APPLICATION OF A 1-D NUMERICAL MODELLING FOR SEDIMENT TRANSORT AND FLUSHING – CASE STUDY OF GULPUR HYDROPOWER PROJECT

By

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(2013-NUST-MS-WRE-62218)

A Thesis submitted in partial fulfillment of

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This is to certify that the

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DEDICATED TO

MY PARENTS

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(Muhammad Adnan Khan)

ABSTRACT

All lakes and reservoirs created on natural rivers are subjected to sedimentation. This represents a great challenge for the dam engineers and reservoirs managers to find appropriate ways and means to slow this phenomenon significantly to improve the sustainability and optimal performance of the reservoir. Worldwide, there are about 50,000 large dams and among them are 25,500 storage reservoirs with a storage volume of about 6464 Bm^3. Annual reservoir capacity loss due to sedimentation varies in between 0.10 and 2.4 percent in the greater parts of the world. Sediment deposition in a reservoir decreases the storage capacity of a reservoir and reduces the life of a hydro power project, which would trigger huge socio-economic impacts. Flushing is one of the best technique to remove these sediments from reservoir.

The current study goals to examine the reservoir sedimentation aspects with the help of numerical simulation. Poonch River encounters a flash flood pattern and hence a large sediment concentration is transported through it. In the case of proposed Gulpur reservoir, which have smaller storage volume compared to the annual inflow and the water depth is also smaller than the high head storage reservoir. So, without any sediment management Gulpur reservoir will, over time, become filled with sediment within 15-17 year. Under this situation more sediment will pass through the turbines compromising their performance and integrity. Therefore, for the sustainability of the project a proper desiltation is required. For this purpose various techniques can be applied like dredging, hydro suction, dry excavation, sediment by passing, density current venting, sediment routing, sluicing and flushing sediment through reservoir. Among these approaches, the most economical method for desilting the reservoir is flushing provided that sufficient water discharge is available. Hence in the present study there is a need to explore different approaches to flush the sediment through reservoir, so that the life of the reservoir is enhanced.

This study focuses on investigation of the sediment accumulation, transportation and flushing in a reservoir. Recorded data of Gulpur Hydro Power Project (HPP) in Pakistan was used for this purpose. A physical model of Poonch River was prepared. The model was built for 5.2 km river length with 51 cross-sections. After base test the model was used to get data for various scenarios of sediment flushing in a case-cade reservoir system. The River geometry, cross-

sections, hydraulic structures, river banks and other physical attributes of river were prepared from topographic survey using AutoCAD. These files were used in HEC-RAS for simulations. Delta profile and flushing were modelled by HEC-RAS 5.0. Delta modelling was supported out via hourly time step for the 20 years of sediment deposition with average flow and sediment discharge conditions, whereas, suitable flushing durations were predicted for various flushing discharges to de-silt the one year deposited sediments. Simulation showed that life of the un-sluiced Gulpur HPP is about 14-15 years. To enhance the life of project, annually 4-5 days is required for flushing with 250 m^3/s discharge conditions.

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LIST OF ABREVATION

No.	Abbreviation	Description
1	AJK	Azad Jammu and Kashmir
2	ASCE	American Society of Civil Engineers
3	Bm^3	Billion cubic meter
4	D or d	Dimensional
5	DDR	Draw Down Ratio
6	DHI	Danish Hydraulic Institute
7	EPCC	Engineering Procurement Construction and Commissioning
8	FWR	Flushing Width Ratio
9	GWh	Gigawatt Hour
10	HEC	Hydrologic Engineering Centre
11	HEC-RAS	Hydrologic Engineering Centre River Analysis System
12	HPP	Hydro Power Project
13	Kg/m^3	Kilogram per cubic meter
14	Km	Kilo meters
15	LOC	Line of Control
16	LTCR	Long Term Capacity Ratio
17	MAF	Million Acre Feet
18	m^3/s	Cumecs
19	$M\ m^3$	Million cubic meter
20	Mt	Million Tones
21	MW	Megawatt
22	MWL	Maximum Water Level
23	n	Manning’s roughness coefficient

LIST OF ABREVATION

No.	Abbreviation	Description
24	NOL	Normal Operating Level
25	RESSASS	Reservoir Survey Analysis and Sedimentation Simulation
26	RUSLE	Revised Universal Soil Loss Equation
27	SBR	Sediment Balance Ratio
28	t/m^3	Tones per cubic meter
29	TE	Trapping Efficiency
30	TWR	Top Width Ratio
31	USLE	Universal Soil Loss Equation
32	WAPDA	Water and Power Development Authority
33	ψ	Coefficient of Erodibility

Chapter 1

INTRODUCTION

1.1 GENERAL

Impounding of inflow from upstream catchment areas of the reservoir carries sediments. finer sediment are carried out by the flow in suspension are called as suspended particles while heavier sediments travel on bed known as bed load. In reservoir generally water has lesser velocity and turbulence which intimately results in deposition of heavier sediments along the bed. Longer time is required by the suspended particles to settle down in the reservoir bed. Sediments establish delta formation after entering into the reservoir. The gradient of sediment bed profile gradually changes and is a function of particle size and its characteristics. River sedimentations have bad impacts on the very useful functions of reservoir; as a result huge economic loss arises. Reservoir efficiency worldwide has been reduced because due to loss in storage capacity the power generation, water usage for water supply and irrigation has been affected. Deposition of sediments near power intakes produces wear and tear of turbine due to their momentum may cause huge financial losses. Deposited sediment in reservoir may cause high risk to the stability of dam (Halcrow Report 2001). Deposition of sediments in reservoirs depends upon many factors i.e. nature of upstream catchment area, flow characteristics, seismic activity, and urbanization etc. catchment characteristics also contributes effectively in deposition of sediment. For medium size reservoirs situated in hilly areas sediment will reach more rapidly in reservoirs which are located in hilly areas due to steep slope of the river bed and fine sediment size.

Worldwide, there are almost 50000 large dams including 25500 are classified as the storage reservoirs having storage volume of about 6, 464 Bm^3 (Caston et al., 2009; White et al., 2000). Every one of the reservoirs usually are subjected to some amount of sedimentation that decreases the storage volumes of the reservoirs as well as other harmful consequences. A total of 20×10^9 tons sediment are estimated to be released from large areas of upper watershed around the world, of which 25% is estimated to be trapped in reservoirs before reaching the ocean (Takeuchi 2004). When a dam is built over a natural river, its flow velocity decreases due to the large area of reservoir for the same flow and thus sediments are deposited within the reservoir contributing to the reservoir sedimentation. From operational and physical perspective of the issue, sedimentation is gradually reducing the storage capacity of reservoirs all over the world. Annual reservoir capacity loss due to sedimentation varies in between 0.10 and 2.4 percent in the greater parts of the world with the average annual storage loss is about 1 percent (Chaudhry, M. A. 2012 and Liu, 2002). Better management of existing reservoir is very important as it is very expanses to construct the new reservoir due to its higher economical and financial aspects. Building new reservoir takes usually took much time period and due to huge environmental losses, it is not advisable to build huge reservoir in order to reduce the storage loss due to depositions of sediments in the reservoir. Major reservoirs in the world have lost their storage capacity annually drastically due to sediment depositions. So need of the hour is to adopt a feasible mechanism for removal of these sediments. Approximately loss of 1% storage capacity is being observed annually due to concentration of deposited sediment (Yoon,1992 and Mahmood, 1987) as shown in Figure 1.1.

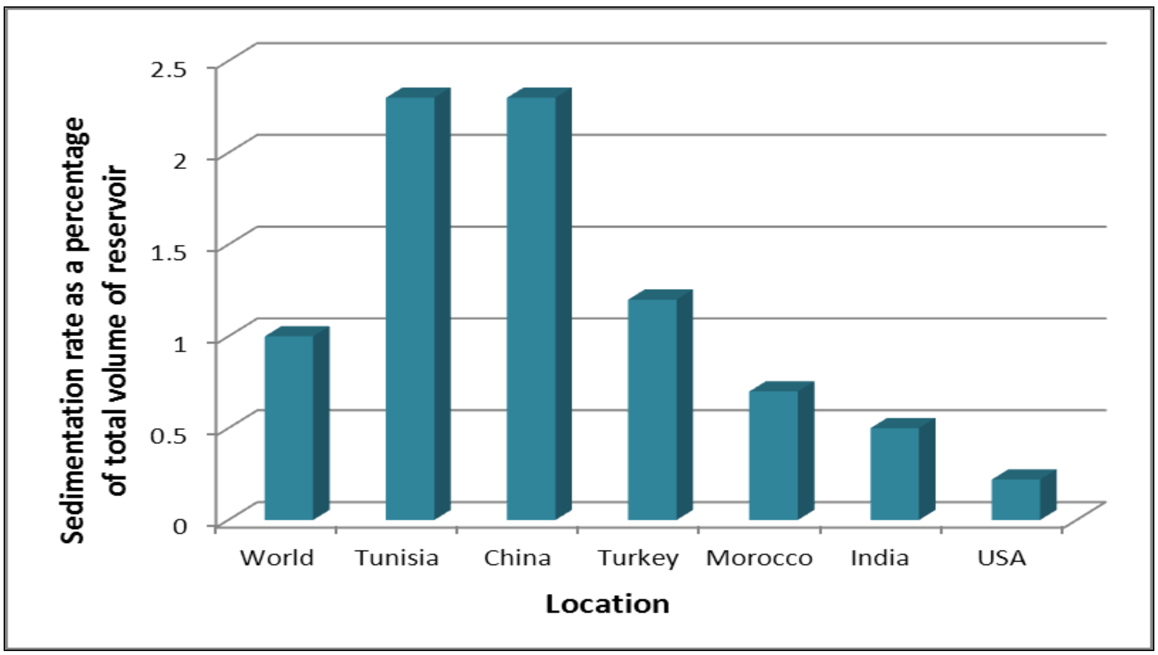

Figure 1.1: Worldwide deposition of sedimentation (Yoon, 1992 and Mahmood, 1987)

Liu (2002) has highlighted the reservoir depletion. According to him this loss varies from 0.1 to 2.3 percent country wise and it accumulates the world annual average loss of 1.0 percent, as shown in Figure 1.2.

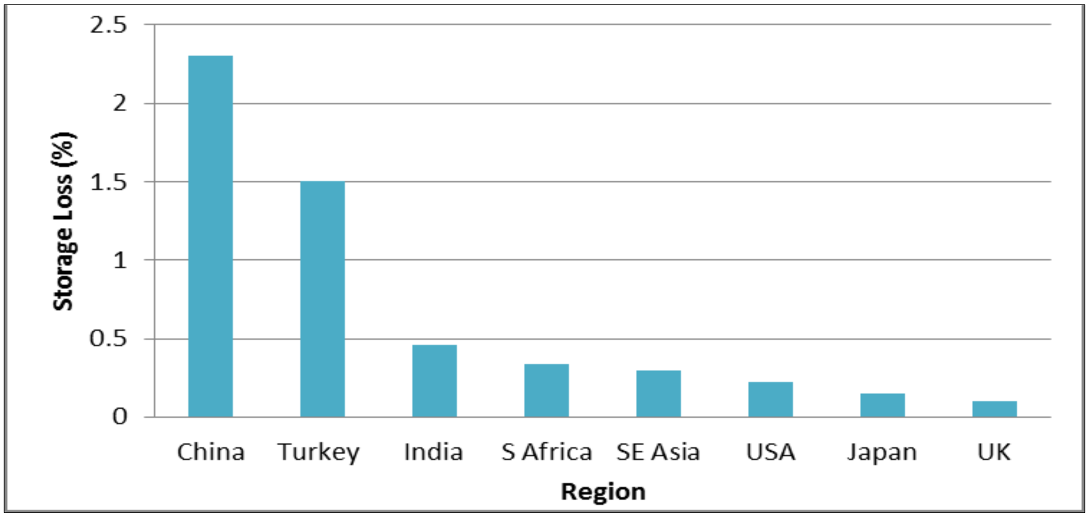

Figure 1.2: Country wise storage losses (Liu, 2002)

Major reservoirs of Pakistan i.e. Tarbela and Mangla have lost up to 28.23% and 20.54% of gross storage capacity respectively (Hydrographic survey, WAPDA 2005). Different methods are used for controlling the sediment depositions in reservoirs. Watershed management (by providing vegetation and check dams) is very useful technique in controlling the sediment depositions in the reservoir. Dredging and excavation is a mechanical method adopted for sediment removal but it is not preferable due to its high cost. Now a day’s removal of deposited sediment is carried out by hydraulic method known as flushing. Flushing technique is in practice since many decades. Flushing is very effective method for the removal of sediment deposited in the narrow reservoir. Flushing is basically the removal of sediments from reservoirs by using low level outlets. Brandt highlighted that flushing is carried out to remove sediments by eroding them while sluicing is carried out to remove arriving sediments towards reservoir not considering the conditions of drawdown. Lai and Shen (1996) has described two types of flushing;

a) Use of high flows to flush the sediment deposited in the reservoir
b) Allow higher sediment concentrated flow during floods

1.2 SEDIMENT DEPOSITION IN RESERVOIR OF PAKISTAN

Major reservoirs of Pakistan are losing their storage capacity very rapidly due to sedimentation. Warsak dam constructed on River Kabul is the first dam built after the independence of Pakistan has lost its total storage capacity, now it is only used for power generation. Table 1.1 shows the loss of storage in reservoir of Pakistan.

Table 1.1: Loss of storage in reservoirs of Pakistan (WAPDA, 2010)

Reservoir	Original Gross Storage Capacity (MAF)	Storage Loss by Year 2010
Tarbela	11.62	34%
Chashma	0.87	55%
Mangla	5.88	27%
Total	18.37	33%

1.3 RESERVOIR SEDIMENT MANAGEMENT

Natural rivers usually attain equilibrium regime with respect to sediment deposition after years and years of flow and there are no major changes in their bed. By developing any obstruction like Dam for Reservoir this balance got changed for similar discharge the area increases and velocity reduces which force the sediment to deposit in the bed. The methods for controlling sediments can be separated in three types:

1.3.1 Preventive Methods

Preventive methods reduce the sediment yield in the watersheds. They are based on erosion of water sheds management techniques usually accessed by USLE (Universal Soil Loss Equation) & RUSLE (Revised Universal Soil Loss Equations) etc.

1.3.2 Routing Methods

In this methods sediments tried to pass through sluicing or bypassing techniques.

1.3.3 Curative Methods

In this method removal techniques e.g. flushing. Dredging & Hydro suctions are used. Figure 1.3 shows the reservoir sedimentation controlling measures.

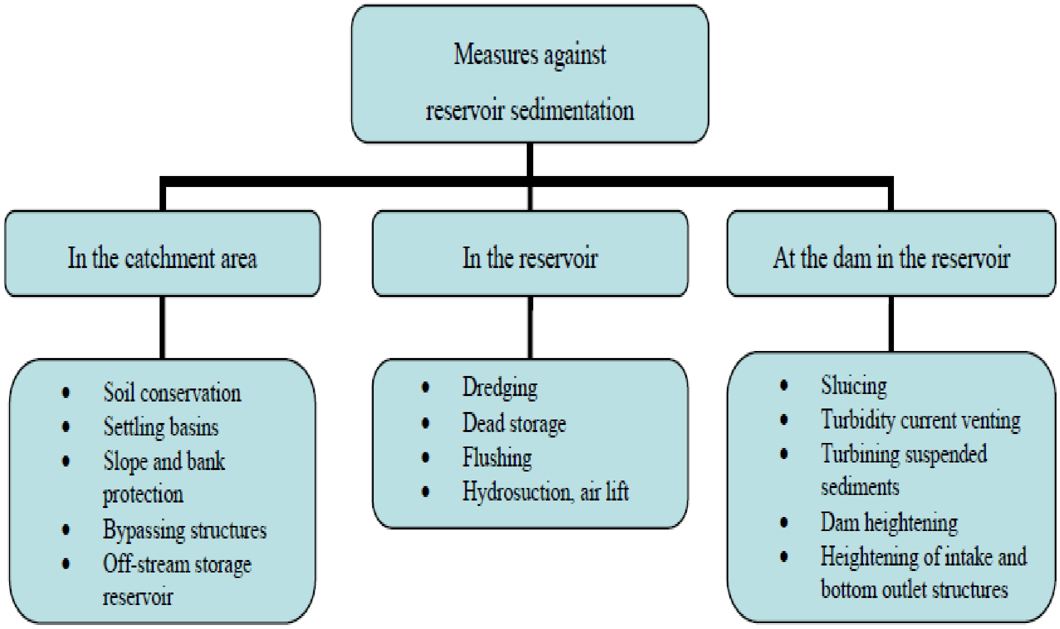

Figure 1.3: Reservoir sedimentation controlling measures (Atkinson, 1996)

1.4 SEDIMENT MANAGEMENT BY RESERVOIR FLUSHING

Flushing consists on remobilizing and transporting deposited sediments through the reservoir using low-level outlets by increasing flow velocities through the reservoir by drawing down the reservoir water level.

White (1999) has described the flushing as a process which remove the stored/ depoisted sediments from a reservoir by hydraulic action, usually by passing the flow through very low level outlets from the dam.

In flushing phenomena the low-level outlets of the dam are unlocked, which results in scouring of the sediments which are depoisted near the outlet.

In Reservoir flushing "'The flow velocities are increased by allowing the water to pass through the low level outlets/tunnels so that deposited sediments can be evacuated" (Castillo, 2015). Riverine conditions in the reservoir should be created for enough time to make flushing more effective. Low level outlets should be near to the river bed and has sufficient capacity to allow full draw down flushing.

Flushing is very effective for narrow reservoir. Effective sediment flushing depends upon the flow velocities as the higher velocities over the entire area are sufficient to erode the sediments at a higher rates. Flushing is very effective in many cases but a lot of water has been consumed by this process (Fi-John et al., 2003).

Study revealed that about 50 reservoirs exist in world, which had been flushed successfully. China has the most number of flushed reservoir in the world. On the other hand many reservoirs have not been flushed successfully. Warsak dam in Pakistan is one of the examples of such reservoir.

Flushing has been successfully carried out at Gebidem-Switzerland, Gmund-Austria, Baira-India, Santo-Domingo-Venezuela, Palagnedra-Switzerland, Hengshan-China Reservoirs, while flushing had also been carried out on the Reservoirs, Heisonglin-China, Ichari-India, Guanting-China, Guernsey-USA, Ouchi-Kurgan-Former USSR, Shuicaozi-China, Sanmenxia-China,Sufid-Rud-Iran, but not successfully flushed (Atkinson, 1996; Emamgholizadeh et al.,2006). The detail of reservoir flushed worldwide has been described/highlighted in the Figure 1.4. As shown in figure 1.4 China has the most number of reservoirs flushed.

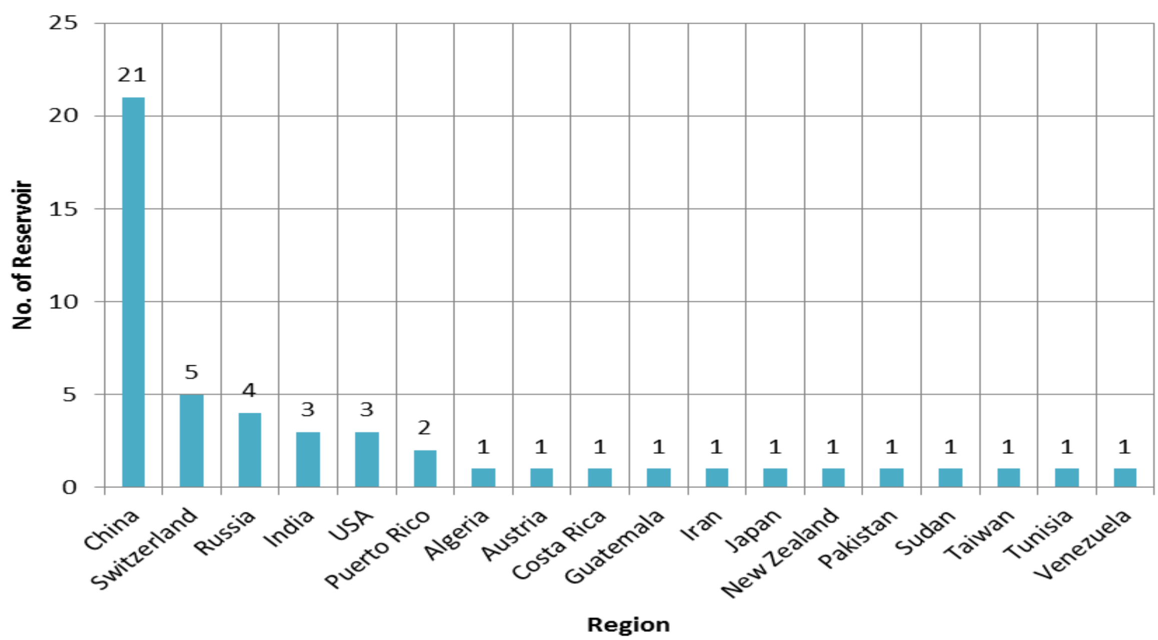

Figure 1.4: Worldwide flushed reservoirs (Atkinson, 1996; Emamgholizadeh, 2006)

Flushing operation is very complex process; it is very difficult to adopt the rules that can be applied on many reservoirs. However the following rules have been made by Shen (1999).

a) Drawn down of water level to enhance the efficiency of the flushing phenomena

b) This phenomenon should be carried out at narrow reservoirs. As the flushing is very effective in narrow reservoirs than wider reservoirs

c) Retrogressive erosion occurs in flushing channel in case of wider reservoirs.

d) The width of the flushing channel was found to be a coefficient of about 11 to 12 times the square root of the bank full discharge inside the flushing channel by Atkinson (1996) from the field data and also by Lai and Shen (1996) as well as Janssen (1999) from laboratory data.

1.5 TYPES OF FLUSHING

Following techniques has been adopted for the flushing phenomena in the reservoirs (fig. 1.5).

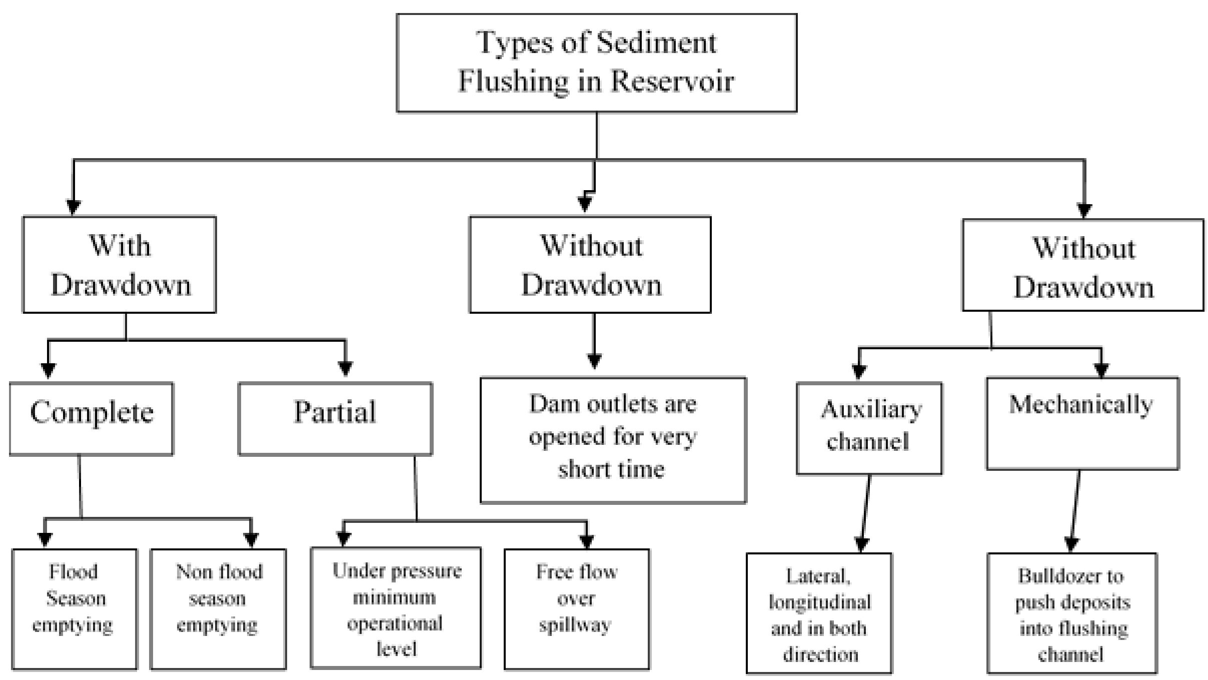

Figure 1.5: Types of flushing

Broadly speaking there are two types of sediment flushing as shown in figure 1.6 (White, 2001).

a) Complete Draw Down Flushing

b) Partial Draw Down Flushing

"Drawdown is the lowering the water levels in a reservoir. Hydraulic flushing involves reservoir drawdown by opening the bottom outlet to generate and accelerate unsteady flow towards the outlet (Morris and Fan, 1998). This accelerated flow possesses an increased stream

power and eroding a channel through the deposits and flushing the fine and coarse sediments through the outlet. During this process a progressive and a retrogressive erosion patterns can occurs in the tail and delta reaches of the reservoir, respectively (Batuca and Jordaan, 2000)". Among reviewed literature, sediment removal from reservoirs (White, 2001) generally only affords the problem of sediment flushing inside the reservoir. However, there is scarce information about experiences on prototypes and especially in respect to flushing channel formation. One of the phenomena in reservoirs that is not well investigated and theoretically explained is the formation of flushing channels in the delta of the reservoir (Sloff et al., 2004).

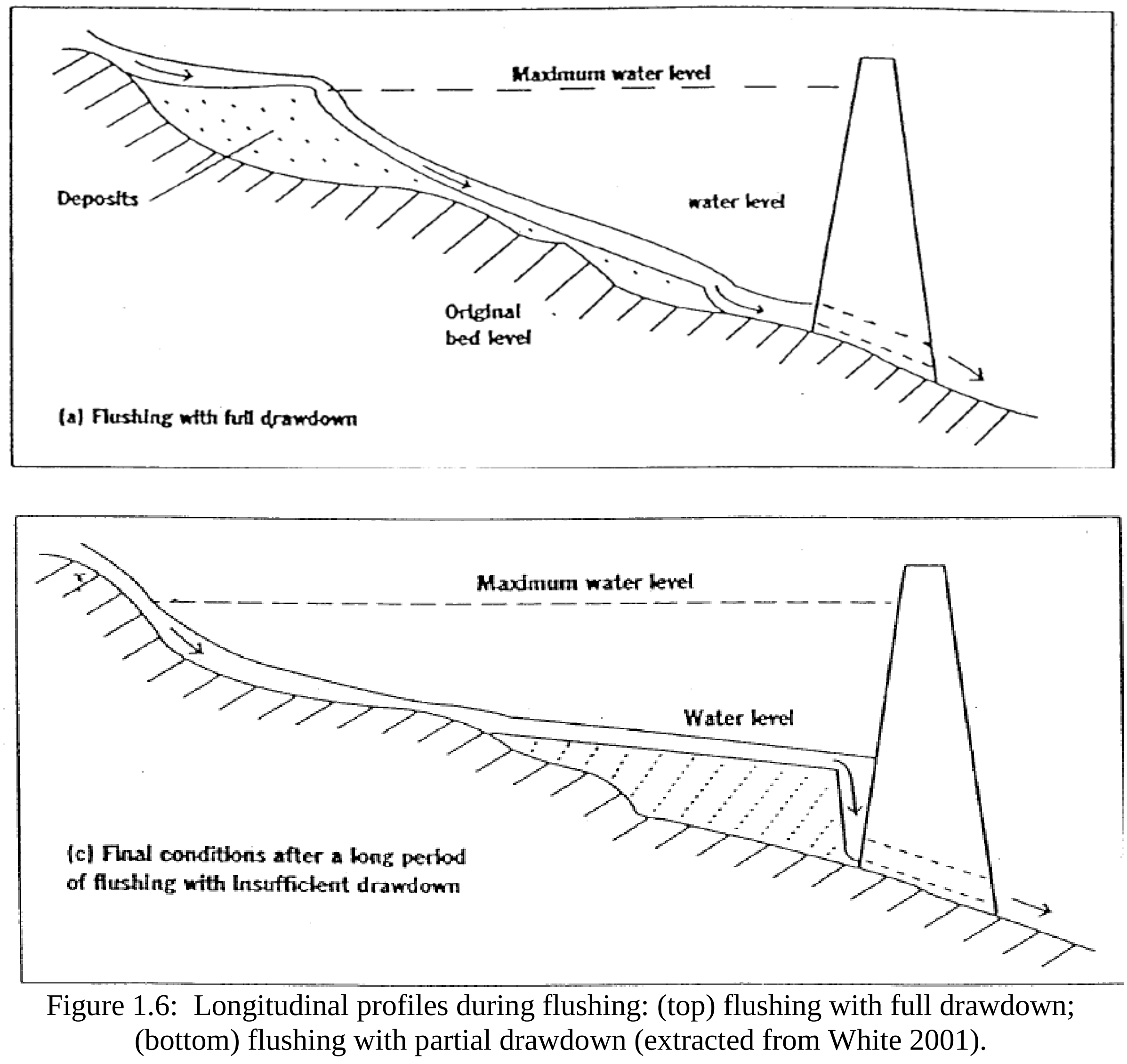

Figure 1.6: Longitudinal profiles during flushing: (top) flushing with full drawdown; (bottom) flushing with partial drawdown (extracted from White 2001).

