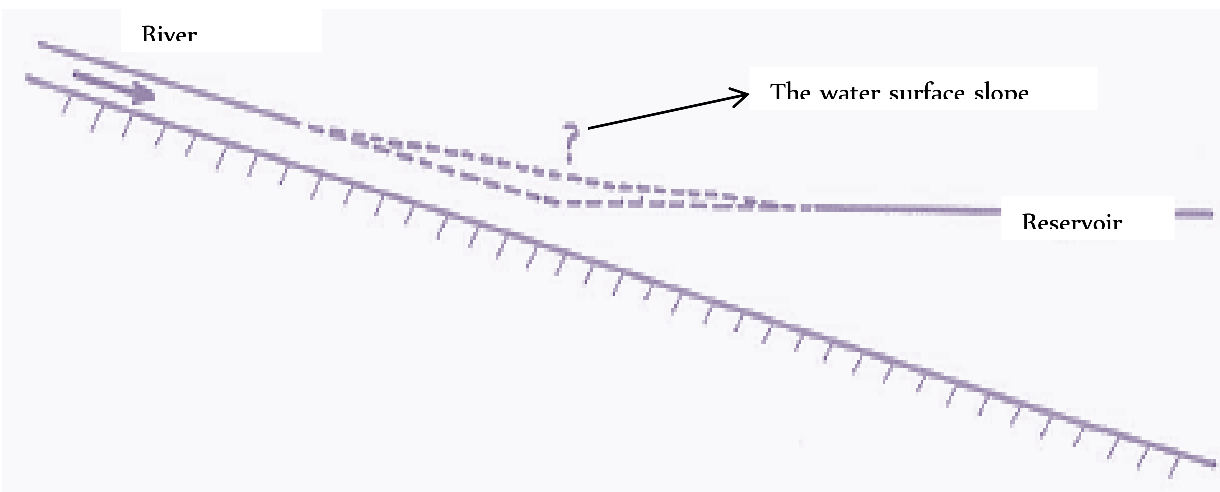


Figure 2.4 A typical dam reservoir and its storage components (MIT OpenCourseWare)

The types of non-uniform flow are innumerable, but certain characteristic types are more common. With subcritical flow, a change in channel shape, slope, or roughness affects the flow for a considerable distance upstream, and thus the flow is said to be under downstream control. If an obstruction, such as a culvert, causes ponding, the water surface above the obstruction will be a smooth curve asymptotic to the normal water surface upstream and to the pool level downstream.

Another example of downstream control occurs where an abrupt channel enlargement, as at the end of a culvert not flowing full, or a break in grade from a mild to a steep slope, causes a drawdown in the flow profile to critical depth. The water surface profile upstream from a change in section or a break in channel slope will be asymptotic to the normal water surface upstream, but will drop away from the normal water surface on approaching the channel change or break in slope. In these two examples, the flow is non-uniform because of the changing water depth caused by changes in the channel slope or channel section. Direct solution of open-channel flow by the Manning equation or by the charts in this section is not possible in the vicinity of the changes in the channel section or channel slope. With supercritical flow, a change in the channel shape, slope, or roughness cannot be reflected upstream except for very short distances. However, the change may affect the depth of flow at downstream points; thus, the flow is said to be under upstream control.

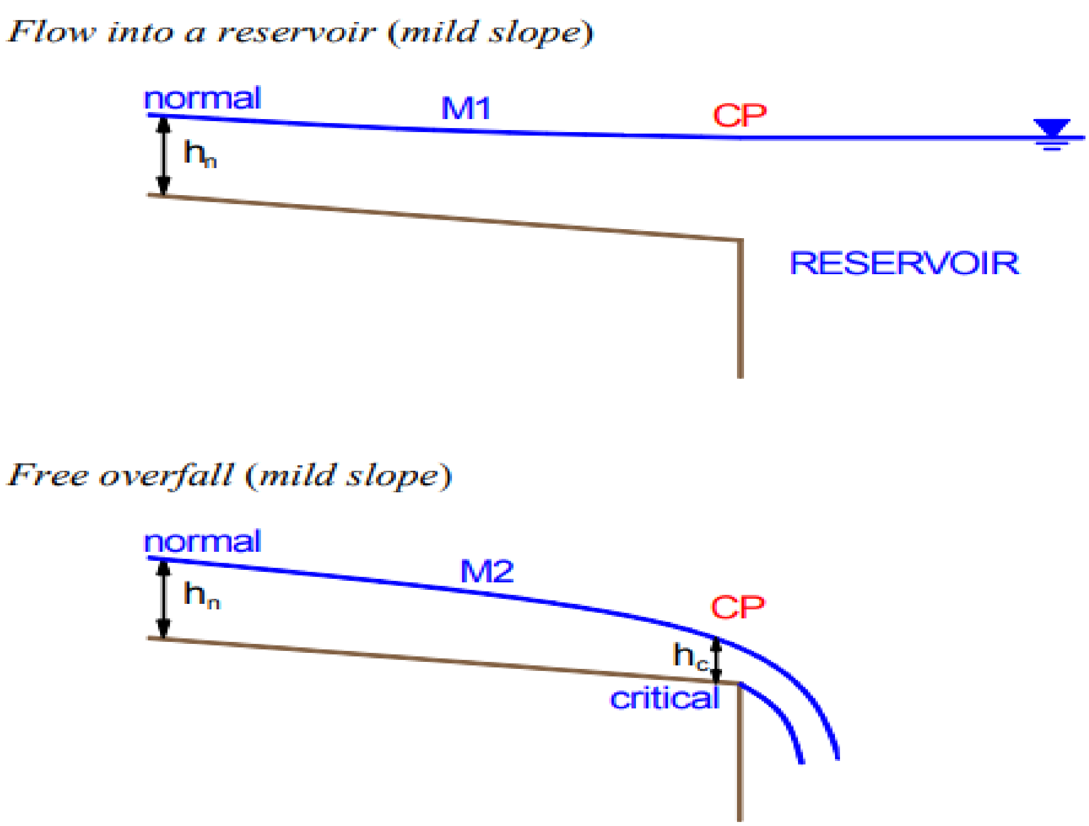


Figure 2.5 Flow in a mild slope with control point (Apsley 2021)

Figure 2.6 Forward in x if the flow is supercritical (Upstream Control) (Apsley 2021)

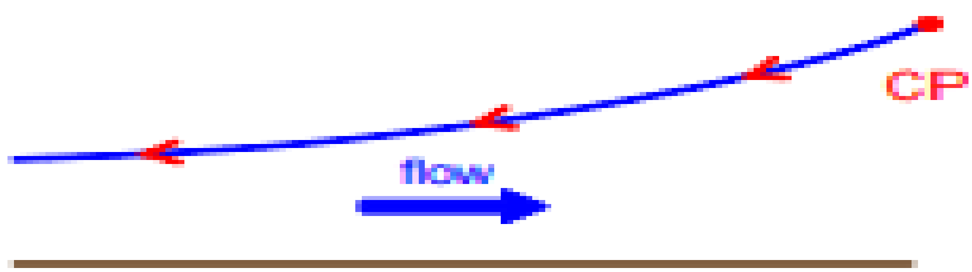


Figure 2.7 Backward in x if the flow is supercritical (Downstream control) (Apsley 2021)

2.8. River Open Channel Flow Behavior

An open channel is a waterway, canal or conduit in which a liquid flows with a free surface. An open channel flow describes the fluid motion in open channel. In contrast to pipe flows, open channel flows are characterized by a free surface which is exposed to the atmosphere. The pressure on this boundary thus remains approximately constant irrespective of any changes in the water depth and the flow velocity. A control point is a location where there is a known relationship between water depth and discharge (aka "stage-discharge relation"). Critical-flow points (weirs, venture flumes, sudden changes in slope, free over fall), sluice gates, entry or discharge to a reservoir. Control points often provide a location where one can start a Gradually Varied Flow (GVF) calculation; i.e. a boundary condition (Rvf, 2021).

Some general rules:

- Supercritical ⇒ controlled by upstream conditions.
- Subcritical ⇒ controlled by downstream conditions.

The Manning and Chezy formulas are resistance equations for steady, uniform, open channel flow. They describe how flow velocity depends on channel friction and slope. The Chezy formula was developed in 1769 by Antoine Chezy and was originally used for the purpose of designing a canal in the Paris water supply (Henderson 1966 and French 1986). The formula can be derived by combining the force balance of any water element in the channel with dimensional analysis of the bottom shear stress (Henderson 1966). It is given by French (1986) as:

$$u = C\sqrt{RS} \quad \text{(8)}$$

where u is the mean flow velocity (m/s), C $(\sqrt{m}/s)$ is a resistant coefficient, R (m) is the hydraulic radius and S (m/m) is the bottom slope.

The Manning formula was developed by Robert Manning in 1889 and was based on empirical curve fitting (French 1986 and Chow, et al., 1988). The formula can be written as (French 1986):

$$u = \frac{1}{n} R^{\frac{2}{3}} s^{\frac{1}{2}} \quad \text{(9)}$$

where $n(s/m^{(1/3)})$ is the Manning roughness coefficient. The Manning formula is widely used, because of its simplicity and reliability, and the Chezy formula is still used in some European countries (Henderson 1996 and Gordon, et al., 1992). The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n values.

2.9. River Roughness Coefficient and Slope over View

Roughness coefficient and slope of the river are the two most preferable natural factors which govern the passage of a discharge throughout a river or a channel (Chow, Maidment & Mays 1988).

Roughness coefficients represent the resistance to flood flows in channels and flood plains. The results of Manning's formula, an indirect computation of stream flow, have applications in flood-plain management, in flood insurance studies, and in the design of bridges and high-ways across flood plains. The roughness values for channels are determined by evaluating the effects of certain roughness factors in the channels. According to the united states geological survey water-supply paper 2339 guideline for selecting manning's roughness coefficients for natural channels and flood plains —the roughness value for a flood plain channels and rivers may vary from 0.011 to 0.07(States, no date).

2.10. Continuity Equation

The conservation of mass principle is simple and is regarded as the most useful physical principal in hydrologic analysis. Equations expressing this conservation principle, so called "continuity equations", can be developed for a fluid volume, flow cross-section, and a point within a flow channel (Chow, Maidment & Mays 1988). The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of steady flow of an incompressible fluid, it assumes the simple form:

$$Q = A_1V_1 = A_2V_2 \qquad (9)$$

where:

A=flow cross-sectional area,ft^2

V=mean cross-sectional velocity,ft/s(measured perpendicular to cross-section)

The subscripts 1 and 2 refer to successive cross-sections along the flow path.

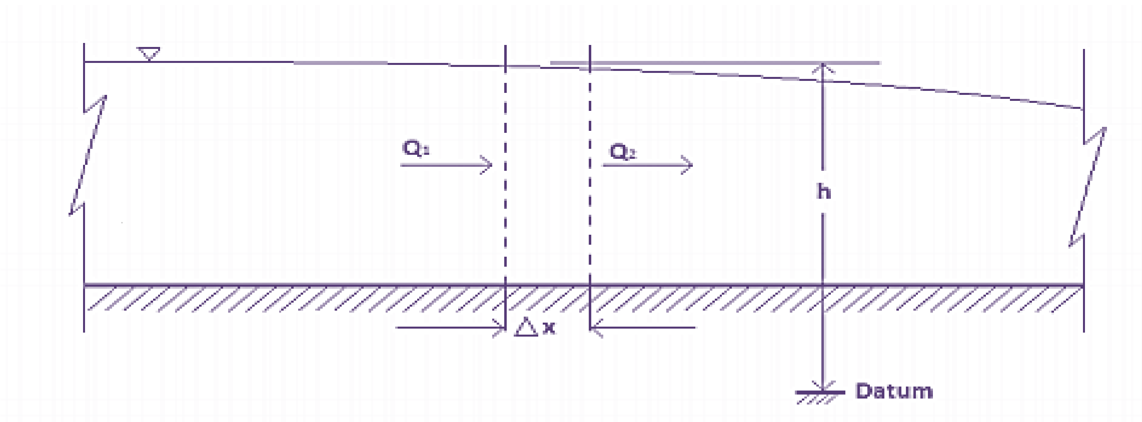


Figure 2.8 Definition sketch for the continuity equation(Henderson, 1966)

An example of where the continuity principle can be applied is that of a river stretch with a changing water-surface level channel (Henderson, 1966). Consider the river section shown in Figure2.8 with a section of a very short length Δx . The discharges at the two ends(Q_1, Q_2) do not have to be the same and will differ according to Equation_10:

$$Q_2 - Q_1 = \frac{\partial Q}{\partial x}\Delta x \quad \ldots\ldots (10)$$

The term on the right-hand side of Equation_10 gives the rate at which the volume within this region is decreasing. If h is the height of the water surface above a certain datum level, then the volume of water in the section with length is increasing with a rate according to (Henderson, 1966):

$$B\frac{\partial h}{\partial t}\Delta x \quad \ldots\ldots (11)$$

where B is the water surface width. The two terms that were presented in the expression above and Equation_10 must be equal in magnitude but with opposite signs, which together result in the equation of continuity for unsteady open channel flow (Henderson, 1966)

$$\frac{\partial Q}{\partial x} + B\frac{\partial h}{\partial t} = 0 \quad \ldots\ldots (12)$$

The continuity equation can be used with Manning's equation to obtain steady uniform flow velocity as:

$$V = \frac{Q}{A} = \frac{1.49\left(R^{\frac{2}{3}}\right)\left(A^{\frac{1}{2}}\right)}{n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

2.11. Flow Routing

Flow routing is a procedure to determine the time and magnitude of flow (i.e. the flow hydrograph) at a point on a watercourse from known or assumed hydrographs at one or more points upstream if the flow is a flood, the procedure is specifically known as flood routing. In a broad sense, flow routing may be considered as an analysis to trace the flow through a hydrologic system, given the input. As the flow through a water body changes due to any kind of disturbance, so does the water level and thereby the water storage in the water body. The change in flow over time can be visualized in a hydrograph and seen as a flood wave that propagates in the direction of the flow. The hydrograph will, depending on the properties of the reservoir or channel, change as it propagates. Describing this change of the hydrograph along the water course is referred to as routing the flow along its course. The routing is often divided into two effects, time lag and flow attenuation. The time lag refers to the time lag between two hydrographs caused by the fact that it takes time for a water volume to travel from an upstream location to a downstream location. The attenuation refers to the change in the shape of the hydrograph as it propagates caused by the storage capacity of the reservoir or friction and irregularities of a channel reach (Henderson 1966 and Subramanya 2008). The difference between lumped and distributed system routing is that in a lumped system model, the flow is calculated as a function of time alone at a particular location. While in a distributed system routing the flow is calculated as a function of space and time throughout the system. Routing by lumped system methods is sometimes called hydrologic routing, and routing by distributed systems methods is sometimes referred to as hydraulic routing (Chow, Maidment & Mays 1988). Reservoir routing and channel routing are the two broad categories of flow routing. Reservoir routing is a procedure derives the outflow hydrograph from a reservoir from the inflow hydrograph into the reservoir with consideration of elevation, storage, and discharge characteristics of the reservoir and spillways which is called lumped routing. The conservation of mass equation is solved with the assumption that outflow discharge and volume of storage are directly related (210–VI–NEH, Amend. 69, April 2014). Channel routing simulates the movement of water through a channel used to predict the magnitudes, volumes, and temporal patterns of the flow (often a flood wave) as it translates down a channel.

2.11.1. Hydrologic Routing Method

Hydrologic Routing uses the continuity equation to relate inflows, outflows and storage to sole for out-flows simpler, more empirical, parameters estimated at application scale from experience and data. In flood forecasting, hydrologists may want to know how a short burst of intense rain in an area upstream of a city will change as it reaches the city. Level pool routing is the procedure for calculating the outflow hydrograph from a reservoir with a horizontal water surface, given its inflow hydrograph and storage characteristics (Chow, Maidment & Mays 1988). Hydrologic routing method is simple and quick in terms of mathematical procedure and necessary input data (Ionescu, Elena and Nistoran, 2019). Hydrologic routing is based on the simple concept of continuity, which means that the change in storage (S) over time t in a water body equals the difference between the inflow (I) and the outflow (O) (Chow, Maidment & Mays 1988).

2.11.2. Reservoir Routing Method

Reservoir routing is the technique by which the outflow hydrograph is computed at the downstream of a reservoir with the known inflow hydrograph. In Reservoir routing the effect of a flood wave entering a reservoir is studied. Knowing the Volume-Elevation characteristics of the reservoir and the outflow-elevation relationship for the spillway and other out-let structures in the reservoir, the effect of a flood wave entering the reservoir is studied to predict the variations of reservoir elevation and outflow discharge with time. This form of reservoir routing is essential in the design of the capacity of spillways and other reservoir outlet structures, in the location and sizing of the capacity of reservoir to meet specific requirements (Husain, 2018).

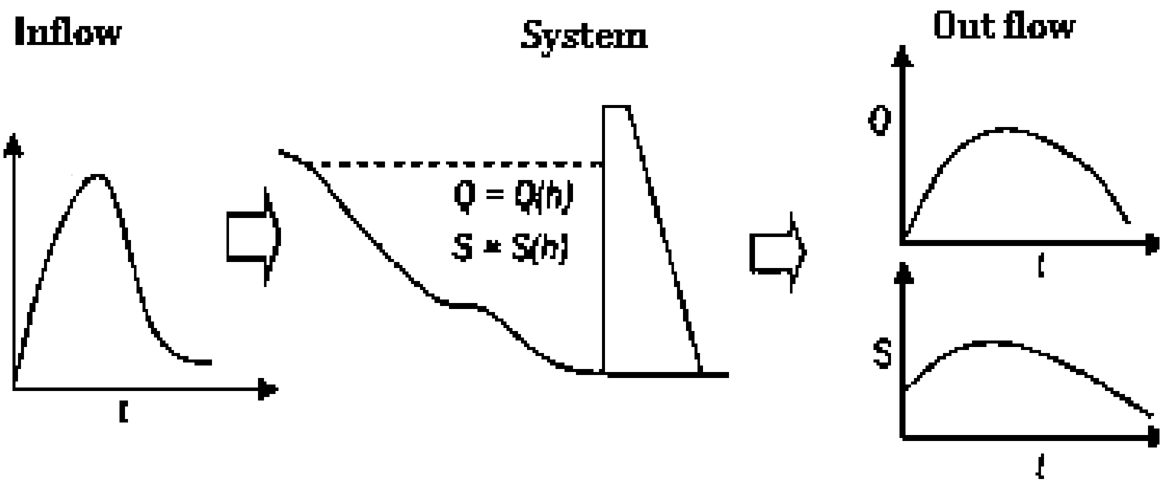


Figure 2.9 Reservoir routing

The passage of flood hydrograph through a reservoir is an unsteady flow phenomenon. The equation of continuity (water or mass balance) is used in all hydrologic routing methods as primary equation. According to this equation, the difference between the inflow and outflow is equal to the rate of change of storage, i.e.

$$I - Q = \frac{dS}{dt} \quad \text{(14)}$$

$where\ I = inflow, Q = outflow, S = storage, and\ t = time.$

Alternatively, in a small time interval Δt, the difference between the total inflow volume and total outflow volume in reach is equal to the change in storage in that reach:

$$I_m \Delta t - Q_m \Delta t = \Delta S \quad \text{(15)}$$

$where\ I_m, Q_m\ and\ \Delta S$ denote average inflow, average outflow and change in storage during time period Δt, respectively. Assuming

$$I_m = \frac{I_1 + I_2}{2}, \qquad Q_m = \frac{Q_1 + Q_2}{2} \quad \text{(16)}$$

$$\Delta S = S_2 - S_1 \quad \text{(17)}$$

$where\ suffixes$ 1 and 2 denote the beginning and end of time interval Δt, equation (14) is written as:

$$\frac{(I_1 + I_2)\Delta t}{2} - \frac{(Q_1 + Q_2)\Delta t}{2} = S_2 - S_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

Here, the time interval Δt must be sufficiently small so that the inflow and outflow hydrographs can be assumed to be linear in that time interval. Further, Δt must be shorter than the time of transit of flood wave through the reservoir.

Equation_18 can be rearranged as:

$$\frac{(I_1 + I_2)\Delta t}{2} + \left(S_1 - \frac{Q_1\Delta t}{2}\right) = \left(S_2 + Q_2\frac{\Delta t}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

In order to solve the continuity equation (Equation_14) a second relation between storage and flow is required. One type of routing is when a flood wave passes through an unregulated lake or reservoir. For a reservoir with an uncontrolled outflow and a level water surface, there is, depending on the properties of the discharge point, a fixed relation between lake elevation (h) and the outflow from the lake (Chow, Maidment & Mays 1988). Thus:

$$O = f_1(h) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

As the lake elevation changes, so does, depending on the topographical properties of the lake, the volumetric storage in the lake (Chow, Maidment & Mays 1988). Thus:

$$S = f_2(h) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Since both storage and outflow are functions of the lake stage, there is an indirect relation between outflow and storage (Chow, Maidment & Mays 1988):

$$S = f(O) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

The relation between stage and outflow can sometimes be determined using hydraulic equations. For example, flow over several types of weirs can be described as (French, 1986):

$$O = C * h^m \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

where m and C depend on the physical properties of the weir.

2.11.3. The Pul's Method

In Pul's method, the continuity Equation_19 is used. The computations are performed as follows. At the starting of flood routing, the initial storage and outflow discharge are known. In Equation_19 all the terms in the left hand side are known at the beginning of time step Δt. Hence the value of the function $(S_2 + Q_2 \Delta t/2)$ at the end of the time step is calculated by Equation_19. Since the relation $S = S(h)$ and $Q = Q(h)$ are known, $(S_2 + Q_2 \Delta t/2)$ will enable one to determine the reservoir elevation and hence the discharge at the end of the time step. This procedure is repeated to cover the full inflow hydrograph(Fenton and Fenton, 2016).

2.11.4. Modified Pul's Method

This is also referred to as the Storage-Indication Method and also known as a level pool method which is a more simplistic, numerically stable approach that can be used successfully under certain circumstances and requires only a simple stage-storage curve for the reservoir. Level pool routing method is a procedure for calculating the outflow hydrograph from a reservoir with a horizontal water surface, given its inflow hydrograph and storage outflow characteristic. The method assumes there is a horizontal water surface in the reservoir from the upstream towards the downstream (see Figure2.10 & 2.11) (Chow, Maidment & Mays 1988).

