Chapter I
INTRODUCTION

1.1 BACKGROUND

The fast growing energy demand in Pakistan needs to be met in order to allow the economic and social development of the country. Overcoming the present short fall in electric power quickly can only be achieved with the implementation of generation projects of relative short gestation time. Thermoelectric projects satisfy partially this condition, but have been the drawback of requiring fossil fuels, which are available limited quantities in the country and must be imported at the expense of scarce foreign exchange earnings.

The search for renewable resources of energy of short gestation period led to the consideration of various hydroelectric schemes at barrages and canal falls in the Indus river basin. Although the head available is low but the discharges are high enough to consider the development of low head hydropower electric projects at different sites.

Tarbela and Mangla dams are losing their storage capacities due to sedimentation and losing their power generation accordingly. Large dams are not being constructed in near future whereas water and power generation demand is increasing day by day. In current scenario we need to focus on the existing projects to get their full power generation. Chashma hydropower is one of the major projects which is facing problem of low generation in flood season due to back water effect of the river section.

1.2 GENERAL

The Chashma Barrage was completed in 1971 as a part of the Indus Basin Project. It is located on the Indus River about 56 km downstream of Jinnah Barrage. The barrage supplies water to the Chashma Jhelum Link (CJ Link) Canal on the left bank and Chashma Right Bank Canal (CRBC) on the right bank. The cooling water supplies for the Chashma Nuclear Power Plant (CHASHNUP) are also taken from the barrage through the CJ Link. A 184 MW hydropower plant was constructed subsequently on the right bank and was commissioned in the year 2001.