A flushing operation will scour an incised channel within the accumulated sediment material within the reservoir (Figure 1.7 as example). The dimensions of this scoured channel are related primarily to the flushing discharge.

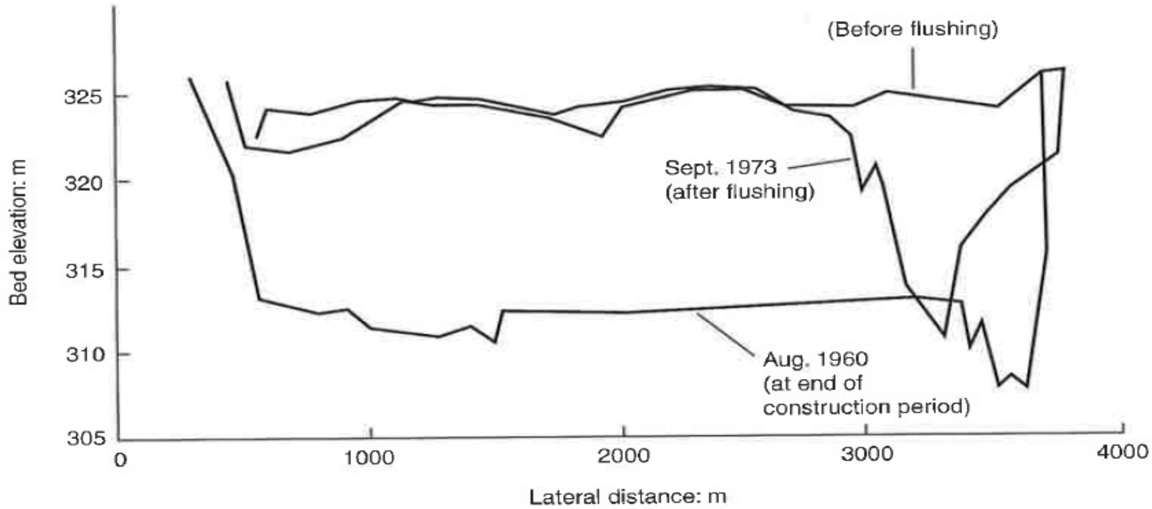

Figure 1.7: Cross-section of flushing channel at Sanmenxia reservoir in China (from White, 2001)

1.6 SLUICING

Sluicing consist on the release of sediment along with the high flows. The main goal is to minimize the sediment that can settle down during high flows, which are the ones transporting most of the material. This can be achieved closing the intakes and opening the sluices to let the material pass through. If water levels at the reservoir are lowered, the action of sluicing can be combined with the remobilization of material as in a flushing operation.

1.7 OBJECTIVE OF THE STUDY

This study aims to explore reservoir life under different future scenario of sediment deposition and analyzing the impacts of sediment on reservoir life by computing long term flushing scenarios.

The key research objectives are;

1. To evaluate and design flushing for the reservoir by using HEC-RAS 5.0 sediment transport model.

2. To carry out sensitivity analysis for calibration of sediment transport function and for different fractions of sediment inflow.
3. To determine the flushing characteristics under various operation scenarios.
4. To evaluate flushing indicators for assessing the feasibility of sediment flushing through Gulpur reservoir.
5. To formulate flushing strategy/plan for Gulpur HPP, Pakistan.

1.8 SCOPE OF THE STUDY

The study will identify the present distribution of the deposited sediment to obtain best estimates of sediment volume in the different area of reservoir and will carry out sediment simulation studies, prediction of the future delta formation and its movement. It will also review the potential for enhanced sediment removal including flushing and determine the effectiveness of such measures and their merits / demerits, including calibration and validation against the physical model of the project. As a result of the research work the movement of the delta due to geologic sediment inflow to the reservoir would be predicted. The result will also provide the information about the period after which dam would become ineffective in generating power. Study is limited to the results obtained using Reservoir Survey Analysis and Sediment Simulation (HEC-RAS) which is one dimensional sediment transport model.

1.9 LAYOUT OF THESIS

Chapter 1 will provide the introduction, objective and scope of the research. Chapter 2 will give details of the literature review, which describes in detail reservoir sedimentation occurring in the world. And also strategies for sediment flushing through the reservoir have been described. Chapter 3 will provides a general description of the weir and reservoir characteristics. And also describes the flow data and estimation of sediment inflow to the reservoir. Chapter 4 will explain the physical modelling of Gulpur HPP with model laws and scale. Chapter 5 will presented the deposition pattern in the reservoir and in the influence zone immediately upstream with no sediment management option is assessed with a 1D model, HEC-RAS. The description of the numerical work and some possible management options are also described in chapter 5. Chapter 6 will conclude the research with conclusions and recommendations.

Chapter 2

LITERATURE REVIEW

2.1 GENERAL

The rate of sediments transport towards a Reservoir depends a lot upon the sediment characteristics (i.e. particle sizes, shapes and concentrations) and other factors are flow characteristics (i.e. velocity, depth, viscosity, geometric shape of channel and slope). In erosion phenomenon sediment particles from the river bed are moved by the shear force of water. Reservoir Sedimentation is a broad topic no definite estimates of reservoir sedimentation are available till date. Zhou in 1993 highlighted that China with maximum no. of reservoirs (82000) has highest storage capacity rates @ 2.3% per year.

The planning and design of a reservoir requires the accurate prediction of sediment transport, erosion and deposition in the reservoir. For existing artificial lakes more, and wider knowledge is still needed to better understand and solve the sedimentation problem, and hence improve reservoir operation. In past many attempts had been in order to access the flushing phenomena in detail. Calculation of sediments deposited in the reservoir is very important and following method can be used in order to better estimate the depositing sediments (White 1999).

a) End area methods (based on range line surveys)

b) Contour methods (based on contour information)

c) Combined methods (based on range line and pre-impoundment contour information)

2.2 SEDIMENT MOVEMENT IN RESERVOIR

Sediment supply towards the reservoir from the water shed depends a lot upon the flow velocities and turbulence. Sediments are present in wide range. Bed load consist of course materials and suspended particles are generated by superficial erosion as well as due to the collision and abrasion of bigger size particles. During the events of flood 70 to 90 % of sediments flows towards the river. When the river flows towards the reservoir velocity of particle reduces and they settle down and form gradually a delta in head water area. Finer particles being suspended flow through the delta stream and passes the lip point enters in to non-stratified flow zone and then deposits along the path due to decrease in velocity and increased cross section.

The quasi-homogeneous flow is shorter in cases of smaller discharge and/or higher sediment concentration. On the contrary, the region of quasi-homogeneous flow is longer in case of larger discharge and/or lower sediment concentration. As a consequence close to the dam the deposition of the finest particles takes place.

Morris & Fan (1997) studied the movements of sediments in the reservoir as shown in fig 2.1 and concluded four basic type of deposition (fig 2.2)

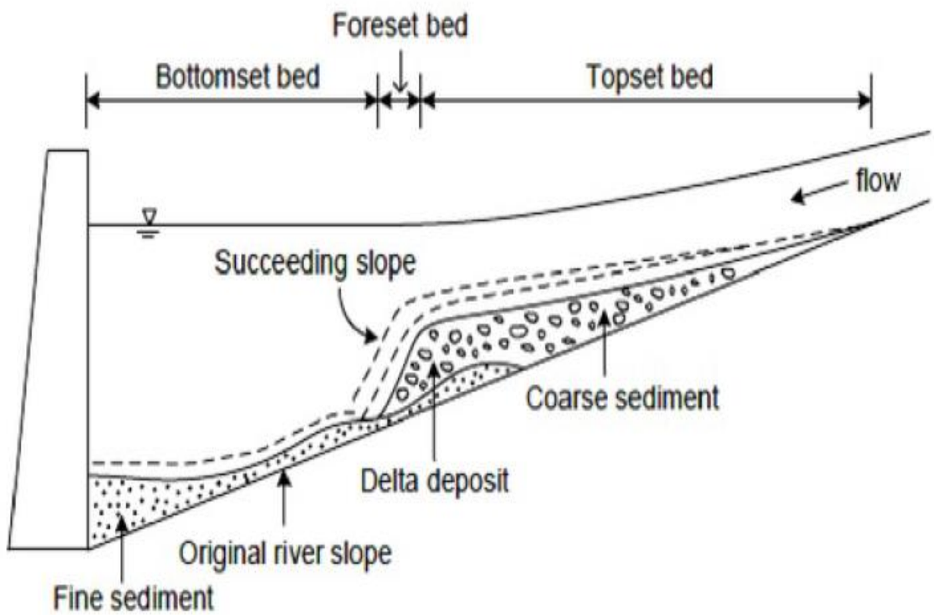

Figure 2.1: Movements of sediments in the reservoir (Morris & Fan 1997)

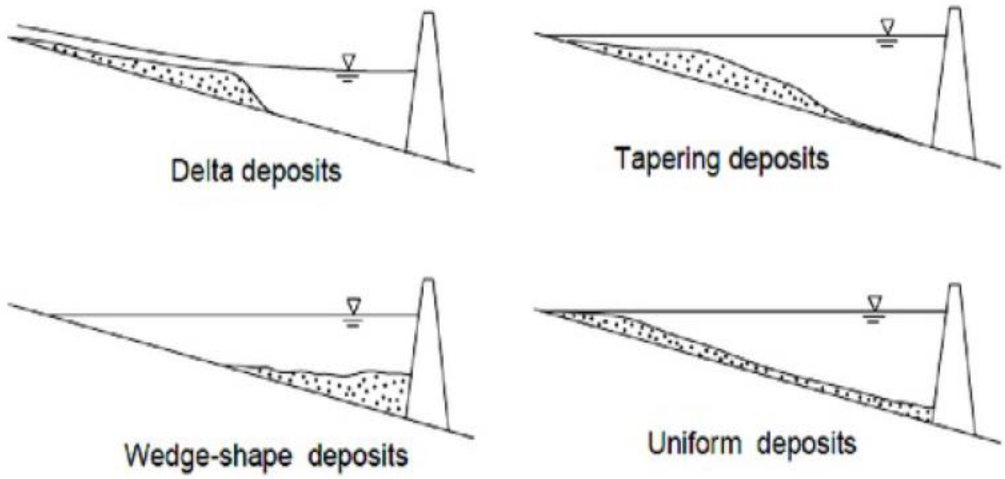

Figure 2.2: Basic types of deposition (Morris & Fan 1997)

2.3 PREVIOUS STUDIES OF RESERVOIR SEDIMENTATION AND FLUSHING

Historically, reservoir sedimentation is known since the earliest construction of dams, but has not been fully understood, described systematically and published. The subject has been already discussed on an ad hoc basis in 1936 in a special session entitled "The silting of reservoirs formed by large dams; its measurements and prevention" during the 2nd Congress on Large Dams in Washington DC. One of the first overall descriptions has been published by the United Nations in 1954. Partial phenomenon observation related to turbidity currents, which are the major transporting factor of sediments in reservoirs, were presented earlier by Grover and Howard (1938) for Lake Mead. Bell (1942) has carried out extensive experimental work in flumes and has emphasized the importance of turbidity currents in sedimentation of reservoirs. Geza (1953) and again Grover (1953) also reported density currents observations related to reservoirs. During the 1950's and later, reservoir sedimentation problems created by turbidity currents in Algeria and France fostered observations, experimental and theoretical studies of reservoir turbidity currents (Thévenin, 1960, Nizéry et al., 1953) and similar studies were performed by Middleton in 1966. Lara (1960) was among the first to report reservoir sedimentation survey.

Some other observations and measurements before 1990 are reported in Japan (Chikita, 1989), Canada (Weirich, 1984, 1986), Ecuador (Jervez, 1985), USA (Ford and Johnson, 1981) and Tadjikistan (Nurek, former Soviet Union - Pyrkin et al., 1978). The number of published studies on turbidity currents and their influence on reservoir sedimentation were limited until the eighties, before mainly by Ellison and Turner (1959) and Fan (1960 and 1962). From 1980 on, the number of publications has significantly increased and the subject is still now under investigation.

Julien (1998) showed some field measurements of the Tarbela Reservoir in Pakistan. The life expectancy of that reservoir is about 100 years. Yang and Simoes (2002) used GSTARS3 model to compute the Tarbela Reservoir geometric change over 21 years, from 1975 to 1996. The simulated bed profile using GSTARS3 is in good agreement with the measured profile. White (2001) did a similar numerical model study of sedimentation in the Tarbela Reservoir. His model predicted more deposition than the measurement.

Yang and Marsooli (2010) applied GSTARS3 to sedimentation studies of the Ekbatan Reservoir and Kardeh Reservoir in Iran. The computed bed profiles with calibrated coefficients are generally in good agreement with the measurements.

Chang et al (1996) evaluated the efficiency of sediment-pass-through for low level outlets in reservoirs on the North Fork Feather River using Fluvial-12 (Chang, 1988). The fluvial-12 simulation indicated that the sediment-pass-through operation is feasible to maintain sediment equilibrium for the river/reservoir system, and sediment released from the reservoir would not have adverse impacts on fish habitat in the river.

Morris and Hu (1992) simulated sediment flushing in the Loiza Reservoir in Puerto Ricousing a one-dimensional model HEC-6 (US Army, 1991). The reservoir was assumed as one-dimensional, because the lateral variation of the channel was not significant.

Flushing with intermediate water surface drawdown has been conducted for the jensanpei Reservoir (Hwang 1985). The storage capacity was reduced due to sedimentation in the find 15 years of operation. After 15 years of operation, flushing with intermediate water surface drawdown was conducted once a year, and no further reduction of storage capacity was observed.

White (2001) summarized 22 case studies of flushing based on field measurement. Most of them were successful, but some of them were not successful due to downstream constraints. Morris and Fan (1997) indicated some limitations of flushing. First, there should be sufficient water to be used for flushing. Second, flushing causes a sodden release of higher sediment concentration than occurs naturally in the river. High sediment concentration may create unacceptable downstream impacts, such as clogging of channel due to deposition and damaging fish habitat.

2.4 EMPIRICAL METHODS OF PREDICTING SEDIMENTATION

Until recently reservoir sedimentation could only be assessed using simple, empirical methods. To estimate the volume of deposited material the notion of trapping efficiency was introduced. The trapping efficiency of a reservoir is defined as a ratio of the quantity of deposited sediment to the total sediment inflow. Gottschalk (1948), Churchill (1948) and Brune (1953) provided simple graphical means to determine trapping efficiency and these have been used extensively. Since, however, the trapping efficiency must depend upon the sediment size, the flow through the reservoir, the distribution of flows into the reservoir and the way that the reservoir is operated, it follows that such estimates of trapping efficiency can only provide approximate values which may, on occasions, be seriously in error.

"After analyzing several reservoirs in the USA, Churchill (1948) came to the conclusion that along with the retention time, the transit velocity, i.e., the velocity with which the water flows in the reservoir, governs the trap efficiency. If the water held in the reservoir is moving fairly rapidly in the reservoir, very little sedimentation will occur because the turbulence associated with the higher velocity hinders settling, even though the retention time may be high. He introduced a parameter known as sedimentation index which is the ratio of the period of retention to the mean transit velocity".

The trap efficiency of the reservoirs was found to increase with increase in the sedimentation index. If a large percentage of the sediment in the stream is moving in the form of density currents, then the concept of mean transit velocity introduced by Churchill is questionable. In such a case, the velocity of density currents may be very different from the mean transit velocity of the flow.

Brune (1953) analysed the records of 44 different reservoirs in the USA (41 of which were normal ponded reservoirs) and found that the capacity to inflow ratio gives better correlation with trap efficiency than the capacity to watershed area ratio.

2.5 MODELLING RESERVOIR STORAGE

When flow enters a reservoir, its velocity drops dramatically and it is no longer capable of transporting the coarser sediment fractions. If the water level in the reservoir is near full supply level the sediment will be deposited near the head of the reservoir. If the reservoir is partially full then deposition will occur further into the reservoir basin and at a lower elevation. It is thus important to model the reservoir water level. The finer sediment will be carried further into the reservoir where its deposition is controlled by the opposing effects of particle weight and turbulence. To predict the reservoir water level a storage sub-model is used. The sub-model uses a continuity equation to relate the inflow of water into the reservoir to any outflows plus the change in storage in the reservoir.

2.6 MODELLING OF FLOW

The water flow in the reservoir and the upstream river is determined using a backwater calculation. The water level predicted in the reservoir storage simulation, described above, is used as an initial downstream boundary condition at the dam to enable the backwater calculation to proceed upstream. This calculation provides water depths, velocities and slopes at each cross section along the length of the reservoir and up the incoming river.

2.7 MODELLING SEDIMENT TRANSPORT

In modelling sediment movement the primary concern is deposition. The trapping efficiency and the location of the deposition depend on the volume of water stored in the reservoir. Since, however, the water level in the reservoir fluctuates and the inflowing discharge varies, sediment that has previously been deposited may be subsequently eroded. It is necessary, therefore, to be able to model both the deposition and erosion of sediment. In performing such calculations, which are of a volumetric nature, due allowance is made both for initial density, and for subsequently increased density due to compaction by overlying deposition. From the calculated velocities, depths and slopes, the sediment concentrations at each section may be calculated, but when modelling the sedimentation process it is necessary to treat the sand and silt fractions separately. This is because sand movement depends only upon local hydraulic conditions, whereas silt movement is also influenced by preceding flow history.

2.8 SAND MOVEMENT

The transported sand sizes at each section are calculating using one of the many established sediment transport theories for non-cohesive materials, e.g., Ackers and White (1973). The movement is dependent upon the sediment diameter. For sediments which do not contain too broad a range of different sizes a representative sediment diameter, D_{35} is often used. For widely graded sediments the range of sediment sizes is divided into a number of classes each with a representative diameter.

2.9 SILT MOVEMENT

The concentrations of the silt fractions entering the reservoir depend on the drainage basin's sediment yield and total annual runoff. The silt is converted with the flow but its concentration reduces as some of the material settles out of suspension onto the bed. The rate of settling is dependent upon the fall velocity which in turn is dependent on concentration and flow conditions. The calculation of the silt concentrations requires more closely spaced sections than for flow. The resulting transport rates of the silt fractions at each section are added to the sand transport rates to obtain the total sediment transport rate. The change in bed level at each section due to the variations in sediment transport rate along the reach can then be determined.

At the head of a reservoir different sediment sizes are sorted according to the ability of the flow to transport the material. Studies have shown that to represent conditions in this region it is best to use a number of representative sand and silt sizes.

Predicting reservoir sedimentation using numerical modelling offers a significant improvement over other methods based on simple estimates of trapping efficiency. The method can readily take account of:

a) Variable flows

b) Variable water levels

c) Sediment size

d) Reservoir geometry

e) Reservoir operating rules

And can provide:

f) Volume, location and compaction of sediment deposited over a

g) Specified time period

h) Annual stage-storage curves

i) Longitudinal profile of the reservoir at any given time

j) Effectiveness of using sediment flushing

2.10 EQUATIONS FOR SEDIMENT EVACUATION FROM RESERVOIR

Wu in 1998 developed an equation for the sediment flushed on the basis of the data of Gen-shen Pie reservoir. It has storage capacity of 7.7 million m^3 with some problems of storage capacity loss due to sedimentation. The following relationship was used to predict the sediment concentration flushed for full storage and empty storage condition, respectively.

$$C = 64.9\left[\frac{v^3}{gdw}\right] - 0.45 \qquad \rightarrow (2.1)$$

$$C = 847\left[\frac{v^3}{gdw}\right] - 0.49 \qquad \rightarrow (2.2)$$

Where;

C = concentration of sediments flushed (kg/m^3).

V = Flow velocity (m/s).

g = Gravity constant (m/s^2).

d = Flow depth (m).

w = Sediments fall velocity (m/s).

Tsinghua equation (2.3) is simple but efficient technique for finding the sediment evacuation from reservoir. It has been used by various researchers for sediment evacuation from the reservoir. It has also been experimentally verified by Lai & Shen 1996.

$$Q_s = \frac{\psi Q_f^{1.5} S^2}{W^{0.6}} \qquad \rightarrow (2.3)$$

Where;

Q_S = Discharge of sediments in ton/sec.

Q_f = Reservoir outflow (m^3/s).

S = Reservoir longitudinal slope energy.

W = Reservoir eroded channel width.

Ψ = Coefficient of erodibility.

Coefficient of erodibility (Ψ) depends on characteristics of suspended sediment and sediment load. IRTEC (1985) and Atkinson (1996) have done significant work for selection of Ψ values. Chang et al (2003) used above equation for sediment flushing from Tapu reservoir in Taiwan. Kawashima in 2003 used these values on the reservoir in China also Khan in 2011 has used this equation to Tarbela reservoir in Pakistan.

IRTEC in 1985 derived these values from flushing data of reservoir in China having discharge Q_f = 0.1 to 5730 m^3/s, S = 0.06-16%, W = 10-1000 m and Q_s = 0.006-777 tons/s. Atkinson in 1996 confirmed the values of erodibility coefficient (Ψ) for the flushing data for 4 reservoir of India and USA. It was concluded that IRTECS value only satisfies in China and its results are not applicable to the other regions in the world. It was recommended that if the water depth of reservoir during flushing is kept more than 30% of the maximum water depth Ψ values will be further adjusted.

Lai and Shen in 1995 concluded that the sediments transport rate is the function of out flow discharge (Q_o), energy slope (S), flushing channel width sediment size etc. following equations have been used for the out flow sediment discharge (Q_s), & Flushed sediment volume.

$$Q_{os} = \frac{EQ_oS^{1.2}}{B^{0.6}} \qquad \rightarrow \quad (2.4)$$

Where;

E = Erodibility of sediment deposited.

$S = S_w = S_o$

S_w = water surface slope.

S_o = Bed Slope.

2.11 NUMERICAL MODELLING (1D, 2D AND 3D)

Numerical models are classified as one-dimensional (1D), two-dimensional (2D), and three-dimensional (3D) models. Most numerical modelling uses a ID model, which is more robust than 2D and 3D models (Morris and Fan 1997). White (2001), and Molinas and Yang (1986) noted that a ID model is suitable for long-term simulation of reservoir sedimentation, while 2D or 3D models require much more field data for calibration.

Complicated 2D and 3D models can be used to assess localized impact of flushing near a 15 low level outlet. Generally speaking, a ID model is suitable for long-term simulation of a long reach of river or reservoir with elongated channel geometry. 1D model requires the least amount of data for calibration and verification and their numerical solutions are relatively simple and stable. 2D or 3D models are suitable for short-term simulations of localized phenomena of a short reach of a river or reservoir. 2D or 3D models require large amounts of data for calibration and verification, and their numerical solutions are complex. Yang (2010) suggested that a qusai-2D model for hydraulic simulation and prediction and a qusai-3D simulation and prediction of channel geometry and profile adjustment is more suitable for long-term simulation and prediction of morphologic changes of a long reach of a river and reservoir with limited field data for engineering purposes.

2.12 PREVIOUS NUMERICAL MODELS

Several numerical model are performing reservoir sedimentation. These models can simulate sediment inflows, sediment outflows, active storage capacity and trap efficiency bed profile and flood simulations. Most numerical reservoir models are time-stepping models. For given initial conditions, the equations are used to predict what happens in the reservoir over a short time-step. The process is then repeated a number of times over the required time period. The time-step used depends upon the size and nature of the reservoir but is typically of the order of a day. Once the geometry of the reservoir and incoming river, and the nature of the flow and sediment are specified, three modelling stages can be carried out: firstly of the reservoir's storage, secondly of its flow and thirdly of the sediment transport within it. The capabilities of the available software are discussed below.

a) GSTAR 4 Model

Generalized Sediment Transport model for Alluvial River. Simulations was generated by US Bureau of Reclamation in 1985. Its latest version available is GSTAR 4 that was released in 2011. The approach used in model is conservation of mass and energy equations. It simulates water surface profile for both steady and unsteady flow also it simulates critical, subcritical and supercritical flows. In Semi 2-D manner it simulates transfer and longitudinal flow condition. Special features of stream tube concept has been added, which divides channel into number of small channels across the river, thus enabling it to better simulate the lateral distribution of sediments. It can also simulate bed profile and changes in the geometry of the channel both depth and width can be simulated. The model has been successfully applied to Tapu reservoir in Taiwan, Rio Grande reservoir in Mexico and also Tarbela reservoir in Pakistan.

b) MIKE II Model

The model was developed by the Danish Hydraulic Institute (DHI) from Denmark. It is a one dimensional model can also perform planning and operational simulations of reservoir. This model is also used in flood forecasting and controlling measures, design of channels, surface drainage and irrigation systems and river operations. It uses saint venant equations to compute water surface profiles i.e., conservation of momentum and continuity equations. This model has been successfully applied to reservoirs in Indian subcontinent.

c) SHARC Model

It is a 1-D sediment transport model developed by HR Wallingford. Its basic purpose is to diagnose the problems of river and channel system. Six modules of the model are available.

- Problem diagnostic and initial data input

- Preliminary economic screening
- Design Tool
- Hydraulic simulations
- Economic analysis
- Environmental analysis

SHARC is used for performing sediment deposition, Bed Profiles and flushing of deposited sediments in both Reservoir and channels it works on Westrich and Juraschek sediment transport equation. The Equation only depends on suspended sediments as it does not consider bed materials (Yang 2006).

d) RESSASS MODEL

The assessment of the general sedimentation pattern in the reservoir considering long time scales (several years) and the changes in storage capacity for trapping the sediment is performed with a one dimensional numerical model, RESSASS.

RESSASS is based upon physically based equations that describe flow and sediment movement in open channels based on steady state backwater computations, and sediment transport calculations for a range of sediment sizes. The model is usually applied to make predictions over periods of several years with a time step of one day. The flow and sediment transport simulations are one-dimensional, that is only variations along the length of the reservoir are considered, and all the quantities calculated are averaged over a cross-section. Detailed descriptions of deposition patterns around particular structures may require the use of other types of modelling.

The model requires a time series of the discharges entering the reservoir as input. If the water level variations at the dam are not also specified then they are calculated using a storage routing method. Velocities and depths through the reservoir are calculated using a backwater computation. The effects of the shape of the reservoir in generating turbulence and mixing are accounted for by adding a term to the shear velocity calculated from a friction relationship derived from 3-D turbulence modelling.

Sediments are divided into different size ranges. The transporting capacities for the sand and larger sizes are calculated separately from finer sediments, silts and clays, in the cohesive size range. Corrections are applied to both sand and silt concentrations to allow for non-equilibrium transport conditions. The sediment masses deposited or eroded at each section are converted to volumes taking consolidation effects into account. The distribution of sediment deposits across the reservoir sections is varied according to user-defined functions. An important aspect of the model is that it calculates the composition of the sediments on the bed of the reservoir from the deposition that has taken place during the simulation. Thus the sediment sizes of the deposited sediment are predicted, rather than being specified initially.

e) HEC-RAS MODEL

HEC-RAS is designed by US Army Corps of Engineers which perform one-dimensional hydraulic calculations for a full network of constructed and natural channels (HEC-RAS, 2010). HEC-RAS have following major capabilities as described below.

1) Graphical user interface

The user interacts through a graphical user interface (GUI) with HEC-RAS. The main focus was to make it easy to use the software, and also maintaining a high level of efficiency for the user.

2) Hydraulic analysis components

The HEC-RAS system contains four one-dimensional river analysis components for:

i. Unsteady flow simulation
ii. Steady flow water surface profile computations
iii. Water quality analysis
iv. Movable boundary sediment transport computations

3) Data Storage and Management

This modelling system component is proposed for calculating water surface profiles for steady gradually varied flow. The system can handle a dendrite system, a full network of channels, or a single river reach. The steady flow component is capable of modelling supercritical, subcritical and mixed flow regimes water surface profiles.