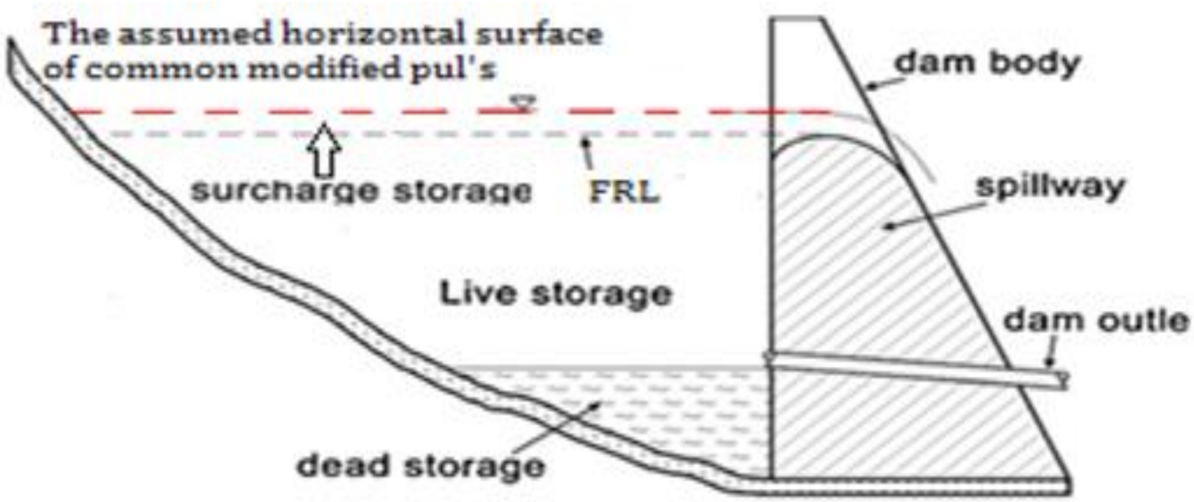


Figure 2.10 The assumed horizontal surface of the common Modified Pul's Method

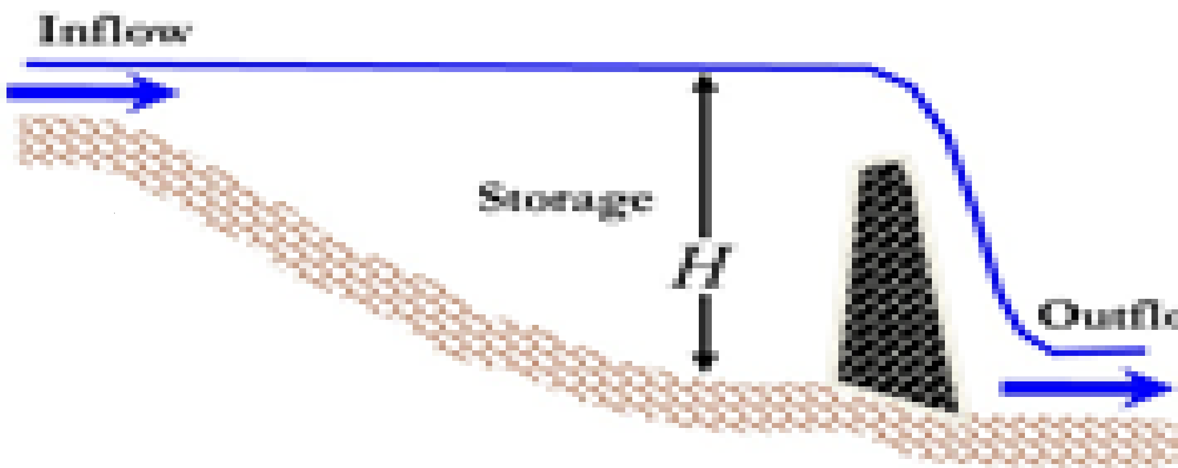


Figure 2.11 The common Modified Pul's routing method (McKinney)

This method represents Equation_18 in finite difference form:

$$S_2 - S_1 = \frac{(I_1 + I_2)\Delta t}{2} - \frac{(Q_1 + Q_2)\Delta t}{2} \quad \ldots\ldots (24)$$

In which Q may incorporate controlled discharge as well as uncontrolled discharge. Separating the known quantities from the unknown ones and rearranging:

$$(I_1 + I_2) + (2S_1/\Delta t - Q_1) = (2S_2/\Delta t - Q_2) \quad \ldots\ldots (25)$$

The left side contains the known terms and the right side is unknown. The inflow hydrograph is known. The discharge Q which may pass through the turbines, outlet works, or over the spillway is also known. The uncontrolled discharge goes freely over the spillway. It depends upon the depth of flow over the spillway and the spillway geometry. Further, the depth of flow over the spillway depends upon the level of water in the reservoir. Therefore:

$$S = S(Y),\ Q = Q(Y) \quad \ldots\ldots (26)$$

where $'Y'$ represents the water surface elevation. The right side of Equation_21 can be written as:

$$\frac{2S}{\Delta t} + Q = f(Y) \quad \ldots\ldots (27)$$

In order to utilize Equation_21, the elevation storage and elevation-discharge relationship must be known. Before routing, the curves of $(2S/\Delta t + Q)$ versus Q are constructed. The routing is now very simple and can be performed using the above equation. The steps involved in the reserving routing using Pul's method are first develop storage, S vs. discharge, Q from the given elevation vs. storage, and the elevation vs. discharge relationships. Then select the routing interval Δt in such a manner that a linear variation of discharge exists within the time interval. Develop $(S + Q\,\Delta t/2)$ and $(S - Q\,\Delta t/2)$ curves with the help of developed $S\ vs\ Q$ relationship as developed.

The routing computations can now be started. To begin with compute average inflow values over Δt by averaging the successive inflow values viz. $(I_2 + I_1)/2$. Compute the total inflow volume entered into the reservoir by multiplying $(I_2 + I_1)/2$ withΔt, i.e., $(I_2 + I_1)\,\Delta t/2.$ Now estimate from the developed plot between $(S - Q\,\Delta t/2)\ vs\ Q$ the quantities $(S_1 - Q_1\,\Delta t/2)$ corresponding to the dischargeQ_1. The discharge Q_1 at $t = 0$ is considered zero corresponding to spillway crest level for uncontrolled flow condition or a non-zero value corresponding to a controlled flow situation. The value of $((I_2 + I_1)\,\Delta t/2 + (S_1 - Q_1\,\Delta t/2)$ can be computed which is equal to the value $(S_2 + Q_2\,\Delta t/2)$. Estimate Q_2 from the developed relationship between $Q\ vs\ \left(S + Q\frac{\Delta t}{2}\right)$.

With the above computation steps the value of discharge Q_2 corresponding to the known I_2 is computed. Now this Q_2 becomes Q_1 for the next routing interval and the corresponding $(S_1 - Q_1\,\Delta t/2)$ is computed from the $Q\ vs\ (S - Q\,\Delta t/2)$ plot.

By repeating the computations steps the entire routed outflow hydrograph from the reservoir is computed (Mays 2012).

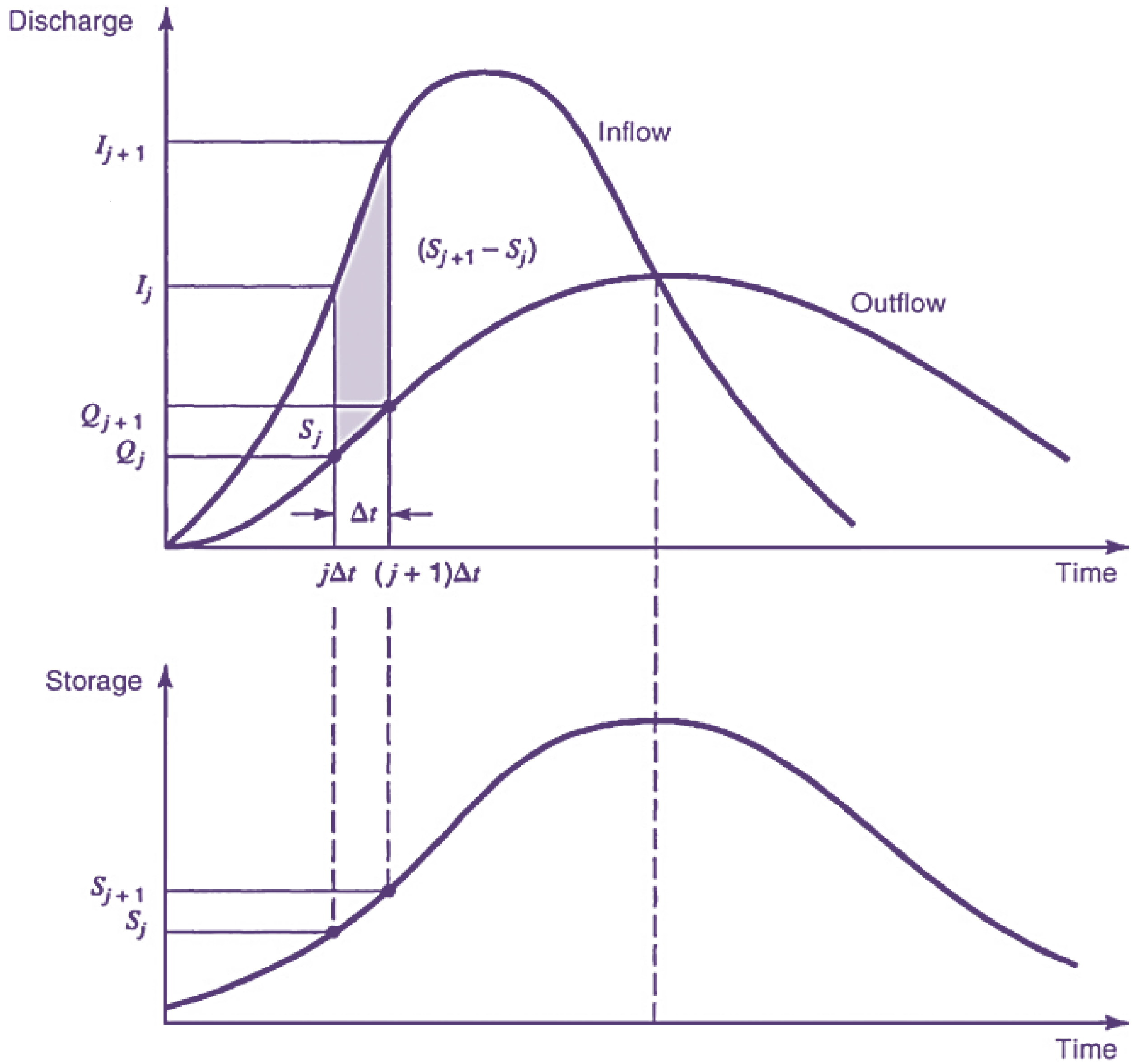
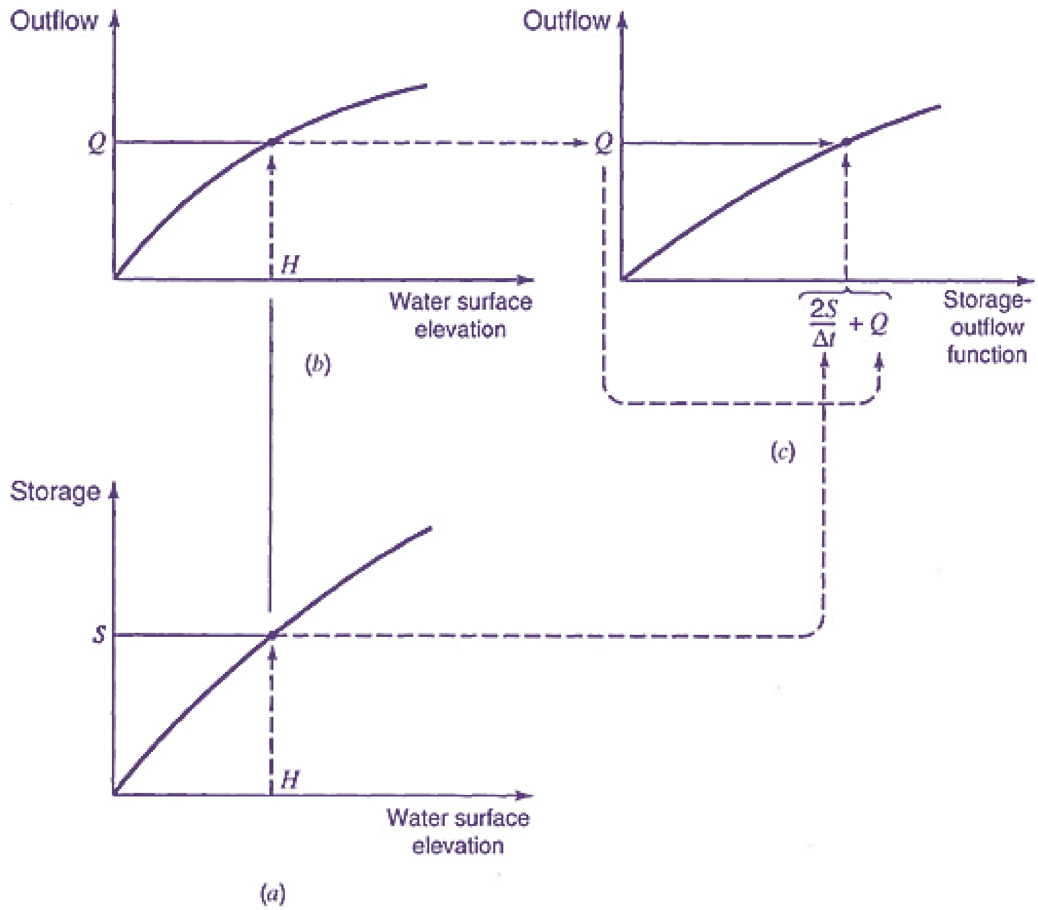


Figure 2.12 Relationships between the inflow, outflow, and storage over time (Mays 2012)



a) Develop an elevation-storage (or elevation-area) curve, H vs S.

b) Develop an elevation-outflow curve, H vs Q.

c) Compute the storage and outflow for the given elevations to relate $(2S/\Delta t+Q)$ to Q.

Figure 2.13 Development of the storage-outflow function (or routing relationship) for level pool routing (Mays 2012)

3. CHAPTER THREE: - Research Method, Material and Procedure

3.1. Description of the Study Area

The research was performed in the Kesem Dam watershed, which is the part of the Awash River basin. The Kesem catchment upstream of the dam site has different topographical, climatological and land use condition.

3.1.1. Topography

There are many surveyed works are done for the main dam, main saddle, spillway and tunnel area topography. The Kesem River catchment to dam site covers about 3135km^2 to its gauging station at Awara Melka and extends from an altitude of almost 3600m to 860m elevation. It rises on the high Ethiopian plateau and descends the western scrap of the Great Rift Valley to join the Awash River. The most prominent feature of the Kesem Valley is the steepness of slopes linking the plateau of the north, west and south extremities of the catchment with the gorges of the central area. The channels of the Upper Kesem, being narrow and highly sinuous, feature in fine sediments. They cascade into narrow, flat-floored trenches where they braid amongst the boulder and cobble bed materials. For a large part of the central Kesem Catchment the valley floor broadens, up to half a kilometer wide. Often, in the upper parts of this section, traditional irrigated agriculture interposes between the channels and side slopes (WWDSE, 2006).

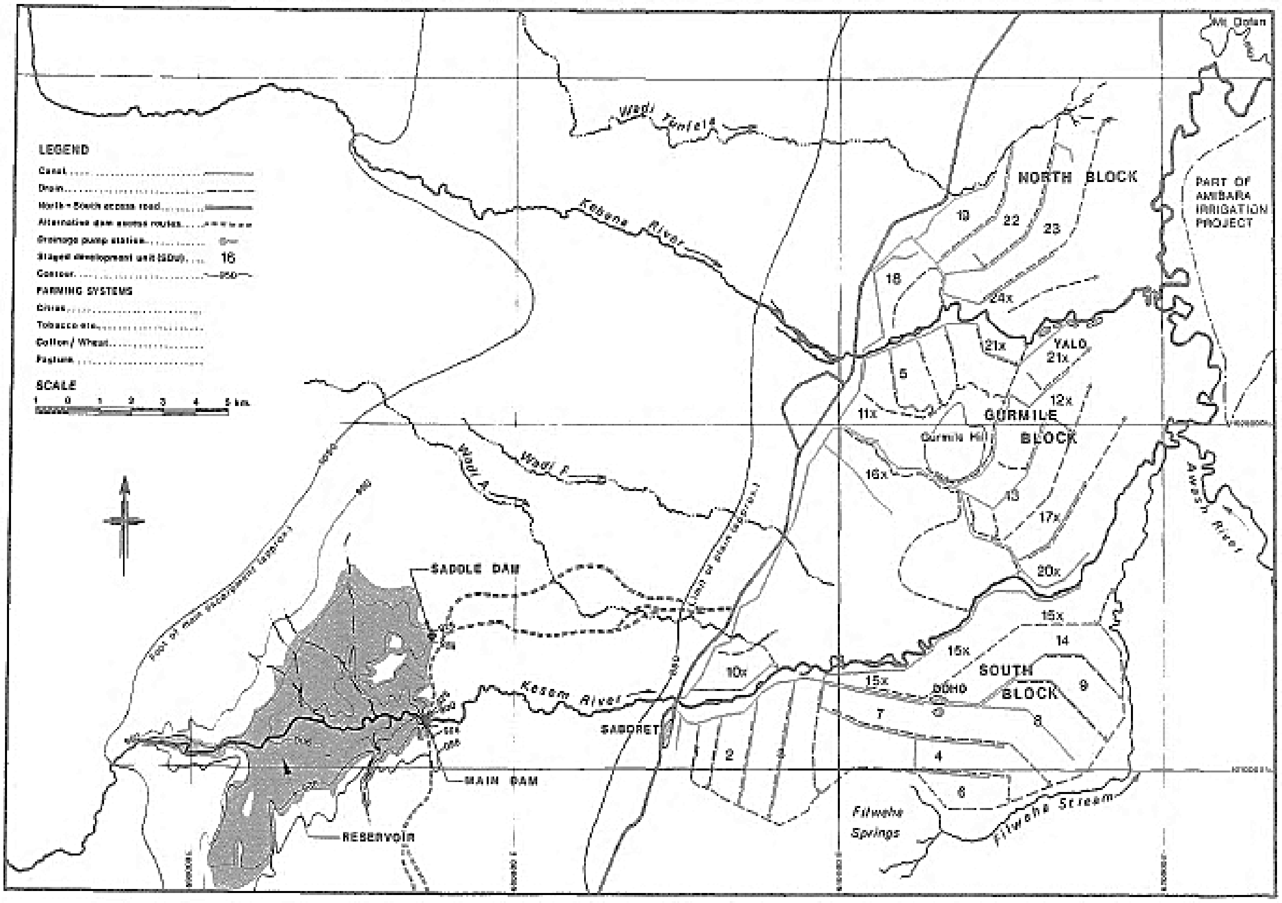


Figure 3.1 Kesem irrigation project layout ('VOLUME 1 Development of Irrigated Agriculture', no date)

3.1.2. Climate

The climate of the study area varies from arid in the lower altitudes to semi-arid in the high altitudes of the watershed. The area experiences a typically tropical semi-arid climate with rainfall normally in the range 350mm to 600mm. The rainfall distribution of the Kesem Dam watershed is low around the outlet of the watershed as compared with the middle and the highest altitude of the watershed (high rainfall). Temperature varies from mean minima of 15ºc and 21ºc to mean maxima of 23ºc and 38ºc in December and June respectively; frost is unknown. Mean relative humidity are lowest in July 36% and highest in August 58%. Mean daily sunshine reported on an annual basis is 8.5hr. The overall pattern of rainfall approximately 60% of the annual total fall. as the main rains between July and September. There is, however, another minor and reliable period of rainfall peaking sometimes during the period March to May, these short rains account for a further 25% of the annual total. Rainfall in the basin increases three-fold with the rise in altitude from the Awash basin into the highland plateau, though this increase is asymmetrical, the north of the basin being much wetter. To the south only the high plateau area has extensive agriculture which, looks like the other plateau, consists of a mosaic of cultivated plots. The lower parts of the basin are too dry to settled agriculture. The Kesem watershed has to mean maximum temperature of 38 °C and mean minimum temperature of 15 °C. However, the weather condition upstream of the watershed is cool and moist with elevation variation between 1500 and 3488 m above mean sea level.

3.1.3. Land Use and Land Cover

The land use condition in Awash catchments upstream of Kesem includes mainly of cultivated agricultural land, grass land, forest land, rural and urban settlements. In the upper most part where there is high rainfall, land use is complete in May with barley and teff. On the lower most part, however, rainfall is too unreliable and the sparse dry acacia scrub gives way to wide stretches of bare ground. The most common soil types are clay, sand, clay-loam, silt-clay-loam, sand-clay, silt-clay (Shemeles, 1989). Land use and soil type have a direct impact on the roughness coefficient of the natural channel.

3.1.4. Kesem River Flow Data

- The rating curve updated for good flow months July to September, are defined by the following set of relationship
  i. $Q = 50,886\ (H-0.1)^{1.8078}$ f or years 1983 to 1988
  ii. $Q = 24.536(H-0.2)^{2.6565}$ for the years 1989 to 2003

where Q is the discharge as connected in m^3/sec

H is observed gauge in meters.

- The historically observed and recast flow series was for 41 years length (1963- 2003) with 3.6% gaps. The gap filling was more encouraging by linear bivariate modeling with catchment rainfall. Than runoff- runoff correlation with Awash flows as observed at Awash station was also attempted but, it did not give good results.

3.1.5. Kesem Dam and Its Reservoir

The Kesem Dam and its reservoir are located on the Kesem river channel; it is located at the southern end of the Afar depression (rift) in Afar regional state of Ethiopia. It lies between the UTM 37 zone coordinates of 580,000–608,000m E and 9,810,000–1,020,000mN. It is a rock fill dam with a central clayey core.