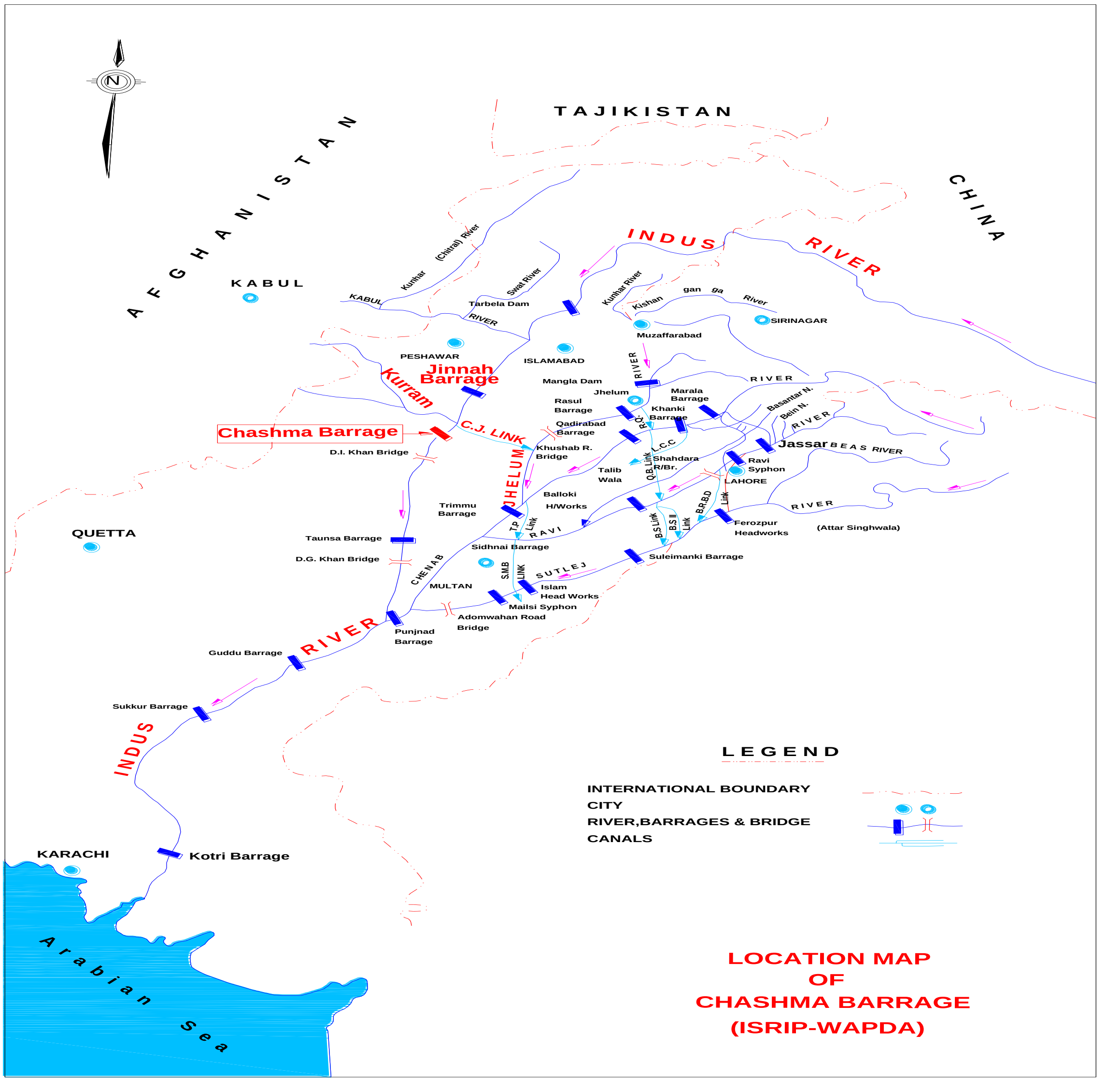

Fig. 1.1 Location Map of Chashma Barrage

The Barrage has 52 bays of 60ft wide each including 41 standard bays, 7 under sluice bays on the left side and 4 under sluice bays on the right side. In addition, a fish ladder and navigation lock form part of the Barrage. On the left bank, the CJ Link has a regulator with 8 bays of 40ft each, while on the right, the CRBC has a regulator with 2 bays of 40ft each. Further on the right bank the Chashma Hydel Power Project was constructed and commissioned in 2001. The installed capacity of the Chashma Hydel Power Project is 184 MW comprising of 8 bulb type turbine units each of 23 MW capacity. The bulb turbines have been installed for the first time in Pakistan. The first unit was commissioned in January 2001, while final commissioning of all units was completed in July 2001.

Fig. 1.2 Chashma Barrage Reservoir

The Chashma Barrage has a very wide and shallow reservoir, like a run-of river hydro-project. The Barrage is more like a lake-type reservoir, not a river-type reservoir. The reservoir is 7 miles wide at the Barrage and its maximum width reaches 13 miles (about 21 km). The surface area of the reservoir is about 139 sq. miles, about 360 km^2 (GTZ-WAPDA, 1987).

The maximum design discharge for the Barrage is 950,000 Cusecs (about 26900 m^3/s). An exceptionally high flood of 1,038,873 Cusecs passed through the Barrage gates in August 2010 which exceeded the design discharge capacity by 9.4%. Fortunately the Dam and Barrage were generally safe with minor damages after the exceptional flood. Frequency analysis shows the return period for the design flood of 950,000 Cusecs is about only 70 years, whereas the flood in 2010 corresponds to a return period of 126 years.

The maximum and minimum designed reservoir water levels are 649ft and 637 ft respectively. The Barrage initially had a gross storage capacity of 0.87 MAF (about 1.07 billion m^3) with live and dead storage capacity of 0.72 MAF and 0.15 MAF respectively. The storage of the Barrage was designed to re-regulate the flow released from Tarbela reservoir and floods from tributaries below Tarbela dam including Kabul, Haro, Soan, Kohat, Toi and Kurram. The re-regulation capacity of the Barrage has significantly reduced due to the reduction of storage capacity and the need of power generation. According to the last hydrographic survey done in 2012, the gross storage capacity reduced to 0.348 MAF and the live storage capacity reduced to 0.289 MAF, i.e. 60% lost of gross storage capacity since its operation in 1971 due to sedimentation (WAPDA, 2011).

1.3 COMPONENTS OF CHASHMA HYDROPOWER PROJECT

Following is the brief of Chashma Hydropower Project components.

1.3.1 Head Race

The Intake of the headrace channel is located at the upstream end of the right guide bank. The right guide bank is reshaped to act as island and the right embankment of the channel is constructed to function as a guide bank. The head race channel off takes from the right guide bank by puncturing and removal of its head portion. The intervening width of the powerhouse portion of the right closure bank is demolished. The bed width has been fixed as 136 m and side slope 4:1. The bed and slopes of the channel are stone protected. It is designed for a discharge of 2000 m^3/s. it can also carry 20% additional discharge over and above the rated capacity.