4) Graphics and Reporting

Graphics include X-Y plots of the river system schematic, profiles, cross-sections, hydrographs, rating curves and many other hydraulic variables. Multiple cross-sections of three-dimensional plot are also provided and Tabular output is available.

After comparison (Table 2.1) of several sediment transport models HEC-RAS 5.0 was selected for River Simulation and bed profile for the case study in the next chapters due to its several advantages over other.

Table 2.1: Comparison of different sediment transport model

Sr. #	Description	MIKE II	GSTARS	HEC-RAS	RESSASS	SHARC
1	Developer	DHI	USBR	HEC USACE	HR Walling ford	HR Walling ford
2	Capability (with reference to present study)	Sediment Transport	Sediment Transport	Sediment Transport	Sediment Transport	Sediment Transport
		Reservoir Operation	Sediment Transport	Sediment Transport	Sediment Transport	Sediment Transport
3	Dimension	1-D	Semi 2-D	1-D	1-D	1-D
4	Availability	Freely Available	Commercial Model	Freely Available	Not Available	Not Available

2.13 FEASIBILITY OF FLUSHING SEDIMENT FROM THE RESERVOIR

In order to determine the feasibility of undertaking flushing, a study has been undertaken based on a methodology developed by HR Wallingford. The document sets out a methodology to predict the likelihood of being able to successfully flush sediment from a reservoir. This methodology was followed and the indicators for successful flushing calculated based upon reservoir geometry, annual flow and sediment load. The following text that describes the theory and methodology has been taken from extracts from the HR Wallingford report.

2.13.1 Factors in Flushing Sediment

Reservoir flushing will be feasible when the following conditions prevail:

a) The sediment quantities transported through the low level outlets during flushing are sufficient to enable a long term balance between the sediment inflow and the sediment flushed,
b) The volume of deposits remaining in the reservoir after a sediment balance has been achieved is sufficiently small to enable a specified storage requirement to be met, and
c) The cost of flushing does not exceed the benefits.

2.13.2 Description of the Process

'Complete drawdown' is defined as the lowering of the water level in a reservoir until the reservoir is empty, and the river flow passes through the reservoir at depths similar to river depths before impoundment. In general, flushing without complete drawdown of water levels will be ineffective. When low level outlets in a dam are first opened high flow velocities are produced in the immediate vicinity of the outlet. Sediment deposits are thus scoured from a region close to the outlets. Flow velocities further away from the outlets are small and hence no scour occurs. It is only when the reservoir is nearly empty that significant sediment quantities are passed through the outlets. When flushing is first attempted at a dam where the low level outlets have insufficient capacity to achieve full drawdown, then little sediment is removed from the reservoir. The flushing will produce full drawdown in the upper reaches of the reservoir, where bed elevations are higher, and sediment will be scoured from this region. The sediment will be deposited again upstream from the dam where drawdown is incomplete. After several flushing operations of this kind, sediment levels in the reach immediately upstream from the dam will have risen to a little below the drawn down water level. Drawdown during flushing will then be complete because it will lower water levels to the new higher bed elevations. Thus flushing will eventually remove significant sediment quantities from the reservoir and further rises in bed elevations will be prevented. Whether full drawdown is established immediately, or after a period of bed level rise upstream from the dam, eventually the quantities of sediment deposited in the reservoir between flushing operations will balance the quantities removed by flushing. This sediment balance can be expressed:

$$Q_s T_f = N M_{in} T E \quad \rightarrow (2.5)$$

Where:

Q_s = Sediment transporting capacity of the flow in the drawn down reservoir (which is a function of the discharge, the channel width and slope, and the deposited sediment properties).

T_f = Duration of flushing.

N = Interval between flushing in years.

M_{in} = Sediment inflow rate

TE = Trapping Efficiency

2.13.3 Transporting Capacity of Flushing Flows

The transporting capacity of flushing flows can be estimated using an empirical method. The method is based on observations of flushing at reservoirs in China, where the predominant practice is annual flushing and so relatively little consolidation occurs between flushing operations. The method is based on the equation:

$$Q_s = \psi \frac{Q_f^{1.5} S^2}{W^{0.6}} \quad \rightarrow (2.6)$$

Where:

Q_s = Sediment transporting capacity (t/s)

Q_f = Flushing discharge (m^3/s)

S = Bed slope

W = Channel width (m)

ψ = Constant set from the sediment type:

i. 1600 for loess sediments
ii. 650 for other sediments with median size finer than 0.1mm
iii. 300 for sediments with median size larger than 0.1mm

iv. 180 for flushing with a low discharge.

Equation 2.6 was attributed to Tsinghua University and is referred to here as the 'Tsinghua University method'. The discrepancy between an individual observation and its prediction is within half to twice for 87% of the observations. Such discrepancy is common for predictions of sediment transport processes.

This prediction method for the transporting power of reservoir flushing flows has been used. However it appears that the equation will produce an overestimate of transporting power (by a factor of perhaps three or even more) when applied in conditions dissimilar to those in China where the original data were collected. However this can be allowed for by the use of a correction factor.

2.13.4 Channel Widths during Flushing

As channel width is an input to the sediment transport prediction method it must be predicted before the method can be applied. Width prediction is also vital when estimating the sustainable capacity which can be achieved in a flushed reservoir. The channel which cuts into sediment deposits during flushing is self-formed, and so its width can be expected to be controlled principally by the discharge, slope and sediment properties. However, channel width during flushing has been found to correlate well with the flushing discharge alone, with no apparent sensitivity to slope or sediment properties.

$$W_f = 12.8Q_f^{0.5} \qquad \rightarrow (2.7)$$

Where:

Q_f = Flushing discharge (m^3/s)

W_f = Flushing width (m)

2.13.5 A Criterion for Successful Flushing

An equation expressing the sediment balance (Equation 2.5) and methods which can be used to calculate the sediment loads during flushing have been presented in the preceding sections. Here the application of these methods to set a criterion for successful flushing is described. A Sediment Balance Ratio (SBR) is defined as;

$$SBR = \frac{sediment\ mass\ flushed\ annually}{sediment\ mass\ depositing\ annually} \qquad \rightarrow (2.8)$$

Using the variables defined for Equation 2.5, Equation 2.8 can be expressed as;

$$SBR = \frac{Q_s T_f}{N M_{in} TE} \qquad \rightarrow (2.9)$$

If SBR > 1.0 then it is expected that a sediment balance can be achieved and so this criterion is satisfied.

The calculation procedure for SBR would be:

a) Decide the likely frequency and duration of flushing. This could be an extended period of partial drawdown to pass high sediment loads in the flood season without deposition (sluicing), or complete drawdown for flushing for periods of days or weeks each year, or occasional flushing carried out every few years. The choice would depend on factors such as the purpose of the impoundment, the reservoir capacity relative to inflow, and the incoming sediment loads.

b) Estimate the sediment quantity to be removed from the reservoir by each flushing operation. In most cases this will be the product, N M TE, of the number of years between flushing, the annual sediment inflow and the reservoir trapping efficiency. Trapping efficiency can be estimated using Brune's or Churchill's methods presented in the ASCE Sedimentation Manual. For sluicing operation a trapping efficiency of 100% is appropriate for the drawdown period, as all the incoming sediment must be passed through the reservoir.

c) Select an initial value for the design flushing discharge. After the reservoir is drawn down the discharge during flushing will be the river discharge, and thus it will depend upon the time of year when flushing operations are planned. Discharge can be estimated from hydrological records for the rivers entering the reservoir. The flushing discharge must be passed through the dam with water levels close to their full drawdown levels and so the size of the low level outlets may limit the flushing discharge. There may be times of year when the reservoir should not be flushed. The penalty of providing small outlets that may limit flushing discharges should be compared with the greater costs of larger more effective outlets.

d) If the reservoir is to be flushed over a long period then the expected discharge hydrograph should be estimated. When the discharge in this hydrograph exceeds the design flushing discharge, sediments will deposit in the backwater region upstream from the dam. Therefore, periods with high river discharges should not be included in the computations of sediment removal by flushing.

e) Estimate the width of channel produced by flushing using Equation 3. For cases with a discharge hydrograph, rather than a single value, a means of combining the discharges to predict a "dominant discharge" is required. A method which provides some weighting towards the higher discharges in the range would be suitable. Other uncertainties in the method do not warrant more precise calculations. The width predicted from Equation 3 should be compared with the original bed width in narrow reservoirs, as the width before impoundment at the bottom of the reservoir may limit the channel width that can be achieved.

f) The purpose of flushing is usually to maintain the lowest bed elevation across each section at the original river bed elevation before impoundment. Therefore the slope of the channel at the end of flushing can be taken as the original river bed slope.

g) The three inputs to the calculation of sediment transporting capacity, Q_s, are now determined: discharge, width and slope. Transporting capacity can be calculated using Equation 2, noting that the sediment sizes are those depositing in the reservoir, not the river bed material size. If conditions are different from those for which the prediction method was developed, a factor of 3 should be applied to reduce the predicted Q_s value. An even greater factor should be applied where median sediment size is much larger than 0.1mm or where flushing is to be attempted after a long period of deposition and consolidation. The factor will allow for the expected overestimate in Q_s.

h) Estimate the duration of flushing, Tf, the flushing discharge and inflow hydrographs during the season when flushing will be undertaken will affect this estimate, together with considerations of the costs in interrupting normal reservoir operation.

i) Values for all the variables input to Equation 4 have been derived above. The equation can be used to derive the sediment balance ratio, SBR. If SBR> 1.0 then the criterion is satisfied.

j) If SBR is too low, then flushing may only be feasible at higher discharges, which may be possible by changing the period when the reservoir is to be flushed, or by providing larger flushing outlets in the dam.

2.13.6 Sustainable Reservoir Capacity

Sustainable reservoir capacity is defined as the storage capacity of a reservoir which can be sustained by flushing in the long term. If the lowest bed levels at each section of a reservoir that has been flushed have been returned to the original river bed, and the reservoir is narrower than the width of a self-formed channel produced by the flushing flow, then very little sediment will have remained in the reservoir. Small pockets of sediment may remain where the shape of the reservoir protects sediment from the flushing flow, but the sustainable reservoir capacity will be approximately equal to the original capacity.

The shape of cross sections that will eventually develop in flushed reservoirs can be determined on the basis of these observations. Firstly, fiat deposits will form at the reservoir operating level, secondly flushing will produce a scoured channel with an approximately trapezoidal section. The depth of the scoured channel will equal the reservoir operating level minus the original river bed elevation, the bottom width will equal the flushing width and the flushed channel will have uniform side slopes.

A total reservoir volume can be calculated from these assumed final cross sections. This volume will be the reservoir capacity which can be assumed to be sustainable in the long term: the ‘sustainable capacity’. In order to carry out the calculation of sustainable capacity a method for predicting the side slopes of the flushed channel is needed.

2.13.7 Side Slope Prediction

Side slopes of channels which cut down through reservoir deposits can vary enormously. At one extreme vertical side can form where the sediment is fully consolidated, at the other extreme slopes as low as 2.5% have been observed for poorly consolidated material. A variation within a reservoir is commonly observed.

The side slope which will develop during flushing depends on the sediment properties, the degree of consolidation, and the depth of deposits and perhaps also the extent of water level fluctuation during flushing. The last effect applies particularly to sand deposits. A sand deposit exposed by flushing will initially form near vertical sides due to the development of negative pore pressures. However the banks will later collapse after re-submergence because the pore pressures will then equalize.

The following equation has been developed based on simple theory and observations:

$$tan\,\alpha = \frac{31.5}{5}\,\rho_d^{4.7} \qquad \rightarrow (2.10)$$

Where:

α = Angle of slope which is just stable.

ρ_d = Dry density in t/m^3.

2.13.8 Successful Flushing Criterion Based on Sustainable Capacity Concept

Above described the assumed shape of the cross sections that will eventually develop in flushed reservoirs, as derived from observations of reservoirs which have been flushed:

- Flat deposits at the reservoir operating level, with
- A trapezoidal shaped scoured channel with its bottom at the original river bed elevation, and
- The bottom width equal to the flushing width.

The total reservoir volume which can be calculated from these assumed final cross sections has been termed the 'sustainable capacity'. A Long Term Capacity Ratio (LTCR), can be defined as;

$$LTCR = \frac{sustainable\ capacity}{original\ capacity} \qquad \rightarrow (2.11)$$

The reservoir capacities in Equation 2.6 are based on a simplified geometry. Values of LTCR greater than about 0.5 would indicate that the capacity criterion is partially satisfied; values approaching unity indicate that the criterion is fully satisfied. An acceptable value for LTCR will depend on the costs associated with flushing. In this methodology, a value of 0.5 is arbitrarily taken as the minimum for the criterion to be satisfied.

2.13.9 Use of Criteria to Assess Constraints to Successful Flushing

Four separate constraints to effective flushing have been considered, and a quantitative criterion can be applied for each one. The four constraints are:

1) Incomplete drawdown of the reservoir. The extent of drawdown can be expressed as a ratio, DDR:

$$DDR = 1 - \frac{flow\ depth\ for\ the\ flushing\ water\ level}{flow\ depth\ for\ the\ normal\ impounding\ level} \qquad \rightarrow (2.12)$$

The depths in Equation 8 are depths above the dam base. Drawdown could be insufficient if DDR is less than about 0.7.

2) Insufficient flushing flows for a sediment balance. Because incomplete drawdown can also compromise the sediment balance, a new definition of sediment balance ratio is required. The SBR ratio can be made independent of drawdown by calculating it for conditions when thalwegs are at the original river bed elevations that are for conditions of full drawdown. This new ratio is termed SBR_d.

3) Insufficient channel width formed by flushing. The scoured valley formed by flushing will have a bottom width approximately equal to the flushing width calculated from Equation3, unless this calculated width exceeds the width of the reservoir at that elevation. Flushing channel width should also be assessed independently of the extent of drawdown, so a Flushing Width Ratio (FWR) can be defined as;

$$FWR = \frac{predicted\ flushing\ width\ from\ equation\ 3}{representative\ bottom\ width\ of\ reservoir} \qquad \rightarrow (2.13)$$

If FWR is significantly less than unity then flushing width can be considered an important constraint. An exception will arise, however, for reservoirs where the side slope of the exposed deposits is shallow, this is discussed in (4) below.

4) Side slope too steep. A steep side slope in the scoured valley formed by flushing will be a constraint when either constraint (3) above applies, or when reservoir bottom widths are small when compared to the top widths (that is width at full storage level). Side slope can be quantified as a constraint by means of a reservoir Top Width Ratio (TWR):

$$TWR = \frac{top\ width\ of\ scoured\ valley}{actual\ top\ width} \qquad \rightarrow (2.14)$$

The scoured section should be assumed to be constrained only by the reservoir bottom width for the calculation of this ratio. Any lack of drawdown should not be considered in the calculation of top widths. If constraint (3) is important then TWR should comfortably exceed 1 (say TWR > 2) to overcome that constraint. If (3) is not a constraint, then TWR values approaching 1 are sufficient.

2.13.9.1 Calculation Method in Establishing Feasibility Criteria

The ratios defined in the above text, can be calculated using the following parameters:

C_o	the original storage capacity of the reservoir (m^3)
L	reservoir length (m)
El_{max}	elevation of top water level (m)
El_{min}	the minimum bed elevation, which is usually the river bed elevation immediately upstream from the dam (m)
W_{bot}	a representative bottom width for the reservoir (m)
SS_{res}	a representative side slope for the reservoir
SS_s	a representative side slope for the deposits exposed during flushing, it can be derived using Equation 2.10 with density computed using Lane and Koelzer's (1953) method
V_{in}	mean annual inflow volume (m^3)
M_{in}	mean annual sediment inflow (tonnes)
Q_f	representative discharge passing through reservoir during flushing (or sluicing if appropriate) (m^3/s)
T_f	duration of flushing (days)
El_f	water surface elevation at the dam during flushing, derived from of, outlet sill elevation and outlet design (m)
Type	sediment type for the Tsinghua University method for predicting sediment loads in flushing flows

2.13.9.2 Sediment balance ratio

The sediment balance ratio, SBR, is defined as;

$$SBR = \frac{M_f}{M_{dep}} \qquad \rightarrow (2.15)$$

Where:

M_f = Mass of sediment flushed annually from the reservoir, and

M_{dep} = The mass of sediment which deposits annually in the reservoir

The sediment masses M_f and M_{dep} are mean values which would apply to a typical year.

The calculation of SBR is performed as follows:

i. Derive a representative reservoir width in the reach upstream from the dam at the flushing water surface elevation:

$$W_{res} = W_{bot} + 2SS_{res}(El_f - El_{min}) \quad \rightarrow (2.16)$$

ii. Calculate the flushing width using Equation 2.7:

$$W_f = 12.8Q_f^{0.5} \quad \rightarrow (2.17)$$

iii. Take the minimum of W_{res} and W_f as the representative width of flow for flushing conditions, W.

iv. Estimate the longitudinal slope during flushing

$$s = \frac{El_{max} - El_f}{L} \quad \rightarrow (2.18)$$

v. Determine the parameter L in the Tsinghua University method for sediment load prediction

1. Ψ = 1600 for fine loess sediments
2. Ψ = 650 for D50 < 0.1mm
3. Ψ = 300 for D50 ≥ 0.1mm
4. Ψ = 180 if the flushing discharge is low (say less than $50m^3/s$)

vi. Calculate the sediment load during flushing

$$Q_s = \psi \frac{Q_f^{1.5} s^2}{W^{0.6}} \quad \rightarrow (2.19)$$

Reduce Q by a factor of 3 for reservoirs where conditions are dissimilar to those in China.

vii. Determine the sediment mass flushed annually (86,400 is the number of seconds in a day)

$$M_f = 86{,}400\, T_f Q_s \quad \rightarrow (2.20)$$

viii. If the reservoir is sluiced, i.e. a long draw down period to pass the high sediment loads without deposition, then a Trapping Efficiency (TE) of 100% should be selected, otherwise predict TE using Brune's curves and the values for C_o and V_{in}.

ix. Calculate the mass depositing annually which must be flushed.

$$M_{dep} = \frac{M_{in}\, TE}{100} \quad \rightarrow (2.21)$$

x. Determine SBR

$$SBR = \frac{M_f}{M_{dep}} \qquad \rightarrow (2.22)$$

xi. Determine W_{bf} the bottom width of the scoured valley at full drawdown. It is the minimum of W_{bot} and $W_{f;}$

xii. Calculate W_{td} from the side slope SS which is discussed in Section

$$W_{td} = W_{bf} + 2SS_s(El_{max} - El_{min}) \qquad \rightarrow (2.23)$$

Both El_{max} and El_{min} are defined in the input to the calculations.

xiii. Determine TWR

$$TWR = \frac{W_{td}}{W_t} \qquad \rightarrow (2.24)$$

2.13.9.3 Long term capacity ratio

The Long Term Capacity Ratio (LTCR) is defined using a simplified reservoir shape model. Firstly the reservoir is assumed to approximate to a prismatic shape with trapezoidal cross sections. Therefore, a reservoir cross section at the dam site is representative of conditions within the reservoir. At this section, the ratio of cross sectional area for the channel formed by flushing to the original reservoir cross sectional area is determined. The ratio is taken to be indicative of the capacity ratio for the entire reservoir. LTCR can be calculated from these parameters: EL_{max}, El_{min}, W_{bot}, SS_{res}, SS_s and El_f used as input to the procedure, from W_{res} and W derived in steps (i) and (iii) of the SBR calculation.

$$LTCR = \frac{A_f}{A_r} \quad \rightarrow \quad (2.25)$$

$$W_{tf} = W + 2SS_s\left(El_{max} - El_f\right) \quad \rightarrow \quad (2.26)$$

$$W_t = W_{bot} + SS_{res}(El_{max} - El_{min}) \quad \rightarrow \quad (2.27)$$

If $W_{tf} < W_t$

Then

$$A_f = \frac{W_{tf} + W}{2}(El_{max} - El_{min}) \quad \rightarrow \quad (2.28)$$

If $W_{tf} > W_t$

Then

$$A_f = Wh_f + \left(h_f + h_l\right)h_m SS_s + h_l^2 SS_{res} \quad \rightarrow \quad (2.29)$$

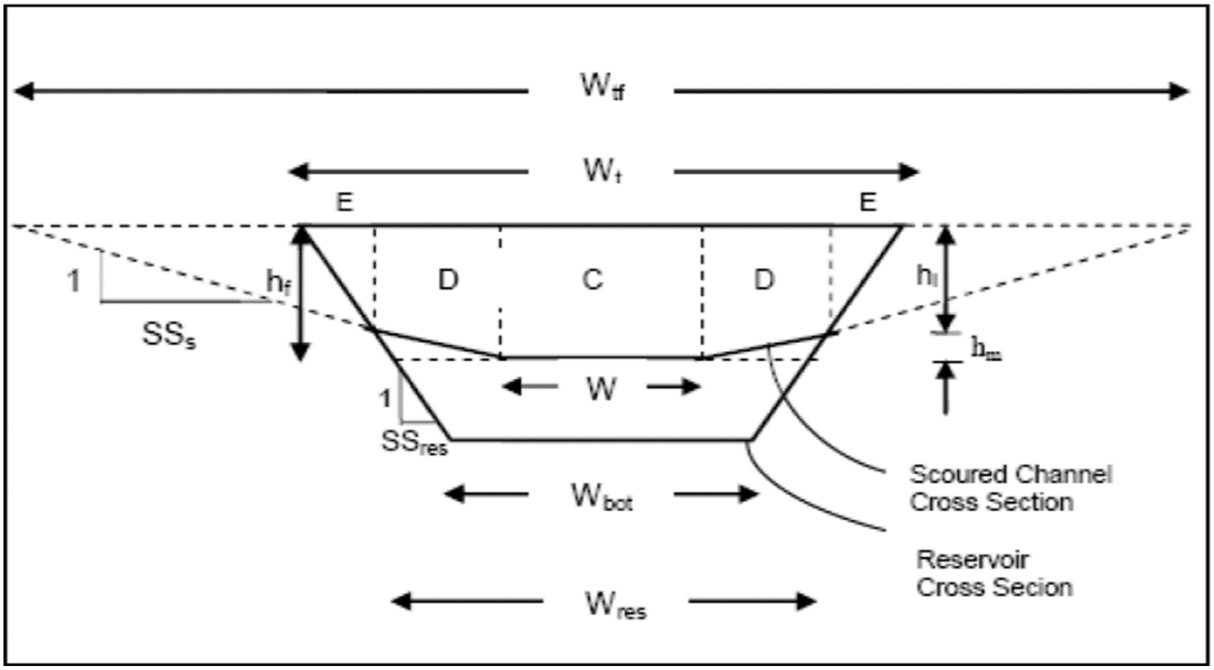

Figure 2.3: Cross section immediately u/s of the dam for simplified reservoir geometry (Atkinson, 1996)

Where, h_m , h_l and h_f are defined in figure 2.3 and calculated below as;

$$h_m = \frac{W_{res} - W}{2(SS_s - SS_{res})} \quad \rightarrow \quad (2.30)$$

$$h_l = El_{max} - El_f - h_m \quad \rightarrow \quad (2.31)$$

$$h_f = El_{max} - El_f \quad \rightarrow \quad (2.32)$$

$$A_r = \frac{W_t + W_{bot}}{2}(El_{max} - El_{min}) \quad \rightarrow \quad (2.33)$$

Where;

A_f = Cross sectional area of valley scoured out by flushing (m^2).

A_r= Cross section area of reservoir in reach immediately upstream from dam (m^2).

El_f = Water surface elevation at the dam during flushing (m).

El_{max} = Elevation of top water level (m).

El_{min} = Minimum river bed elevation immediately upstream from the dam (m).

SS_{res} = Representative side slope for the reservoir.

SS_s = Side slope for the deposits exposed by flushing.

W = Width of Flow flushing condition (m).

W_{bot} = Bottom width of the reservoir (m).

W_{res} = Reservoir width in the reach upstream from the dam at flushing water surface elevation (m).

W_{tf} = Top width of the scoured valley at the top water level (m).

2.13.9.4 Drawdown ratio

This ratio, termed DDR, is defined;

$$DDR = 1 - \frac{El_f - El_{min}}{El_{max} - El_{min}} \quad \rightarrow (2.34)$$

2.13.9.5 Sediment balance ratio with full drawdown

This ratio, SBR_d is defined and calculated in the same manner as SBR. The only difference is in steps (i) and (iv) which use El_f ; its value for full drawdown should be used. That is:

$$El_f = El_{min} \quad \rightarrow (2.35)$$

2.13.9.6 Flushing width ratio

The Flushing Width Ratio (FWR) is:

$$FWR = \frac{W_f}{W_{bot}} \quad \rightarrow (2.36)$$

Where:

W_f = Calculated using aforementioned equation

W_{bot} = Input parameter

2.13.9.7 Top width ratio

The top width ratio for a flushed reservoir, TWR, is calculated:

$$TWR = \frac{W_{td}}{W_t} \quad \rightarrow (2.37)$$

Where:

W_{td}= The value for scoured valley width at top water level if complete drawdown is assumed.

W_t = The reservoir top width calculated in previous Section.

Chapter 3

GULPUR HYDROPOWER PROJECT

3.1 GENERAL

The Gulpur Hydropower Project (HPP) is a run-of-river scheme located on Poonch River about 25 km upstream of very famous Mangla Reservoir in Pakistan. The proposed Gulpur HPP scheme is located in NE Pakistan on the Poonch River near Kotli town. The coordinates of the location are 394781 m E, 3702129 m N. General location of the weir in the Poonch River is shown in figure 3.1.

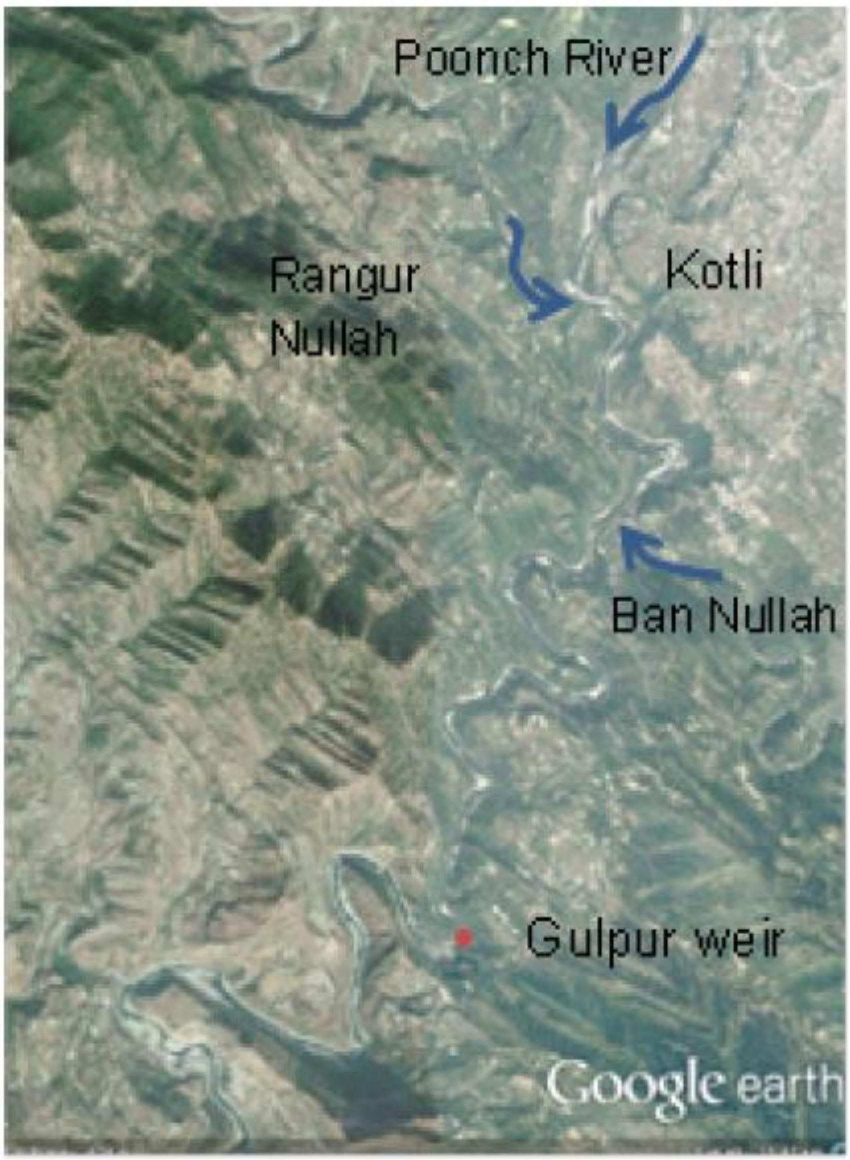

Figure 3.1: General location of the weir in the Poonch River

The weir is located at the Poonch River that flows into the Mangla reservoir 35 km downstream. Approximately 7 km upstream the weir there is a tributary on the left side, Ban Nullah, and approximately 10 km upstream the weir there is another tributary on the right side, Rangur Nulla.