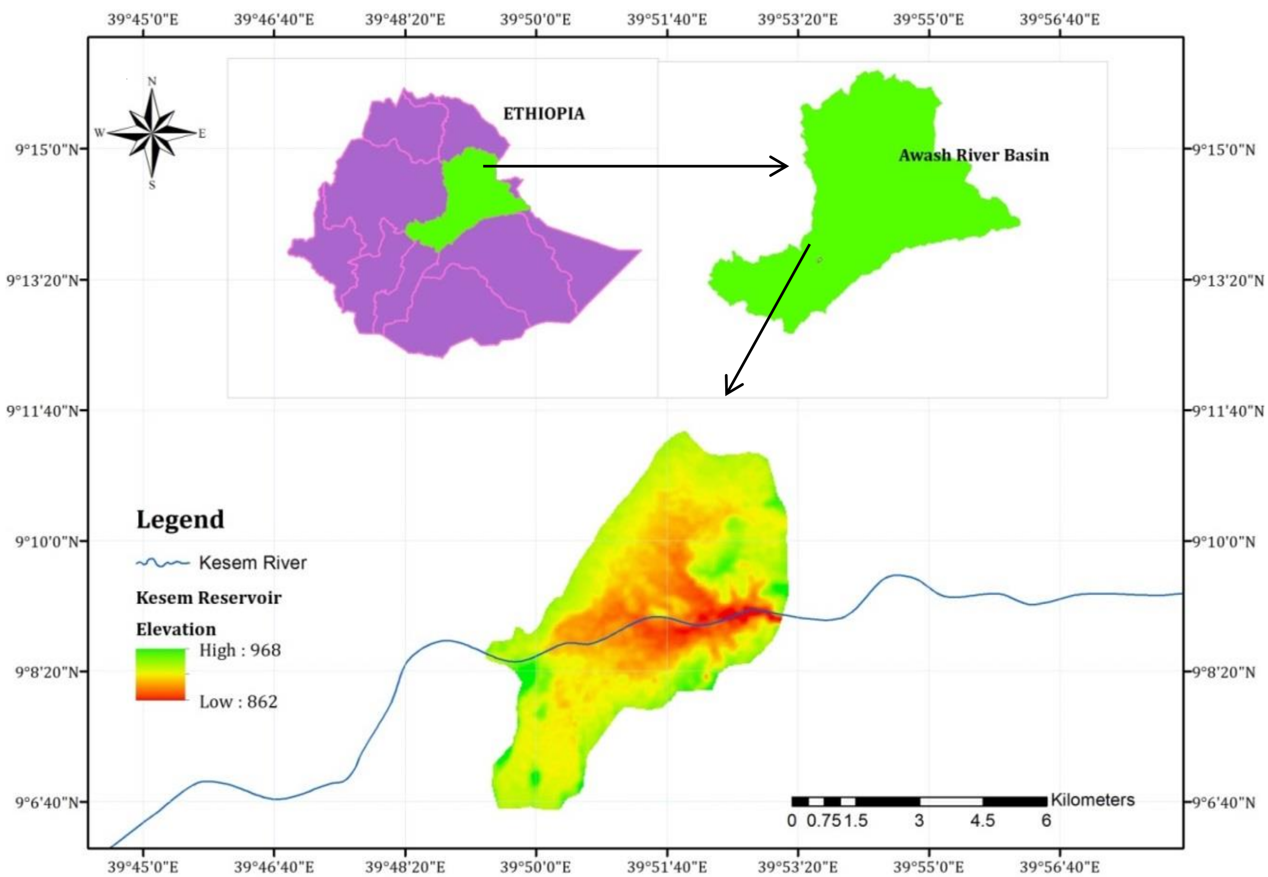


Figure 3.2 Location map of the study area

The dam has a maximum construction height of 96m. The crest is 720m long and 10 m wide with an elevation of 941.00m from the mean sea level. Upstream dam slope in an order of 1:2.25 and downstream slope of 1:2.0. The core has a slope of 1:0.25 with a minimum width of 4m. Kesem River is passing through Kesem Dam watershed; before draining into the Awash River it has been contributing water for Kesem irrigation project available lower of Dam. The Kesem Dam watershed has a watershed drainage area of about 2660 km2 up to the outlet point of the Kesem Dam. The chute spillway comprises an approach channel, a control structure, a discharge channel and a plunge pool to be created by the scouring action of flow passing over the spillway. The approach channel is planned to have a curved alignment to suit the topographic situation, the angle from the weir centerline being nearly 14.5 degrees. The approach channel takes a straight alignment as it reaches the spillway axis so that the flow arrived at the control structure is normal to its axis. The bottom width of the approach channel is 180 m at the inlet and 123 m near the control structure. The floor of Approach Channel is at level of 925m.The control structure is in the form of an Ogee weir. The design estimates the peak flow discharge for different spillway length. For 100 m ogee weir length, the crest of ungated weir is at El. 930.0 m, same as the FRL and the maximum probable flood level will be 939.5m. The routed flow over the weir corresponding to PMF works out to 6180 cumecs. The overflow head in passing the routed flow works out to 9.5 m (WWDSE, 2006). The reservoir area exposes the Tertiary volcanic rocks of Nazret Group along with the western and southwestern boundary, layered Ignimbrite forming the most important and widely distributed rock unit, basalts on eastern margin along with older alluvium deposits and Kesem River alluvium. Northwest trending faults system control the geology of part of reservoir area. Most of the reservoir area is contained within a 'graben' structure partially in filled with alluvial deposits. Numerous formulas and hot springs have been identified in the reservoir area.

Figure 3.3 Kesem River and its dam (GKM Consultant project summery)

The Kesem Dam is proposed to store about 500 Mm^3 in its reservoir, has to be designed for PMF inflow hydrograph which estimated by convoluting the unit hydrograph with appropriate Probable Maximum Precipitation which is called "Hydrometeorological Approach". The essential requirement for derivation of the design PMF inflow hydrograph is a unit hydrograph. The designed PMF inflow hydrograph for the entire catchment was routed through the reservoir flood routing which is concerned with the transformation of the flood wave during its passage through a storage reservoir using "storage indication method". This method, also known as the 'Modified Pul's Method' which assume a linear relation between inflow and outflow during a time interval Δt. The flood routing method followed this method using the elevation-area-capacity curve (Figure3.4) and the elevation out-flow with alternative spillway length is exercised. The designed inflow and out-flow hydrograph for the entire catchment with assumed spillway length of 120meter and peak PMF hydrograph 9237.77m^3/sec gets routed and produce the routed out-flow hydrograph with a peak of 6530m^3/sec. The maximum water level reached is 938.76m.

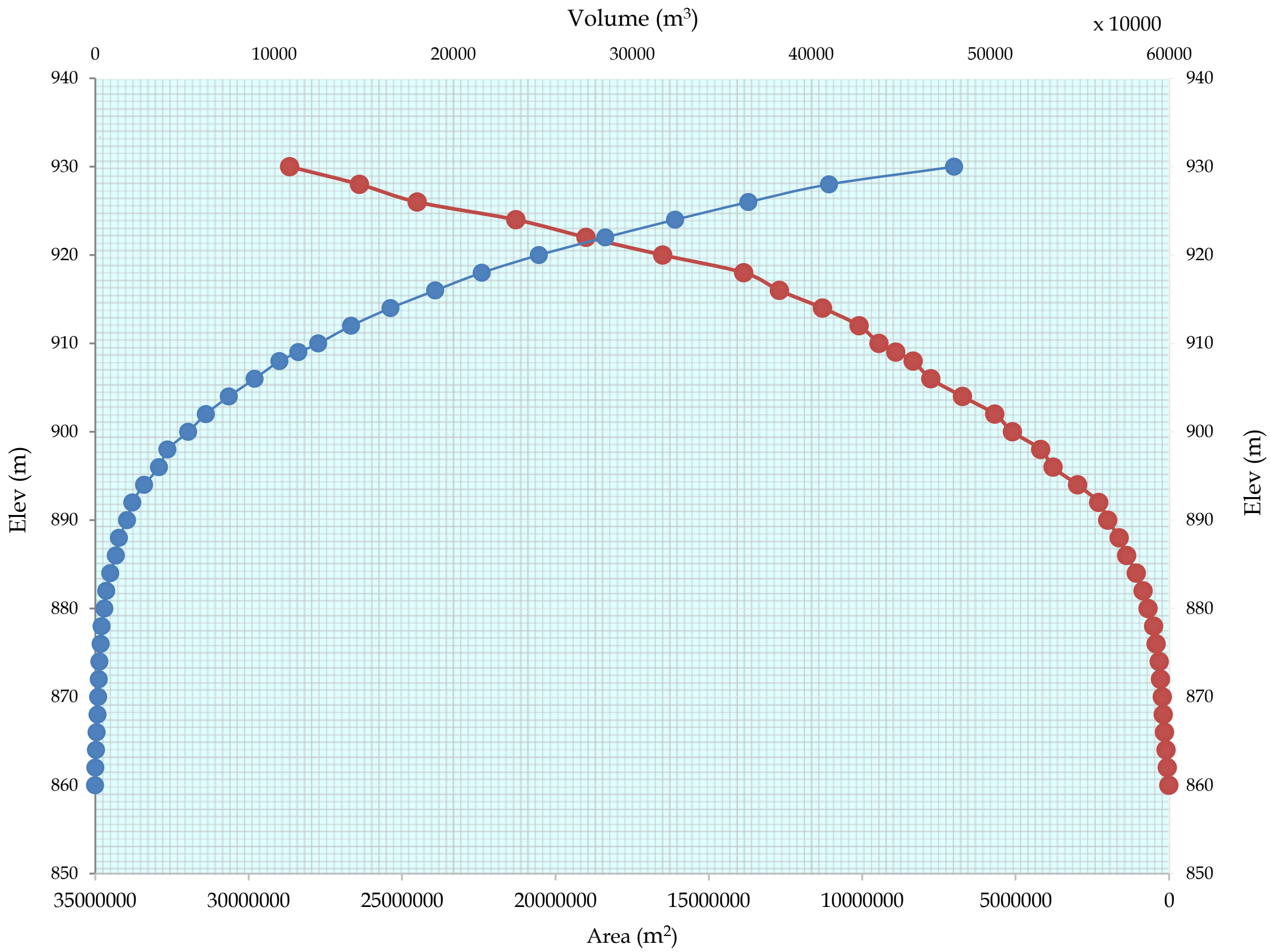


Figure 3.4 Elevation area capacity curve

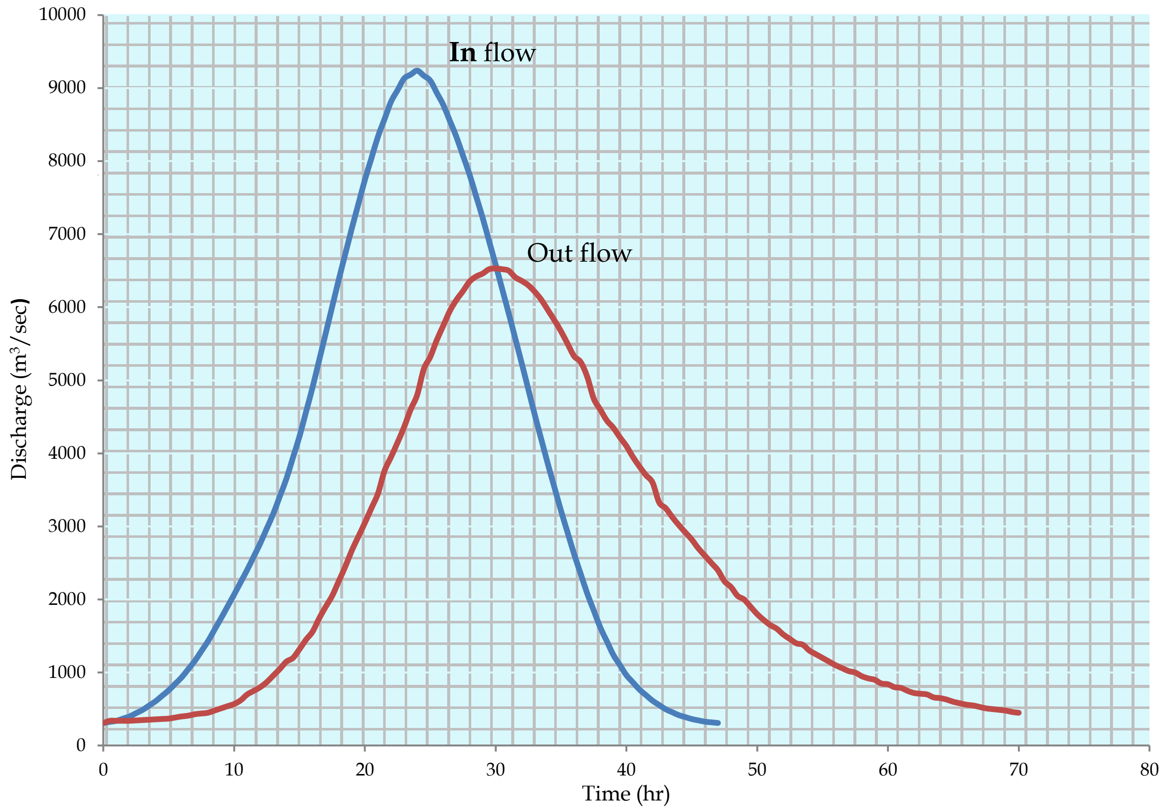


Figure 3.5 Inflow Outflow hydrograph

3.2. Data used and Its Analysis

There are different flood routing methods in open channels which require varying amounts of data, depending upon the routing technique used. In order to obtain the most accurate results by modifying the common Modified Pul's Method there are some additional data's required rather than the common method. As stated earlier the Manning formula which is uses at the upstream requires water surface slope, cross-sectional area, and wetted perimeter of the inlet cross section for control the inflow at the entrance. The modified method adopted in this research uses similar technique with the common method for the downstream control when the elevation at the downstream is higher than the upstream; rather it uses the manning equation to control the inflow hydrograph at the entrance.

$$Q = \frac{1}{n} R^{\frac{2}{3}} A \sqrt{S_o} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Necessary data (input data's) to build a numerical finite difference model using MATLAB software are:

(i) The geometry of the reservoir at the inlet and outflow structures. The top bathymetry of the reservoir must be described at the inlet of the reservoir for developing upstream rating curve $I(h)$.

(ii) Upstream boundary condition consists of the inflow hydrograph, $I(t)$, at the tail of the reservoir (from the Kesem Dam reservoir report is provided in the Annex 1, whereas as the downstream boundary condition is considered the elevation-discharge equation of the dam outflow structures, $O(h)$;

(iii) The elevation capacity curves as part of the elevation area capacity curve data must enter to describe the reservoir. Usually used for reservoir flood routing, reservoir operation, determination of water surface area, and capacity corresponding to each elevation, reservoir classification, and reservoir sediment distribution.

(iv) The discharge coefficient (C),

(v) Bottom slope and roughness coefficient of the river channel at the point where the reservoir full supply level upstream end.

For the spatial data the shape file of the Kesem Reservoir was created on Google Earth (Figure3.6). The Digital Elevation Model (DEM) which describes a digital representation of a topographic surface and its elevation of the study area at any point in a given spatial resolution. It is available in a form of raster or regular grid of spot heights of the study area (Figure3.8) was clipped using Arc GIS 10.4 by overlapping the created shape file of the Kesem Reservoir on Ethiopia's DEM which downloaded from USGS, ASTER (Advanced Spaceborne Thermal Emission and Reflection Radiometer) and GDEM (Global Digital Elevation Model) website (http://gdem.ersda c.jspac esyst ems.or.jp/) for a resolution of 30 m by 30 m. Using this reservoir DEM, the bed slope of the river at the inlet of the reservoir was determined. In addition, time series inflow hydrograph and elevation capacity curve of Kesem Reservoir taken from the design hydrology draft final report.

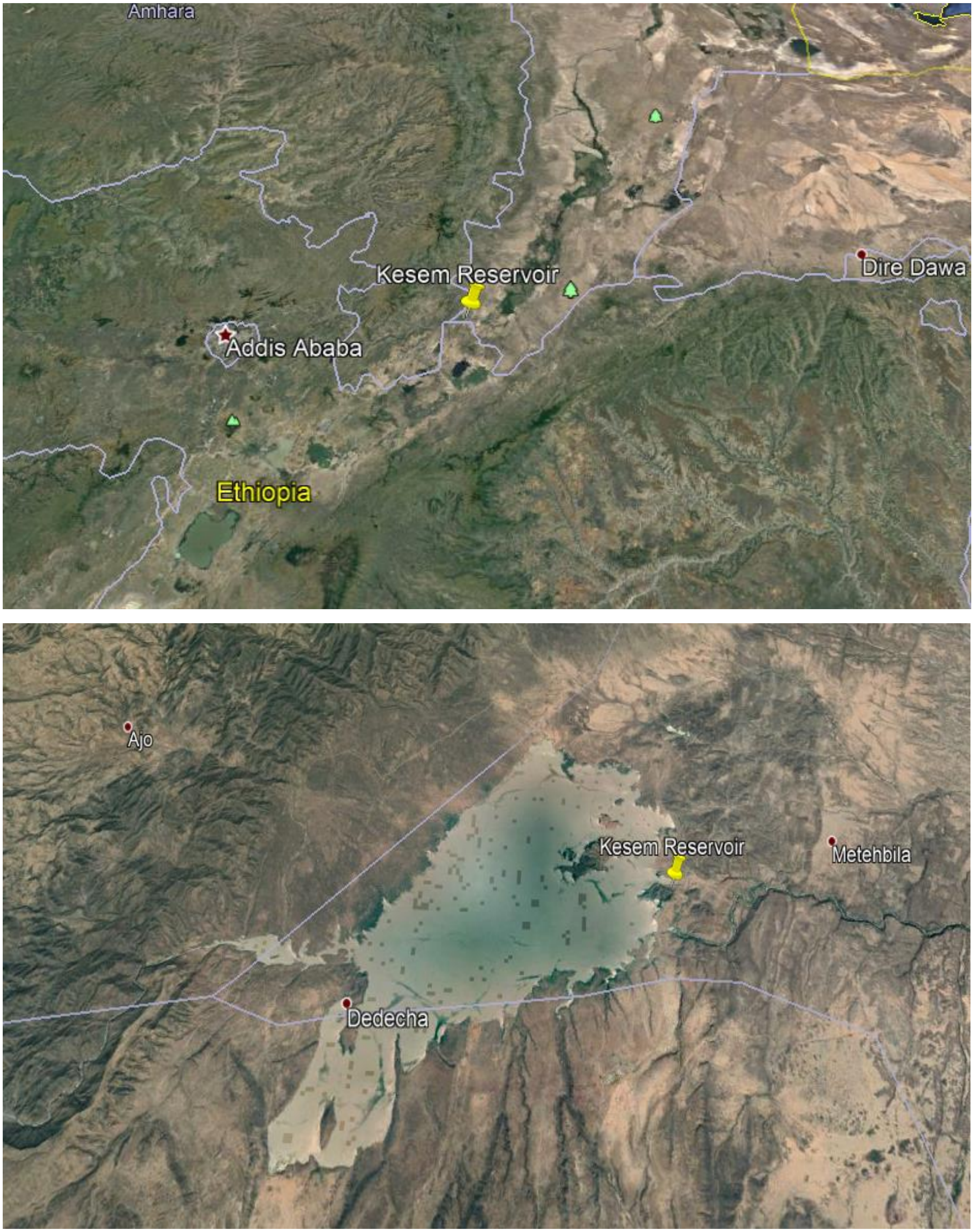


Figure 3.6 Kesem Reservoir and its location on Google Earth

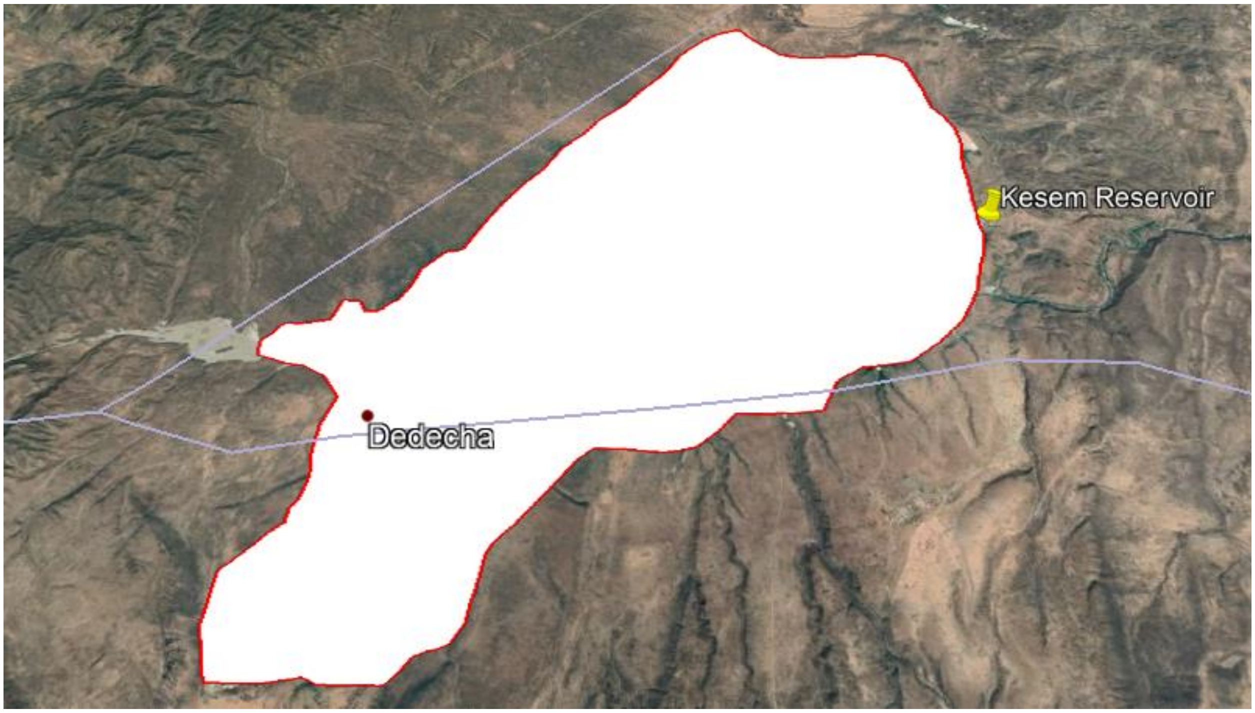


Figure 3.7 Kesem Reservoir shape file on Google Earth

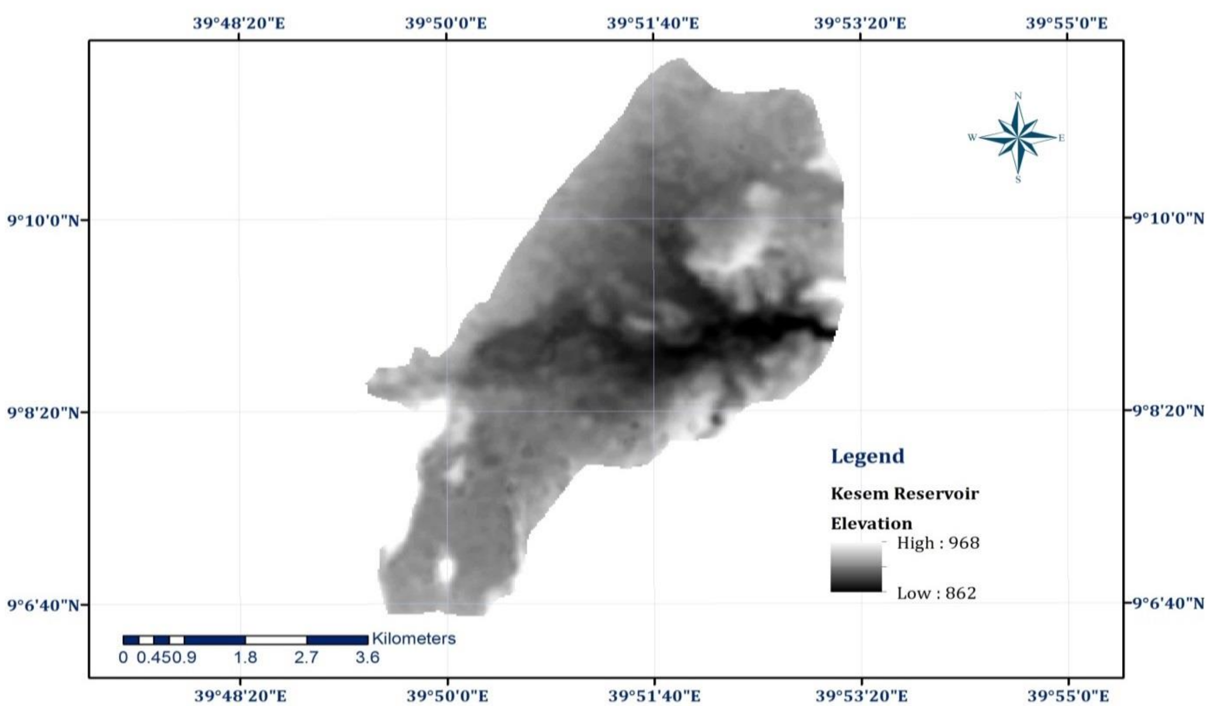


Figure 3.8 Kesem Reservoir DEM

3.3. Research Method Procedures

For spillway design discharge estimation, this research focuses on modifying the common Modified Pul's flood routing method. This new approach is concerned with the transformation of the flood wave through a storage reservoir. The approach considers the bottom roughness, bed slope and inlet cross sectional geometry at the upstream of the reservoir. In this new approach the common methods horizontal surface assumption above the spill way crest is avoided and replaced by a sloped water surface (see Figure3.9). The upstream boundary conditions control this sloped water surface at the upstream entrance cross-section, and its elevation is determined by the manning equation, which is an empirical equation that applies to uniform flow in open channels and is a function of the channel velocity, flow area, and channel slope. The improvement to account the additional storage of the reservoir that is not considered in the common method solve numerically using a finite-difference method of the continuity equation is implemented in MATLAB computer program is proposed and developed in to a standalone application.