1.3.2 Power House

The power house is located downstream of the existing right closure bund at a distance of 320 m. the distance from the downstream corner of right undersluices of the barrage to the centre of the power house pit has been fixed as 400 m in order to avoid any settlement of the barrage during the construction of the powerhouse and dewatering of the powerhouse pit. The power house has a length (Left Bank to Right Bank) of 136 m to accommodate the 8 bulb units. There is one service block at each and expedite erection of the units. The overall length of the power house including the 2 service blocks is 202 m and the width is 61 m.

1.3.3 Tailrace

The releases from the power house will join the Indus River downstream of the barrage through a Tailrace. It has length of 1200 m, 136 m bed width and 4:1 side slopes stone protected. It is designed for a discharge of 2000 m^3/s. it can also carry 20% additional discharge over and above the rated capacity.

1.3.4 Crossing of CRBC

The CRBC is relocated by providing it a suitable diversion and crossing over the draft tubes portion of the power house through an aqueduct.

1.3.5 Turbines

The Chashma hydropower has 8 double regulated bulb units with a nominal rating of 23MW each and runner diameter of 6.3 m.

Fig. 1.3 Bays of Chashma Barrage

1.4 STUDY AREA

Chashma Hydel Power Project (CHPP) is located on the right abutment of Chashma Barrage. The barrage is located on the Indus River near the village Chashma in Mianwali District. The installed capacity of CHPP is 184 MW comprising of 8 bulb type turbine units each of 23 MW capacities. The bulb turbines have been installed for the first time in Pakistan. The first unit was commissioned in January 2001, while final commissioning of all units was completed in July 2001. The salient features are listed below

Reservoir (Existing):

Maximum Pond Level	197.80m
Normal pond level	195.70m
Minimum pond level	194.20m

Hydrological Data:

Maximum Discharge	2,584 m^3/s
Rated discharge	2,000 m^3/s
Unit discharge	250 m^3/s
Head available	4 to 11.6 m
Rated head	8.35m

Power House:

Type	semi- outdoor
Length (Right to Left)	202 m
Turbine type	Bulb/Horizontal
Rated Output	23 MW
Installed Capacity	184 MW

Headrace and Tailrace Channel:

	Upstream Channel	**Downstream Channel**
Length	1000 m	1200 m
Bed Width	136 m	136 m
Side Slope	4 : 1	4 : 1
Max. Pond Level	197.80 masl	192.70 masl
Min. Pond Level	194.50 masl	183.70 masl