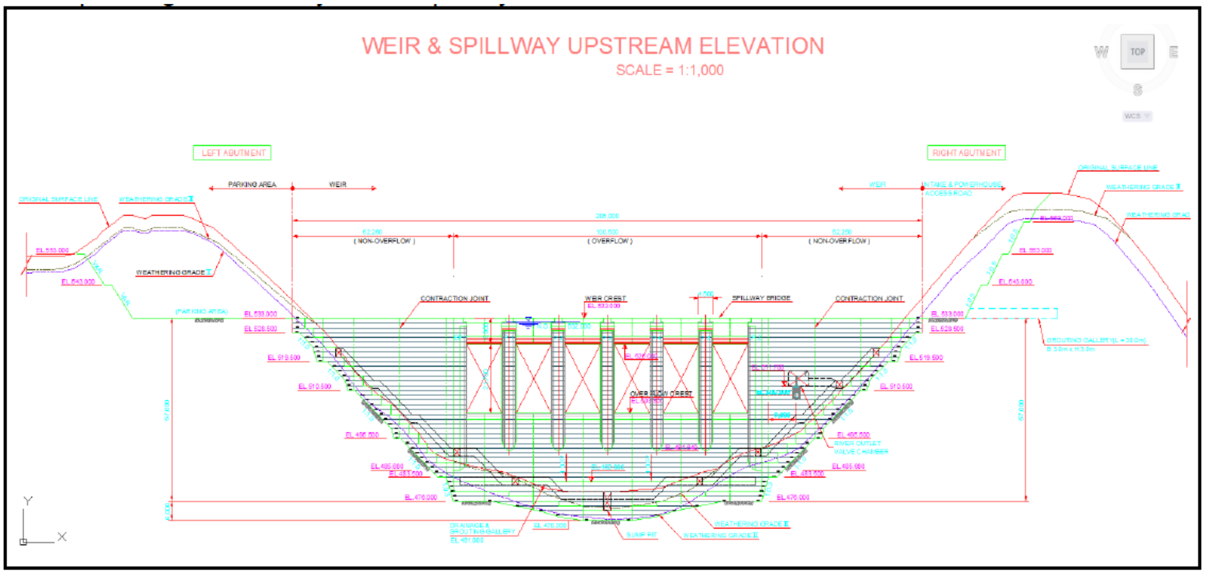

Figure 3.2: Cross-section of the weir

The Gulpur HPP has a 66 m high dam with a crest level at 533.5 masl having 205 m length as shown in Figure 3.2. The weir structure is provided with 6 ogee type spillways with free overflow having crest level at 503.3 masl and a width of 11.5 m each. The design flood of the weir is 17208 m^3/s that corresponds to 500 years frequency flood.

The power intake is located on the right abutment approximately 30 m upstream the weir. It has two openings 12 m wide, 10 m high and 30 m long each. The bottom level of the intake is 519 m. The design capacity is 100.5 m^3/s. The Project with 57.45 m head has installed capacity of 102 MW and will generate 465 GWh of energy per annum. The turbine operation plan is described in the table 3.1.

An upstream and downstream cofferdam will be built during construction of the weir to divert the river. The crest level of the upstream cofferdam is 503 m.

The maximum water level (MWL) at the reservoir created by the weir is 532.2 m and the normal operating level (NOL) is 532 m. The length of the reservoir with normal operating level is about 10 km and therefore, it creates a backwater effect on the two tributaries upstream.

Table 3.1: Turbine operation plan for inflow

Classification	Inflow range	Turbine operation	Remark
Case 1	$Q < 20.1\ m^3/s$	Non-operation	Minimum unit discharge
Case 2	$20.1 \leq Q < 100.5\ m^3/s$	1 unit operation	-
Case 3	$100.5 \leq Q \leq 1{,}000\ m^3/s$	2 unit operation	Inflow evenly distributed to the turbines
Case 4	$1{,}000\ m^3/s < Q$	Non-operation	-

3.2 CATCHMENT DATA

The Poonch River is one of the four main tributaries of the Jhelum River (Neelum, Kunhar, Kanshi and Poonch are the Jhelum River tributaries). Unlike the Jhelum River, which originates in the Himalayas and passing through the Wular Lake, the Poonch River rises in the Pir Panjal ranges at an elevation of about El. 4700 and it has its confluence with the Jhelum River in Mangla Reservoir at an elevation of about El. 366. The total length of the Poonch River is about 141 kilometers (km), with 59 km in Indian occupied Kashmir. The length in Pakistan territory is about 82 km, from the line of control (LOC) to Mangla Reservoir.

In its initial reach, the Poonch River is very steep and drops down from an elevation of about El. 4000 to El. 1000. Near Poonch village, it enters a comparatively wide valley. The river again enters a narrow canyon near Sehra, and after flowing for nearly 20 km through a narrow and quite steep valley, it emerges at Kotli into a broad valley. The average slope of the Poonch River from Tatta Pani to the powerhouse location near Dakhari is about 0.57 percent. Downstream of Kotli, the Poonch River flows about 40 km, again through a very narrow valley, down to village Rajdhani, the starting point of Mangla Reservoir. The Poonch River has a relatively flat gradient of about 0.3 percent in its lower part and it navigates its way through a series of meanders in between sandstone ridges.

The mean elevation of the catchment area is about El. 1800. High mountains above elevation of El. 4000 remain snow covered almost throughout the year. The catchment includes a few permanent glaciers in the Pir Panjal ranges, located in Indian occupied territory. The area above El. 1800 receives considerable snowfall, which substantially contributes to the Poonch River's river flow. The drainage is roughly round in shape. After a heavy rainfall, the river swells quickly and gives rise to high peak discharges within a short period. The bulk of Poonch River runoff, however, results from monsoons.

The river has the following main tributaries:

1) Suran River
2) Mandi River
3) Betar Nullah
4) Rangun Nullah
5) Rangar Nullah
6) Rangur Nullah with tributaries of Bobra and Sarsawah
7) Mendhor River
8) Nail Nullah
9) Ban Nullah with tributaries Nail, Kharban and Banaban
10) Mamhuli Nullah
11) Duliah Nullah

The mean elevation of these tributaries and nullahs decreases from El. 2500 in the northern part, down to El. 700 in the southern part of the Poonch River catchment. The flat areas in southern parts are heavily cultivated. Northern areas are covered with vegetation and forests. At higher elevations some uncovered areas can also be seen.

The total catchment area of the Poonch River is about 4196 km^2 out of which about 2446 km^2 lies in Indian occupied Kashmir, and remaining 1750 km^2 in Azad Jammu and Kashmir (AJK). The catchment area is located between latitude 33°-13' to 34°-00' North and longitude 73°-39' to 74°-34' East. Whereas, the catchment area up to the proposed dam site is about 3,648 km^2. Mean weighted elevation of the catchment is about 2380 masl. The catchment area along with its contributing nullahs is shown in Figure 3.3.

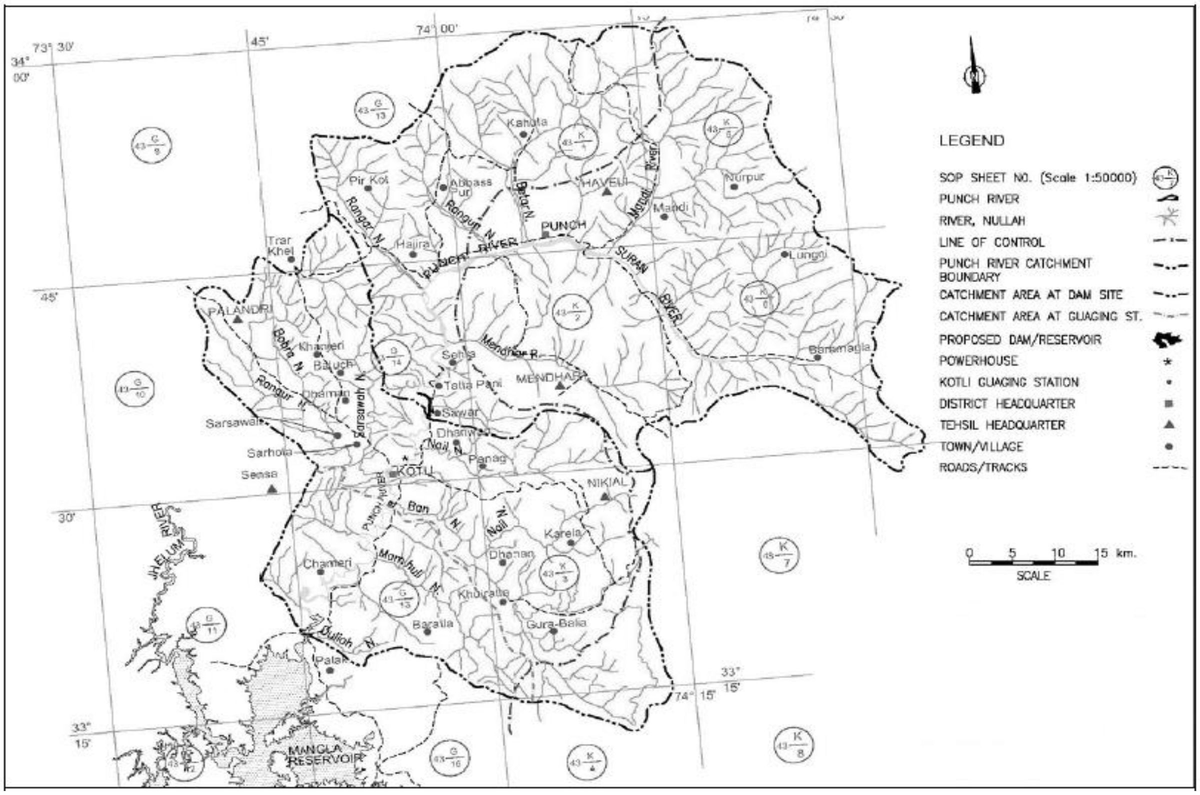

Figure 3.3: Catchment area of Poonch River

3.3 ANALYSIS OF FLOW DATA

Water discharges at Rehman bridge station have been provided for the period 1960 to 2011 for monthly average discharges and for the period 1960 to 2009 for daily values. The station is located on the Poonch River just downstream the confluence with Ban Nullah. The average mean annual flood in the recorded period is 125.4 m^3/s shown in Figure 3.4.

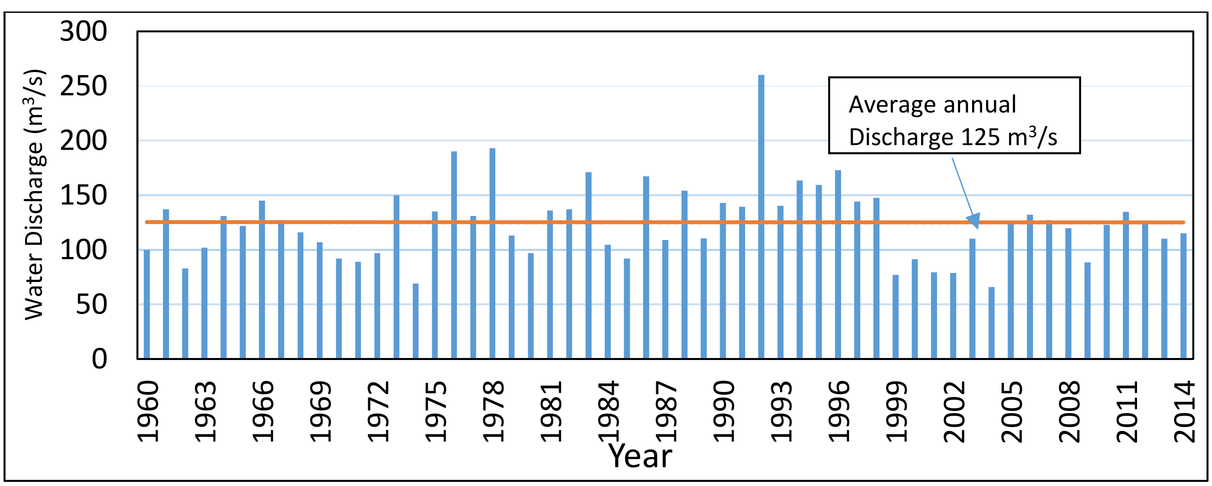

Figure 3.4: Ave. annual discharge from 1960-2014

The average monthly values show two annual peaks of discharge, a minor one in March-April (of the order of 180 m^3/s) related to the snowmelt period, and a larger one in July-August (with an average value of 264 m^3/s). The monthly variation of flow within an average year is shown in Figure 3.5.

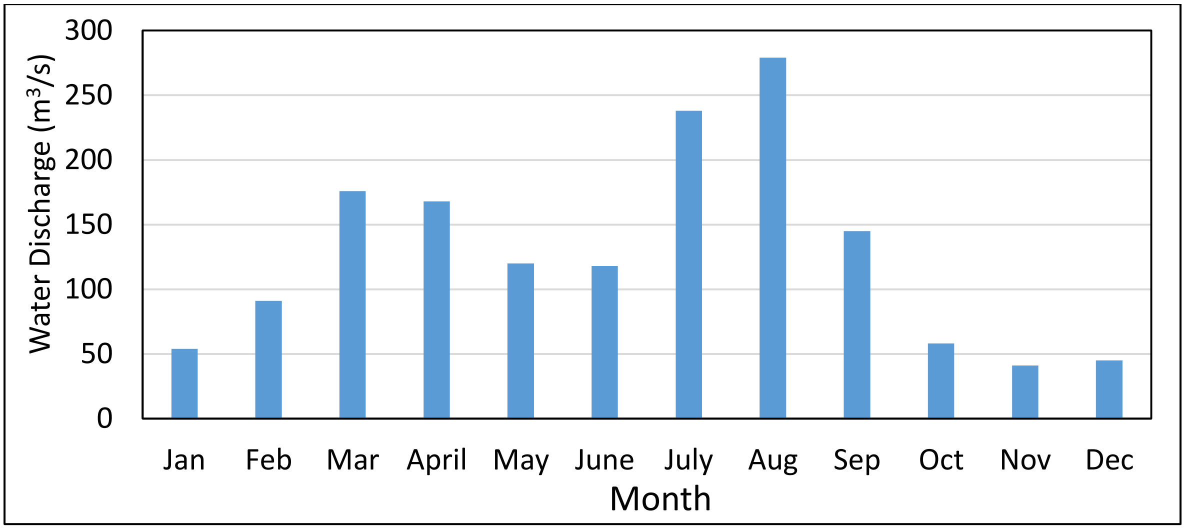

Figure 3.5: Ave. monthly water discharges from 1960-2014.

The maximum and minimum monthly water discharges follow a similar pattern to the ones shown for the average values. The maximum monthly discharge is 830 m^3/s, recorded in September 1992.

This trend is also shown in Figure 3.6 with the average daily discharges.

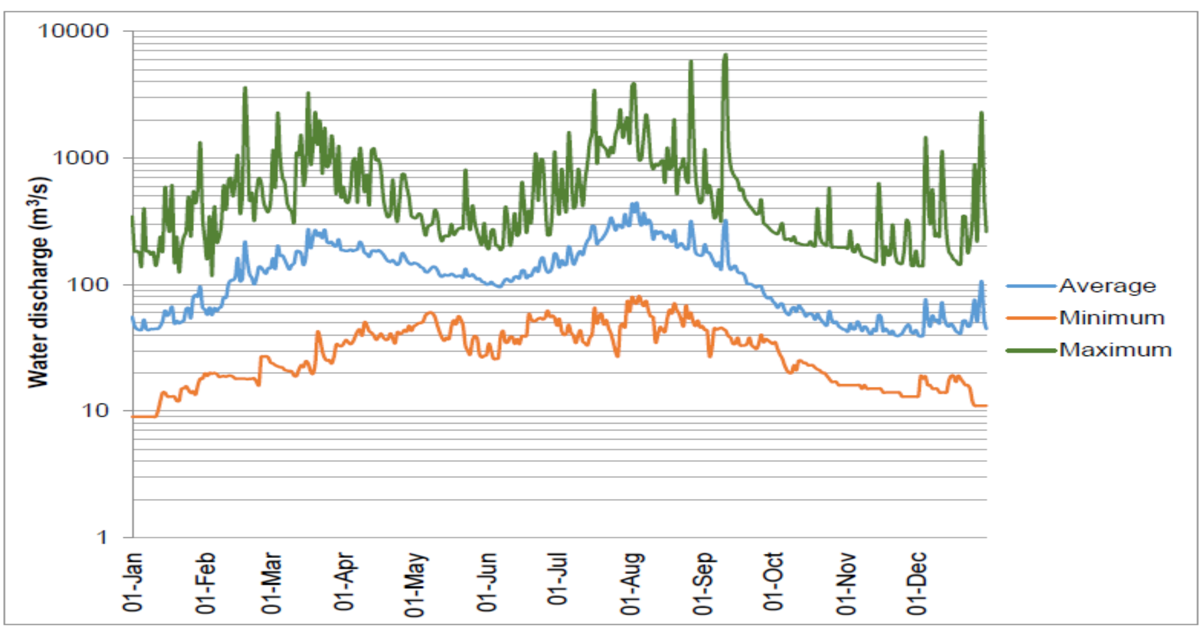

Figure 3.6: Average daily discharges

Daily values show large variations of discharge. As example, daily values for the 1992 event (in September) are shown in Figure 3.7

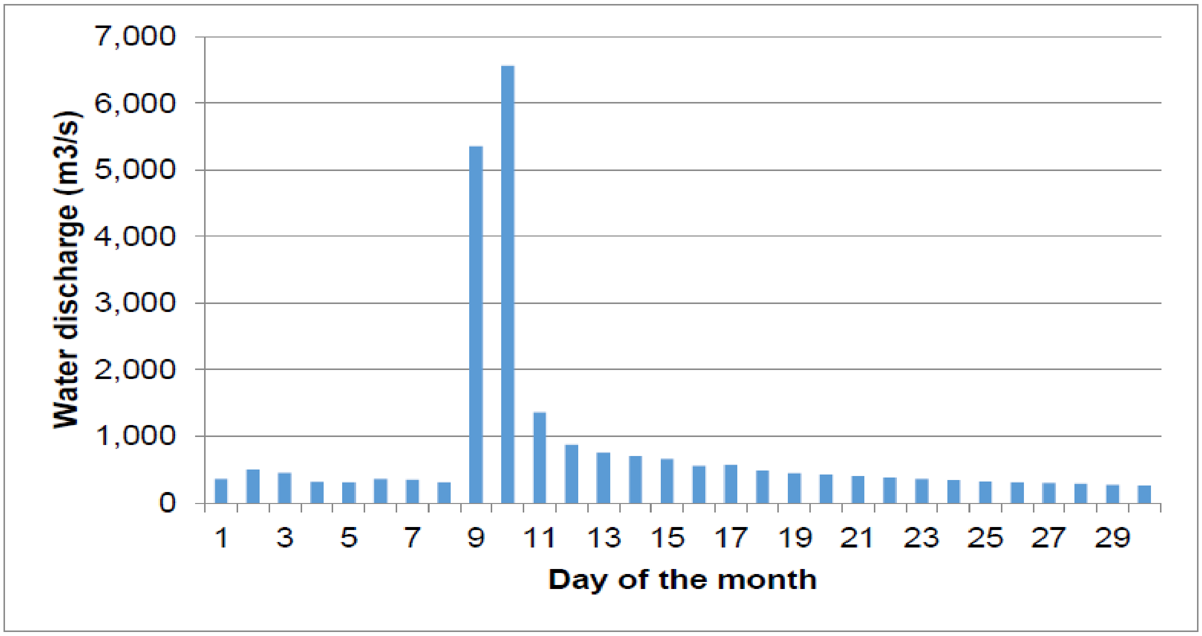

Figure 3.7: Daily discharges in September 1992

3.4 SEDIMENT INFLOW ESTIMATION

Surface Water Hydrology Project (SWHP) of WAPDA observe the suspended sediment concentration at Rehman Bridge gauging station at a regular interval in addition to the daily flow measurements. Suspended sediment concentration data for the period 1960 to 2014 has been collected and used in the analysis.

3.4.1 Sediment Rating Curve

Suspended sediment concentration data is used to develop correlation between the rate of flow and sediment load. This relationship is termed as sediment rating curve. As per UBSR Guidelines (1987) sediment rating curve can best be computed by least square method and usually defined by one of three relationship:

$Q_s = 0.006048\ Q_w^{2.95}$ For $Q_w < 153$ m^3/s ………… (3.1)

$Q_s = 0.25056\ Q_w^{2.22}$ For $153 < Q_w < 530$ m^3/s ……. (3.2)

$Q_s = 94.0032\ Q_w^{1.276}$ For $Q_w > 530$ m^3/s ……..… (3.3)

Where;

Q_s = Sediment load, Tons/day

Q_w = Water discharge, Cumecs

The annual suspended sediment load is calculated for the existing flow series, from 1960 to 2014, as shown in Figure 3.8.

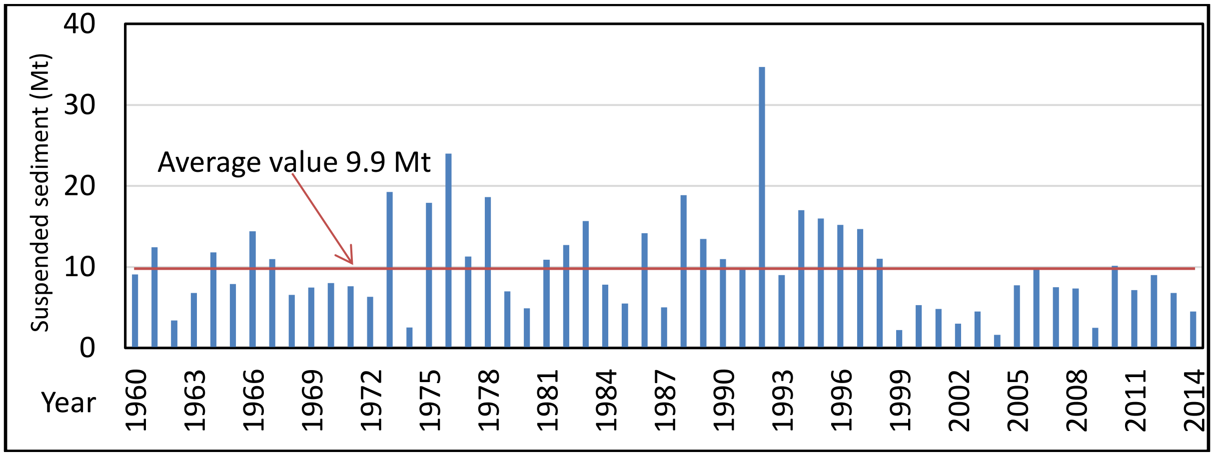

Figure 3.8: Annual suspended sediment load from 1960-2014

It is estimated that mean annual suspended sediment load is about 9.9 million tons. The minimum value of 1.6 millon tons was estimated for year 2004 and maximum value of 34.7 millon tons was estimated for year 1992. It is further noted that about 65% of the annual suspended sediment load comes during the three month: July through September and about 15% in the month of March. Only 20% of the annual suspended load comes during the remaining eight month.

To account for unmeasured load close to bed, the sedimentation report suggests to increase the total amount by 20%. This is considered to be a good estimate and likely to be on the safe side. This estimate can be supported by procedure of Strand and Pemberton (1987) for estimating bed load. According to this procedure, for the cases when there is no significant amount of sand present in bed (bed consist of compacted clay, gravel, cobbles and boulders – compare with photographs of Poonch river bed in Figure 3.9) and percentage of sand in the suspension is less than 25% (the average percentage of sand in Poonch suspended samples is 11%), the percentage of bedload is given in the range of 5-15%. However for the cases when streambed is composed of sand material and percentage of sand in suspended material is above 25%, this range is given as between 10 and 35%.


Figure 3.9: Pictures of bed material

Field data show that the percentages of clay, silt and sand vary largely. An attempt to relate these percentages to discharge does not show reliable correlations. The average percentages of clay, silt and sand in suspended load are 27, 62 and 11 per cent.

Assuming that bed load, which is not measured, is 20 % of the suspended load , the average annual sediment inflow into the proposed Gulpur reservoir is 11.9 Mt.

The percentages of sand, silt and clay do not show reliable correlations with water discharge. The average percentages for each fraction are calculated from the suspended field data (ISAN, 2015) and an additional 20% is added to the sand fraction that corresponds to the unmeasured material transported as bed load. The percentages considered in this study are shown in the next table. The values obtained are similar to those used by Mott MacDonald (2011) and cited in Hager & Bailey (2014). The percentages in that study were 12% clay, 60% silt and 28% sand. In the present study, the percentage of clay is higher and the percentage of silt is lower, while the proportion of sand (the fraction that is most easily trapped in a reservoir) is nearly the same.

The numerical simulations presented in the following chapter, require the definition of representative sizes for each of the sediment fractions. There is no information of sediment sizes in the provided data and therefore, several assumptions of sizes and percentages for each fraction are considered.

For the clay fraction one representative size, 0.003 mm, and one percentage, 23% is considered.

For the silt fraction, two representative sizes of fine and coarse silt, 0.01 mm and 0.035 mm, are considered. The total percentage of silt is 50% and it is assumed to be split uniformly between the two fractions considered.

The sand fraction is split in four sizes corresponding to a very fine, medium and coarse sand: 0.1 mm, 0.3 mm and 1 mm respectively and one size corresponding to a medium gravel. Several percentages for each fraction have been considered. The estimations take into account the different transport capacity that depends on the sediment size.

The study of the deposition pattern in the reservoir considers the above as the base case scenario but to take into account the uncertainties related to the assumption presented above, other scenarios with different types of sediment inflow are also considered in chapter 5.

Chapter 4

PHYSICAL MODELLING OF GULPUR HPP

4.1 INTRODUCTION

In order to undertake the aforesaid possible objectives, physical modelling of the Gulpur Hydropower Project is better tool to study the sediment transport behavior and to optimize the energy generated from this project. Physical Modelling was carried out at well reputed Nandipur Laboratory located in District Gujranwala, Punjab Pakistan. It is worth mentioning that Physical Models of many important hydraulic structures of Pakistan have been developed to examine certain phenomena i.e. Sediment Transport & behavior of the hydraulic structures under different flow conditions etc.

It is understandable that distorted/undistorted physical modelling was carried out keeping in view the scale limitation for these cascade hydropower projects. The results obtained from the Physical Modelling were correlated with Numerical Modelling using state of the art software i.e. HEC-RAS, RESSASS. Verification and Calibration of the model were carried out in line with the available data.

The physical model of Gulpur HPP was built with geometric scale ratio of 1:40 at Nandipur Research Institute, Gujranwala, Pakistan. The river bed elevation of 480 masl at just upstream of the dam in Prototype corresponds to bed elevation of 230 masl in the Model.

4.2 THEORY

Whenever it is necessary to perform tests on a model to obtain information that cannot be obtained by analytical means alone, the rules of similitude must be applied. Similitude is the theory and art of predicting prototype performance from model observations.

Here, geometrical similitude, kinematic similitude and dynamic similitude should exist between model and prototype. Dynamic similarity exists between geometrically and kinematically similar systems if the ratios of all forces in the model and prototype are the same. Dimensionless parameters of dynamic similitude are Euler number, Froude number, Reynolds number, Weber number and Cauchy number. Since it is regarded almost impossible to realize a model with the same number of these parameters with prototype, dominant forces are selectively considered when achieving the similitude law for a model experiment.

When gravity force is dominant, Froude model law is adopted. Likewise, viscosity corresponds with Reynolds model law, surface tension with Weber model law, and elastic force with Cauchy model law. If multiple forces are dominant, all the relevant model law should be applied to model testing and analysis.

4.2.1 Model Laws and Scales

Modelling strategy requires due consideration of similitude, appropriate or practicable model scales, model layout, and model construction.