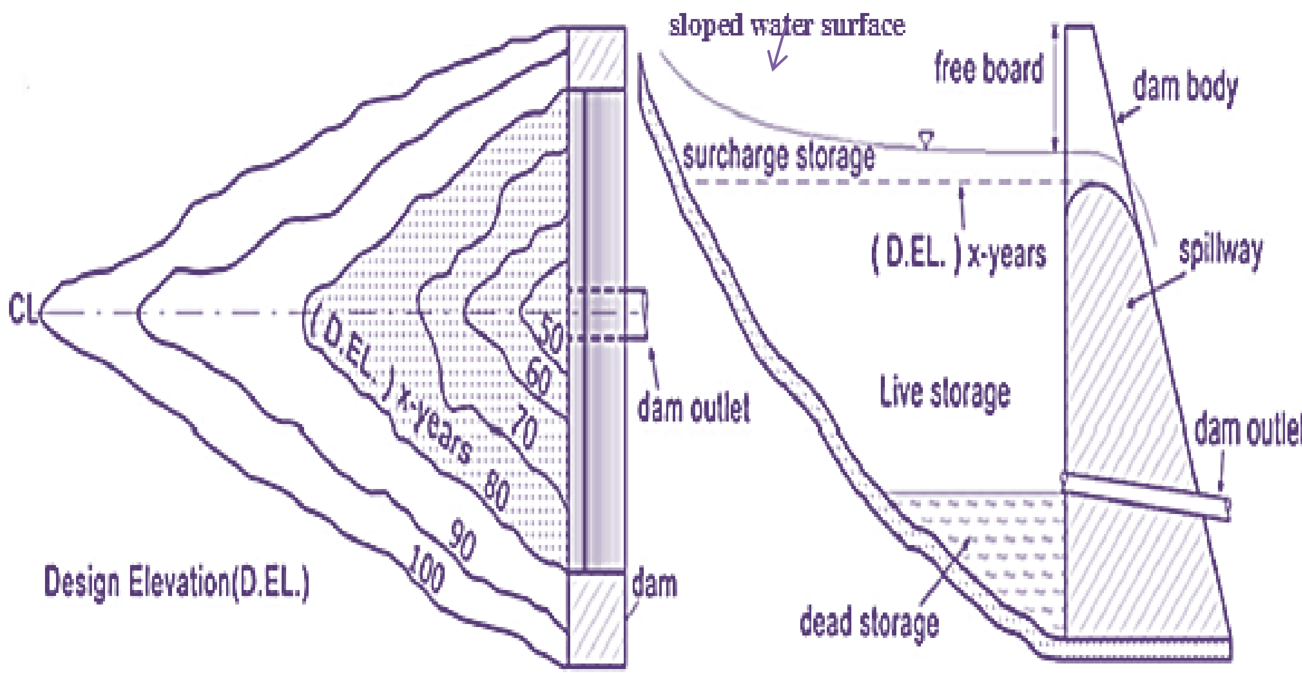


Figure 3.9 Sloped water surface in the modified routing method (Kamis, Bahrawi and Elfeki, 2018)

3.3.1. Determining Reservoir Entrance at the Upstream

The methodology for modifying the common Modified Pul's Method has been done using MATLAB computer program by entering the relevant inputs. The research begins with determining the inlet at the reservoir's entrance, which is upstream of the Kesem River flow (see Figure3.10) and its geometrical data. Calculate the river's bed slope using this determined inlet cross section by taking a profile that passes through the river and is perpendicular to the inlet cross section.

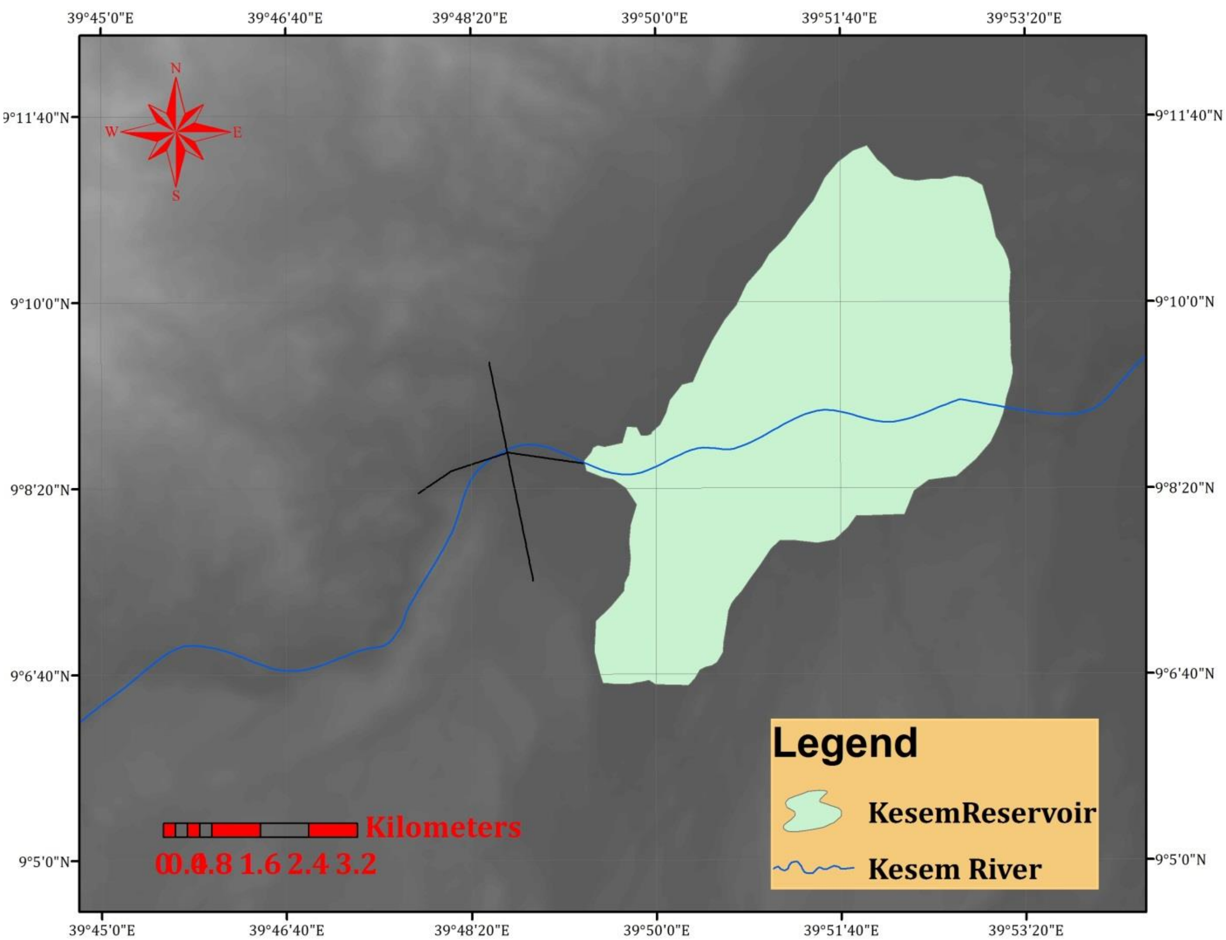


Figure 3.10 Reservoir entrance selection

3.3.2. Upstream Geometry

Using the determined inlet cross-section at section 3.3.1 the wetted perimeter, cross-sectional area and hydraulic radius of the inlet cross section are calculated using 'BentleyFlowMaster' software. It is an easy-to-use windows-based program that aids civil engineers with the design and analysis of pipes, ditches, open channels, weirs, and more. The software computes flows, water velocities, depths and pressures based on several well-known formulas that are: Darcy-Weisbach, Manning's, Kutter's, and Hazen-Williams. Bentley FlowMaster software allows the user to solve for a variable by computing the answer using the parameters provided by the user. Rating tables, rating curves, and cross sectional geometry are all calculated using the program. In put the sectional geometry of the inlet, segment roughness (start station and elevation, end station and elevation and roughness coefficient), channel slope and inflow discharge from the inflow hydrograph to the 'BentleyFlowMaster' software to solve the elevation at that time interval when the inflow flood arrives at the upstream. Using this estimated elevation discharge relation at the inlet cross section the upstream rating curve determines. The stage-discharge relationships, commonly known as discharge rating curves, are usually obtained in natural channels by determining the average flow velocities through the cross-sectional areas corresponding to different stages.

3.3.3. Kesem River Bed Slope and Roughness Coefficient

The river bed slope determined by providing profile from the entrance cross-section some upstream and downstream distance. Using this profile distance geometry a bed slope is determined in excel. The roughness coefficient in natural channels is difficult to determine in the field. Roughness coefficients vary widely with the depth of flow, as well as with distance through the channel. At very low stages the many irregularities of the channel bottom become exposed causing high values of roughness coefficients. The roughness coefficients will usually begin to decrease at deeper flows within the banks of the channel as there are less roughness elements per area of flow. When the stream begins to overflow its banks, a portion of the flow will be in the flood plain where there is usually more vegetation and the flow depths are low. Therefore the values of the roughness coefficients will begin to increase as the stage increases. Due to the very complex nature of roughness coefficients in natural rivers and streams, such coefficients are usually determined either experimentally or by using some empirical equation (Shumie, 2018). Kesem River is a floodplain river and the recommended maximum roughness coefficient by researches for floodplain river is 0.07(Chow, Maidment & Mays 1988). Using the recommended roughness coefficient of 0.07 and the previously estimated bed slope at each time interval of inflow hydrograph checks the elevation at the upstream using the manning equation of the 'BentleyFlowMaster' software which described in section 3.3.2.

3.4. Formulation of MATLAB Script to Modify the Common Method

MATLAB is a high-performance language for technical computation. It integrates computation, visualization, and programming in an easy-to-use environment (Mathworks, 2016). A MATLAB code is needed to apply the modified concept on the common Modified Pul' s method which estimates the outflow discharge by considering the sloped water surface that add an extra storage to the reservoir. First prepare a MATLAB code that estimates the outflow hydrograph using the standard Modified Pul's approach, and then adapt it to solve the modified notion of the standard method. To improve the common Modified Pul's approach and determine the out-flow discharge for varied number of ratios of storage created between the maximum upstream and downstream control storage levels, MATLAB 7.12.0.635 (R2018a) computer program is utilized. The upstream control storage level is reservoir storage created by the upstream control at the entrance of the reservoir and takes this storage volume as the design storage capacity is unacceptable and overestimated because it added unreal storage when it goes to the downstream. At the exit the downstream controls and creates a storage which controls by the downstream structure, because of this a sloped water surface is created both by the upstream control at the entrance and downstream control at the exit (spillway). The difference of these two storages contains the ignored storage in the common method, so take different rations of this storage to consider in this improved research. The variable name 'nrd' in the formed code refers to the varied number of ratio denominators utilized in the code. This research checks the reduced outflow discharge by varying the water surface slope or the number of ratios of the upstream and downstream control storage difference. Different number of ratios gives different storage value and outflow discharge. The minimum ratio denominator (minimum nrd value) begins with 2, indicating that the extra storage is half of the reservoir storage difference caused by the upstream and downstream controls, and that it is the maximum possible added (considered) storage, and so forth.

The computation of the discharge at the inflow x-section at dh interval was done using manning equation which is the upstream rating curve. The inflow flood is expected while the reservoir is full as such the elevation used for this computation is started from the spillway crest level 930m up to the maximum water level 940m with 0.01m interval dh to minimize the error. The volume of water in the reservoir and the inflow discharge for this elevation is obtained from the elevation capacity curve and upstream rating curve (which described in section 3.3.2) respectively using MATLAB interpolation technique (see section 3.4.2). The discharge over the spillway is computed for the elevation above the spillway crest level using the formula below

$$O = CL(h - 930)^{1.5} \dots \dots \dots (29)$$

The following right-hand side of the Modified Pul's method equation is estimated first, and all its values are known.

Rearranging Equation_19 and we get:

$$RHS = \frac{1}{2}(I_1 + I_2) + \frac{S_1}{\Delta t} - \frac{1}{2}(O_1) \dots \dots \dots (30)$$

The numbers for storage and discharge on the left hand side were unknown for the next time interval. Then, using Modified Pul's equation, find the smallest possible difference between the LHS and RHS. Check all the possibilities and take the smallest one from the interpolated elevation capacity discharge matrix, which is defined in the code as the'spd1' variable name using the first 'for loop'.

$$LHS - RHS = \left| \frac{\min \ possible\ reservoir\ capacity\ for\ next\ time\ step}{\Delta t} + \frac{1}{2} * \min possible\ out\ flow\ dischrge\ over\ the\ spillway\ for\ next\ time\ step \right| - RHS \dots \dots \dots (31)$$

Estimate the storage and discharge for the next time interval using the estimated minimum value dimensions (i, j). This computation is continued for the whole inflow hydrograph.

A conditional statement is executed when the common method is modified to take into account the new storage. The ''if statement'' for this modification uses to compare the upstream and downstream control elevations which are computed from manning (Equation_28) and weir (Equation_29) equations respectively. When expressions and statements are combined, a group of statements is executed if the outcome of an expression is nonempty and contains only nonzero components, it is said to be true (logical or real numeric). Otherwise, the expression is false. The else if and else blocks are optional. If the previous expressions in the "if End" block are false, the statements will be executed. As discussed in the previous sections the upstream controls when the inflow flood arrives at the entrance up to some interval of time until the downstream elevation higher than the upstream. The RHS equation is known in both the common and modified methods; the only difference is the storage (S 1) utilized in Equation_19. The new improved method uses the MATLAB interpolation methodology outlined in section 3.4.2 to estimate the upstream elevation for a particular inflow hydrograph from the upstream rating curve, then compares the upstream and downstream elevations using the 'if statement'. If the upstream water surface elevation is higher than the downstream one compute a new storage for that elevation from the elevation capacity curve using the interpolation technique discussed in the next section. Else assume there is no extra storage and take the previous which is the one in the common method. If there is an extra storage estimate the added (extra) storage by subtracting the new storage from the previous one and multiply it with some varying ratio that is $(1/\, nrd)$. Add this extra storage to the previous one. This extra storage is identified in the code with a variable name of 'AddSt'. The 'AddSt' variable in Equation_32 shown below

$$AddSt = \frac{the\ new(upstream\ control)storage - the\ pervious\ (downstream\ control)storage}{nrd} \;..(32)$$

The LHS for the modified method is:

$$LHS - RHS = \left| \frac{\min\ possible\ reservoir\ capacity + exrta\ storage\ or\ AddSt}{\Delta t} + \frac{1}{2} * \min possible\ out\ flow\ dischrge\ over\ the\ spillway \right| - RHS \ldots\ldots\ldots\ldots.(33)$$

Like the common method find the possible minimum difference between the left hand and right hand side using its minimum value dimensions (i, j) determine the elevation and discharge for the estimated storage from 'spd1' matrix.

3.4.1. Check the Acceptability of the Formulated MATLAB Code

To test the acceptability of the formulated MATLAB code its common method based result is compared with the result of the value in the Kesem Dam design report and with Riverware software result. Riverware software is a water resource modeling tool used to model various physical processes using different user selected method. Routing is one of the user selected method in the software which allows us to estimate outflow discharge and design head using different routing technics (methods). From these different routing methods the Modified Pul's method is the one. The peak discharge in the design report for 120m spillway crest length is 6530m^3/sec. First the code develops to solve the common Modified Pul's Method which based on the horizontal water surface assumption, then compare this out-flow discharges result with the designed result of the report and the user friendly software result. Based on this comparison we can evaluate the formulated code whether it solves the modified routing method equation or not. It also shows the acceptability of the formulated code which solves a reservoir routing using a finite difference equation in MATLAB software to estimate the out-flow discharge and stage (design head). The horizontal and sloped water surface assumptions, as mentioned in the preceding sections, are the main differences between the common and new modified methods of this research. The code that solves the common method takes this into account by removing the added storage, which is defined as 'AddSt' in the formulated code, as well as avoiding the estimated elevation at the entrance based on the upstream control assumption, which means we only have an elevation of water surface in the reservoir that is controlled by the downstream structure (spillway) using the weir equation (Equation_29). The common Modified Pul's Method governing equation which is the continuity equation (Equation_9) solves using the approach described in section 3.4 (Equation_30 and 31).

3.4.2. Interpolation Method

The elevation capacity curve from the design report converted in to a discrete data using an online application is called https://automeris.io/WebPlotDigitizer it is a program distributed under the GNU Affero General Public License Version 3, which extract data from a graph, pdf or image to excel and it is a better way to get the discrete data than the manual method. At the downstream the out-flow hydrograph which controls by the downstream structure in our case the flow that passes through the spillway was calculated for the elevation in the elevation capacity curve and get elevation capacity discharge relation and the upstream rating curve, which was described in section 3.3.2, was also estimated for the elevation in the elevation capacity curve to the upstream control. To order to minimize an error in the routing process there elevation difference makes to a minimum $dh = 0.01m$ and to perform this interpolating procedure must be performed in order to obtain these values at some desired point. The MATLAB computer program solves this interpolation using interp1 command with Spline function. The interp1 command interpolated values of a 1-D function at specific query points using linear interpolation and which interpolates between data points. It finds values at intermediate points, of a one-dimensional function that underlies the data. This function is shown below in Figure3.11, along with the relationship between vectors x, Y, xi, and yi. Interpolation is the same operation as table lookup. Described in table lookup terms, the table is [x, Y] and interp1 looks up the elements of xi in x, and, based upon their locations, returns values yi interpolated within the elements of Y. Spline function is one of the most popular interpolation techniques which is a series of polynomials joined at knots. Splines can be useful in scenarios where using a single approximating polynomial is impractical. Curve Fitting Toolbox functions allow to construct splines for fitting to and smoothing data by constructing splines. Splines can be used to smooth noisy data and perform interpolation.