Full model-prototype similitude requires satisfaction of the following conditions:

a) Geometric similitude, whereby the ratio of all length dimensions are same and where only similitude in form is involved.
b) Kinematic similitude (time and velocity), whereby at geometrically homologous points in model and prototype, velocities and accelerations are in a constant ratio.
c) Dynamic similitude (force), whereby, in addition to Kinematic similitude, the force polygons are similar at geometrically equivalent points for model and prototype.

It is known that for a river model where flow is driven by gravity, the similarities of geometry, kinematic and dynamic forces between model and prototype at geometrically similar locations are governed by Froude Number in model law. However, a movable bed test is for observing particle movement and the behavior of particles is determined by viscosity, and thus the similitude of Reynolds number between model and prototype should also be considered. For fully turbulent flow, the effect of surface tension becomes negligible as Reynolds number becomes greater than 4000. Hence, in a river where turbulent flows occur (Re>4000), the representation of the flow system in the river is governed by the law of gravity flow, or the Froude model law. The Froude number is defined as:

$$F = \frac{v}{\sqrt{gd}} \qquad \rightarrow (4.1)$$

That is

$$\frac{v_r^2}{g_r d_r} = 1 \qquad \rightarrow (4.2)$$

Where;

V = velocity of flow (m/s),

g = acceleration due to gravity (m/s^2), and

d = hydraulic depth (m), and the subscript "r" indicates the model to prototype ratio.

In order to satisfy the dynamic similitude requirement, the model and prototype should have the same Froude Number.

Reynold's number of Gulpur project is greater than 4000, therefore, Reynold's model law is not considered. Therefore, the model in Gulpur project use only Froude's model law. The model was built with geometric scale ratio of 1:40. The following scale ratios shown in Table 4.1 are generated according to Froude's model law (Bansal, 1986):

Table 4.1: Scale ratio of physical model

Dimension	Ratio	Scale Relation
Length	L_r	1: 40
Time	$T_r = L_r^{1/2}$	1: 6.32
Velocity	$V_r = L_r^{1/2}$	1: 6.32
Discharge	$Q_r = L_r^{5/2}$	1: 10119
Pressure	$P_r = L_r^{1/6}$	1: 1.85

4.3 SEDIMENT MODEL

Sediment transport is a complex process. Modelling of sediment transport in physical models adds even more complexity. The entrainment, transportation, and subsequent deposition of sediment depend not only on the characteristics of the flow involved, but also on the properties of the sediment itself (ASCE Task Committee, 1977).

Classification of sediment load can be divided into two groups:

1) Suspended load that is carried in suspension with the flow and
2) Bed load that moves on or near the bed.

As suspended sediment in the river is mostly below couple of mm size, the model scale of the suspended sediment is microscopic, and thus only bed load is simulated in the modelling.

4.3.1 Quantity of Sediment

Bed load sampling in a river like the Poonch River is almost impossible. The empirical formulas to calculate the bed load such as Meyer Peter and Meuller (1949), and Engelund and Hansen (1967) and others are not applicable for Himalayan Rivers. Therefore, the bed load is derived based on the measured suspended sediment concentration at the headworks site. Bed load in general is expected to be about 10% of the total sediment load, but has been conservatively increased to 20% for the project (in basic design).

4.3.2 Condition for Selecting Model Particle

The weight and electrostatic force or viscosity among particles are in relation with particle size and affect the selection of the particle size for the model. As the particle size reduces from silt to clay, its influence on the electrostatic force among particles increases. Therefore, clay particles exhibit different form of movement than that of sand or gravel. While the weight of sand and gravel particles is the determinant factor of particle movement, the electrostatic force is a decisive in case of clay particles as they are clumped together and move in chunks, instead of moving independently from one another. When preparing the model particles, the minimum particle size should not be smaller than silt size. And the minimum particle diameter has been set to be about 0.2mm for this test. For the application of similitude law by particle size, refer to Table 4.2:

4.4 MODEL CONSTRUCTION

The model construction was initiated with the excavation of riverbed which is generally carried out manually by experienced labors, leaving in banks in compacted form. But keeping in view the time limits, the river channel of the Gulpur site was excavated both manually and mechanically where necessary, by using an excavator.

Table 4.2: Application of Froude number by particle size

Larger than 2~3mm	Froude model law
0.2~2mm	Froude model law with small inaccuracies
Smaller than 0.2mm	Froude model law cannot be used (owing to cohesive binding force become dominant)

After completion of the excavation of riverbed, the side hills were moulded according to the contours of topographic plan provided by the Engineering Procurement Construction and Commissioning (EPCC). The moulding was followed by hand compaction. Which were then applied with brick lining, the brick layer was grouted by sand cement mortar of 4:1 ratio. The river bed was moulded with locally available sand.

The gated spillway along with the stilling basin, the piers and radial gates were fabricated in Plexiglas transparent material according to the drawings and data provided by the EPCC. The structures were mounted on a steel frame to avoid deformation and unnecessary vibration and then installed on the model. Similarly, the power intake structure was fabricated in Plexiglas material which followed by the PVC pipes as headrace tunnels. The tailrace was constructed precisely according to the design provided by the EPCC. Model as constructed is shown in photographs as Figure 4.1 and Figure 4.2.

Figure 4.1: Model as constructed (scale 1:40)

Figure 4.2: Weir upstream view

For feeding measured discharge in the model, discharge measuring arrangements have been provided at two different locations upstream of the model. At both the locations a suppressed sharp crested weir was installed at the end of a discharge measuring flume. To maintain the desired tail water levels, the tail gate arrangement has been provided at the downstream end of the model as shown in Figure 4.3 and Figure 4.4.

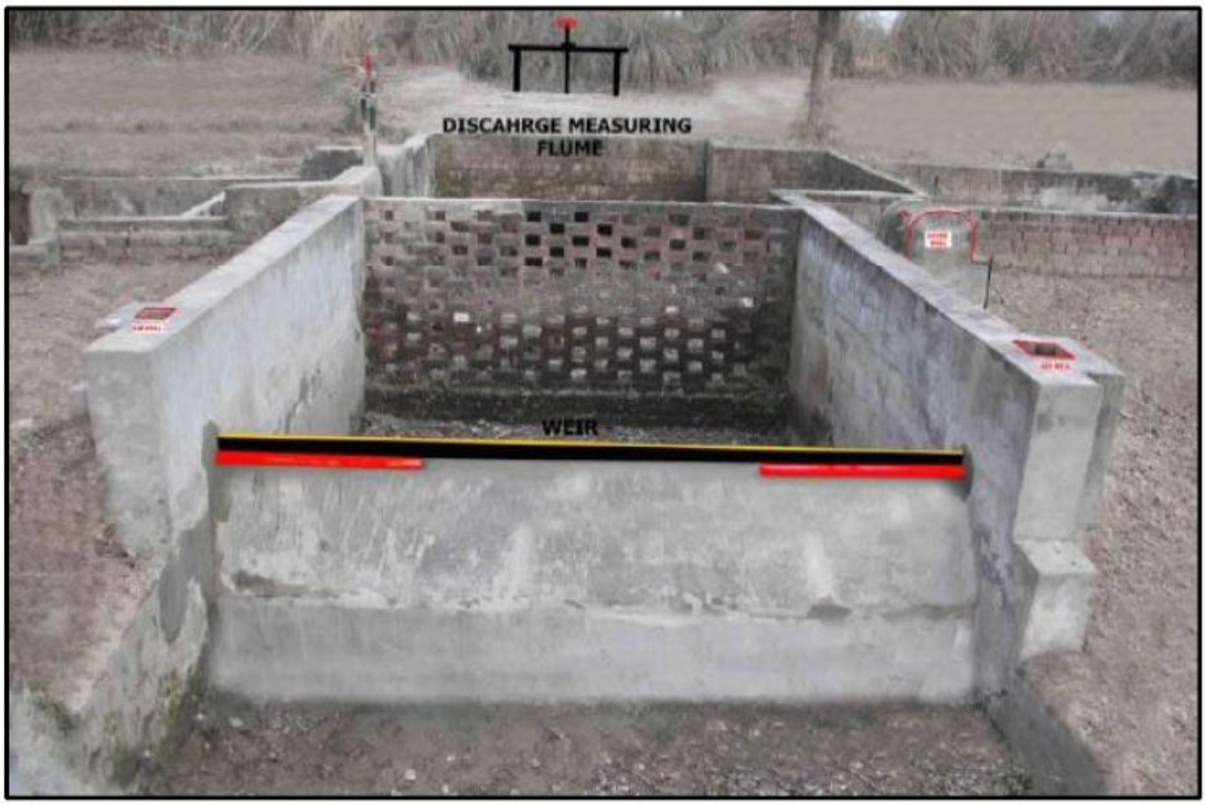

Figure 4.3: Discharge measuring flume

For discharge measurement from the sharp crested weir, the Rehbock formula has been used as recommended in Water Measurement Manual by Unites States Department of Interior, Bureau of Reclamation, Colorado:

$$Q = \frac{3}{2} B h_e \left[3.228 + 0.435 \frac{h_e}{z}\right] \quad \rightarrow (4.3)$$

Where,

Q = Quantity in cubic feet per second,

$h_e = h + 0.0036$,

h = observed head on crest, in feet, without correction for velocity of approach,

Z = height of weir crest above bottom of channel of approach (in feet), and

B = length of weir crest (in feet)

Figure 4.4: Tail gate arrangement

4.5 SEDIMENT FLUSHING TEST

Effective flushing is essential for a long-term operation of a dam as it deals with sedimentation upstream of the weir. In this project, the influence range of the flushing operation will be studied in order to develop an optimal flushing operation plan.

Overall experiment procedure is as follows.

4.5.1 Sediment Deposition before Testing

Sediment deposit level at upstream of dam has been determined to be EL. 230.75 masl (corresponding to EL.510.0 masl of the Prototype) considering HEC-RAS data (Analysis result of long-term riverbed change) and the sedimentation simulation at the upstream of weir by HR Wallingford, and the slope gradient of 1:500. After sedimentation on the riverbed, a constant flow of 0.02609 m^3/s (corresponding to 264 m^3/s of the Prototype) that is mean monthly flow of August) will be running until the equilibrium riverbed profile is formed. Figure 4.5 shows the process of the riverbed sedimentation.

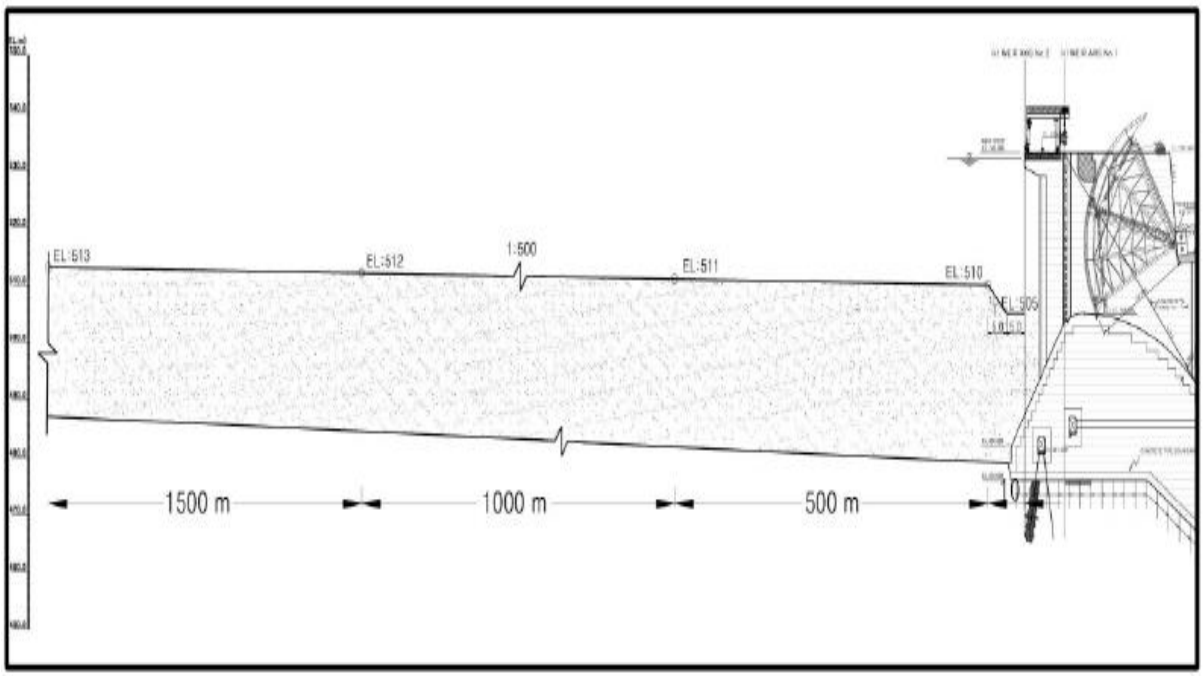

Figure 4.5: Riverbed sediment profile

4.5.2 Test Condition

Flushing operation is generally carried out over 1 to 2 days during flood season. In this experiment, however, three cases have been simulated in line with developing an optimal flushing operation plan. General and boundary conditions for the test are described in Table 4.3 below.

Table 4.3: Test conditions for flushing

Case	Inflow (m^3/s)	Operation time	Initial Reservoir Elevations (El. m)	Gate Opening
1	264 (Mean monthly discharge of August)*	1 Day*	532.0*	6 gates (full open)
2	264 (Mean monthly discharge of August)*	1 Day*	532.0*	3 gates (full open)
3	264 (Mean monthly discharge of August)*	1 Day*	532.0*	1 gate (full open)

*(corresponding to the Prototype scale)

4.5.2.1 Case 1

Before performing this test, delta was moulded according to the figure given in the test programme as shown in Figure 4.6 Model was filled up to elevation of El.532 m very slowly without disturbing the sediment bed moulded in the model.

Figure 4.6: Sediment bed as moulded on model before test run

After maintaining the reservoir level at El.532 m with the incoming constant flow of 264 (m^3/s) all the 6 gates of spillway were fully opened within 5 minutes on model. The reservoir level was lowered down gradually and attained El.505.6 m at the end when the sediment flushing has started. The model was continuously run for duration of 4 hours on model equal to 1-day prototype. The bed configuration upstream and downstream of spillway was observed on next day after drying the model. The model observations observed visually and recorded photographically are shown in Figure 4.7.

Figure 4.7: Case 1 – Flushing-affected area and sedimentation

4.5.2.2 Case 2

After maintaining the reservoir level at El.532 m with the incoming constant flow of 264 (m^3/s) the right 3 gates of spillway were fully opened within 5 minutes on model. The reservoir level was lowered down gradually and attained El.506.7 m at the end when the sediment flushing has started. The model was continuously run for a duration of 4 hours on model equal to 1 day on prototype. The bed configuration upstream and downstream of spillway was observed on next day after drying the model. The model observations observed visually and recorded photographically are shown in Figure 4.8.

Figure 4.8: Case 2 – Flushing-affected area and sedimentation

4.5.2.3 Case 3

After maintaining the reservoir level at El.532 m with the incoming constant flow of 264 (m^3/s) the most left 1 gate of spillway was gradually opened to the fullest. The reservoir level was lowered down gradually and attained El.506.9 m at the end when the sediment flushing has started. The model was continuously run for a duration of 4 hours on model equal to 1 day on prototype. The bed configuration upstream and downstream of spillway was observed on next day after drying the model. The model observations observed visually and recorded photographically are shown in Figure 4.9.

Figure 4.9: Case 3 – Flushing-affected area and sedimentation

4.6 TESTING FOR FLUSHING WITH PHYSICAL MODEL

4.6.1 Calibration and Sensitivity Analysis

Numerical simulations based on Gulpur HPP physical model were carried out using HEC-RAS 5.0. The calibration of calculated data by the HEC-RAS model was performed using the measured bed change values in physical model. The Manning's Roughness Coefficient of flow resistance and constant of Ackers and White were considered as the calibration parameters of the model in such a way that the change of this coefficient makes the error between calculated bed level changes and those of observed bed levels minimum. Sensitivity analysis was performed to get guidance for calibration of the model. Selection of appropriate Manning's Roughness Coefficient and sediment parameters was checked through such sensitivity analysis. HEC-RAS Model was run for Manning's values of 0.02, 0.025, 0.03, 0.035, 0.04, 0.045 and 0.05. The results of change in bed elevation are shown in Figure 4.10. Change in bed elevation for 'n' values of 0.03, 0.035 and 0.04 closely resembles with that of observed in the physical model. Statistical analysis were performed (Table 4.4) which supports value of 0.03. Similar analysis was made with various sediment transport functions and the results of change in bed elevation are shown in Figure 4.11. Statistical analysis was performed to further refinement of the results. The trending line in Figure 4.11 as well as statistical analysis (Table 4.4) supports to select Acker's and White sediment Transport function.

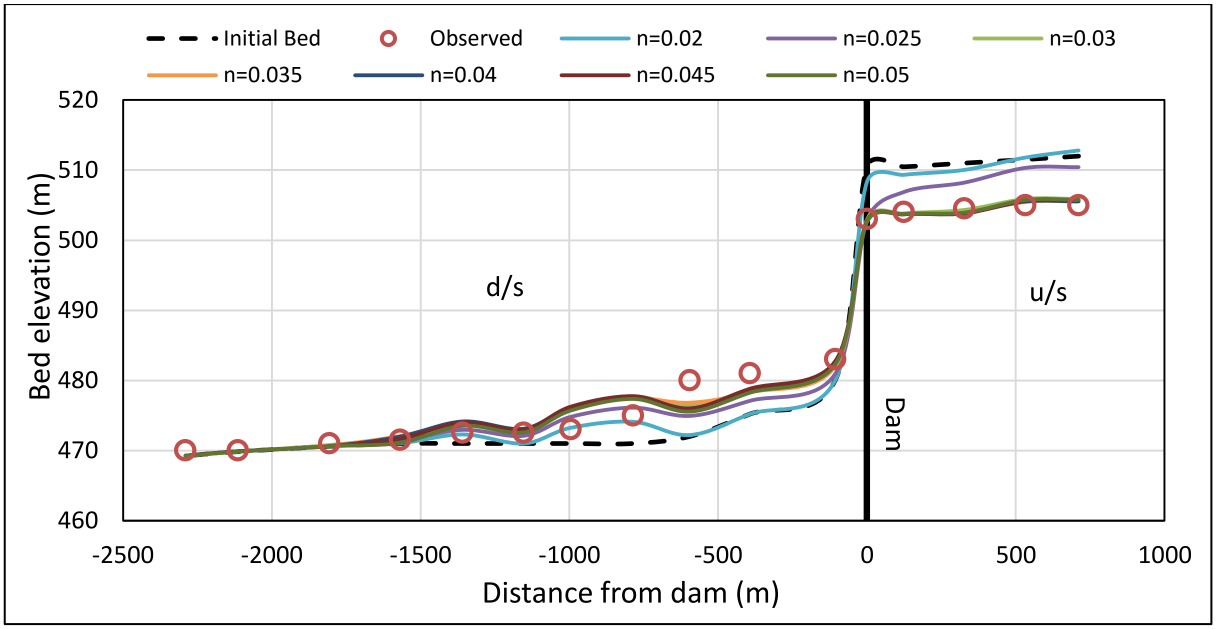

Figure 4.10: Change in river bed level for different values of "n"

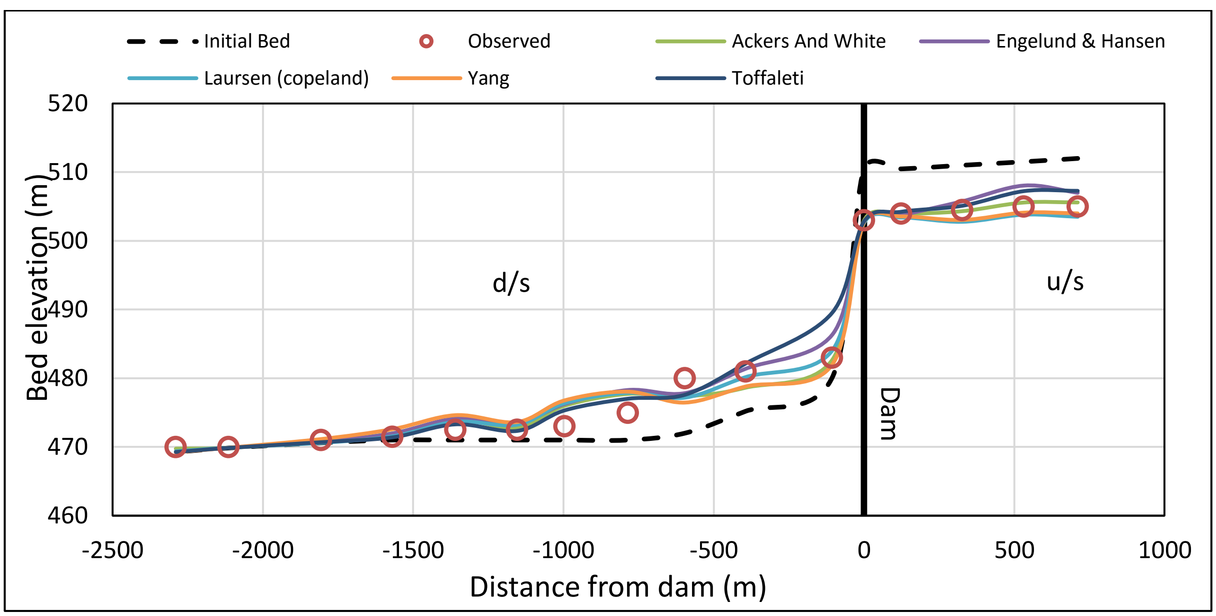

Figure 4.11: Change in river bed level for different sediment transport functions

4.6.2 Validation of Numerical Model

Two tests were performed for validation of the model by considering manning co-efficient of 0.03 and Acker's and White as sediment transport function. First test was performed with 3 gates of spillway fully opened within 5 minute and the second test was performed with1 gate fully opened within 5 minute. Results are shown in Figure 4.12 & 4.13 respectively.

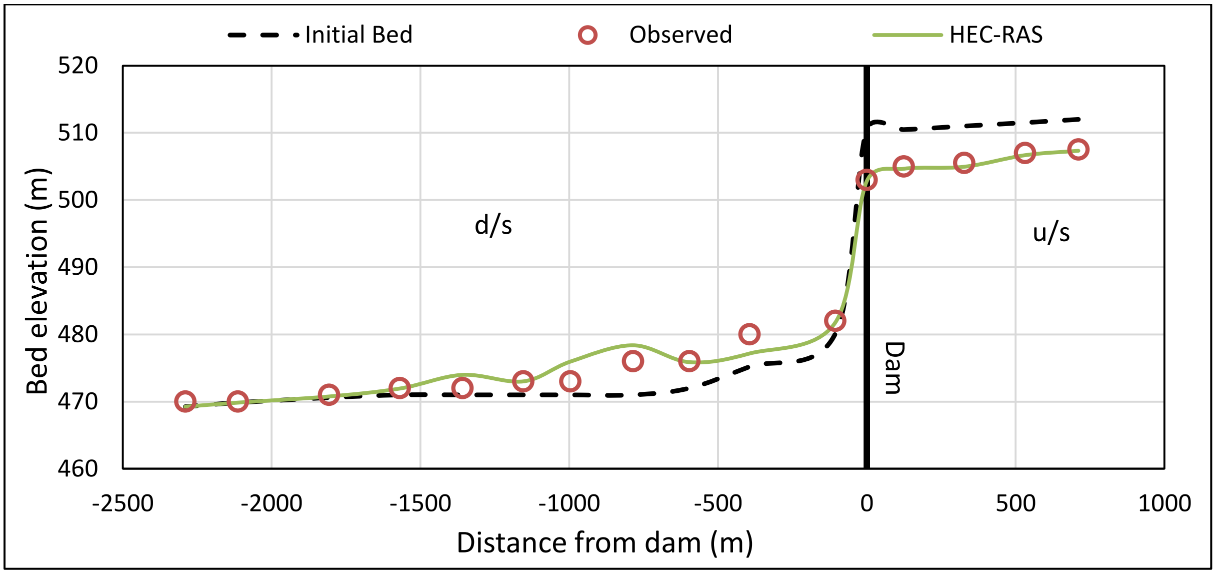

Figure 4.12: Change in river bed level under 3 gate opening condition

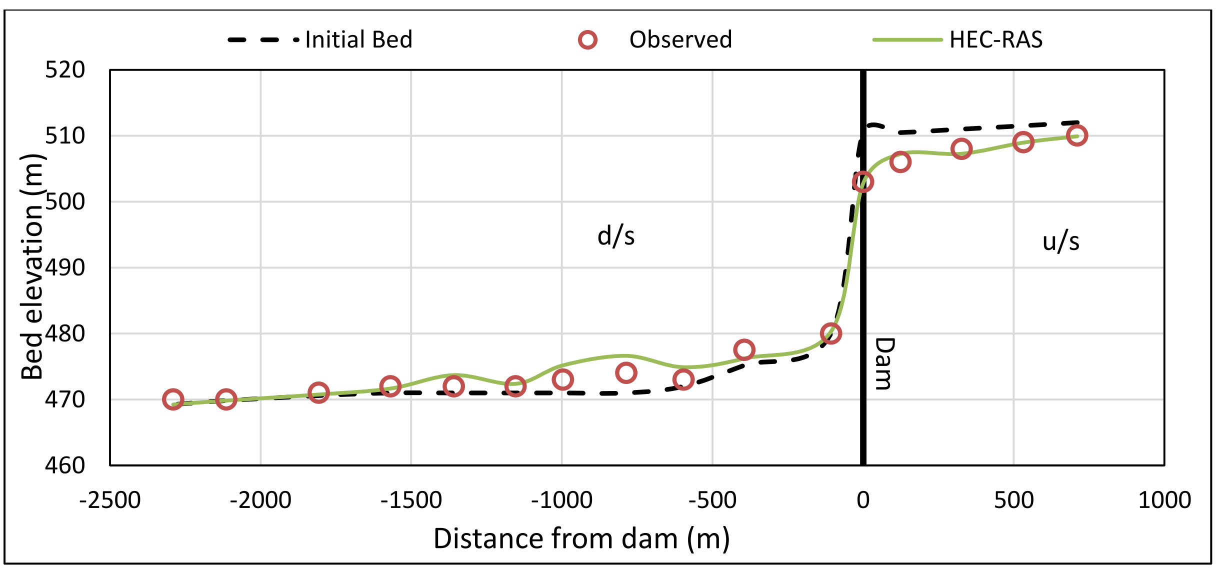

Figure 4.13: Change in river bed level under 1 gate opening condition

Bed configuration upstream and downstream of spillway was observed. The observed reverbed sediment profile was compared with that of observed one with HEC-RAS. The observed and HEC-RAS generated reverbed sediment profiles closely resembles with each other.

4.6.3 Statistical Measure for Comparing the Simulation

The goodness-of-fit measures employed to evaluate different simulations representing different choices of parameter are Nash-Sutcliffe coefficient and Root Mean Square Error (RMSE). The parameters (Mutreja 1986) as given below at equation (4.4) and (4.5) were used to test the model accuracy:

$$\text{Nash-Sutcliff Efficiency} = 1 - \frac{\sum_{i=1}^{n}(O_i - C_i)^2}{\sum_{i=1}^{n}(O_i - \overline{O}_i)^2} \qquad \rightarrow (4.4)$$

$$\text{Root Mean Square Error} = \frac{1}{N}\sum_{i=1}^{n}(C_i - O_i)^2 \quad \rightarrow (4.5)$$

$$\overline{O_\iota} = \frac{1}{N}\sum (Yb_o)_j \qquad \rightarrow (4.6)$$

Where;

N = Length of record

(C_i) = Computed bed level change

(O_i) = Observed bed level change

$\overline{(O_\iota)}$= Mean observed bed level change

Table 4.4: Statistical analysis

<table>
<tr><th colspan="8">Calibration</th></tr>
<tr><td colspan="8">Selection of Manning Co-efficient “n”</td></tr>
<tr><td>‘n’</td><td>0.02</td><td>0.025</td><td>0.03</td><td>0.035</td><td>0.04</td><td>0.045</td><td>0.05</td></tr>
<tr><td>Nash-Sutcliffe Efficiency</td><td>-250.75</td><td>-102.42</td><td>0.02</td><td>-0.44</td><td>-0.54</td><td>-0.96</td><td>-1.17</td></tr>
<tr><td>Root Mean Square Error</td><td>6.24</td><td>4.00</td><td>0.39</td><td>0.47</td><td>0.49</td><td>0.55</td><td>0.58</td></tr>
<tr><td colspan="8"></td></tr>
<tr><td colspan="8">Selection of Sediment Transport Function</td></tr>
<tr><td></td><td>Ackers And White</td><td>Engelund & Hansen</td><td>Laursen (copeland)</td><td>Yang</td><td colspan="3">Toffaleti</td></tr>
<tr><td>Nash-Sutcliffe Efficiency</td><td>0.02</td><td>-18.41</td><td>-8.26</td><td>-4.42</td><td colspan="3">-12.82</td></tr>
<tr><td>Root Mean Square Error</td><td>0.39</td><td>1.73</td><td>1.20</td><td>0.92</td><td colspan="3">1.46</td></tr>
<tr><td colspan="8"></td></tr>
<tr><th colspan="8">Validation</th></tr>
<tr><th colspan="8">Flushing Test</th></tr>
<tr><td></td><td colspan="3">3 gate opening condition</td><td colspan="4">1 gate opening condition</td></tr>
<tr><td>Nash-Sutcliffe Efficiency</td><td colspan="3">0.90</td><td colspan="4">0.89</td></tr>
<tr><td>Root Mean Square Error</td><td colspan="3">0.35</td><td colspan="4">0.65</td></tr>
</table>

Chapter 5

SIMULATION AND ANALYSIS

5.1 GENERAL

Numerical simulation is very much dependent on the quality and detail of field data used in building the model. There are number of parameters like roughness coefficient, active layer thickness, time step, space step, bends and meanders etc., which affect the results of simulations. These factors need careful calibration for obtaining reasonable results.