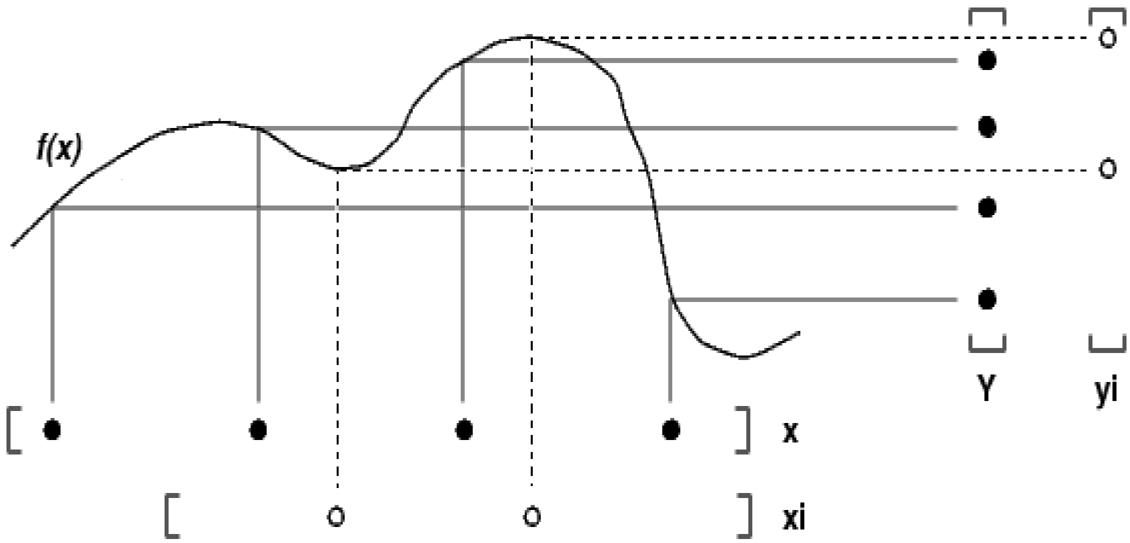


Figure 3.11 Interp1 interpolation

❖ The MATLAB Code

- This part of the code shows the input data

```
clc, clear
hg=load('hgkesem.dat');  m=size(hg,1);   %m number of hours of
inflow     hydrograph data hg
rc=xlsread('new1');      %rc upstream rating curve
cac=xlsread('capacity');    nc=size(cac,1); %Capacity(in MCM)-
Area (in m2)- Elevation (in m) curve [elv capacity elv area]
order
cc1=cac(:,1:2);
cc1(:,2)=1000000*cc1(:,2);% Capacity/Volume (in m3)curve with
elevation
%ac1=cac(:,3:4);
%ac1(:,2)=1000000*ac1(:,2);% Top surface area (in m2) of the
reservoir
% spillway design discharge equation discharge over the
spillway, spd1=CLhmax^1.5
% C is discharge coefficient,
% L is spillway crest length in meters
% hmax the maximum water depth over the spillway
% dh is the increment in depth of flow.
% splc is spillway crest level.
% frl1 and fra1 are reservoir volume and surface area
Respectively at splc
splc=930;   frl1=480000000;   %fra1=22500000;
C=2.1;     L=120;
```

- The following loop computes the volume of water, the top surface area, the discharge over the spillway, and the discharge at the inflow x-section in the spd1 matrix at dh intervals at a given elevation above 'splc' (spillway crest elevation of the code variable).

```
dh=0.01;     hmax=splc+11;   i=1;    spd1=zeros(10,5);
for h=splc:dh:hmax
    spd1(i,1)=h;    spd1(i,2)=interp1(cc1(:,1),cc1(:,2),h,'spline')-
frl1;
    %spd1(i,3)=interp1(ac1(:,1),ac1(:,2),h,'spline');
    spd1(i,4)=C*L*(h-splc)^1.5;
    spd1(i,5)=interp1(rc(:,1),rc(:,2),h,'spline'); i=i+1;
end
```

- The following code tries to find a depth h above the spillway crest level that satisfies the Modified Pul's method, which estimates the stored water and discharge over the spillway at a given time t, as well as the stored water if the upstream water level is used for storage computation at the same time t.

```
out1=zeros(10,4);       st=zeros(10,2);   j=1;  dd=0;   dt=1*60*60;
lhst=zeros(10,1);   rhst=zeros(10,1);   hgg=[hg];
out1(1,1)=splc; out1(1,3)=splc; hu(1,1)=splc;   AddSt(1,1)=0;    nrd=2;
for jj=2:m
%out1 is the designed outflow discharge
%for the common modified pul's method
    lhst(jj,1)=0.5*(hgg(jj-1,2)+hgg(jj,2))+st(jj-1,1)/dt-0.5*out1(jj-
1,2); %LHS of the governing equation...
%the modified puls method where all the variables are known for d/s
%control
     sol1=abs(spd1(:,2)/dt+0.5*spd1(:,4)-lhst(jj,1)); %this finds the
%      minimum possible difference between the LHS and RHS of modified
%      puls equation
       [xx,nn]=min(sol1);
    st(jj,1)=spd1(nn,2);
%      st(jj,2)=spd1(ny,2);
    out1(jj,1)=spd1(nn,1);
    out1(jj,2)=spd1(nn,4);
 %for the new modified pul's method
    lhst(jj,2)=0.5*(hgg(jj-1,2)+hgg(jj,2))+st(jj-1,2)/dt-0.5*out1(jj-
1,4); %LHS of the governing equation......
%      the modified puls method where all the variables are known for
u/s
%      control
           hu(jj,1)=interp1(rc(:,2),rc(:,1),hgg(jj,2),'spline');

%hu the depth of flow at the upstream
if(hu(jj,1)>=out1(jj,1))
        sst=interp1(cc1(:,1),cc1(:,2),hu(jj,1),'spline')-frl1;
        AddSt(jj,1)=(sst-st(jj-1,2))/nsd;
    else
       AddSt(jj,1)=0;
    end
    if (AddSt(jj,1)<0); AddSt(jj,1)=0; end

    sol2=abs((spd1(:,2)+ AddSt (jj,1))/dt+0.5*spd1(:,4)-lhst(jj,2));

[yy,ny]=min(sol2);
    st(jj,2)=spd1(ny,2);
    out1(jj,3)=spd1(ny,1);
    out1(jj,4)=spd1(ny,4);
```

3.5. Developed Standalone Application

A standalone application is an application that runs locally on the device and doesn't require anything else to be functional. The new developed standalone application of this research is built on MATLAB App Designer. App Designer is a development environment that allows us to design an app's layout and program its behavior. It includes a fully integrated MATLAB® Editor as well as a huge number of interactive UI components. It also has a grid layout manager for organizing the user interface, as well as automated reflow features that allow the program to detect and adjust to screen size changes. It allows us to deploy apps by packaging them into installation files right from the Program Designer toolstrip, or by building a standalone desktop or web app (MATLAB CompilerTM is required). By dragging and dropping visual components, App Designer makes it simple to create graphical user interfaces. The well-known MATLAB programming language is used to implement the actions and procedures. Later on, using the free MATLAB Runtime, standalone applications can be written and run on any machine. To specify the behavior of our program, use the integrated version of the MATLAB Editor. App

Build the apps with standard components such as buttons, axes, panel container, table, check boxes, trees, and drop-down lists. App Designer also provides controls such as gauges, lamps, knobs, and switches that let you replicate the look and actions of instrumentation panels. we can also use container components, such as tabs, panels, and grid layouts to organize our user interface. Add component callbacks and custom mouse and keyboard interactions that execute when a user interacts with our app. Use 2D and 3D plots, as well as tables, in our app to allow users to interactively explore data.

We can share our app with other MATLAB users through MATLAB Online and MATLAB Drive, allowing them to run and collaborate on our app design by extending permission to edit our files.

This research's developed software app demonstrates how to estimate the spillway design discharge using the common and enhanced Modified Pul's routing methods, as well as how to compare the findings. The spillway crest length and level, the full reservoir level capacity, the discharge coefficient, and the added storage ratio of the storage space between the maximum upstream and downstream levels are all constant parameters for this app and are stored in an anchored panel container on the top left. The software reacts by plotting the designed spillway discharge and stage, as well as its discrete data, on the table. The plot and the table are both updated as a result of user selections. This app includes these components:

- Axis — UIAxes objects are useful for creating Cartesian plots in apps. They are very similar to the Cartesian Axes objects returned by the axes function. This app has two UIAxes. The first UIAxis show the discharge vs. time graph and the second UIAxes that is the UIAxes2 shows the elevation vs. time graph.
- Button — Buttons are UI components that respond when the user presses and releases them. The button group executes a callback function to update the plot with the appropriate data.
- Panel — used to contain the whole input constant parameters together
- Table — used to view the estimated discharge of the common and the new improved modified pul's routing method with elevation and time.

4. CHAPTER FOUR: - Results and Discussion

4.1. Upstream Rating Curve

The rating curve at the location where the full reservoir level touches the river is computed based on Manning's equation. Where roughness coefficient, river bed slope and river cross section is obtained at this extreme end of the reservoir entrance point. The roughness coefficient was taken from previous works in Floodplain Rivers, which recommended maximum roughness coefficient for flood plain as 0.07. This research also takes this recommended roughness coefficient.

The river bed slope is determined as discussed in section 3.3.3 which estimated by taking some distance upstream and downstream of the entrance cross-section. Using this determined distance cross-section estimate the bed slope. The estimated bed slope becomes 0.0065. Which shown in the Figure4.1.

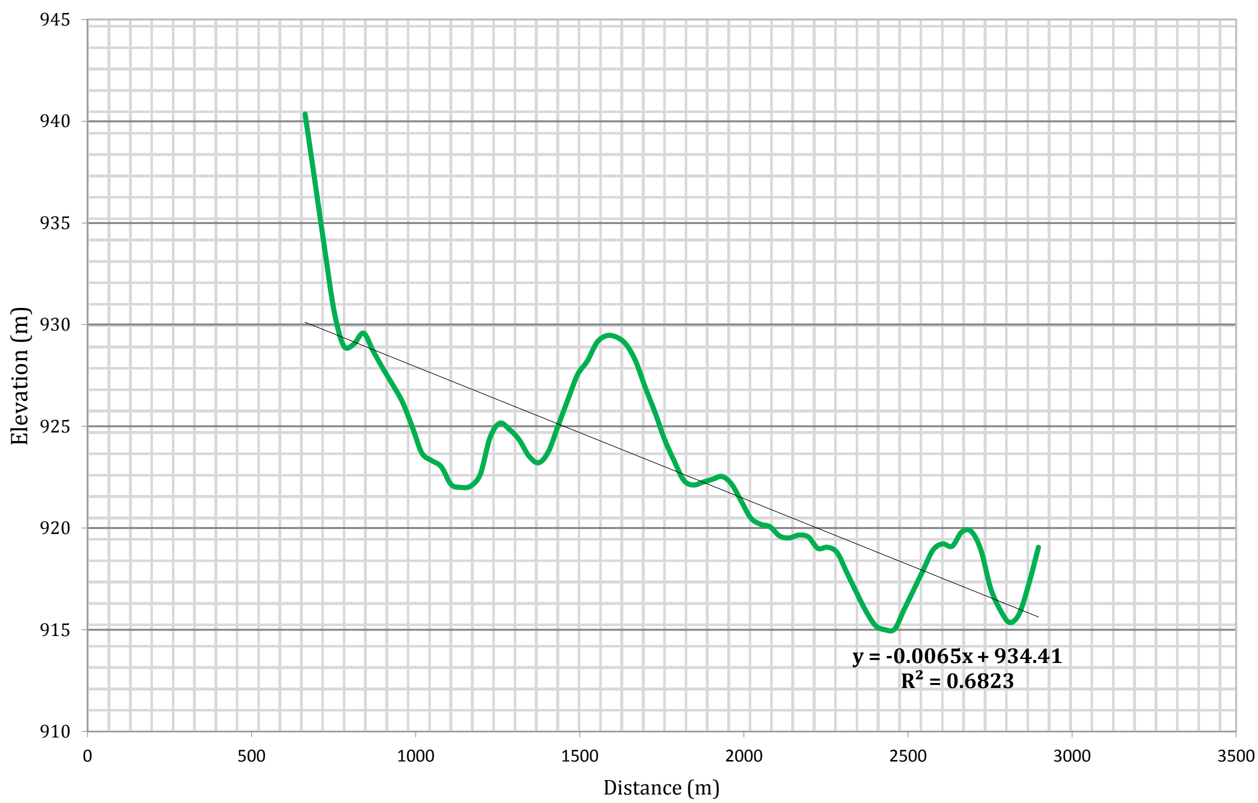


Figure 4.1 Kesem River bed slope

The selected reservoir entrance cross-section discussed in section 3.3.1 has resulted in the Figure4.2.

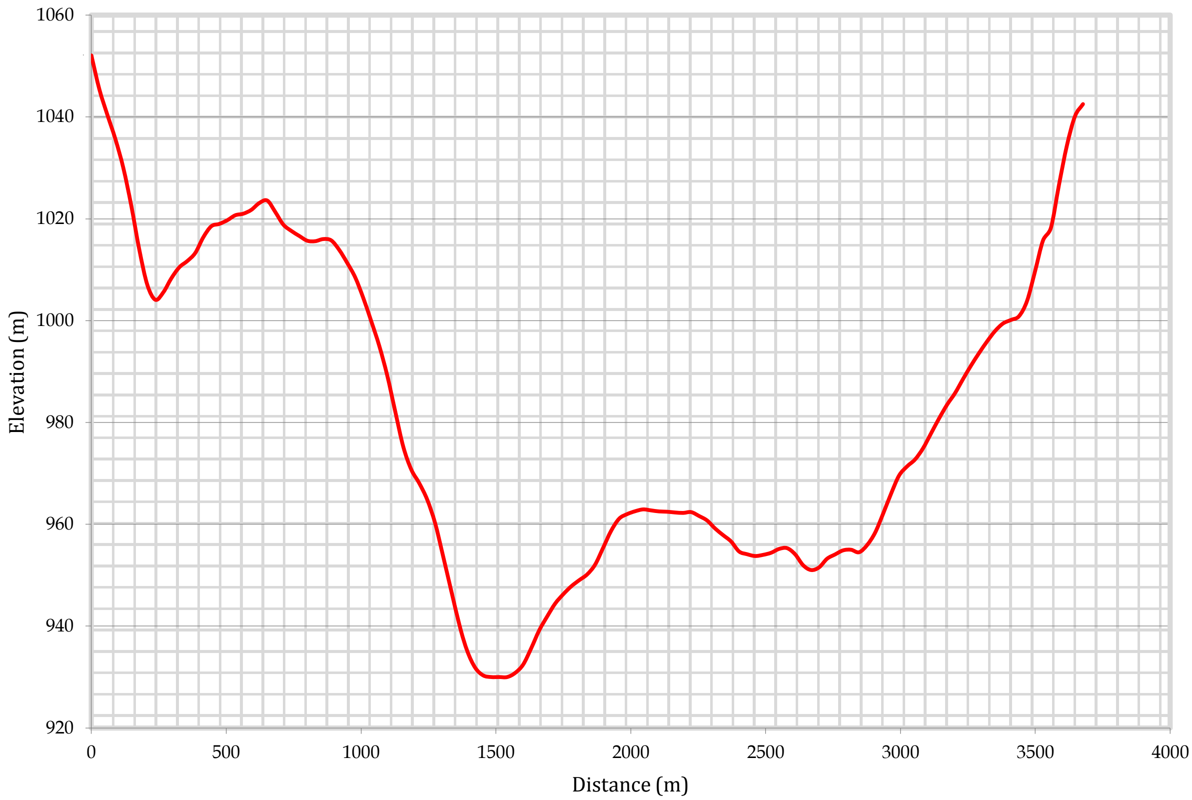


Figure 4.2 Kesem Reservoir entrance cross sectional profile

Using the 'Bentley FlowMaster' software the rating curve obtained for the river characters indicated above is as shown in Figure4.3.

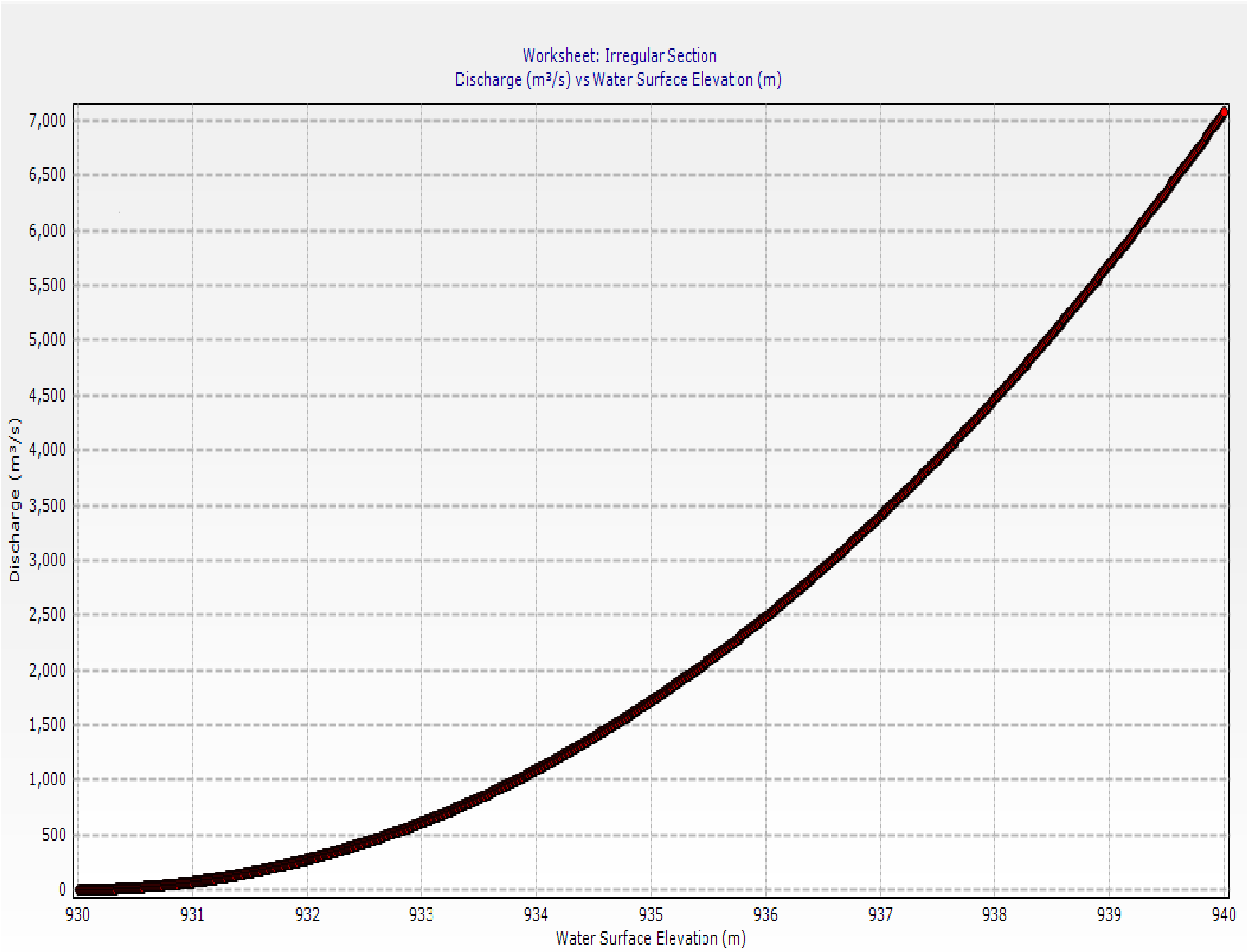


Figure 4.3 Upstream rating curve

4.2. Common Modified Pul's Method Result

To check the acceptability of the formulated MATLAB code the common Modified Pul's method result shown in Kesem Dam designed report and the Riverware software Modified Pul's method result was compared with the result obtained from the code. The trend and the peak discharge results of the this results can be seen in Figure4.4.The designed peak outflow discharge from the formulation MATLAB code with the same spillway length and discharge coefficient is 6410.98m^3/sec while the peak discharge reported in the design report is 6530m^3/sec and the estimated outflow discharge from the Riverware software was 6466.86m^3/sec. The difference between these results is small as shows in the next figure. This small difference might also come from the area capacity curve data interpolation approach. The code for the common Modified Pul's Method follows the same procedure with the new modified method code the only difference between the two is the added storage which identified as 'AddSt' variable name in the code which created by the sloped water surface at the upstream side and extends some distance to the downstream

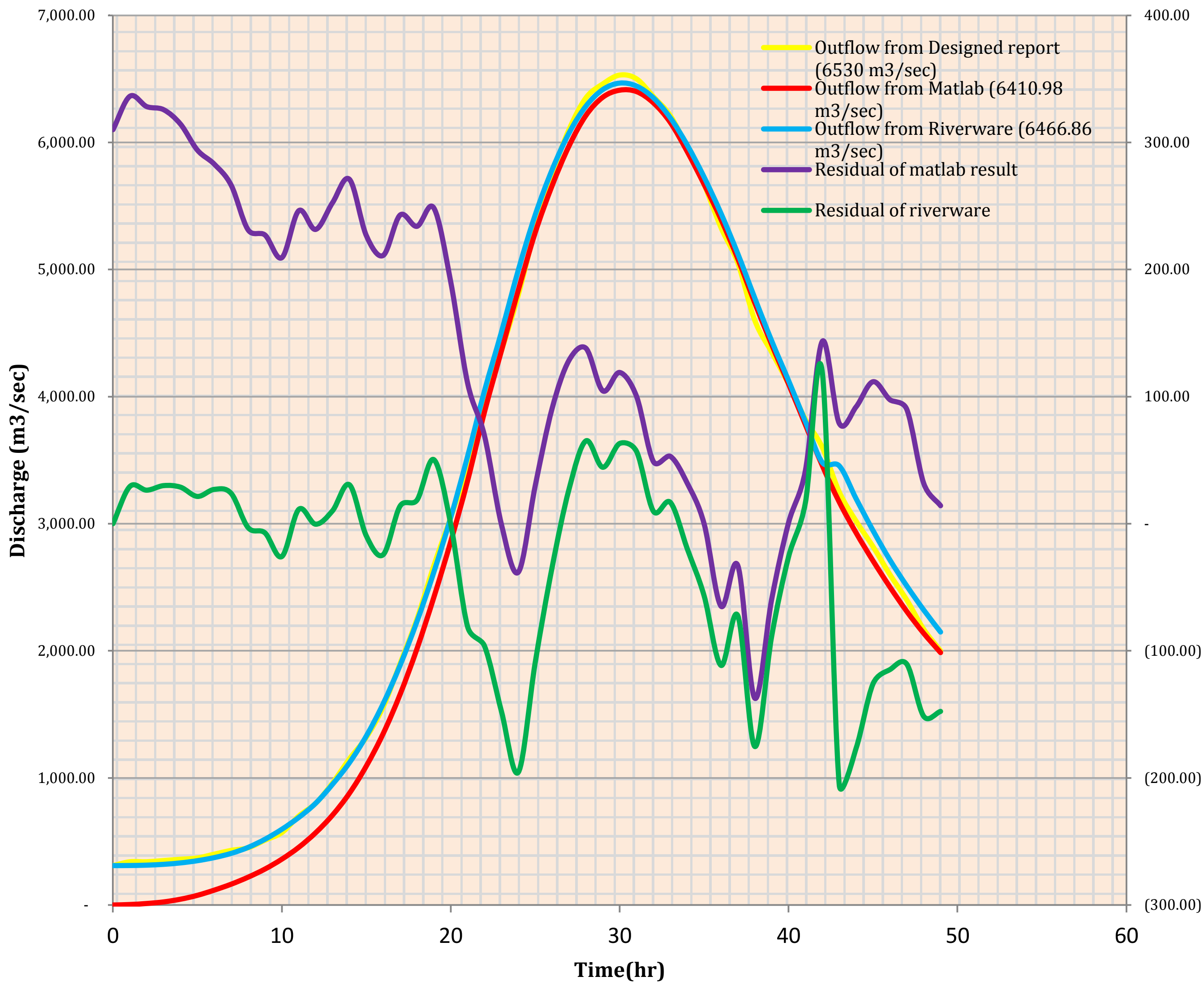
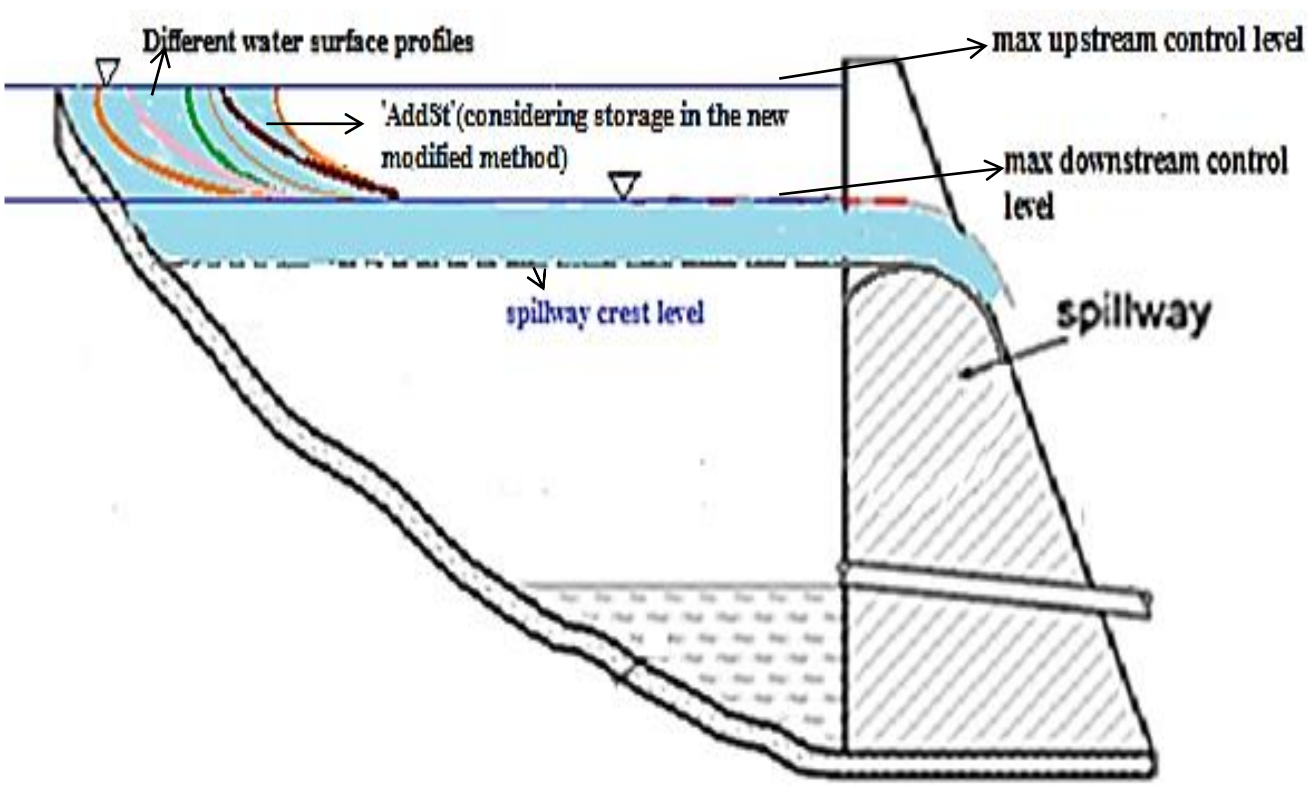


Figure 4.4 Comparison between Designed, formulated MATLAB code and Riverware software outflow discharges

4.3. Spill way Out-Flow Discharge Result

As discusses earlier in section 3.4 the formulated MATLAB code which estimated the outflow discharge considering different possible added storage due to the sloping nature of the reservoir water surface. This additional storage which was not accounted in the common Modified Pul's Method has been handled using a factor defined as in the code as 'nrd'. 'nrd' is indicates the fraction of the ignored storage in the common Modified Pul's method with respect to a storage created by the flood level at the upstream entrance over the whole reservoir surface. This storage is obtained by a horizontal line which extends from the highest upstream flood level over the whole reservoir surface. This newly added storage that is not accounted in the common method is between the two extreme flood levels (upstream (entrance of reservoir) and downstream (exit of the reservoir)) flood levels in the reservoir at a particular time.. To account this considering storage only the upstream control level that is assuming the whole space as an added surcharge storage is unrealistic it may contain some extra storage which is doesn't exist. Similarly consider the downstream control level also ignore some storage like the common method. To solve this limitation as discussed in the previous sections this research take different water surface profiles which brings about different 'nrd' values and determine the outflow discharge and stage.. The schematic drawing shown below shows this clearly its physical meaning in reservoir.