5.2 DATA FOR BUILDUP OF THE MODEL

The data used for building the model can be classified as topographical, hydrological, hydraulic and that related to sediments. Firstly plot all the geometric data available and apply the boundary condition of the stream, bed materials, available flows also incorporate dissolved particles coming in to the reservoir w.r.t seasonal discharges will try to establish the duration of flushing and calculate maximum possible efficiency without disturbing the structure.

5.3 DEFINITION OF INPUT PARAMETERS

5.3.1 Geometry of the Reservoir

The geometry of the reservoir is defined by cross-sections perpendicular to the main flow. Forty-one cross-sections upstream the dam were provided by ISAN as a contour drawing. They cover the whole area of the reservoir that extends up to elevation 532 m (the normal operating level).

The geometry file for HEC-RAS contains information on cross-sections, hydraulic structures, river banks and other physical attributes of the river.

The model was built for 11.46 km river length with 41 cross-sections and the dam site is situated at halfway between Section No. 10 & 11 (Figure 5.1). River reach under study is a mountain stream with vegetation in channel, banks are steep with trees and brush on banks submerged. Choosing the value roughness coefficient say Manning’s n is a challenging task for such a river reach Chow (1959). The calibrated Manning’s n value 0.03 were used for simulation.

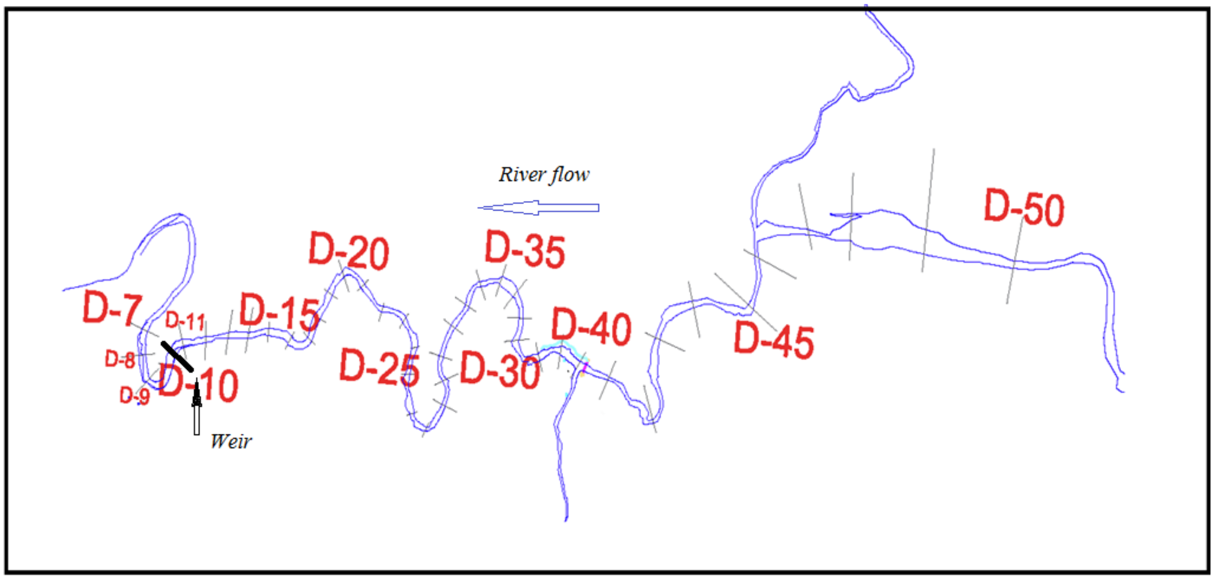

Figure 5.1: Schematic diagram showing the cross sections locations used for the delta modelling

5.3.2 Boundary Condition

After assigning the geometric properties to the model the next step are to assign the boundary conditions to the model boundary conditions can be used in HEC-RAS 5.0. Model are of two types one is flow boundary conditions and the other is sediment boundary condition.

5.3.2.1 Flow boundary condition

Quasi-Unsteady flow boundary conditions were used for sediment transport modelling. The flow data which was synthesized from the previous historical data was entered in the Quasi-unsteady flow data editor which comprised of two boundary condition

a) Upstream boundary condition
b) Downstream boundary condition

a) Upstream boundary condition

Monthly inflow hydrograph for 20 years was assigned as the upstream boundary condition.

Two series of water discharges corresponding to 20 years have been considered. The first series (Series 1 in Figure 5.2) extends from 1964 to 1984 and is representative of the average inflows into the reservoir. The second series starts in 1988 and includes some of the highest discharges in the record, as the 1992 event.

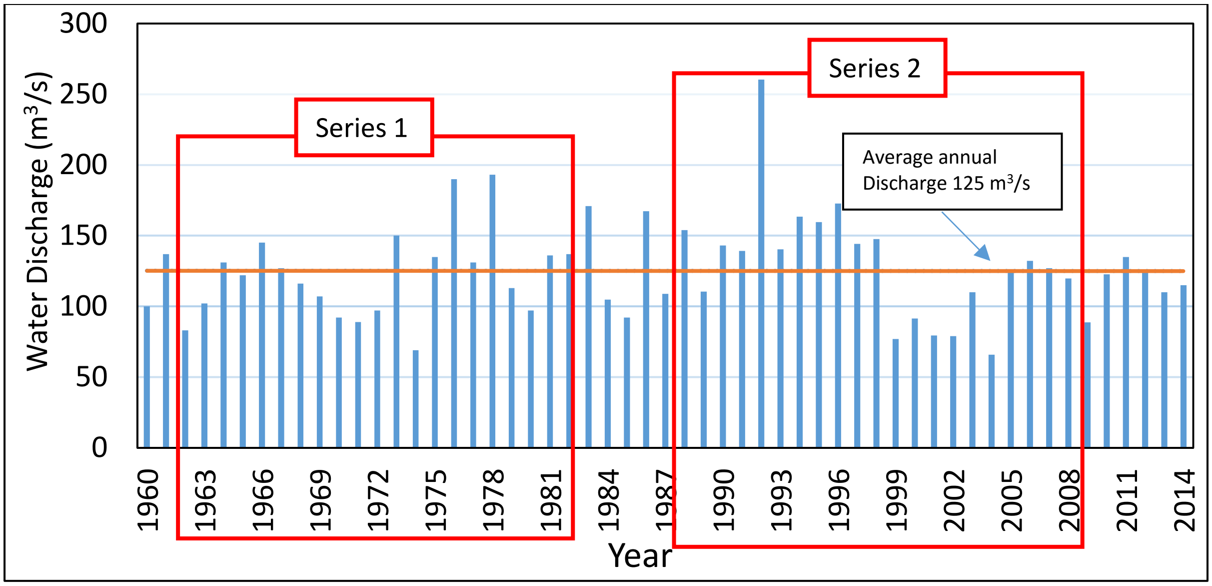

Figure 5.2: Water discharge series used for numerical simulations

b) Downstream boundary condition

HEC-RAS includes three option for setting quasi-unsteady downstream boundary condition: Stage Time Series, Rating Curve, or Normal Depth.

Stage time series were used for deposition of sediments in the reservoir and normal depth was used for flushing as a downstream boundary condition.

For the modelling of deposition pattern in the reservoir, it is considered that the water level at the reservoir is kept constant at 532 m, the Normal Operation Level.

5.3.2.2 Sediment boundary condition

Sediment boundary can be assigned to the model are Sediment Bed Gradation, Maximum Scour Depth, Sediment Transport Function, Rating Curve and Sediment Load Series. These are all discuss with respect to our model.

a) Initial Condition and Transport Parameters
b) Boundary Condition

a) Initial condition and transport function

The initial condition and transport parameter specified for HEC-RAS for Gulpur Reservoir are as following at each cross section.

i. Transport function

A sediment transport function can be selected from the drop down box near the top of the form. For the present study, Acker's and White transport function was selected on the basis of sensitivity analysis of transport functions to model the delta of Gulpur HPP.

ii. Sorting method

The Exner 5 method was used in this study. It is a three layer active bed model that includes the capability of forming a coarse surface layer that will limit erosion of deeper material thereby simulating bed armouring.

iii. Fall velocity

Several method are available for computing fall velocity in HEC-RAS. In present study, Van Rijn fall (1993) velocity was used.

iv. Maximum depth

In the HEC-RAS sediment frame work, a sediment control volume is associated with each cross-section. The control volume starts midway from the next cross section upstream and ends midway to the next cross section downstream. The maximum erodible depth used for model was 10m.

v. Bed gradation

HEC-RAS first requires the creation of bed material gradation curve. Then the bed gradation curve can be associated with the appropriate range of cross section using pick and drag functionalities. In this study, sediment bed gradation is the gradation of material accumulated at the bed of the reservoir which is to be flushed out we will take the gradation as shown in Figure 5.3.

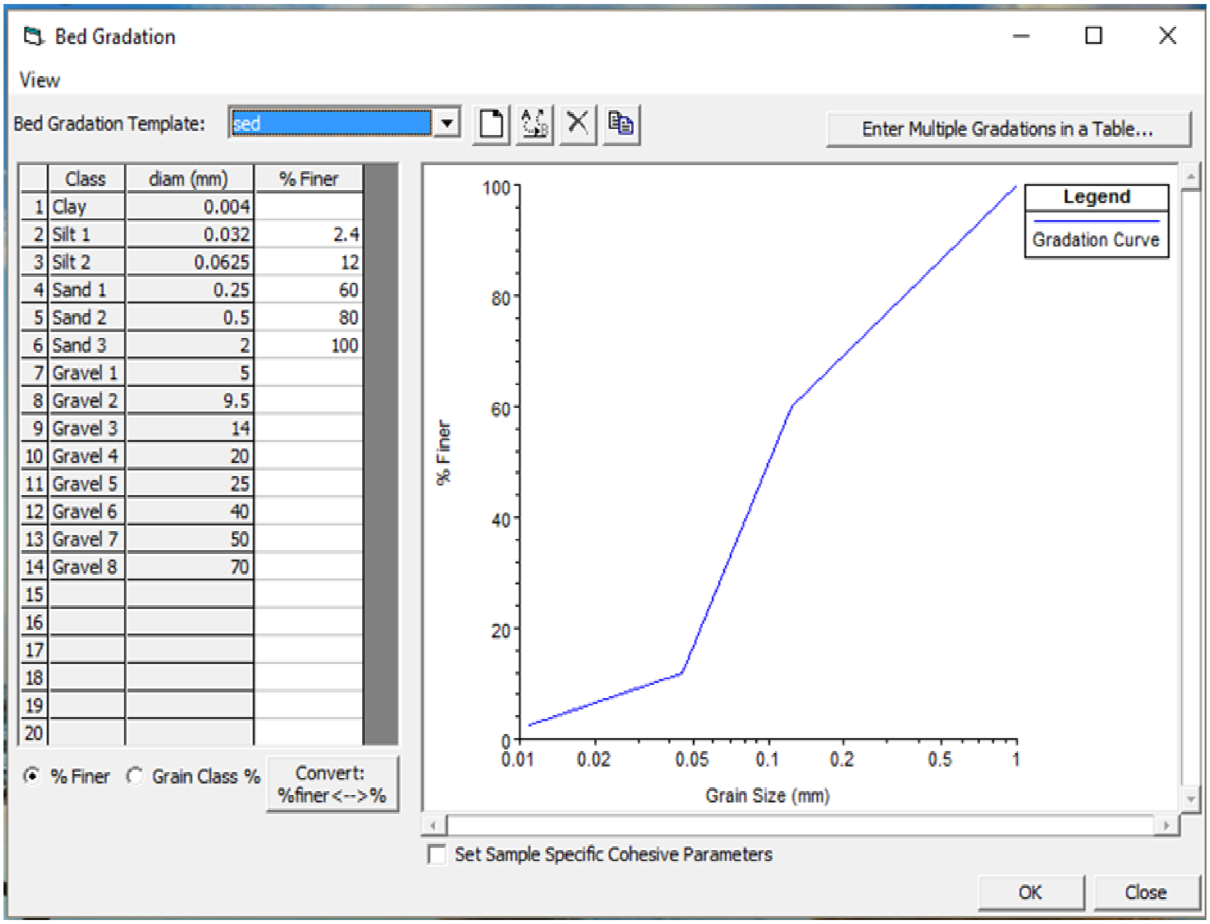

Figure 5.3: Rating curve

b) Sediment boundary condition

i. Rating curve

Rating curve is basically used to incorporate the effect of Suspended Sediments coming with inflow to the reservoir these sediment are expressed as tons/day quantity with respect to flow in Cumecs also we have to specify the particles size w.r.t. share they are contributing to overall volume coming with inflow.

a. Sediment inflow

The sediment inflow is estimated based on water discharge and sediment rating curves. The representative sizes and assumed proportions for each fraction are presented in Table 5.1.

TABLE 5.1: Sizes and %age of fractions in sediment inflow (Assumed as base case for modelling)

Class no	Description	Representative diameter	%age in all sediment
1	Clay	0.003 mm	23%
2	Silt 1	0.01 mm	25%
3	Silt 2	0.035 mm	25%
4	Sand 1	0.1 mm	12.27%
5	Sand 2	0.3 mm	12.27%
6	Sand 3	1 mm	2.01%
7	Gravel 1	3 mm	0.45%

It is assumed that the sizes larger than a medium gravel will be removed upstream from the reservoir by sediment mining (except for one of variation runs described in Table 5.2). It is advised to remove the large sediment in a controlled manner using check dams.

To assess the sensitivity of the results to the sediment values, three additional scenarios have been considered:

Variation 1: It considers an increase in the total sediment inflow. The increase is about 40%. The comparison of sediment inflow between the base case scenario and this variation is shown in the Figure 5.4.

Variation 2: This variation considers the observed suspended flow only, i.e. without the addition of the estimated 20% of bedload. The proportion of fine sediment (silt and clay) that is flowing into the reservoir is thus higher than in base case (89%). The total percentage of coarse material is also as observed in suspended load only (11 %). The comparison of sediment inflow between the base case scenario and this variation is shown in the Figure 5.4. The proportions of material used are presented in Table 5.2.

Variation 3: It considers two more fractions of coarse material (Gravel 2 with d=10 mm and Gravel 3 with d=30 mm) but maintaining the total amount of sediment. The added 20% of unmeasured bedload is split equally between sand (10%) and gravel (10%). The ratios between fractions are obtained by using transport capacity assumption as described earlier in this chapter for sand.

Variation 4: It considers that the proportion of sand is double (54%) while maintaining the same total amount of sediment. The percentage of clay is thus reduced to 14.4% and silt to 31.6%.

Figure 5.4 shows annual sediment inflow for the base case and Variations 1 and 2 and two flow conditions: average flow Series 1 and high flow Series 2 (as shown in Figure 5.2). The total sediment inflow for Variations 3 and 4 is the same as for Base case sediment inflow as these variations only differ in sediment composition and not total amount.

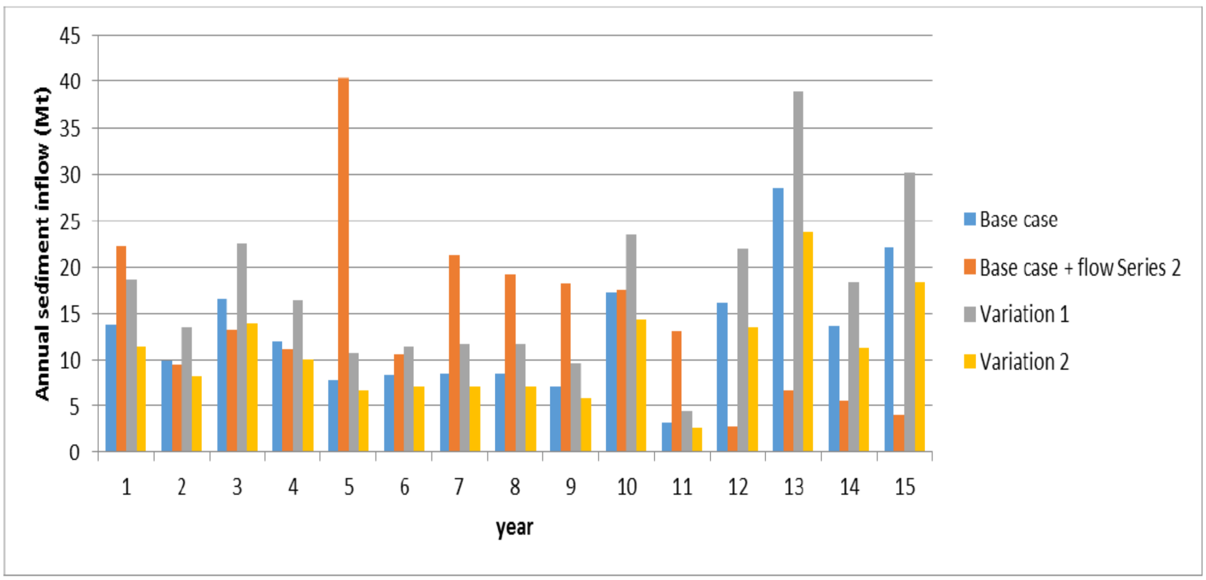

Figure 5.4: Sediment inflow estimated for the base case scenario and variations 1 and 2

Table 5.2: Sizes and percentages of fractions in sediment inflow assumed for the variation scenarios

Class No.	Description	Representative diameter (mm)	Percentage in all sediment (%)			
			Base case & variation 1	Variation 2	Variation 3	Variation 4
1	Clay	0.003	23	27	23	14.4
2	Silt 1	0.01	25	31	25.6	15.8
3	Silt 2	0.035	25	31	25.6	15.8
4	Sand 1	0.1	12.27	5	8.1	24.54
5	Sand 2	0.3	12.27	5	8.0	24.54
6	Sand 3	1	2.01	0.82	1.2	4.02
7	Gravel 1	3	0.45	0.18	4.4	0.9
8	Gravel 2	10	-	-	2.9	-
9	Gravel 3	30	-	-	1.2	-

5.4 DEPOSITION PATTERN IN THE RESERVOIR WITH NO SEDIMENT MANAGEMENT OPTIONS

Deposition in the reservoir is the result of the difference between the inflow of the sediment into the reservoir and its releasing capacity for sediment (the percentage of the sediment that passes through reservoir), which is the opposite of the trapping efficiency (the percentage of the sediment that is trapped in the reservoir). The amount of annual deposition is therefore dependent on:

- Concentration of sediment in the water and proportions of material. As this is one of the most relevant parameters, its influence on the results is explored considering different sediment inflows.

- Water inflow, which is variable within a year and between the years. Water inflows for the future cannot be predicted but must be assumed. This is best done based on the past observed discharges. Two sets of years, Series 1 and 2 are used (Figure 5.2).

- Configuration of the reservoir, which determines the trapping efficiency of a given sediment size. The configuration is defined by its geometry. A roughness coefficient needs also to be prescribed and its influence on the results is explored considering two different values.

5.4.1 Modelling Scenarios

The table 6.3 describes the scenarios considered to model the deposition pattern in the reservoir, considering no sediment management options in the reservoir. As explained when discussing sediment inflow inputs, it is considered that check dams or other mining activities will trap the coarser sediment (larger than a medium gravel) upstream the reservoir. The rating curve described in previous chapter is considered for suspended material.

Table 5.3: Description of tests

Test	Description	Discharge	Sediment inflow	Roughness coefficient
1	Base case	Series 1	Base case	0.03
2	High water flow series	Series 2	Base case	0.03
3	Increased bed load by 40%	Series 1	Variation 1	0.03
4	Suspended inflow only	Series 1	Variation 2	0.03
5	No check dam	Series 1	Variation 3	0.03
6	Increased roughness	Series 1	Base case	0.05
7	Influence of increased sand proportion in a base case scenario	Series 1	Variation 4	0.03
8	Influence of increased sand proportion in a base case scenario	Series 2	Variation 4	0.03

5.4.2 Results of Simulations

5.4.2.1 Base case scenario

The results of the base case scenario (Test 1, Figure 5.5) show that sedimentation will increase gradually in the reservoir that will be completely filled with sediment in approximately 15 years. The deposited sediment at the dam reaches the level of intake (519 m) in 14 years. The numerical results show that approximately after 4 years, the levels at the topset of the delta (from chainage 5,000 and more) are similar to the operating level.

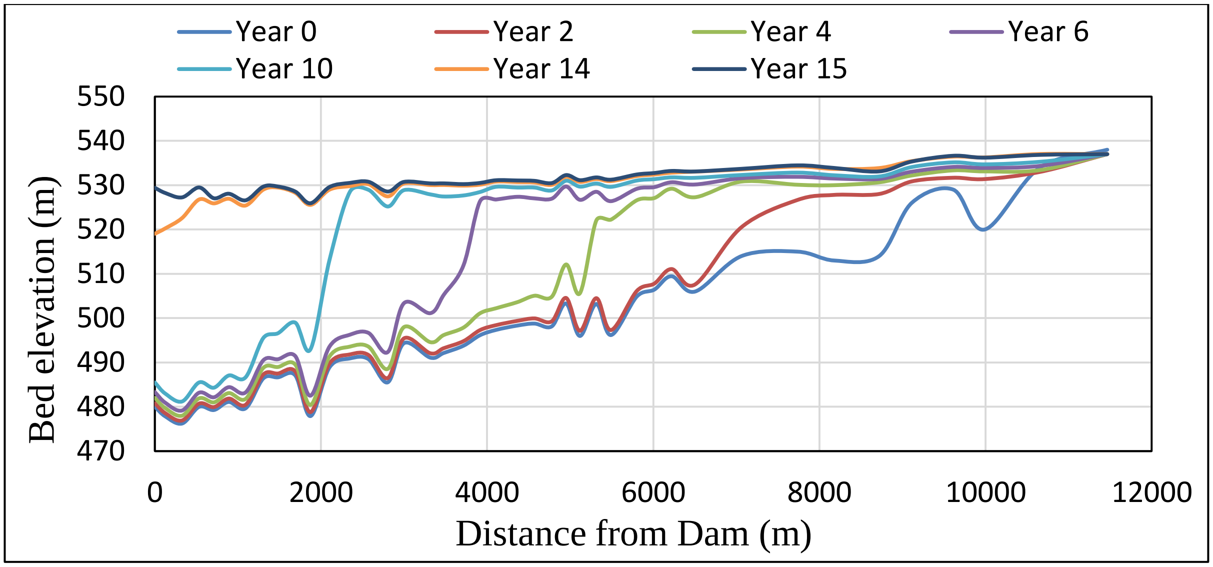

Figure 5.5: Longitudinal Bed Profiles along the Reservoir for the Base Case Scenario (Test 1)

The trapping efficiency of coarser sand fractions (Sand 2 and 3 and Gravel 1) is almost 100% over the first 15 years of operation under inflow conditions assumed for simulation test 1. Thus sediment of diameter 0.3 mm or greater is likely to be trapped in the reservoir. The trapping efficiency of the finer material reduces with time. For the finest fractions (clay and silt 1) it is of the order of 80% in the first years and reduces to less than 10% after 12 years.

5.4.2.2 Influence of high flood series

Test 2 considers a flow series with the 1992 large flood event (Figure 5.6). The impact of the 1992 flood event (4 years from the origin of the flow series) is clear when comparing the bed profiles with the previous ones as more deposition is shown (after the event in year 1993). The filling of the reservoir is slightly quicker than previously and after 10 years, the bed levels near the dam reaches 529.5 m against the base case scenario where this level was achieved in 15 years.

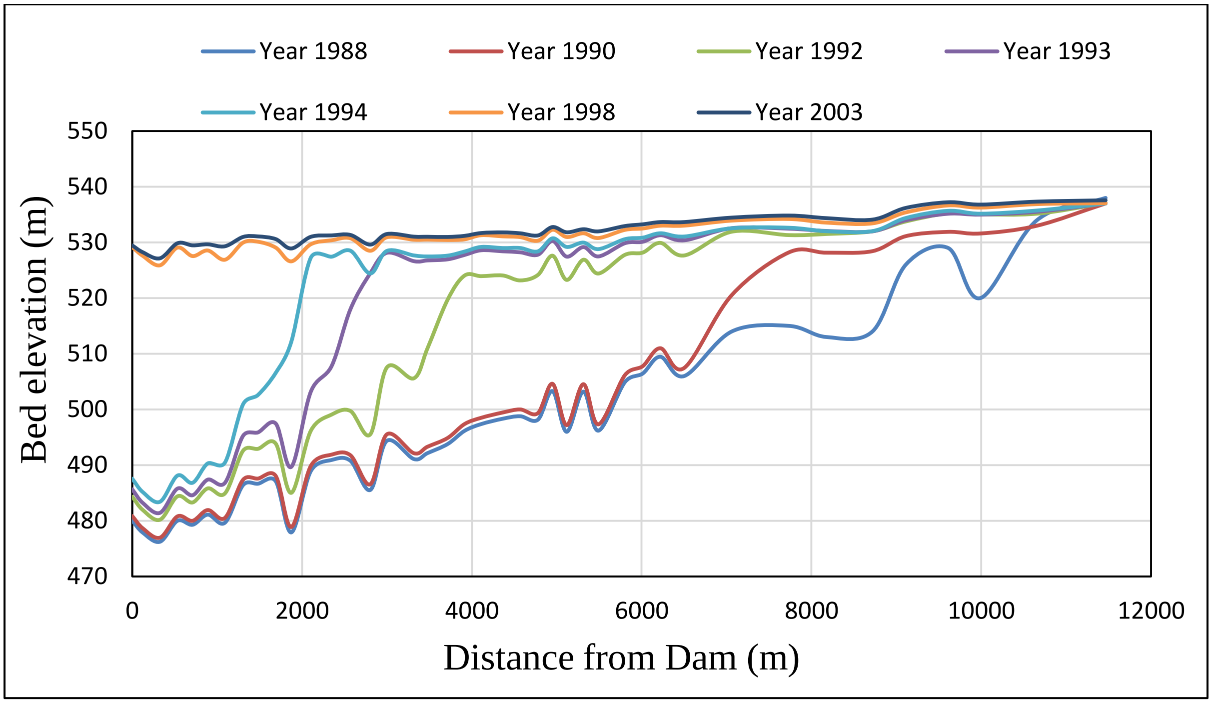

Figure 5.6: Longitudinal bed profiles along the reservoir for the flow series with a large event (Test 2)

The results are not unexpected given the high variation in the flows of Poonch River. It takes about 10 years for the sediment deposits at dam to reach the intake level and for sand particles greater than 0.1 mm and 0.2 mm to start entering the power system in appreciable quantities. However Test 2 shows that this can happen earlier if hydrologic conditions are different. The future water and sediment inflows are uncertain and this fact has to be taken into account in planning.

5.4.2.3 Increased bed load by 40%

The sediment inflow is based on rating curve that relates sediment flow to water discharge. However there is a considerable scatter in the plot of discharge - sediment flow. Thus there is a high degree of uncertainty on sediment amounts into the reservoir (Figure 5.4). How this uncertainty translates into reservoir sedimentation results is investigated with Test 3 (Figure 5.7), where the amount of inflowing sediment is increased by about 40% to reflect the amounts of annual sediment inflow predicted by an earlier study (ISAN).