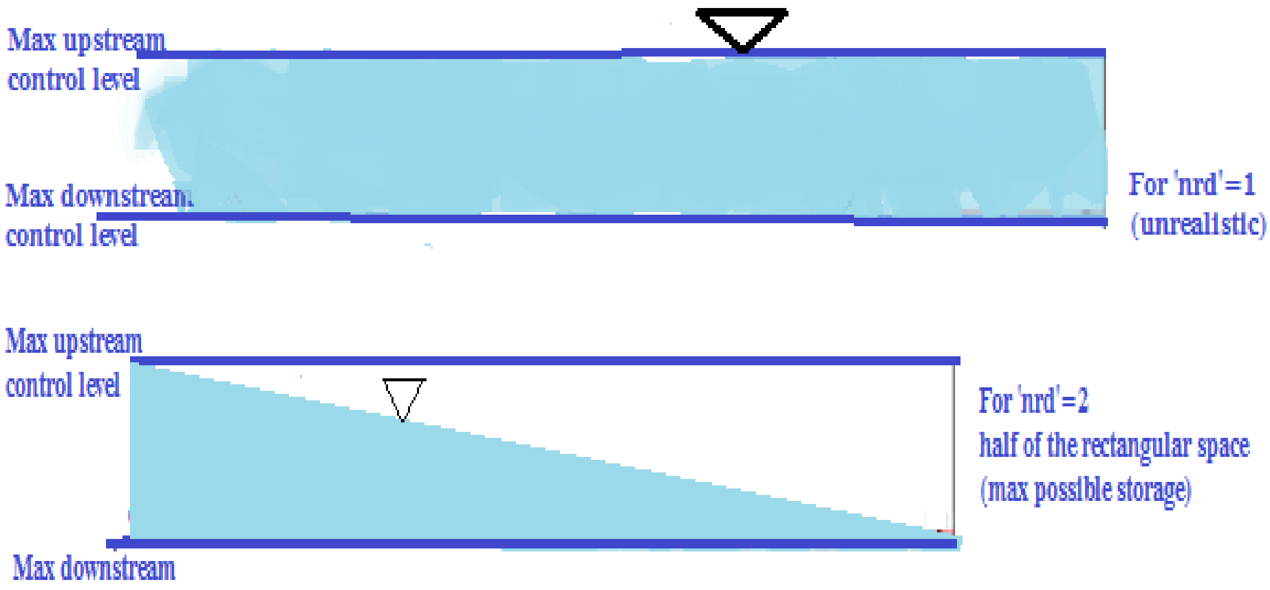


Figure 4.5 Schematic drawings which shows the physical meaning of 'nrd' values in reservoir

In the following two figures (Figure4.6 and Figure4.7) the outflow hydrograph at the spillway location is seen for two cases that (nrd = 2 and nrd = 20). 'nrd' of 2 means taking half of the storage created between the two extreme flood levels in the reservoir at a time and which reduces the outflow discharge extremely. For 'nrd' value 20 the out flow discharge and stage higher than the 'nrd' 2. That implies the larger the 'nrd' value the less the storage gained due to the sloping water surface. Thus, the peak discharge in the outflow hydrograph or the spillway design discharge becomes larger when 'nrd' become larger. The corresponding stage above the spillway crest is also depicted in the next two figures (Figure4.8 and Figure4.9).

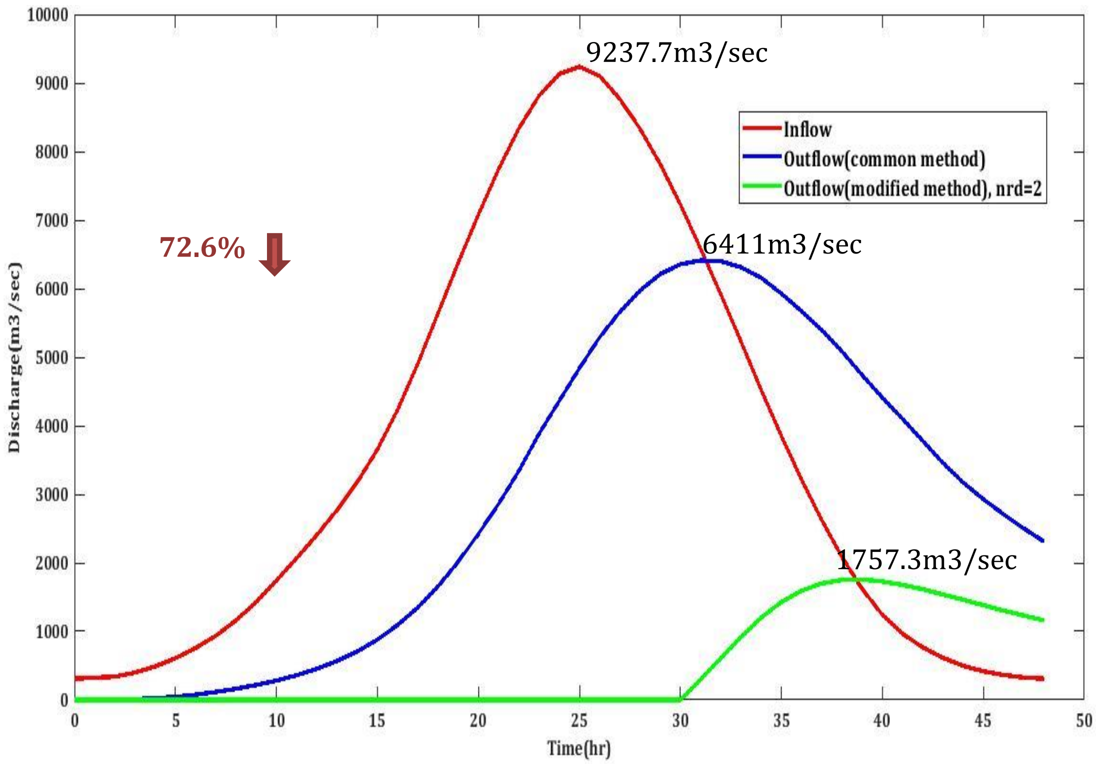


Figure 4.6 Estimated Outflow Discharge for 'nrd' 2

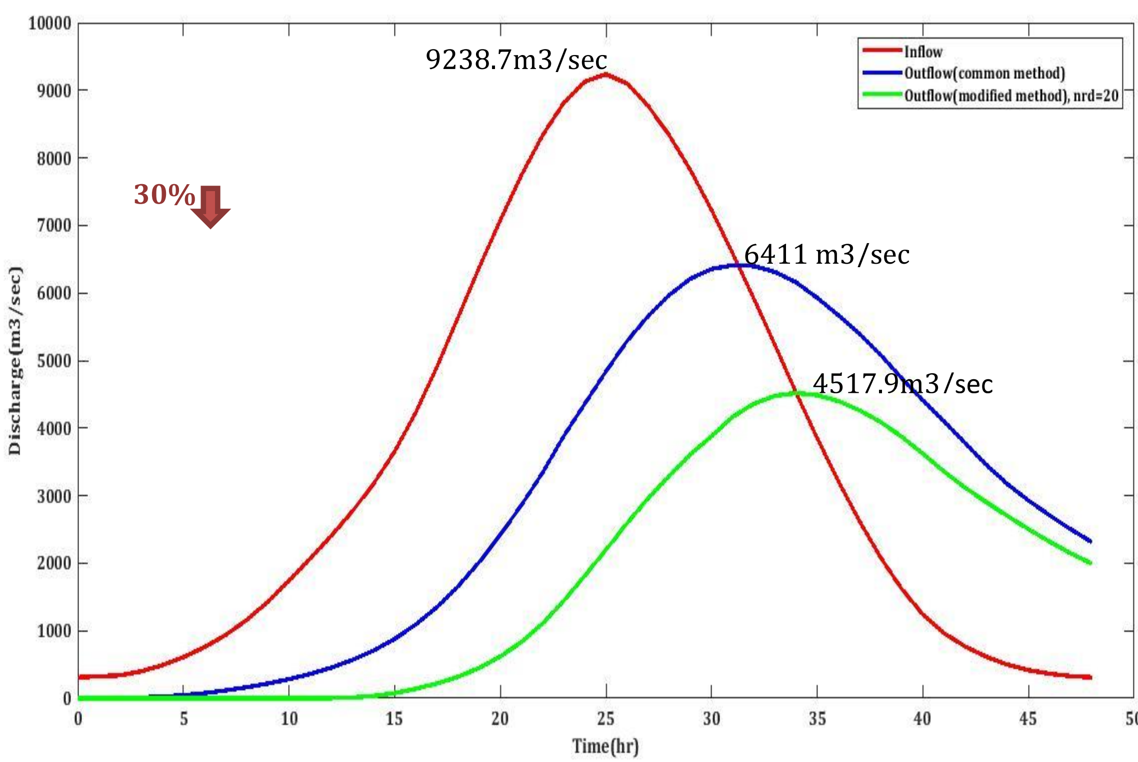


Figure 4.7 Estimated Outflow Discharge for 'nrd' 20

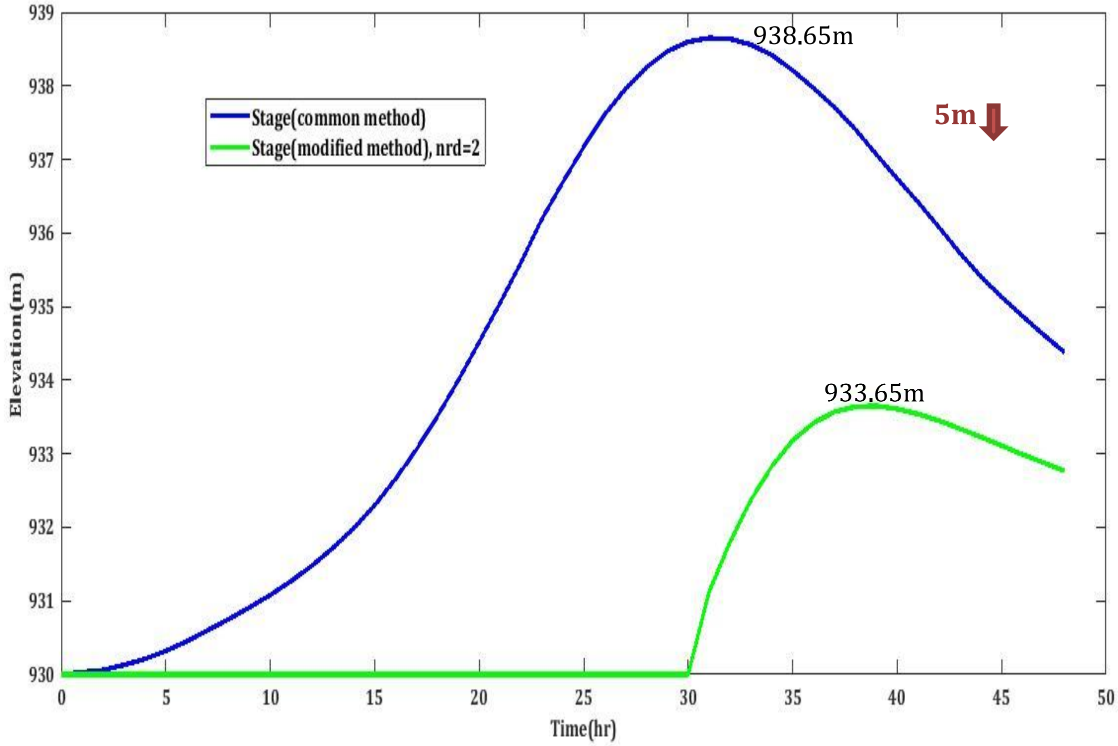


Figure 4.8 Estimated Stage for 'nrd' 2

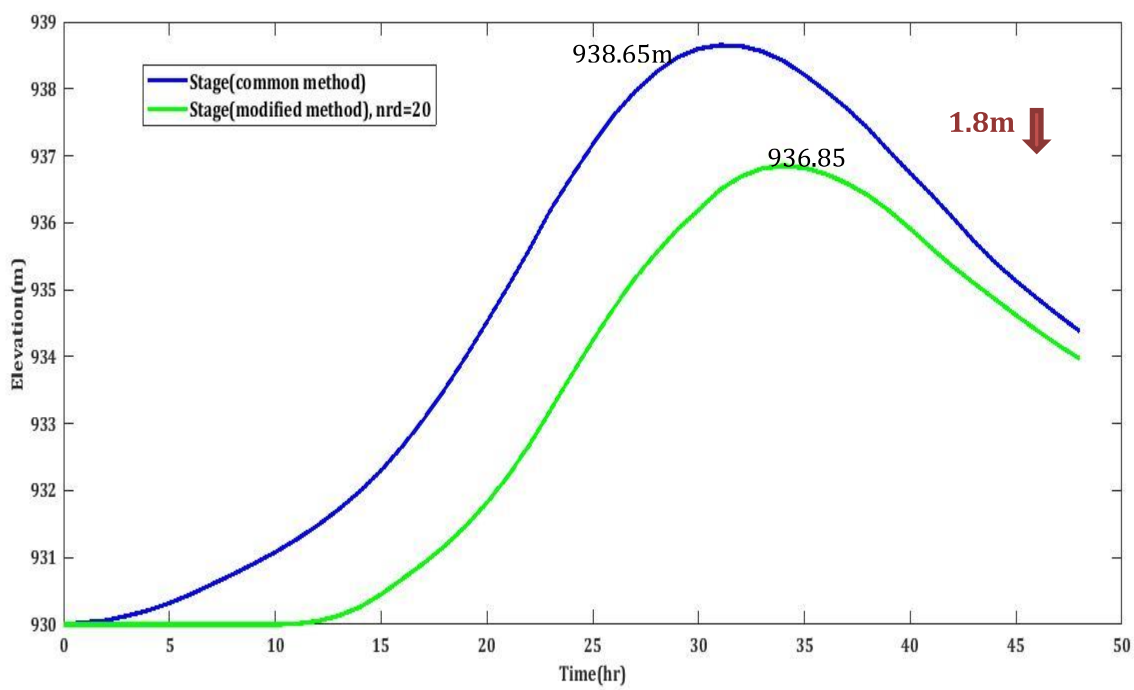


Figure 4.9 Estimated Stage for 'nrd' 20

The outflow discharge results for different 'nrd' values can be seen in Figure4.10 and in Annex 7.

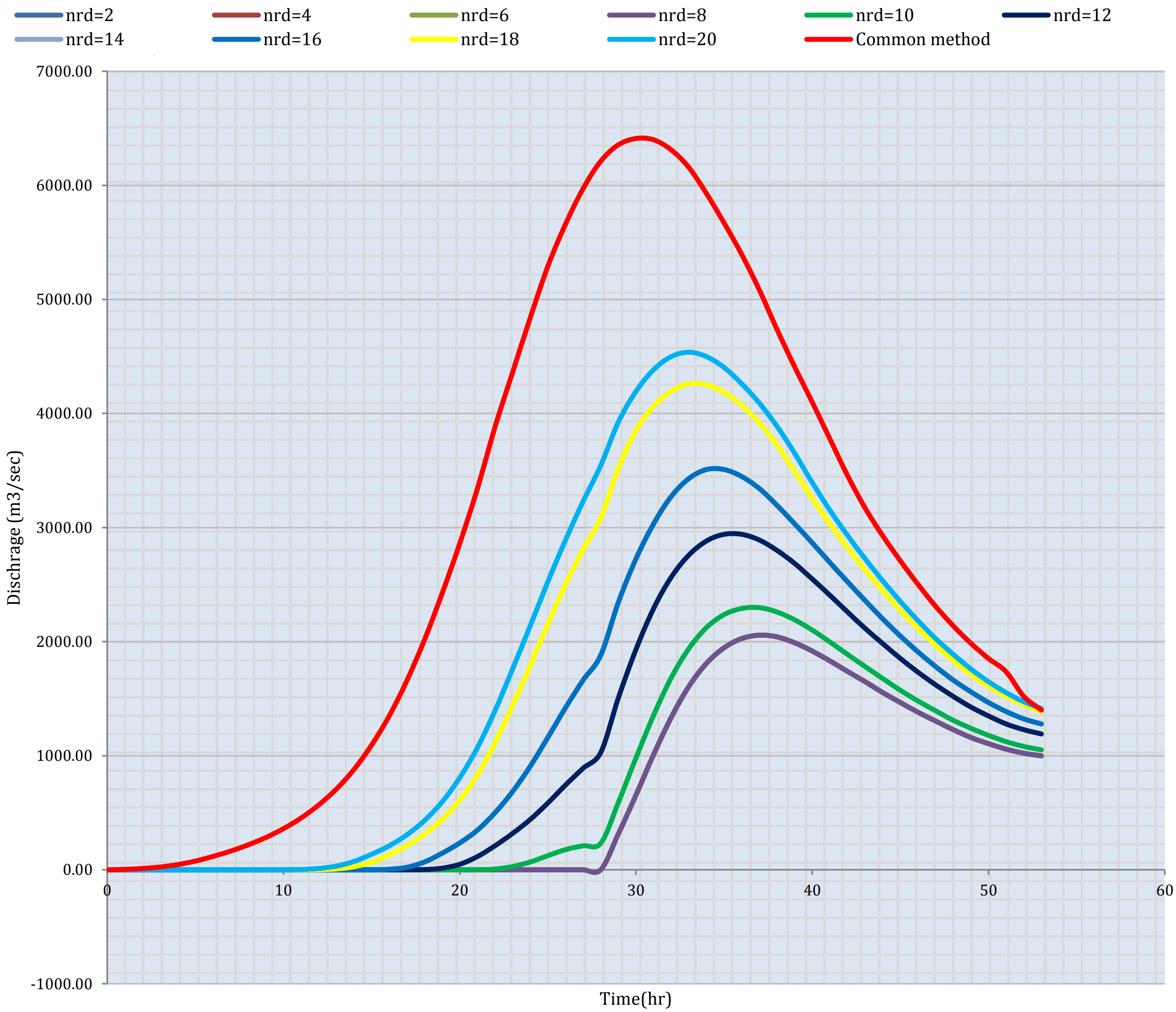


Figure 4.10 Estimated out flow discharge for different 'nrd' values

4.4. The Developed Standalone Application Result

Save an excel file in the same directory as the application file that contains time (hr) and inflow(m3/sec) from the inflow hydrograph, elevation(m) and reservoir capacity(m3) from the elevation area capacity curve, and elevation (m) and discharge(m3/sec) from the upstream rating curve, all in the same order. Include the '.dat' file for the inflow hydrograph in that directory as well. Then, in the app design view, enter the constant parameters and press the run button; the software will automatically generate the outflow discharge, and stage curve with its discrete data.

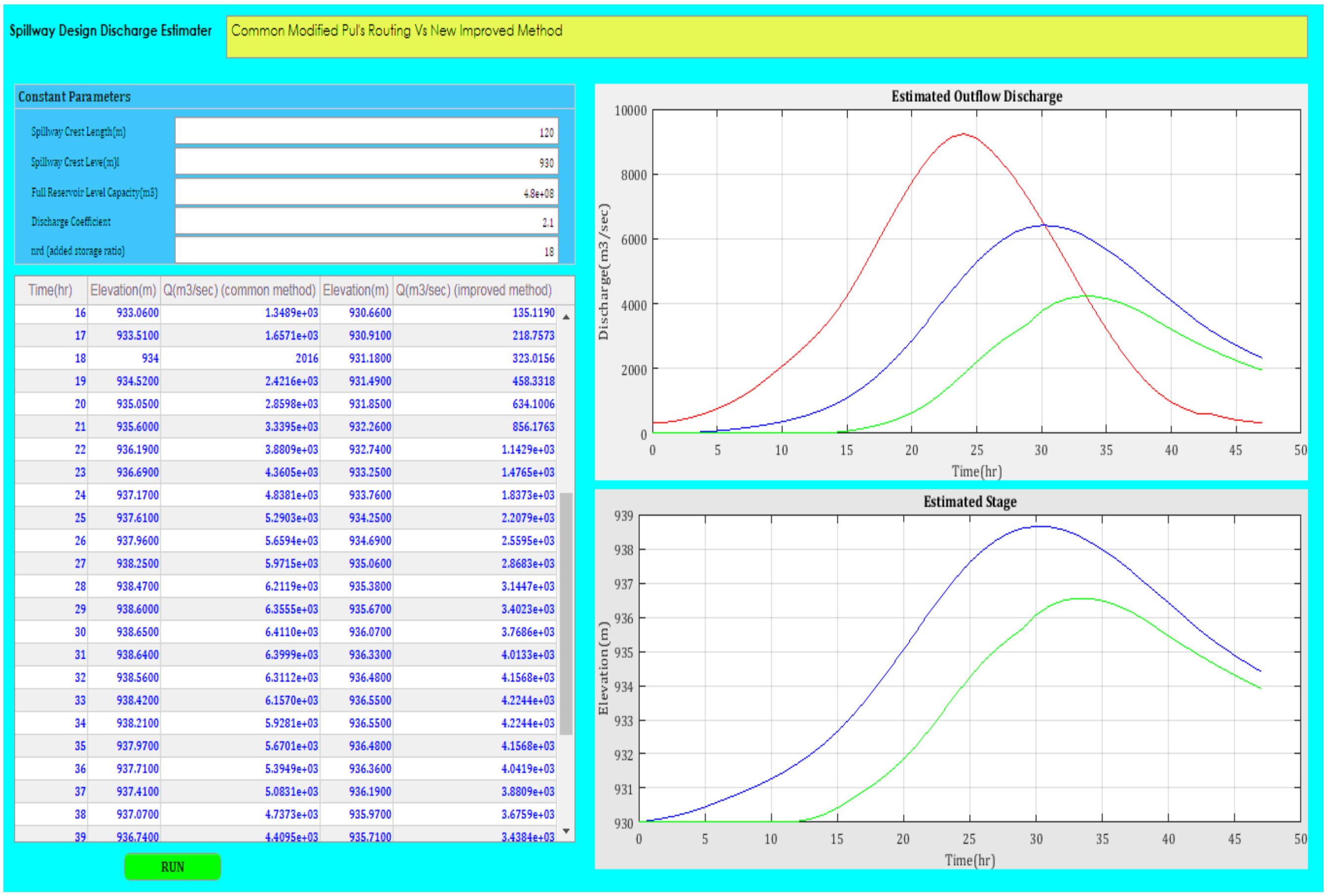


Figure 4.11 Developed standalone application design view

5. CHAPTER FIVE: - Conclusion and Recommendation

5.1. Conclusion

This research assesses the limitation of spillway design discharge estimation by using the common Modified Pul's Method and propose solution by taking Kesem Dam as case study site. The Modified Pul's Method uses a horizontal surface assumption which actually has been a sloping surface across the reservoir created by the water surface elevation at the entrance and exit (spillway). As such considerable amount of reservoir storage is ignored in the method. Considering this ignored storage in the common method by modifying the common routing method is the proposed solution in the research.

The research focuses in considering the ignored storage in the common method to do this estimating an upstream and downstream elevation is needed from an empirical manning equation for the upstream condition and a flow over ogee structure formula for the downstream condition. Compare these two upstream and downstream elevations at the same time t and take the maximum one for estimating the reservoir capacity. Accordingly, the result shows that there is considerable peak discharge (Spillway design discharge) reduction and there by design flood level reduction when this modified approach is adopted.

When the elevation by the upstream condition is higher than the downstream condition, the research attempted to demonstrate the computed additional storage by adding it to the previous storage of the common technique, rather than there being no additional storage. The out-flow discharge should be reduced as a result of this additional storage, and the research reveals that this discharge reduction with varied additional storage slopes, with the most maximum conceivable one being assumed to be half of the additional storage. To be made a simple and quick solution for this limitation a MATLAB code is developed which check the acceptability by comparing its result with design from the design report.

Generally, the research looked for solutions that modify the common Modified Pul's Method by discussing the spillway capacity estimation limitations and consider the ignored storage in this common method. This method will help in reducing the out-flow discharge and estimating the improved spillway discharge. This makes an important role in designing safe and economical structure.

5.2. Recommendation

This research has been conducted in major limitation of defining the exact water surface profile across the reservoir thus additional research should be carried out for estimating the exact slope of the water surface from the upstream to downstream. One approach can be adopting the gradually varied flow assumption.

Besides, this research employs the reservoir's elevation capacity curve, further research should be done to account for changes in the reservoir's surface area induced by changes in elevation across the reservoir generated by the water surface slope.

The result of the research is very helpful for real and more accurate estimation of spillway discharge. Therefore, other new spillway designs should refer and use this paper to design dam and spillway structures.

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Annex 1: Inflow Hydrograph

Time (hr)	Inflow (m3/s)	Time (hr)	Inflow (m3/s)	Time (hr)	Inflow (m3/s)	Time (hr)	Inflow (m3/s)
0	310	12	2769.82	24	**9237.77**	36	2621.89
0.5	324.92	12.5	2972.85	24.5	9169.47	36.5	2353.06
1	339.84	13	3175.87	25	9101.17	37	2084.24
1.5	369.69	13.5	3414.71	25.5	8934.23	37.5	1851.26
2	399.53	14	3653.54	26	8767.28	38	1618.28
2.5	444.3	14.5	3943.11	26.5	8549.4	38.5	1430.1
3	489.07	15	4232.68	27	8331.52	39	1241.92
3.5	548.76	15.5	4568.54	27.5	8074.78	39.5	1103.03
4	608.45	16	4904.41	28	7818.05	40	964.14
4.5	683.06	16.5	5271.6	28.5	7522.5	40.5	865.57
5	757.67	17	5638.79	29	7226.95	41	767
5.5	847.21	17.5	6007.47	29.5	6904.48	41.5	690.83
6	936.74	18	6376.15	30	6582.01	42	614.67
6.5	1048.66	18.5	6726.95	30.5	6249.05	42.5	556.42
7	1160.58	19	7077.76	31	5916.09	43	498.18
7.5	1294.88	19.5	7410.65	31.5	5572.69	43.5	457.85
8	1429.18	20	7743.54	32	5229.3	44	417.53
8.5	1585.88	20.5	8039.17	32.5	4875.44	44.5	390.65
9	1742.59	21	8334.79	33	4521.58	45	363.76
9.5	1905.34	21.5	8573.75	33.5	4185.55	45.5	345.84
10	2068.1	22	8812.71	34	3849.52	46	327.92
10.5	2236.82	22.5	8971.17	34.5	3531.41	46.5	318.96
11	2405.54	23	9129.64	35	3213.3	47	310
11.5	2587.68	23.5	9183.7	35.5	2917.59		

Annex 2: Designed Kesem Outflow Discharge from the Design Report

Time (min)	Outflow (m3/s)	Time (min)	Outflow (m3/s)	Time (min)	Outflow (m3/s)	Time (min)	Outflow (m3/s)
0	310.00	19	2,670.00	38	4,600.00	57	1,020.00
1	340.00	20	3,050.00	39	4,350.00	58	950.00
10	570.00	29	6,460.00	48	2,170.00	67	530.00
11	700.00	30	**6,530.00**	49	2,000.00	68	500.00
12	800.00	31	6,500.00	50	1,800.00	69	480.00
13	960.00	32	6,360.00	51	1,650.00	70	450.00
14	1,150.00	33	6,210.00	52	1,520.00	57	1,020.00
15	1,320.00	34	5,960.00	53	1,400.00	58	950.00
16	1,560.00	35	5,670.00	54	1,300.00	59	900.00
17	1,900.00	36	5,330.00	55	1,200.00	60	840.00
18	2,250.00	37	5,050.00	56	1,100.00	61	790.00
2	340.00	21	3,450.00	40	4,100.00	59	900.00
3	350.00	22	3,950.00	41	3,820.00	60	840.00
4	360.00	23	4,360.00	42	3,600.00	61	790.00
5	370.00	24	4,800.00	43	3,250.00	62	720.00
6	400.00	25	5,320.00	44	3,020.00	63	700.00
7	430.00	26	5,750.00	45	2,820.00	64	650.00
8	450.00	27	6,100.00	46	2,600.00	65	600.00
9	510.00	28	6,350.00	47	2,400.00	66	560.00

Annex 3: Elevation – Capacity Curve

Elevation (m)	Capacity (MCM)	Elevation (m)	Capacity (MCM)	Elevation (m)	Capacity (MCM)	Elevation (m)	Capacity (MCM)
860	0	888	13.3	914	165	935.59	642.74
862	0.2	890	17.8	916	190	936.29	664.29
864	0.5	892	20.67	918	216	936.9	685.84
866	0.9	894	27.26	920	248	937.55	707.39
868	1.4	896	35.72	922	285	938.16	728.94
870	1.7	898	43.5	924	324	938.77	750.48
872	2	898.5	46.25	926	365	939.41	772.03
874	2.4	900	52	928	410	939.96	792.6
876	3.07	902	62.5	930	480	940	800
878	3.6	904	74.68	930.64	491.87		
880	5.1	906	88.9	931.42	513.43		
882	6.13	908	102	932.12	534.98		
884	8.51	910	120	932.84	556.53		
886	11.56	912	143	933.56	578.08		

Annex 4: Profile along the River Flow

Distance (m)	Elevation (m)	Distance (m)	Elevation (m)	Distance (m)	Elevation (m)	Distance (m)	Elevation (m)
663.13009	940.3513	1285.7911	924.8462	1904.5275	922.3958	2518.0313	916.9754
692.78061	937.0629	1315.4416	924.3403	1933.742	922.5356	2547.2458	917.9466
722.43113	933.7525	1345.0921	923.541	1962.9565	922.1531	2576.4602	918.9178
752.08166	930.6085	1374.7426	923.2119	1992.1709	921.3217	2605.6747	919.2211
781.73218	928.9243	1404.3932	923.7506	2021.3854	920.4903	2634.8892	919.1088
811.3827	929.0362	1434.0437	925.0259	2050.5999	920.1898	2664.1036	919.8057
841.03323	929.5804	1463.6942	926.3012	2079.8143	920.0622	2693.3181	919.8292
870.68375	928.6922	1493.3447	927.543	2109.0288	919.6073	2722.5326	918.9155
900.33427	927.8407	1522.9953	928.206	2138.2433	919.5143	2751.747	917.0593
929.9848	927.0443	1552.6458	929.1271	2167.4577	919.6541	2780.9615	915.9869
959.63532	926.197	1582.2963	929.4568	2196.6722	919.5621	2810.176	915.3543
989.28584	924.9877	1611.9468	929.3869	2225.8867	919	2839.3904	915.7889
1018.9364	923.685	1641.5974	929.0221	2255.1011	919.0563	2868.6049	917.2539
1048.5869	923.3187	1670.8118	928.1898	2284.3156	918.7943	2897.8194	919.0529
1078.2374	923.0096	1700.0263	926.8927	2313.5301	917.8231		
1107.8879	922.1413	1729.2407	925.6905	2342.7445	916.8519		
1137.5385	922	1758.4552	924.3599	2371.959	915.9245		
1167.189	922.057	1787.6697	923.3049	2401.1735	915.2068		
1196.8395	922.6385	1816.8841	922.3495	2430.3879	915		
1226.49	924.442	1846.0986	922.1162	2459.6024	915.033		
1256.1406	925.1553	1875.3131	922.256	2488.8168	916.0042		

Annex 5: Reservoir Entrance Cross-Sectional Data

Distance (m)	Elevation (m)	Distance (m)	Elevation (m3/s)	Distance (m)	Elevation (m3/s)	Distance (m)	Elevation (m3/s)
0.00	1052.09	533.74	1020.70	1067.48	995.05	1601.22	932.47
29.65	1045.43	563.39	1021.01	1097.13	989.22	1630.87	935.65
59.30	1040.43	593.04	1021.76	1126.79	982.16	1660.53	939.16
88.96	1035.67	622.70	1023.10	1156.44	975.26	1690.18	941.90
118.61	1029.96	652.35	1023.58	1186.09	970.77	1719.83	944.43
148.26	1022.50	682.00	1021.34	1215.74	968.08	1749.48	946.25
177.91	1013.90	711.65	1018.87	1245.39	964.84	1779.13	947.80
207.57	1007.01	741.31	1017.63	1275.05	960.15	1808.79	949.02
237.22	1004.10	770.96	1016.64	1304.70	953.64	1838.44	950.16
266.87	1005.57	800.61	1015.70	1334.35	946.60	1868.09	952.10
296.52	1008.32	830.26	1015.61	1364.00	940.39	1897.74	955.40
326.17	1010.49	859.92	1016.02	1393.66	935.14	1927.40	958.73
355.83	1011.71	889.57	1015.75	1423.31	931.83	1957.05	961.10
385.48	1013.31	919.22	1013.88	1452.96	930.32	1986.70	962.00
415.13	1016.38	948.87	1011.33	1482.61	929.40	2016.35	962.57
444.78	1018.54	978.52	1008.48	1512.26	929.05	2046.00	962.93
474.44	1019.00	1008.18	1004.46	1541.92	929.26	2075.66	962.73
504.09	1019.72	1037.83	999.88	1571.57	930.82	2105.31	962.53

Distance (m)	Elevation (m)	Distance (m)	Elevation (m3/s)	Distance (m)	Elevation (m3/s)
2134.96	962.47	2668.70	951.02	3202.44	985.77
2164.61	962.32	2698.35	951.55	3232.09	988.53
2194.27	962.24	2728.01	953.24	3261.75	991.18
2223.92	962.39	2757.66	954.09	3291.40	993.63
2253.57	961.63	2787.31	954.88	3321.05	995.90
2283.22	960.75	2816.96	955.00	3350.70	997.95
2312.87	959.18	2846.62	954.51	3380.36	999.44
2342.53	957.87	2876.27	955.92	3410.01	1000.14
2372.18	956.63	2905.92	958.34	3439.66	1000.88
2401.83	954.66	2935.57	962.06	3469.31	1003.85
2431.48	954.13	2965.22	966.01	3498.96	1009.65
2461.14	953.78	2994.88	969.57	3528.62	1015.74
2490.79	954.00	3024.53	971.37	3558.27	1018.29
2520.44	954.41	3054.18	972.74	3587.92	1026.68
2550.09	955.16	3083.83	974.93	3617.57	1034.47
2579.75	955.32	3113.49	977.86	3647.23	1040.15
2609.40	954.12	3143.14	980.79	3676.88	1042.53
2639.05	951.97	3172.79	983.48		

Annex 6: Upstream Rating Curve Data from BlentleyFlowmaster Software

Rating Table for Irregular Section

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.007
Normal Depth	11.3

Section Definitions

Roughness Segment Definitions

Start Station & Elevation	End Station & Elevation	Roughness Coefficient
(0+00, 1,019.91)	(34+99, 1,038.02)	0.070

Water Surface Elevation (m)	Discharge (m^3/s)	Velocity (m/s)	Flow Area (m^2)	Wetted Perimeter (m)	Top Width (m)
930.00	(N/A)	(N/A)	(N/A)	(N/A)	(N/A)
930.10	0.80	0.22	3.6	42.0	42.03
930.20	2.79	0.33	8.4	54.4	54.39
930.30	6.01	0.42	14.5	66.8	66.76
930.40	11.00	0.51	21.5	72.6	72.57
930.50	17.40	0.60	29.0	76.9	76.88
930.60	25.08	0.68	36.9	81.2	81.19
930.70	34.03	0.75	45.2	85.5	85.50
930.80	44.25	0.82	54.0	89.8	89.81
930.90	55.75	0.88	63.2	94.2	94.12
931.00	68.81	0.95	72.8	97.9	97.81
931.10	83.78	1.01	82.7	100.2	100.12
931.20	100.04	1.08	92.8	102.5	102.44
931.30	117.56	1.14	103.2	104.8	104.75
931.40	136.35	1.20	113.8	107.1	107.07
931.50	156.40	1.26	124.6	109.5	109.38
931.60	177.70	1.31	135.6	111.8	111.70
931.70	200.25	1.36	146.9	114.1	114.01
931.80	224.06	1.41	158.4	116.4	116.33

931.90	249.13	1.46	170.2	118.8	118.64
932.00	275.46	1.51	182.2	121.1	120.96
932.10	303.05	1.56	194.4	123.4	123.27
932.20	331.92	1.60	206.8	125.7	125.59
932.30	362.06	1.65	219.5	128.1	127.90
932.40	393.49	1.69	232.4	130.4	130.22
932.50	426.20	1.74	245.5	132.7	132.53
932.60	460.22	1.78	258.9	135.0	134.85
932.70	495.60	1.82	272.5	137.3	137.14
932.80	533.14	1.86	286.3	139.3	139.08
932.90	572.00	1.90	300.3	141.2	141.01
933.00	612.16	1.95	314.5	143.2	142.95
933.10	653.65	1.99	328.9	145.1	144.88
933.20	696.45	2.03	343.5	147.1	146.82
933.30	740.59	2.07	358.3	149.0	148.75
933.40	786.06	2.11	373.2	151.0	150.69
933.50	832.86	2.14	388.4	152.9	152.62
933.60	881.02	2.18	403.8	154.8	154.56
933.70	930.52	2.22	419.3	156.8	156.49
933.80	982.22	2.26	435.1	158.5	158.22
933.90	1,036.07	2.30	451.0	160.1	159.75
934.00	1,091.28	2.34	467.0	161.6	161.27
934.10	1,147.83	2.38	483.2	163.1	162.80
934.20	1,205.75	2.41	499.6	164.7	164.32
934.30	1,265.01	2.45	516.1	166.2	165.85
934.40	1,325.64	2.49	532.7	167.8	167.37
934.50	1,387.63	2.52	549.6	169.3	168.90
934.60	1,450.98	2.56	566.5	170.8	170.42
934.70	1,515.70	2.60	583.6	172.4	171.95
934.80	1,581.79	2.63	600.9	173.9	173.47
934.90	1,649.26	2.67	618.3	175.5	175.00
935.00	1,718.10	2.70	635.9	177.0	176.52
935.10	1,788.32	2.74	653.6	178.5	178.05
935.20	1,859.93	2.77	671.5	180.1	179.58
935.30	1,932.93	2.80	689.6	181.6	181.10
935.40	2,007.32	2.84	707.7	183.1	182.63
935.50	2,083.10	2.87	726.1	184.7	184.15
935.60	2,160.29	2.90	744.6	186.2	185.68
935.70	2,238.88	2.93	763.2	187.8	187.20
935.80	2,318.88	2.97	782.0	189.3	188.73
935.90	2,400.29	3.00	801.0	190.8	190.25
936.00	2,483.12	3.03	820.1	192.4	191.78
936.10	2,567.38	3.06	839.3	193.9	193.30
936.20	2,654.42	3.09	858.7	195.3	194.67
936.30	2,742.99	3.12	878.3	196.7	196.03

936.40	2,833.00	3.16	897.9	198.0	197.39
936.50	2,924.42	3.19	917.7	199.4	198.74
936.60	3,017.28	3.22	937.7	200.8	200.10
936.70	3,111.58	3.25	957.8	202.2	201.46
936.80	3,207.31	3.28	978.0	203.5	202.81
936.90	3,304.48	3.31	998.3	204.9	204.17
937.00	3,403.09	3.34	1,018.8	206.3	205.53
937.10	3,503.15	3.37	1,039.4	207.6	206.88
937.20	3,604.66	3.40	1,060.2	209.0	208.24
937.30	3,707.63	3.43	1,081.1	210.4	209.60
937.40	3,812.05	3.46	1,102.1	211.8	210.96
937.50	3,917.93	3.49	1,123.3	213.1	212.31
937.60	4,025.28	3.52	1,144.6	214.5	213.67
937.70	4,134.10	3.55	1,166.0	215.9	215.03
937.80	4,244.40	3.57	1,187.6	217.2	216.38
937.90	4,356.16	3.60	1,209.3	218.6	217.74
938.00	4,469.41	3.63	1,231.1	220.0	219.10
938.10	4,584.15	3.66	1,253.1	221.4	220.45
938.20	4,700.37	3.69	1,275.2	222.7	221.81
938.30	4,818.68	3.71	1,297.4	224.1	223.12
938.40	4,939.59	3.74	1,319.8	225.3	224.36
938.50	5,061.99	3.77	1,342.3	226.6	225.60
938.60	5,185.88	3.80	1,364.9	227.8	226.83
938.70	5,311.27	3.83	1,387.7	229.1	228.07
938.80	5,438.16	3.86	1,410.6	230.3	229.30
938.90	5,566.54	3.88	1,433.5	231.6	230.54
939.00	5,696.44	3.91	1,456.7	232.8	231.77
939.10	5,827.84	3.94	1,479.9	234.1	233.01
939.20	5,960.75	3.97	1,503.3	235.3	234.24
939.30	6,095.17	3.99	1,526.8	236.6	235.48
939.40	6,231.11	4.02	1,550.4	237.8	236.71
939.50	6,368.58	4.05	1,574.1	239.1	237.95
939.60	6,507.56	4.07	1,598.0	240.3	239.18
939.70	6,648.08	4.10	1,621.9	241.6	240.42
939.80	6,790.12	4.13	1,646.0	242.8	241.65
939.90	6,933.70	4.15	1,670.3	244.1	242.89
940.00	7,078.81	4.18	1,694.6	245.3	244.13

file1.fm8	Bentley Systems, Inc. Haestad Methods Solution Center	FlowMaster [10.00.00.02]
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Annex 7: Outflow Discharge for Different 'nrd' Values