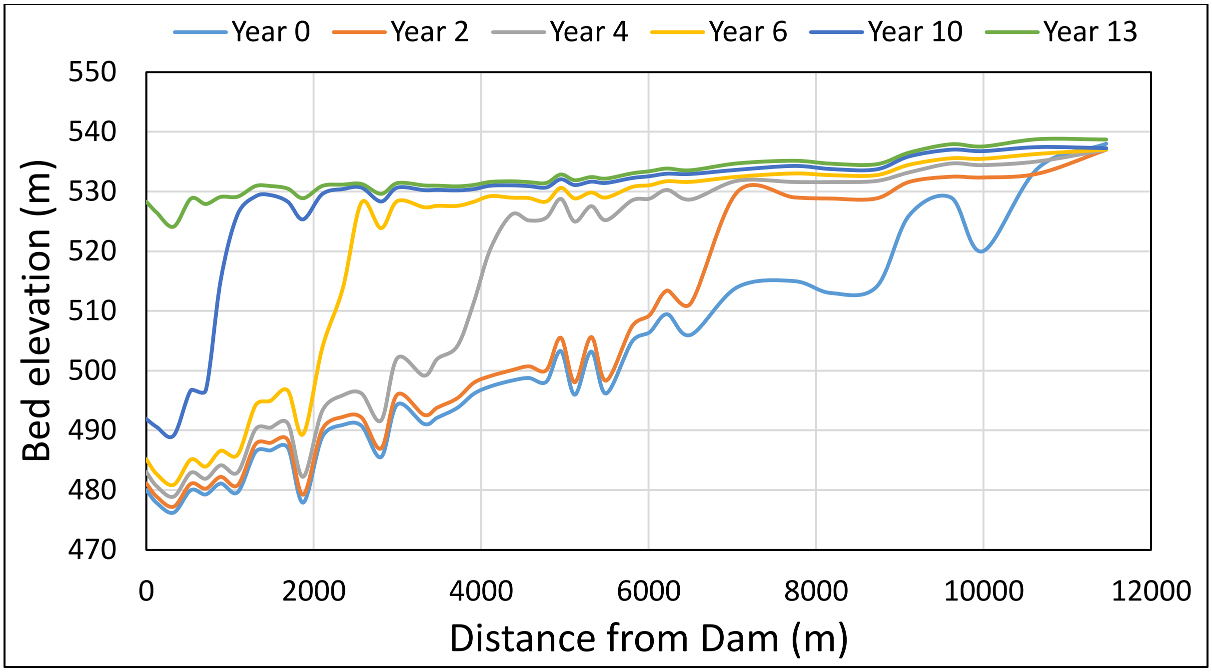

Figure 5.7: Longitudinal bed profile along the reservoir (Test 3)

The numerical results shows that approximately after 4 years in test 3, the level at the topset of the delta reaches to 4000 m from the dam face while in base case in test 1 more than 5000 m. Due to higher sediment inflow, the model predicts that in test 3 the sediment reaches the normal operating levels three years earlier (in 12 years) than in test 1 where it was in year 14.

5.4.2.4 Suspended inflow only

The sediment inflows were estimated based on a set of observed suspended sediment flows. To account for unmeasured bedload, this was increased in the Base Case scenario (Test 1) by 20%. This additional amount was distributed to coarse fractions, which move close to bed in higher concentration than they do further away from bed. However, to take into account the uncertainties related to this assumption, Test 4 was run to evaluate the sedimentation dynamics in the reservoir if no increase due to bed load was considered.

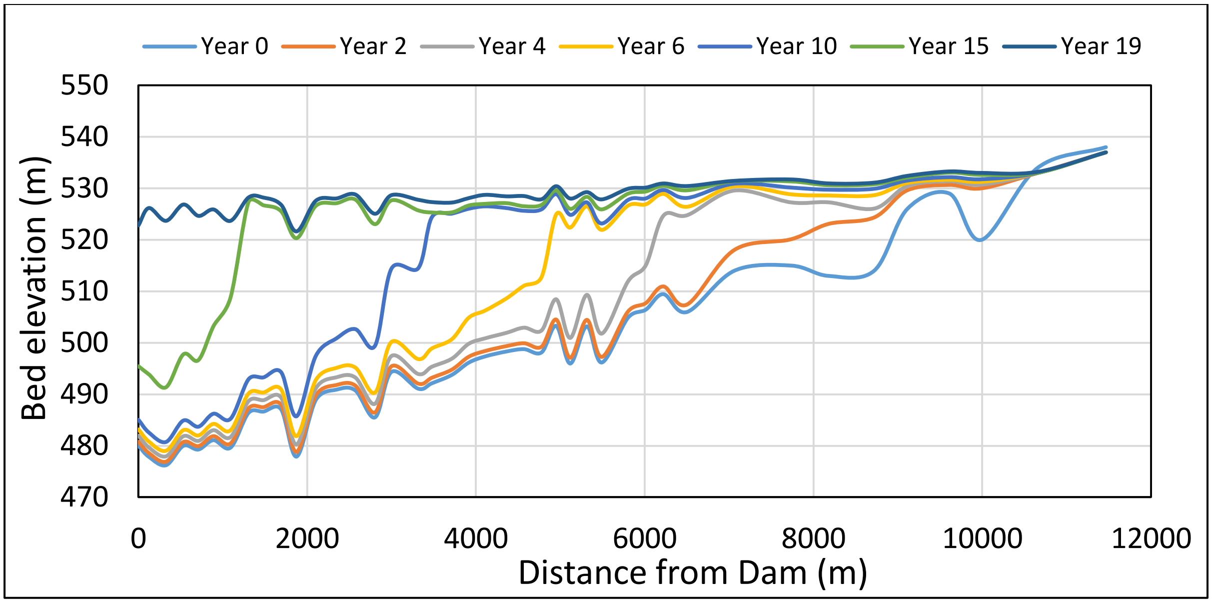

Figure 5.8: Longitudinal bed profile along the reservoir (Test 4)

It can be seen in the Figure 5.8 that with lower amount of sediment and in particular sand, the rising of bed levels near dam as well the inflow of sediment into power intake is slowed by 4-5 years in Test 4 compared to base case Test 1.

5.4.2.5 No check dam

Test 5 (Figure 5.9) analyses the case when gravel is not extracted by sediment mining upstream of the reservoir and is allowed to reach the reservoir. Gravel sizes up to d=30 mm were taken into account for this simulation. The evolution of the longitudinal bed profile is show below. The model results show that the bed level in the upstream end of the reservoir rises significantly. This can cause severe problems with flooding upstream of the reservoir. The results indicate that gravel extraction, preferably in a controlled manner, with one or more check dams upstream of the reservoir, is necessary to prevent the bed level rising in the upper end of the reservoir.

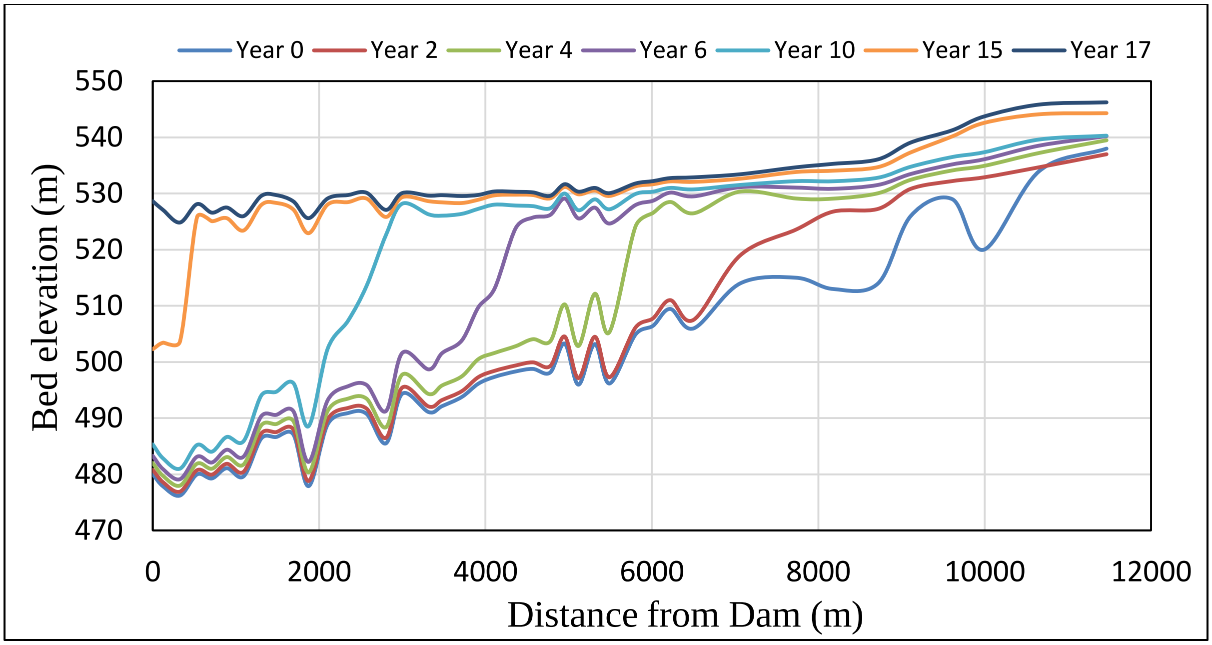

Figure 5.9: Longitudinal bed profiles along the Reservoir (Test 5)

5.4.2.6 Influence of roughness coefficient

The influence of the roughness coefficient used in the simulations is minor (shown in Figure 5.10) with respect to bed level at dam and small with respect to inflow of sediment d>0.1 mm into the power system.

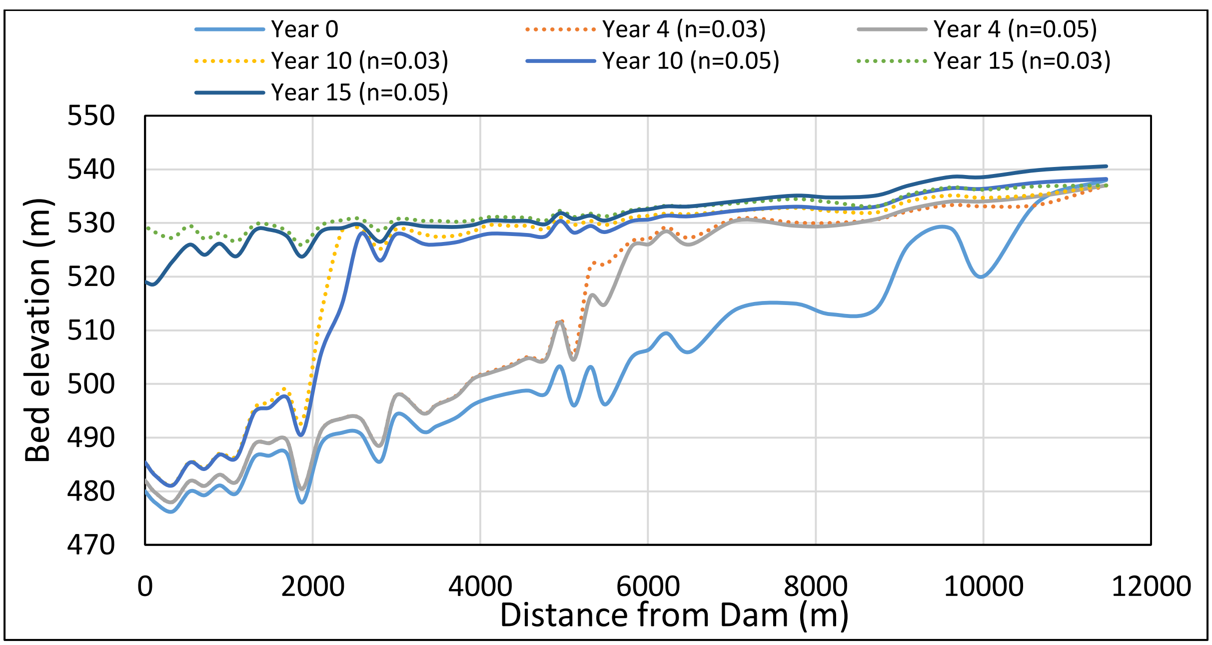

Figure 5.10: Comparison of longitudinal profile for Test 1 and Test 6

5.4.2.7 Influence of increased sand proportion in a base case scenario

There are considerable uncertainties related not only to the estimation of unmeasured bed load but also to the suspended load and its composition.

As the amount of sand is higher, the proportion of the trapped sediment can be expected to be higher as well as and the deposition pattern to be different. Sand is less easily transported and thus deposits earlier. This can be seen from the model results presented in Figure 5.11 for Test 1 and Test 7 cases. A steeper delta slop can be observed which indicates a potential problem at the upstream end of the reservoir if flushing does not start when required to keep the overall level of deposits in the reservoir low. In terms of general progress of the rate of sediment deposits towards the dam, however, there is no significant difference. This can be explained by the fact that while trapping efficiency of sand is higher (Test 7), the deposition density is also higher than that of silt and clay (by about 50%) and therefore, the deposited material occupies less volume.

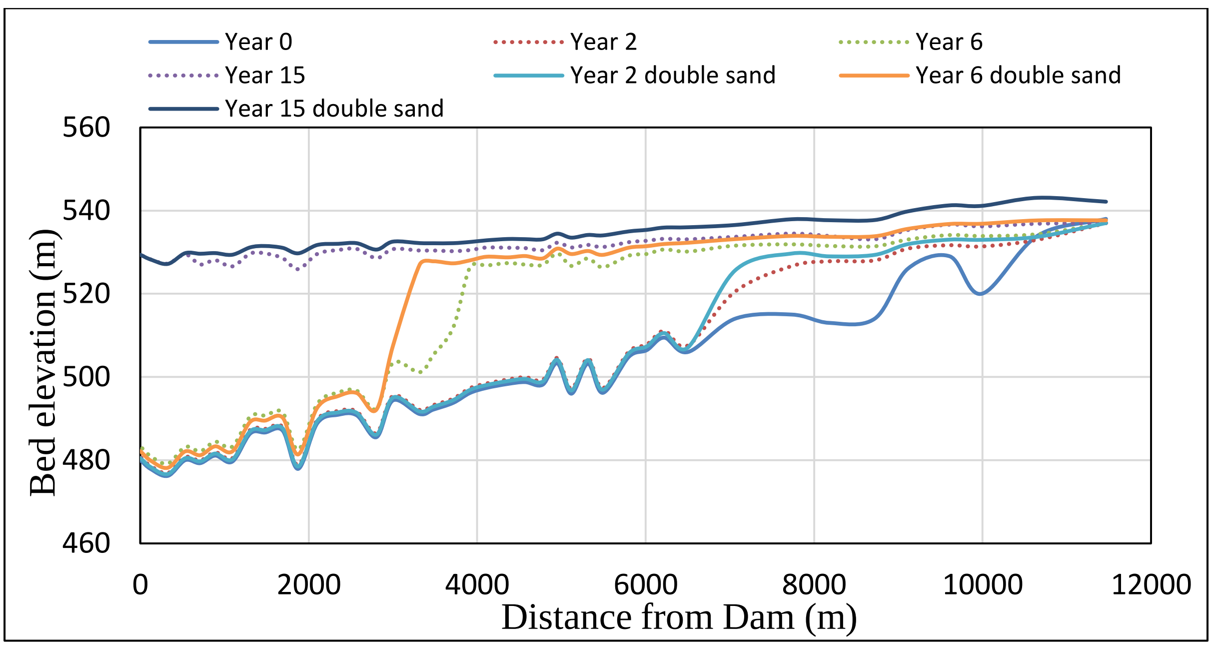

Figure 5.11: Comparison of longitudinal profiles for Test 1 and Test 7

A comparison between the high flow series with base case sand amount (Test 2) and high flow series with double sand amount (Test 8) is presented in Figure 5.12. In terms of the deposition pattern, it still holds that the model predicts a steeper delta slope in the double sand case.

However, in this case there is some difference in the rate of progress after the extremely high flows (which occur in year 5 of high flow series, i.e. Series in Figure 5.2.

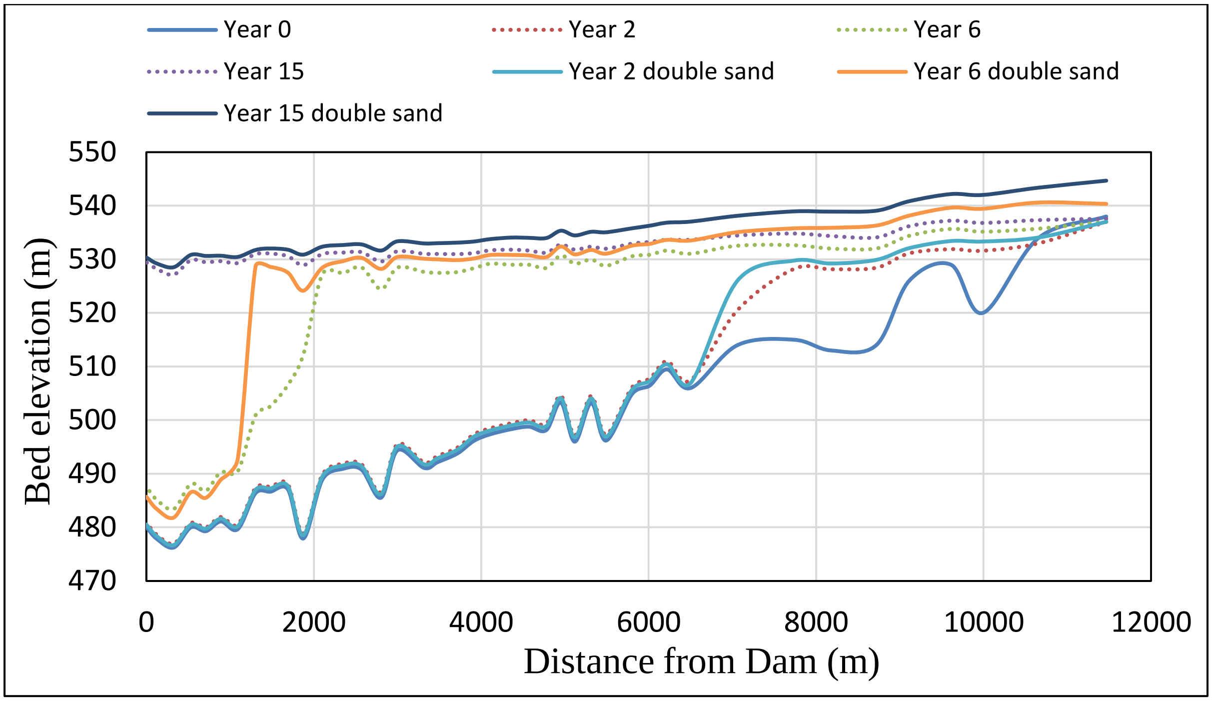

Figure 5.12: Comparison of longitudinal profiles for Test 2 and Test 8

A comparison of bed levels at dam with the level of intakes for all performed test is shown in Figure 5.13. This sensitivity analysis shows that depending on hydrological conditions (Test 2) and sediment inflow and composition (Tests 3 and 4) the level of sediment deposits can reach the intake level several years earlier or later compared to what is predicted in the base case (Test 1) where it is 14 years. An impact of a single event (Test 2 in year 4) is evident. These variations have to be taken into account for planning purposes. Sensitivity of this parameter to Manning coefficient (Test 6) is small.

In Test 3 scenario, with increased amount of sediments particularly sand, the model predicts that the sediment reaches the intake level in about 9 years. Similarly, in Test 4, with lower sediment inflow particularly sand, it takes 18 years and in Test 5 &6, it reaches in about 16 years.

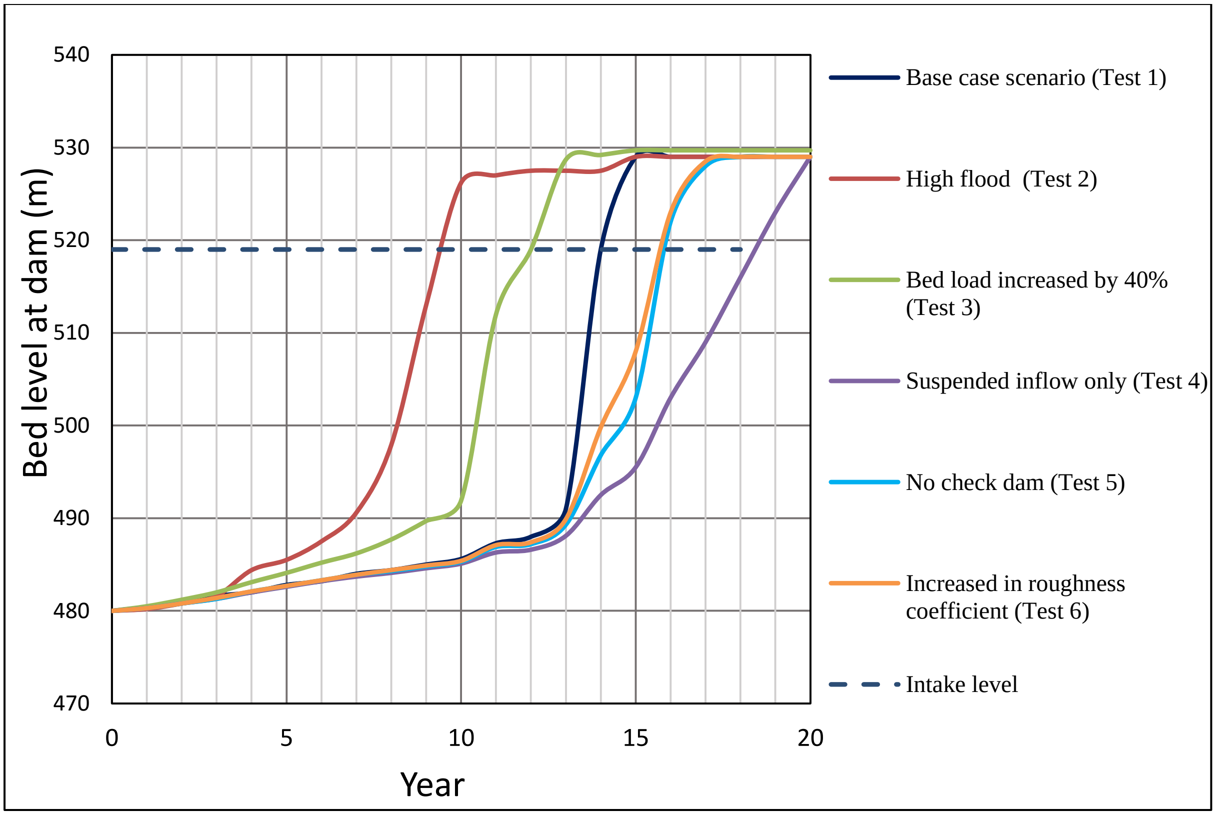

Figure 5.13: A comparison of bed levels at dam with the level of intakes (Tests 1-6)

5.5 MODELLING SEDIMENT FLUSHING THROUGH GULPUR RESERVOIR

The Gulpur HPP has relatively smaller reservoir with high sediment deposition rate, therefore for its technical viability, a successful flushing of the deposited sediment is required.

5.5.1 Definition of Modelling Scenarios

The main goals of flushing of the sediment from the reservoir are:

- Prevent sediment from accumulating at the position of the power intakes (close to the dam) to the level where it approaches the invert level of the intakes (519 m);
- Prevent sediment from entering the power intakes, in particular larger fractions (in this study taken as d>0.1 mm as recommended in e.g. Morris and Fan, 1998). This can be done by removing sediment deposits from reservoir and thus prepare space for newly

arriving sediment to deposit – restoring trapping capacity of the reservoir as explained in Chapter 2.

Depending on the size of reservoir, water and sediment inflow characteristics and operation rules, flushing can be performed in different ways. The proposed Gulpur reservoir is of run-of-river type and after the water level drawdown required for flushing it can refill within a day or so. Water discharges are irregular but with two annual flood seasons, a smaller one in March/April with an average daily peak around 500 m^3/sand a bigger one in July/August with an average daily peak of around 1300 m^3/s. These are the periods when most sediment is expected to arrive and deposit in the reservoir.

5.5.1.1 Suitable time for flushing

Given these parameters, the most suitable time for flushing would be on the falling limb of the second flood period. In that period, water discharges will be high enough to ensure an efficient flushing operation that would be able to remove recently deposited sediments (giving no time to possible consolidation). From Figure 5.14 it can be seen that a suitable timing is before calendar day 222 (10th of August).

Due to high concentration of sediment, it has been proposed that the power plant will not operate when water discharges are higher than 1,000 m^3/s. Sluicing and flushing can also be performed during these periods getting additional benefit of available water discharge being high. In the observed period of 55 years (between 1960 and 2014) there were 60 events where water discharge was higher than 1,000 m^3/s. There duration was seldom longer than one day.

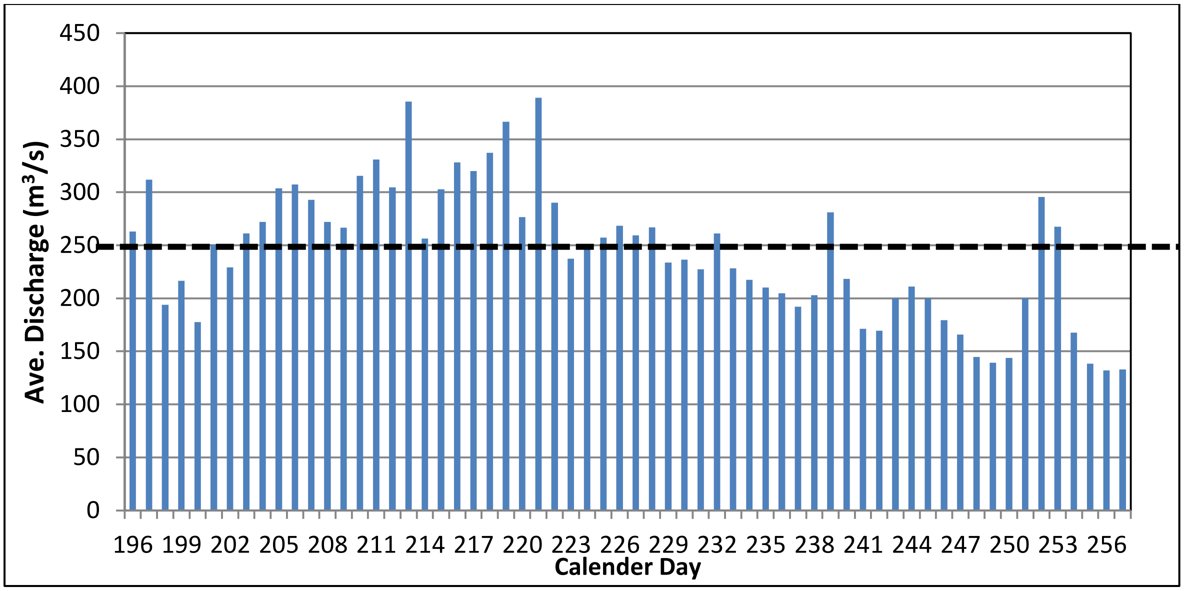

Figure 5.14: Average discharges during second flood period between 15th of July (196) and 15th of September (257)

5.5.1.2 Definition of main parameters of flushing operation

To investigate the flushing performance of the reservoir, flushing scenario is modelled using HEC-RAS 5.0 Model. For modelling, The geometry of the model for the simulation of flushing scenarios was the same as that for deposition except that four additional cross sections were added downstream of the dam and these were modified as obtained after 7 year delta modelling to simulate potential deposition in this area. It was assumed that the first flushing becomes essential after 7 years and further after every one year sediment deposition in the reservoir.

For flushing operation modelling, a quasi-unsteady file was prepared in the HEC-RAS. The constant daily flushing discharges of 250, 500, 800, 1000 m^3/s, as boundary condition, were used for the complete flushing duration and resultant durations for yearly sediment flushing were determined. The normal depth (a value of friction slope as 0.0052) was also considered in the mode as a downstream boundary condition.

Bed material gradation curve at the dam site was used as an initial condition. Transport function of Ackers-white (1973) along with Rubey (1933) fall velocity method was used. By using long term historical data, sediment rating curve was prepared for the dam site which was used as sediment boundary condition. Further, fraction of the gravel, sand silt and clay was also allocated.

First flushing was performed after 7 years using complete drawdown flushing approach and it was observed that it takes 5 days to flush the deposited sediments at a flushing discharge of 250 m^3/s. After further sediment deposition every year, it takes 4 days to flush it on 8th year, 4 days for 9th year flushing; thereafter, a dynamic equilibrium condition in the reservoir bed is achieved requiring 4 days flushing every year. Bed profile after 7th, 8th and 9th years flushing is shown in Figure 5.15.