Time (hr)	Outflow Discharge (m^3/sec)									
	nrd=2	nrd=4	nrd=6	nrd=8	nrd=10	nrd=12	nrd=14	nrd=16	nrd=18	nrd=20
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.24
11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.71
12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.26	11.36
13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	7.67	32.14
14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	26.74	73.17
15	0.00	0.00	0.00	0.00	0.00	0.00	0.24	0.24	65.98	135.92
16	0.00	0.00	0.00	0.00	0.00	0.00	4.49	4.49	129.97	210.42
17	0.00	0.00	0.00	0.00	0.00	0.00	21.68	21.68	210.42	306.77
18	0.00	0.00	0.00	0.00	0.00	1.94	68.35	68.35	310.71	432.02
19	0.00	0.00	0.00	0.00	0.00	14.08	145.02	145.02	440.87	595.17
20	0.00	0.00	0.00	0.00	0.00	48.06	235.16	235.16	609.94	807.22
21	0.00	0.00	0.00	0.00	0.00	115.49	346.93	346.93	829.03	1069.45
22	0.00	0.00	0.00	0.00	6.54	210.42	499.81	499.81	1111.47	1394.09
23	0.00	0.00	0.00	0.00	28.50	318.64	685.61	685.61	1439.94	1760.27
24	0.00	0.00	0.00	0.00	68.35	440.87	906.90	906.90	1788.51	2138.82
25	0.00	0.00	0.00	0.00	124.11	585.38	1160.14	1160.14	2153.87	2525.28
26	0.00	0.00	0.00	0.00	176.71	742.94	1420.23	1420.23	2501.50	2890.94
27	0.00	0.00	0.00	0.00	210.42	890.02	1662.62	1662.62	2808.26	3229.51
28	0.00	0.00	0.00	0.00	231.57	1027.98	1881.32	1881.32	3084.09	3544.74
29	314.67	314.67	314.67	314.67	590.27	1506.29	2344.86	2344.86	3518.10	3924.66
30	654.99	654.99	654.99	654.99	981.26	1931.93	2726.39	2726.39	3851.31	4194.42
31	1010.37	1010.37	1010.37	1010.37	1355.18	2290.84	3033.30	3033.30	4063.45	4383.91
32	1335.87	1335.87	1335.87	1335.87	1683.39	2565.09	3272.71	3272.71	4194.42	4498.93
33	1607.66	1607.66	1607.66	1607.66	1939.20	2759.04	3429.78	3429.78	4260.42	4537.50
34	1809.79	1809.79	1809.79	1809.79	2123.81	2882.63	3509.23	3509.23	4250.97	4498.93
35	1946.48	1946.48	1946.48	1946.48	2237.24	2940.92	3509.23	3509.23	4185.02	4403.01

36	2027.12	2027.12	2027.12	2027.12	2290.84	2940.92	3447.38	3447.38	4072.76	4260.42
37	2056.71	2056.71	2056.71	2056.71	2298.53	2890.94	3342.21	3342.21	3915.47	4091.40
38	2041.89	2041.89	2041.89	2041.89	2260.15	2800.04	3195.10	3195.10	3724.05	3887.93
39	1990.33	1990.33	1990.33	1990.33	2191.63	2685.76	3033.30	3033.30	3491.53	3651.97
40	1917.42	1917.42	1917.42	1917.42	2101.36	2549.14	2866.05	2866.05	3255.41	3394.66
41	1831.15	1831.15	1831.15	1831.15	1997.67	2407.11	2693.87	2693.87	3033.30	3152.25
42	1739.19	1739.19	1739.19	1739.19	1888.52	2260.15	2525.28	2525.28	2824.73	2932.57
43	1648.82	1648.82	1648.82	1648.82	1781.44	2116.32	2360.37	2360.37	2629.22	2726.39
44	1553.32	1553.32	1553.32	1553.32	1676.46	1983.00	2199.21	2199.21	2446.29	2533.23
45	1466.36	1466.36	1466.36	1466.36	1573.63	1852.59	2049.30	2049.30	2275.48	2352.61
46	1381.08	1381.08	1381.08	1381.08	1479.63	1732.18	1910.18	1910.18	2116.32	2184.06
47	1303.89	1303.89	1303.89	1303.89	1394.09	1621.34	1781.44	1781.44	1968.36	2027.12
48	1228.19	1228.19	1228.19	1228.19	1310.26	1519.68	1662.62	1662.62	1831.15	1888.52
49	1160.14	1160.14	1160.14	1160.14	1240.70	1426.79	1560.08	1560.08	1711.21	1760.27
50	1105.43	1105.43	1105.43	1105.43	1178.57	1348.74	1466.36	1466.36	1607.66	1648.82
51	1057.55	1057.55	1057.55	1057.55	1123.57	1278.49	1387.58	1387.58	1519.68	1553.32
52	1022.10	1022.10	1022.10	1022.10	1081.40	1228.19	1323.05	1323.05	1446.53	1472.99
53	998.69	998.69	998.69	998.69	1051.61	1190.91	1278.49	1278.49	1387.58	1413.68
54	987.06	987.06	987.06	987.06	1039.77	1166.27	1246.97	1246.97	1348.74	1368.11
55	753.53	867.67	906.90	929.57	992.87	1129.64	1215.72	1215.72	1316.65	1335.87
56	397.24	701.09	807.22	862.12	940.97	1093.39	1184.74	1184.74	1291.17	1310.26
57	0.00	495.19	685.61	780.22	884.41	1051.61	1154.02	1154.02	1272.16	1297.52
58	0.00	257.09	546.80	690.76	829.03	1016.23	1129.64	1129.64	1265.85	1291.17
59	0.00	0.00	392.96	595.17	769.51	981.26	1111.47	1111.47	1265.85	1297.52
60	0.00	0.00	231.57	495.19	711.48	952.43	1099.41	1099.41	1278.49	1316.65
61	0.00	0.00	39.83	397.24	660.06	929.57	1099.41	1099.41	1303.89	1355.18
62	0.00	0.00	0.00	298.94	609.94	918.21	1111.47	1111.47	1348.74	1413.68
63	0.00	0.00	0.00	193.32	565.98	918.21	1141.81	1141.81	1413.68	1492.94
64	0.00	0.00	0.00	75.63	527.83	929.57	1190.91	1190.91	1506.29	1600.84
65	0.00	0.00	0.00	0.00	499.81	963.92	1265.85	1265.85	1635.06	1739.19
66	0.00	0.00	0.00	0.00	481.41	1022.10	1368.11	1368.11	1795.59	1917.42
67	0.00	0.00	0.00	0.00	481.41	1105.43	1506.29	1506.29	1997.67	2138.82
68	0.00	0.00	0.00	0.00	490.58	1209.50	1676.46	1676.46	2229.61	2391.49
69	0.00	0.00	0.00	0.00	518.44	1348.74	1881.32	1881.32	2501.50	2677.66
70	0.00	0.00	0.00	0.00	580.51	1539.83	2131.31	2131.31	2808.26	3008.01
71	0.00	0.00	0.00	0.00	660.06	1753.23	2407.11	2407.11	3152.25	3385.90
72	0.00	0.00	0.00	0.00	753.53	1983.00	2693.87	2693.87	3535.85	3778.41
73	0.00	0.00	0.00	0.00	862.12	2229.61	2991.19	2991.19	3897.10	4128.76
74	0.00	0.00	0.00	0.00	975.47	2469.90	3290.04	3290.04	4203.83	4450.89
75	0.00	0.00	0.00	0.00	1069.45	2669.56	3562.54	3562.54	4479.69	4741.74

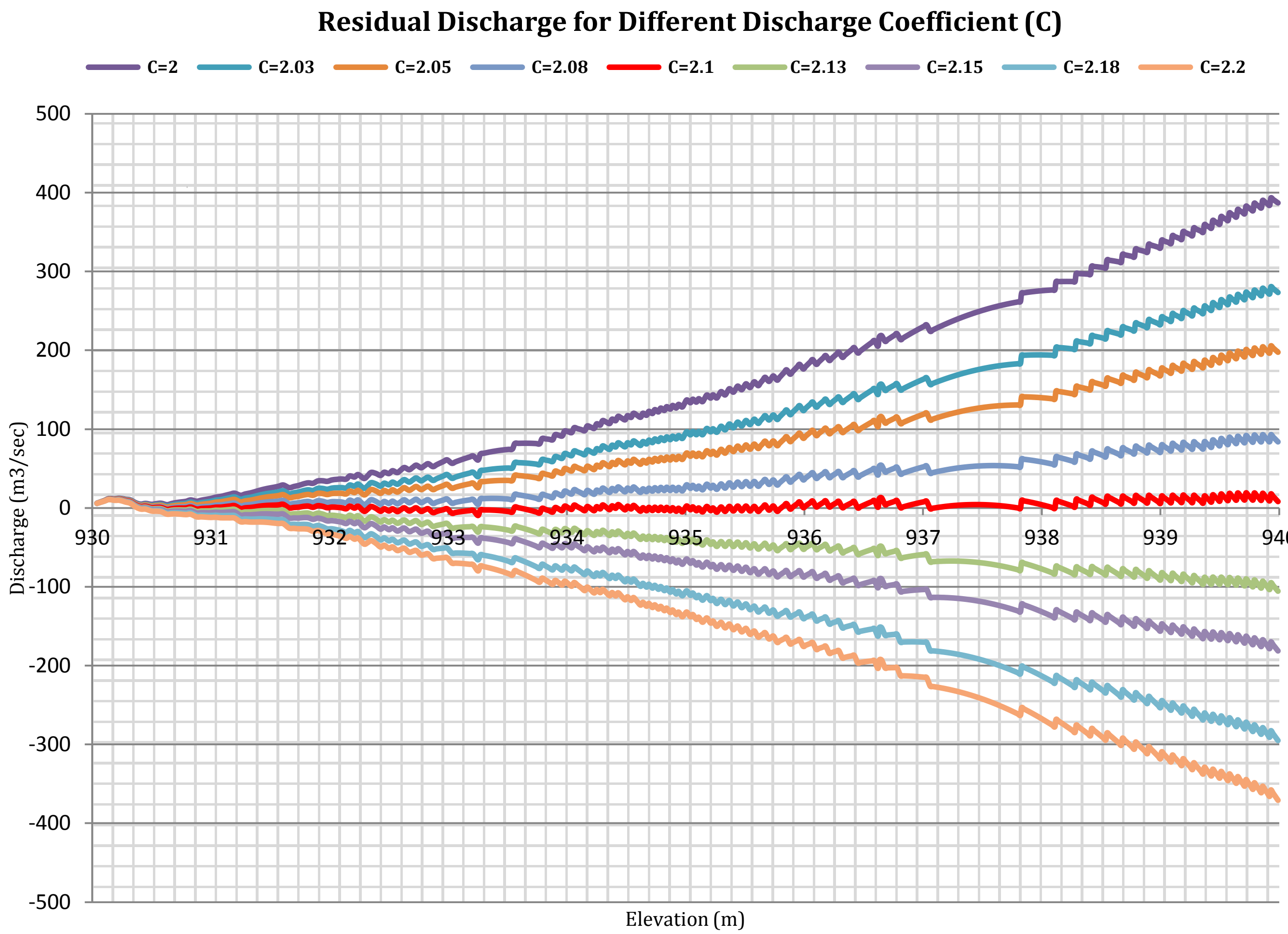
76	0.00	0.00	0.00	0.00	1147.91	2849.50	3796.59	3796.59	4722.16	4988.76
77	314.67	314.67	314.67	314.67	1628.20	3272.71	4138.12	4138.12	5008.70	5239.93
78	654.99	654.99	654.99	654.99	2049.30	3634.03	4393.46	4393.46	5189.36	5402.82
79	1010.37	1010.37	1010.37	1010.37	2399.30	3887.93	4576.17	4576.17	5290.65	5484.89
80	1335.87	1335.87	1335.87	1335.87	2669.56	4035.56	4673.32	4673.32	**5321.16**	**5495.17**
81	1607.66	1607.66	1607.66	1607.66	2857.77	4110.06	**4692.84**	**4692.84**	5280.49	5433.55
82	1809.79	1809.79	1809.79	1809.79	2974.40	**4119.41**	4644.11	4644.11	5179.27	5310.99
83	1946.48	1946.48	1946.48	1946.48	3024.87	4063.45	4527.85	4527.85	5018.68	5138.97
84	2027.12	2027.12	2027.12	2027.12	**3016.44**	3961.52	4374.37	4374.37	4800.64	4919.17
85	**2056.71**	**2056.71**	**2056.71**	**2056.71**	2957.65	3814.80	4185.02	4185.02	4547.15	4653.84
86	2041.89	2041.89	2041.89	2041.89	2857.77	3625.06	3979.98	3979.98	4288.81	4374.37
87	1990.33	1990.33	1990.33	1990.33	2734.54	3403.43	3742.14	3742.14	4035.56	4110.06
88	1917.42	1917.42	1917.42	1917.42	2597.09	3177.93	3482.68	3482.68	3769.33	3842.17
89	1831.15	1831.15	1831.15	1831.15	2454.15	2966.02	3220.90	3220.90	3482.68	3553.64
90	1739.19	1739.19	1739.19	1739.19	2306.23	2767.23	2991.19	2991.19	3203.69	3264.05
91	1648.82	1648.82	1648.82	1648.82	2161.40	2581.08	2783.62	2783.62	2966.02	3016.44
92	1553.32	1553.32	1553.32	1553.32	2019.74	2399.30	2589.08	2589.08	2750.87	2791.82
93	1466.36	1466.36	1466.36	1466.36	1888.52	2229.61	2407.11	2407.11	2549.14	2589.08
94	1381.08	1381.08	1381.08	1381.08	1767.32	2071.56	2237.24	2237.24	2368.14	2399.30
95	1303.89	1303.89	1303.89	1303.89	1655.72	1931.93	2079.00	2079.00	2199.21	2229.61

Annex 8: Downstream Rating Curve with Different Discharge Coefficient

Downstream rating curve from the design report		Calculated downstream rating curve for different C values								
Elevation	Out Flow	Outflow m^3/sec								
(m)	(CMS)	C=2	C=2.03	C=2.05	C=2.08	C=2.1	C=2.13	C=2.15	C=2.18	C=2.2
930.1	7.94	7.589	7.703	7.779	7.893	7.969	8.083	8.159	8.273	8.348
930.2	16.9	21.47	21.79	22	22.32	22.54	22.86	23.08	23.4	23.61
930.3	24.4	39.44	40.03	40.42	41.01	41.41	42	42.39	42.99	43.38
930.4	31.53	60.72	61.63	62.23	63.14	63.75	64.66	65.27	66.18	66.79
930.5	38.78	84.85	86.13	86.97	88.25	89.1	90.37	91.22	92.49	93.34
930.6	46.64	111.5	113.2	114.3	116	117.1	118.8	119.9	121.6	122.7
930.7	53.69	140.6	142.7	144.1	146.2	147.6	149.7	151.1	153.2	154.6
930.8	60.34	171.7	174.3	176	178.6	180.3	182.9	184.6	187.2	188.9
930.9	67.84	204.9	208	210	213.1	215.2	218.2	220.3	223.4	225.4
931	78.16	240	243.6	246	249.6	252	255.6	258	261.6	264
931.1	89.41	276.9	281	283.8	288	290.7	294.9	297.7	301.8	304.6
931.2	100.66	315.5	320.2	323.4	328.1	331.3	336	339.1	343.9	347
931.3	111.92	355.7	361.1	364.6	370	373.5	378.9	382.4	387.8	391.3
931.4	124.11	397.6	403.5	407.5	413.5	417.4	423.4	427.4	433.3	437.3
931.5	137.23	440.9	447.5	451.9	458.5	463	469.6	474	480.6	485
931.6	150.36	485.7	493	497.9	505.2	510	517.3	522.2	529.4	534.3
931.7	163.49	532	539.9	545.3	553.2	558.6	566.5	571.9	579.8	585.2
931.8	176.87	579.6	588.3	594.1	602.8	608.6	617.3	623.1	631.8	637.5
931.9	191.62	628.6	638	644.3	653.7	660	669.4	675.7	685.1	691.4
932	206.62	678.8	689	695.8	706	712.8	722.9	729.7	739.9	746.7
932.1	221.63	730.4	741.3	748.6	759.6	766.9	777.8	785.1	796.1	803.4
932.2	236.63	783.2	794.9	802.7	814.5	822.3	834.1	841.9	853.6	861.5
932.3	251.96	837.1	849.7	858.1	870.6	879	891.6	899.9	912.5	920.9
932.4	268.18	892.3	905.7	914.6	928	937	950.3	959.3	972.6	981.6
932.5	283.84	948.7	962.9	972.4	986.6	996.1	1010	1020	1034	1044
932.6	300.39	1006	1021	1031	1046	1056	1072	1082	1097	1107
932.7	317.27	1065	1081	1091	1107	1118	1134	1145	1161	1171
932.8	334.15	1124	1141	1153	1169	1181	1198	1209	1226	1237
932.9	351.42	1185	1203	1215	1233	1245	1262	1274	1292	1304
933	369.39	1247	1266	1278	1297	1309	1328	1341	1359	1372
933.1	387.05	1310	1330	1343	1362	1375	1395	1408	1428	1441
933.2	405.41	1374	1394	1408	1429	1443	1463	1477	1497	1511
933.3	424.17	1439	1460	1475	1496	1511	1532	1547	1568	1583

933.4	442.92	1505	1527	1542	1565	1580	1602	1617	1640	1655
933.5	461.67	1571	1595	1611	1634	1650	1674	1689	1713	1729
933.6	480.43	1639	1664	1680	1705	1721	1746	1762	1787	1803
933.7	499.63	1708	1734	1751	1776	1794	1819	1836	1862	1879
933.8	519.36	1778	1804	1822	1849	1867	1893	1911	1938	1956
933.9	539.01	1848	1876	1895	1922	1941	1969	1987	2015	2033
934	559.19	1920	1949	1968	1997	2016	2045	2064	2093	2112
934.1	579.38	1992	2022	2042	2072	2092	2122	2142	2172	2192
934.2	600.45	2066	2097	2117	2148	2169	2200	2221	2252	2272
934.3	621.08	2140	2172	2194	2226	2247	2279	2301	2333	2354
934.4	640.06	2215	2248	2270	2304	2326	2359	2381	2414	2437
934.5	662.79	2291	2325	2348	2383	2406	2440	2463	2497	2520
934.6	682.97	2368	2403	2427	2463	2486	2522	2545	2581	2605
934.7	702.41	2445	2482	2507	2543	2568	2604	2629	2666	2690
934.8	724.56	2524	2562	2587	2625	2650	2688	2713	2751	2776
934.9	745.85	2603	2642	2668	2707	2733	2772	2798	2837	2864
935	767.14	2683	2724	2750	2791	2817	2858	2885	2925	2952
935.1	787.87	2764	2806	2833	2875	2902	2944	2971	3013	3041
935.2	807.8	2846	2889	2917	2960	2988	3031	3059	3102	3130
935.3	827.72	2928	2972	3002	3045	3075	3119	3148	3192	3221
935.4	846.95	3012	3057	3087	3132	3162	3207	3238	3283	3313
935.5	869.06	3096	3142	3173	3220	3250	3297	3328	3374	3405
935.6	888.83	3180	3228	3260	3308	3340	3387	3419	3467	3499
935.7	909.64	3266	3315	3348	3397	3429	3478	3511	3560	3593
935.8	930.52	3352	3403	3436	3486	3520	3570	3604	3654	3688
935.9	950.37	3439	3491	3525	3577	3611	3663	3697	3749	3783
936	971.31	3527	3580	3615	3668	3704	3757	3792	3845	3880
936.1	992.25	3616	3670	3706	3760	3797	3851	3887	3941	3977
936.2	1013.03	3705	3761	3798	3853	3890	3946	3983	4039	4076
936.3	1033.66	3795	3852	3890	3947	3985	4042	4080	4137	4175
936.4	1054.29	3886	3944	3983	4041	4080	4138	4177	4236	4274
936.5	1074.92	3977	4037	4077	4136	4176	4236	4276	4335	4375
936.6	1095.55	4069	4130	4171	4232	4273	4334	4375	4436	4476
936.7	1116.18	4162	4225	4266	4329	4370	4433	4474	4537	4578
936.8	1136.81	4256	4320	4362	4426	4469	4532	4575	4639	4681
936.9	1157.44	4350	4415	4459	4524	4567	4633	4676	4741	4785
937	1178.07	4445	4512	4556	4623	4667	4734	4778	4845	4889
937.1	1198.7	4540	4609	4654	4722	4767	4836	4881	4949	4994
937.2	1219.33	4637	4706	4753	4822	4869	4938	4984	5054	5100
937.3	1239.95	4734	4805	4852	4923	4970	5041	5089	5160	5207

937.4	1260.58	4831	4904	4952	5024	5073	5145	5194	5266	5314
937.5	1281.21	4930	5003	5053	5127	5176	5250	5299	5373	5422
937.6	1301.84	5028	5104	5154	5230	5280	5355	5406	5481	5531
937.7	1322.47	5128	5205	5256	5333	5384	5461	5513	5590	5641
937.8	1343.1	5228	5307	5359	5437	5490	5568	5620	5699	5751
937.9	1363.73	5329	5409	5462	5542	5596	5675	5729	5809	5862
938	1384.36	5431	5512	5566	5648	5702	5784	5838	5919	5974
938.1	1404.99	5533	5616	5671	5754	5809	5892	5948	6031	6086
938.2	1425.62	5635	5720	5776	5861	5917	6002	6058	6143	6199
938.3	1446.25	5739	5825	5882	5968	6026	6112	6169	6255	6313
938.4	1466.88	5843	5931	5989	6077	6135	6223	6281	6369	6427
938.5	1487.5	5948	6037	6096	6185	6245	6334	6394	6483	6542
938.6	1508.13	6053	6144	6204	6295	6355	6446	6507	6598	6658
938.7	1528.76	6159	6251	6313	6405	6467	6559	6621	6713	6775
938.8	1549.39	6265	6359	6422	6516	6578	6672	6735	6829	6892
938.9	1570.02	6372	6468	6532	6627	6691	6787	6850	6946	7010
939	1590.65	6480	6577	6642	6739	6804	6901	6966	7063	7128
939.1	1611.28	6588	6687	6753	6852	6918	7017	7082	7181	7247
939.2	1631.91	6697	6798	6865	6965	7032	7133	7199	7300	7367
939.3	1652.54	6807	6909	6977	7079	7147	7249	7317	7419	7487
939.4	1673.17	6917	7021	7090	7193	7263	7366	7436	7539	7608
939.5	1693.8	7027	7133	7203	7309	7379	7484	7554	7660	7730
939.6	1714.43	7139	7246	7317	7424	7496	7603	7674	7781	7853
939.7	1735.05	7251	7359	7432	7541	7613	7722	7794	7903	7976
939.8	1755.68	7363	7473	7547	7657	7731	7842	7915	8026	8099
939.9	1776.31	7476	7588	7663	7775	7850	7962	8037	8149	8224
940	1796.94	7589	7703	7779	7893	7969	8083	8159	8273	8348



Residual Discharge for Different Discharge Coefficient (C)
C=2
C=2.03
C=2.05
C=2.08
C=2.1
C=2.13
C=2.15
C=2.18
C=2.2
500
400
300
200
100
0
-100
-200
-300
-400
-500
Discharge (m3/sec)
930
931
932
933
934
935
936
937
938
939
940
Elevation (m)