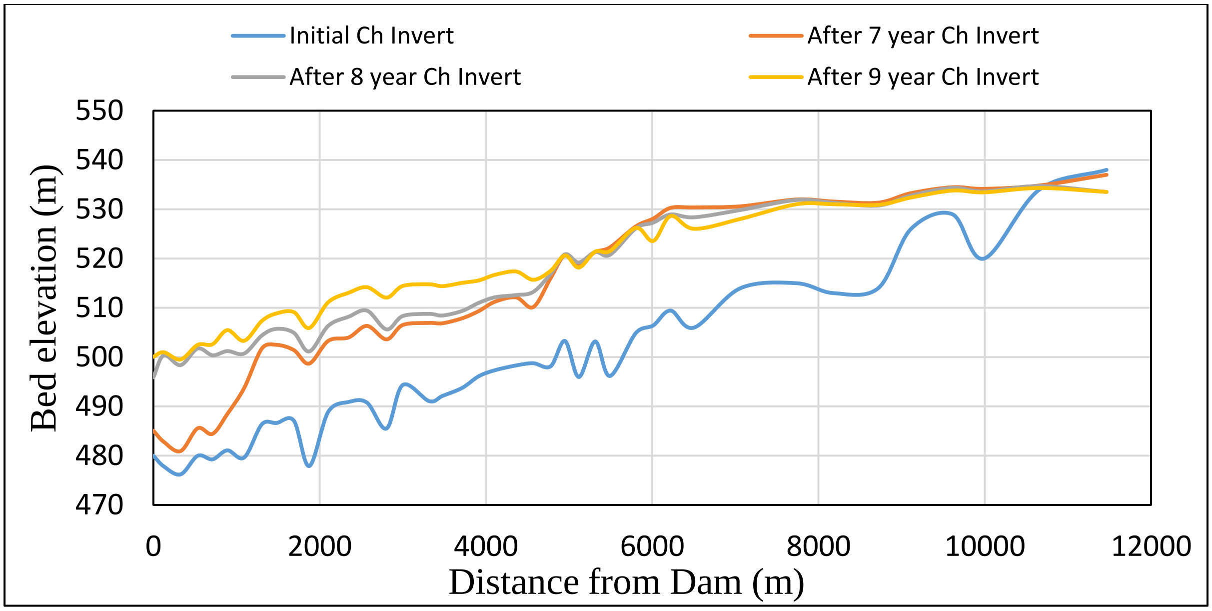

Figure 5.15: Bed profile after 7th, 8th and 9th years flushing

To flush the sediments, the reservoir was emptied, continuously flushed and refilled in a sequential order. A sediments aggradation was attained on the upstream of the dam site by flushing operation because of the reason that sill level of flushing gates is at 503 masl with corresponding original bed level at 480 masl thus creating a dead storage that was silted permanently. Moreover, bed profile degradation was increased on the downstream side of the dam from 1st to 7th year of operation. It is due to the fact that the height of the dam up to sill level of flushing sluices acted as a barrier in the transport of bed load and much of the suspended load, and hence relatively silt free water caused scouring on downstream side. This scouring on the downstream of the dam site was checked once the bed level rose up to the flushing gates sill level. Then instead of degradation on downstream side of the dam, aggradation started after an equilibrium condition was achieved.

Before starting flushing, the reservoir should be emptied at around 9th August every year using average flow; however, this date will be refined based on the actual temporal distribution every year. After emptying, certain days are required for continuous flushing at riverine flow condition, after flushing operation, it should be refilled. To achieve drawdown, the flushing gates will be opened and a riverine flow will be obtained.

After achieving the equilibrium, one year (i.e. 10th year) delta deposited was taken as input to the HEC-RAS flushing model (Figure 5.16). It takes 4 days for emptying, flushing and refilling of the reservoir.

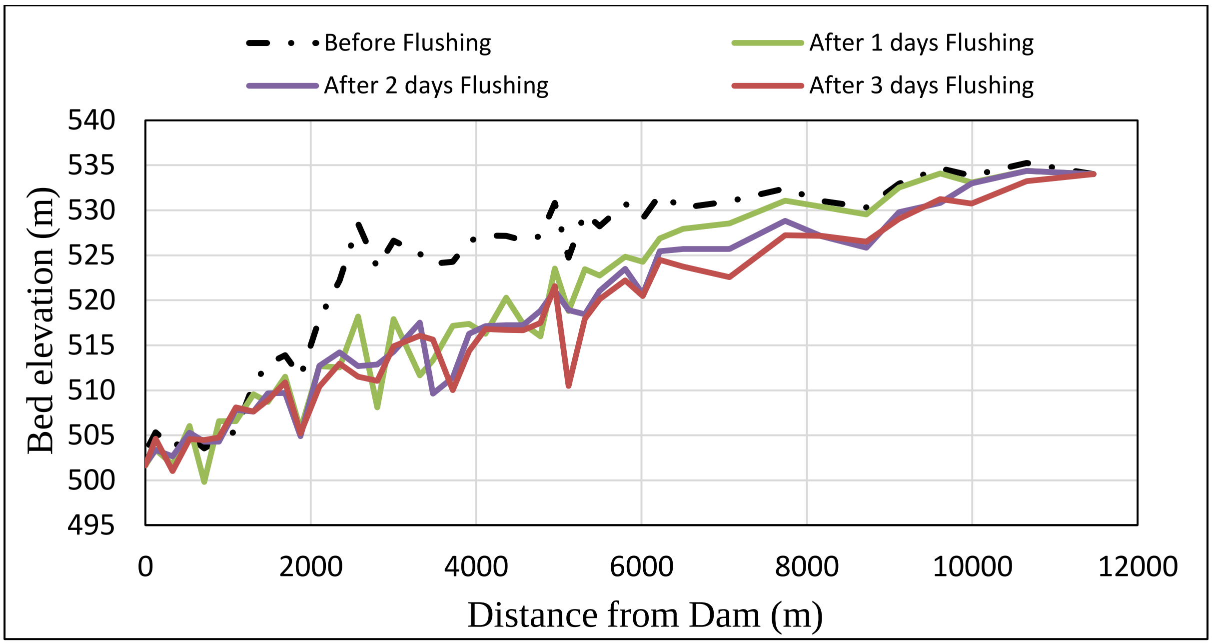

Figure 5.16: Bed profile of 1 year deposition after equilibrium

5.5.2 Strategies for Flushing Sediments through the Reservoir

Different strategies are consider for sediments flushing through reservoir, these are discussed below;

5.5.2.1 Suitable flushing discharge and flushing duration

According to Sayah (2014) and Castillo (2014), a suitable flow for reservoir flushing is that when it has the order of double of the mean annual flow. The Poonch River has mean annual flow of 125 m^3/s (Figure 3.4), hence a discharge of 250 m^3/s would be a recommended one for reservoir flushing. Mean daily flow hydrograph for the Gulpur dam site has been shown in Figure 5.14 along with the flushing discharge of 250 m^3/s constant line. It depicts that 10th of

August is the suitable flushing time when the flows are highest. However, some variation in flushing time may be there every year based on the availability of suitable discharge.

The model was run for different discharges ranging from 125 to 1000 m^3/s and corresponding flushing durations were determined as shown in Figure 5.17. It was observed that the flushing durations required are from 2 to 6 days corresponding to various discharges.

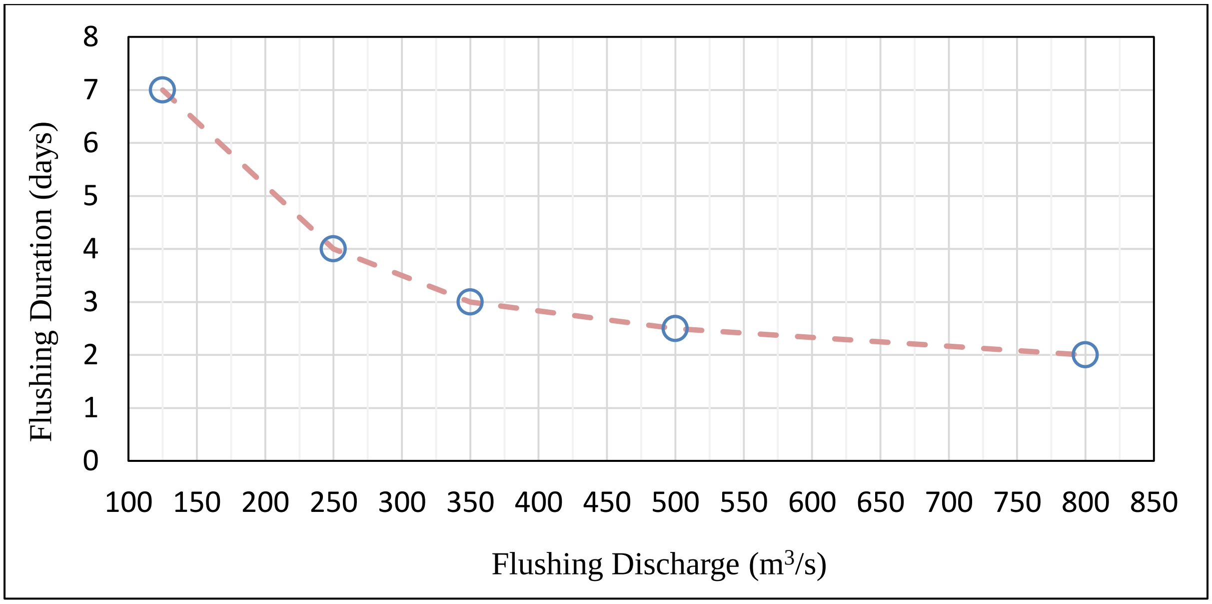

Figure 5.17: Flushing durations for various discharges

5.5.2.2 Time required to empty and refill the reservoir

Before starting flushing, the reservoir should be emptied at around 9th August every year using average flow; however, this date will be refined based on the actual temporal distribution every year. After emptying, certain days are required for continuous flushing at riverine flow condition, after flushing operation, it should be refilled. To achieve drawdown, the flushing gates will be opened and a riverine flow will be obtained.

For the Gulpur dam the emptying time is 5 Hours and the time required to refill the reservoir with 250 m^3/s discharge is shown in Figure 5.18.

Figure 5.19 shows the time required to refill the reservoir with different discharges.

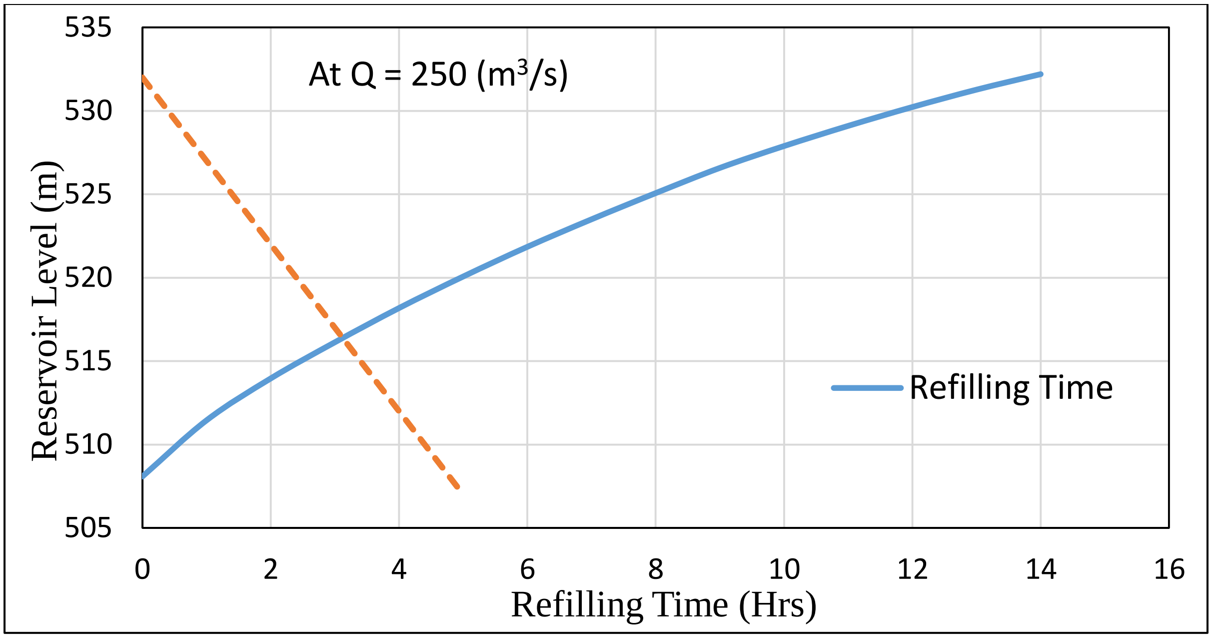

Figure 5.18: Reservoir emptying time and refilling time at 250 m^3/s discharge

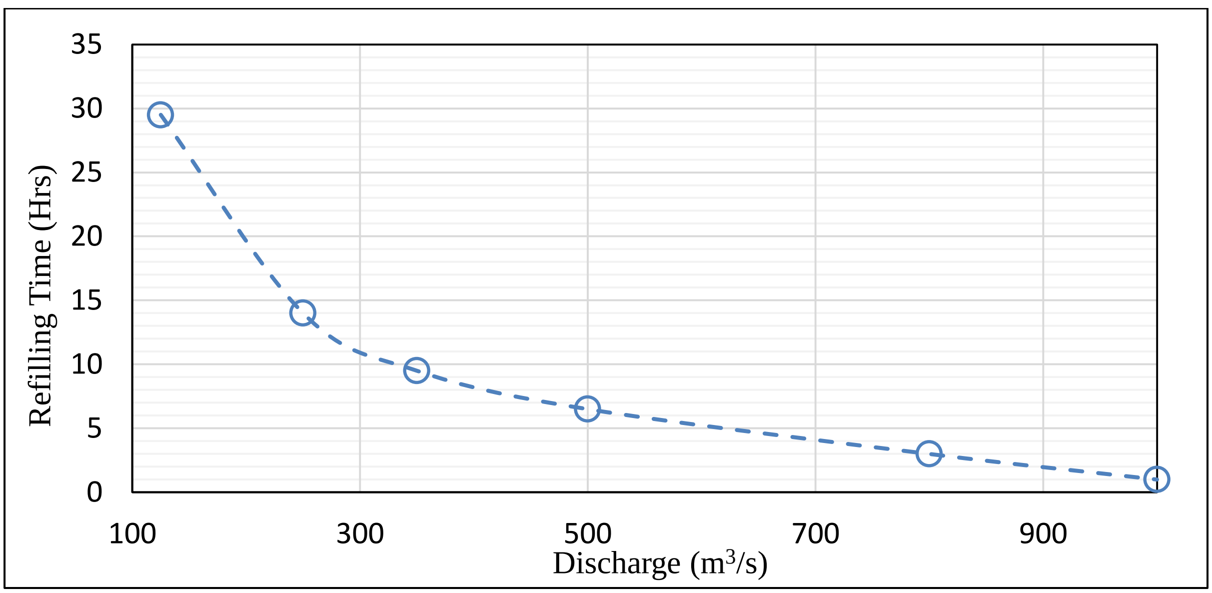

Figure 5.19: Time required refilling the reservoir with different discharges

5.5.2.3 Flushable sediment size

For discharge of 250 m^3/s, the velocities of flows at various sections are given in Figure 5.20. The maximum velocity is attained at river station No. 24, i.e., 3.93 m/s and the avg. velocity is 1.99 m/s (6.5 ft./s) for this critical velocity, maximum sediment size that can be flushed is of 15 mm diameter as determined by the Figure 5.21 (findings of ASCE Task Committee, 1967).

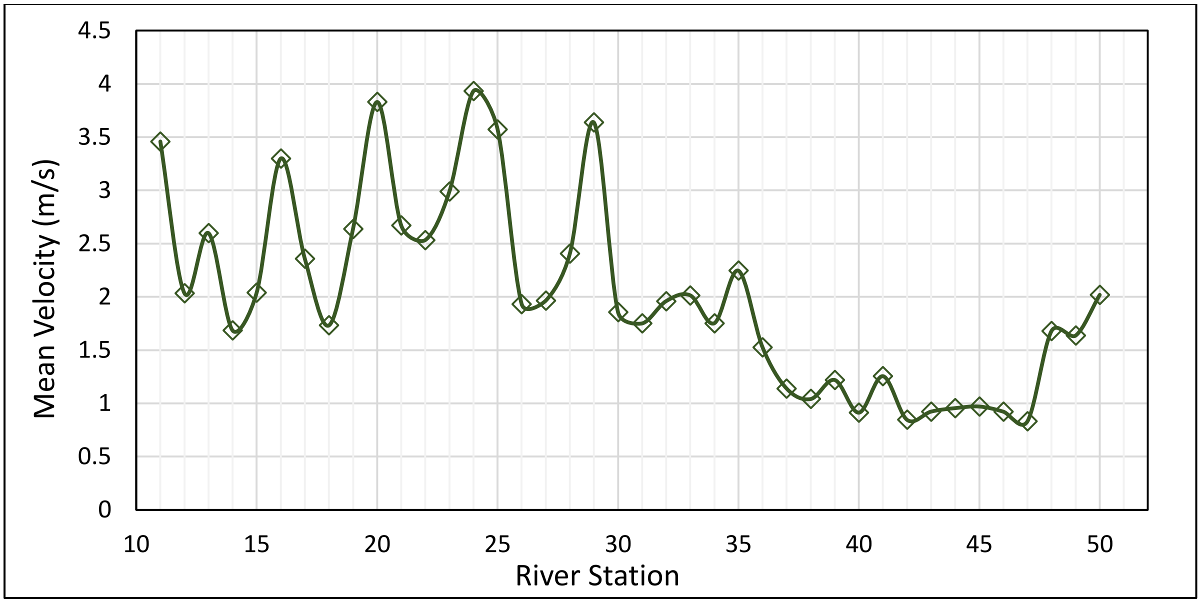

Figure 5.20: Mean velocities at various river stations during annual flushing operation

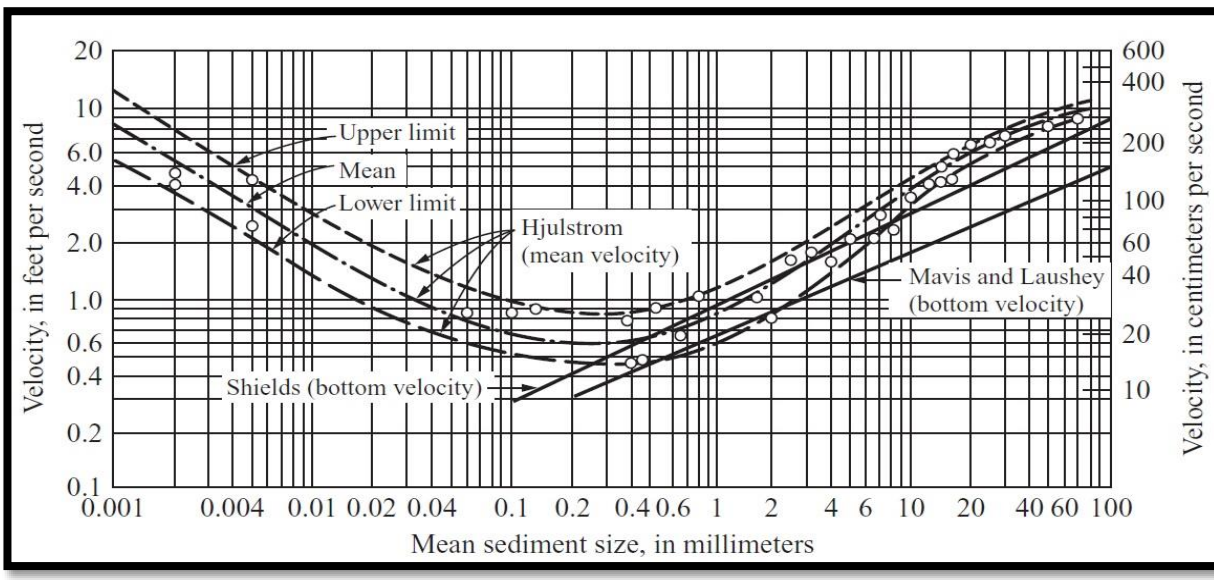

Figure 5.21: Critical water velocities as function of mean grain size (ASCE Task Committee, 1967)

5.5.3 Trapping Efficiency

The amount of sediment deposited within a reservoir depends on the trap efficiency. Trap efficiency is the ratio of amount of sediment deposited to the amount of sediment inflow into the reservoir. Trap efficiency of Gulpur reservoir was 52%, calculated from Brown's curve (1943) as shown in Figure 5.22.

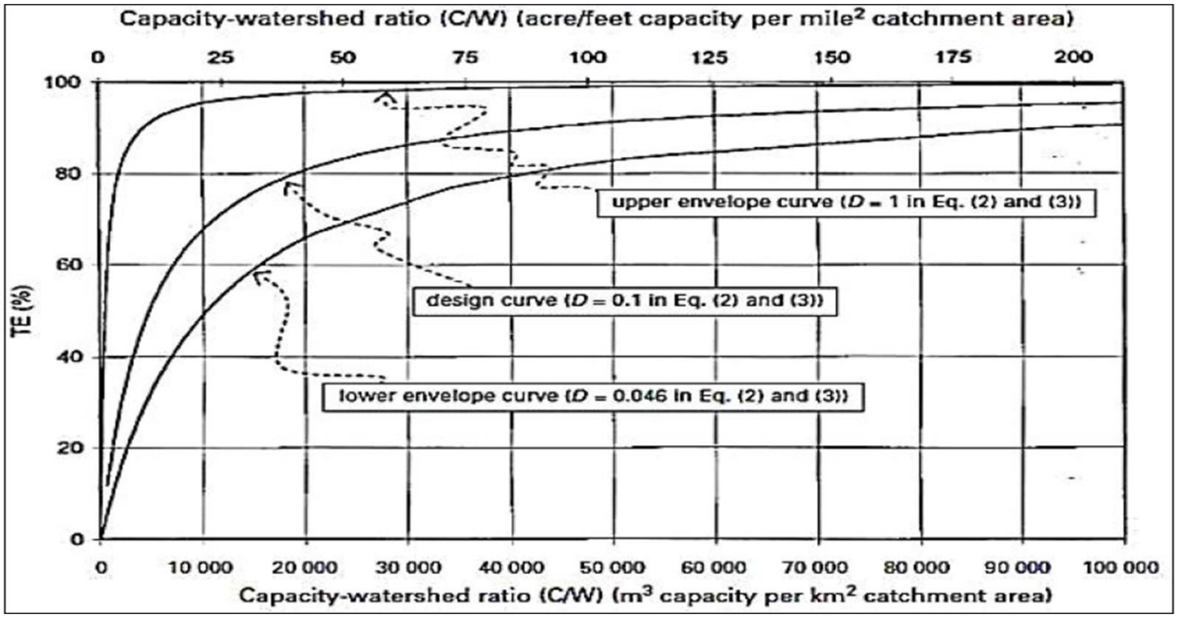

Figure 5.22: Brown’s curve

5.5.4 Flushing Indicator

The value of five flushing indicator have been computed for the Gulpur HPP reservoir and are reported in Table 5.4 along with input data required for these calculations. The parameters are computed for a flushing discharge of 250 m^3/s with a flushing duration of 3 days. The output values (Table 5.5) shows that Gulpur HPP reservoir fulfills all the criteria, hence the flushing efficiency of the reservoir would be around 90 to 100%.

Table 5.4: Input data for flushing analysis

Input Data for Flushing Analysis					
S. No	**Parameter**	**Symbol**	**`Empirical Value**	**Model Value**	**Units**
1	Original Storage Capacity	Co		43.44	Mm^3
2	Reservoir Length	L		9,000	m
3	Elevation of top water level at dam	El_{max}		532.5	m
4	River Bed Level at Dam site	El_{min}		503	m

Input Data for Flushing Analysis					
S. No	**Parameter**	**Symbol**	**Empirical Value**	**Model Value**	**Units**
5	Water surface elevation at dam during flushing	El_f		506	m
6	Representative bottom width	W_{bot}	133.81		m
7	Representative side slope	SS_{res}	1.631		
8	Representative side slope for sediment	SS_s	1.628		
9	Mean annual water inflow	V_{in}		3942	Mm^3
10	Mean annual sediment inflow	M_{in}		10,567,412.59	Tons
11	Tsinghua University factor for sediment type	ψ	300		
12	Sediment load factor (if different China)		3		
13	Sediment Transport Capacity	Q_s	32.21	25.41	Tonnes/s
14	Capacity - Inflow ratio	C_o/V_{in}	0.01		
15	Capacity – Watershed ratio	C_0/w	4985.96		m^3/Km^2
16	Trap Efficiency	TE	52	57	%
17	Flushing Discharge	Q_f		250	Cumecs
18	Flushing Duration	T_f		3	Days

Table 5.5: Output flushing parameter

Output Flushing Parameter				
S. No	**Parameter**	**Symbol**	**Value**	**Criteria**
1	Sediment Balance Ratio	SBR	1.38	> 1*
2	Long Term Capacity Ratio	LTCR	0.845	> 0.5*
3	Drawdown Ratio	DDR	0.898	> 0.7*
4	Flushing Width Ratio	FWR	1.51	> 1*
5	Top Width Ratio	TWR	1.015	(1 to 2)*

* (Atkinson 1996)

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

Gulpur hydropower scheme is proposed on the Poonch River close to the town of Kotli in Pakistan. The discharge regime of the river is highly variable and so is the sediment inflow. The sediment inflow was estimated to be, on average, 11.9 Mt per year. This is calculated from a three-stage rating curve that is in turn derived from a set of observations of concentrations of suspended sediment and discharges and assuming an increase of 20% to take into account unmeasured bedload. The annual amount however, varies between 2.5 and 38 Mt, depending on the year.

Considering the observations and the above mentioned unmeasured bedload assumption the proportion of clay, silt and sand considered is 23%, 50% and 27% respectively. A number of assumptions to estimate a refined sediment gradation was made for the purpose of constructing a reservoir sedimentation model as no further division into fractions was available. Scenarios with different proportions of material, including one with 54% of sand were also considered

Reservoir sedimentation simulations were performed with HEC-RAS, a 1D reservoir model that makes predictions over periods of several years. The numerical results provide a good understanding of the general deposition issues in the reservoir as well as the impacts of several management scenarios. However, uncertainties related to the input data and the model approach, always associated to any numerical simulation, must be considered when analyzing the results.

6.2 CONCLUSIONS

The numerical model validation on the physical model results showed reasonable agreement, indicating its potential to simulate reservoir flushing.

The numerical results show that in average hydrological conditions the reservoir fills completely within 14-15 years. If hydrological conditions are more severe (as in the 1992 flood event) this can happen several years earlier. If the amount of sand in the sediment inflow is

54%, these process may develop earlier. The coarser sediment may also increase bed and water levels upstream of the reservoir. Check dam was found to be useful in blocking the entry of gravel size particles upstream of the reservoir.

It was observed that the initial storage capacity of the Gulpur Reservoir was 43.44 MCM, which after attaining equilibrium after 9th year, reduced to 16.15 MCM.

Due to unpredictable behavior of the river flow regime, a combination of flushing at high discharges (1,000 m^3/s) and in absence of this, by the mid-August discharges (of the order of 250 m^3/s or more) can prevent reservoir bed levels from rising close to the intake level as well as keeping the sediment inflow into the intakes low. From the model results it appears that about 3-4 days per year plus the time required to refill the reservoir will be required for flushing.

The trap efficiency determined from the model is 57% which agrees well with 52% value from empirical curve.

Numerical values obtained for five number of flushing indicator for the Gulpur HPP reservoir are well satisfied, hence flushing efficiency of the reservoir to flush the sediment would be 90 to 100%.

6.3 RECOMMENDATIONS

The coarser sediment may also increase bed and water levels upstream of the reservoir; therefore, to obstruct the coarser sediments, a check dam is recommended to be built just upstream of the reservoir.

Plan of hydrographic surveys of the reservoir to estimate quantity of deposited sediment should be made at least twice per year, after the spring and summer high flow season. Immediate initial survey should also be performed after the impoundment.

For observing position of sediment delta in the reservoir, hydrographic survey are required. Hence the cost of establishing range lines, boats and echo depth sounder etc. to monitor the sediment delta may be included in the cost of the project at the detailed design stage.

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