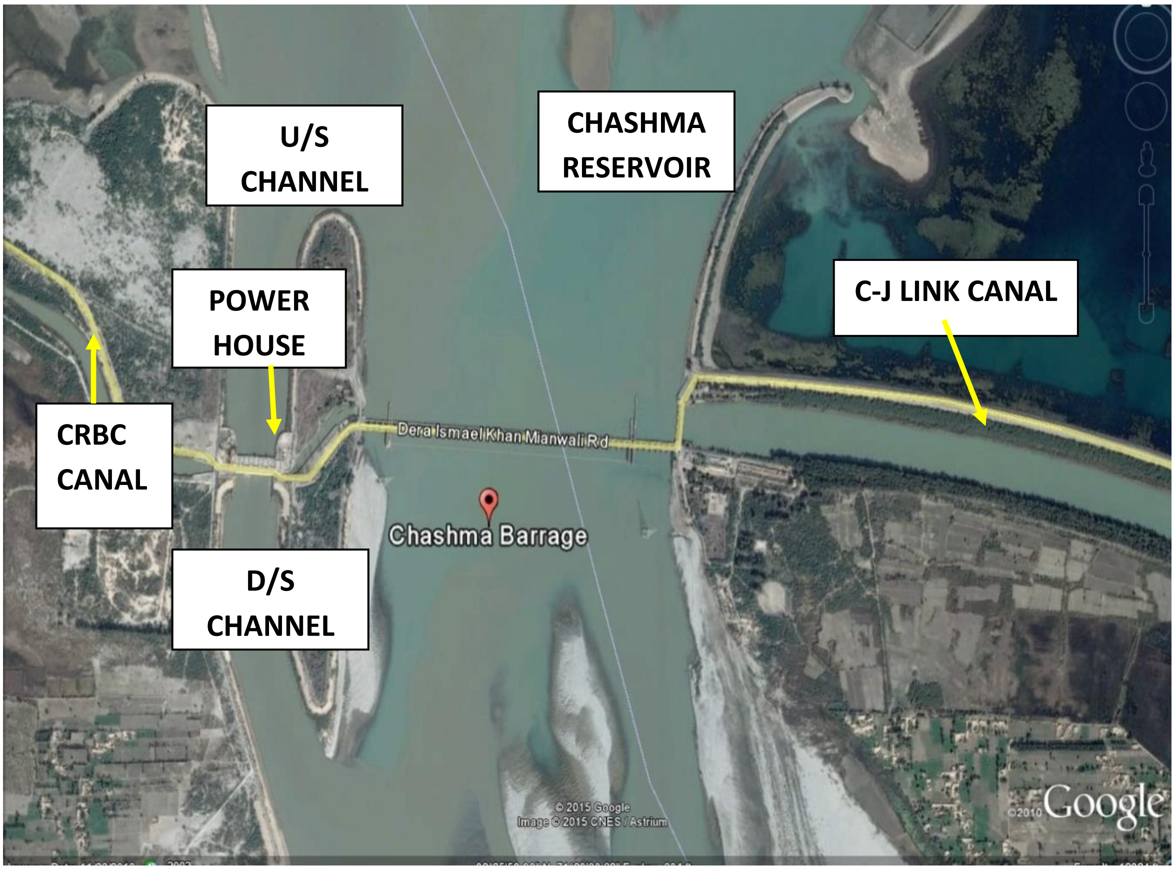

Fig. 1.4 Layout Plan of Chashma Barrage

1.5 PROBLEM STATEMENT

The normal operating range of Net Head of Chashma Hydel Power Project is 4 to 12 meters and maximum discharge through eight turbines at a rated net head of 8.4 meters is 84,755 cusecs. Chashma Barrage originally had a storage capacity of 0.86 MAF which with the passage of time and due to siltation of ponds has been reduced to 0.334 MAF. Due to low head and siltation of ponds, the range of operation of power plant has been narrowed and is heavily dependent on hydrological conditions.

The outflow of power house falls into the Indus River after 1200 meters at an approximate angle of 30 degrees. The optimum output is achieved from the power plant when Chashma Barrage discharge is less than the Tailrace channel discharge.

When irrigation indent from barrage is increased beyond maximum discharge of power house the Hydropower generation is affected due to short length of d/s Channel and acute angle of power house discharge falling in Indus river causing increase in tailrace water level and resultant decrease in net head. The following table recorded from actual data reflects the impact of Chashma Barrage discharges on Tailrace water level/net head.

Table 1.1 Summary of Discharges vs Net Head Levels in CHPP

Sr. No.	**Date**	**Discharge (Cusecs)**			**Head race Water Level (masl)**	**Barrage d/s Water Level (masl)**	**Barrage d/s Water Level (ft)**	**Tailrace Water Level (masl)**		**Net Head (m)**
		Power House	**Barrage**	**Total**				**(m)**	**(ft)**	
1.	01.10.12	81,280	3475	84,755	194.68	186.28	611.20	186.34	611.38	8.34
2.	15.06.12	70,069	65,533	1,35,602	193.71	186.64	612.35	186.76	612.75	6.95
3.	25.07.13	79,183	1,49,957	2,29,140	194.35	187.81	615.18	187.53	615.30	6.82
4.	26.07.12	80,218	3,07,626	3,87,844	194.46	188.83	619.55	188.53	618.56	5.93
5.	14.08.13	82,418	4,82,479	5,64,897	194.83	189.84	622.87	189.28	621.03	5.55

Table 1.1 clearly indicates that with increase of outflow through Barrage, Tailrace water level also increases, therefore, affecting the power generation of Chashma Hydropower Project. It is evident from Sr. # 1 that net head is maximum when outflow through the Barrage is low, however, with increase in outflow through barrage, the net head falls from 8.34 m to 5.55 m.

1.6 OBJECTIVE

Numerical modeling of Tailrace Channel of Chashma Hydropower to study and explore different scenarios using HEC-RAS modeling, in order to enhance the net head.

1.7 UTILIZATION OF STUDY

The basic goal of study is to contribute towards improved operation of Chashma Hydropower Project during high flow season by sharing the results of the research with the concerned Authority. It will also be beneficial for other barrages across the Pakistan encountering similar issues.

1.8 SUMMARY

Major focus has been laid on the introduction, significance and importance of the research work in this chapter. It also provides an insight into the objectives, overview, scope of the work and the limitations involved therein.

Chapter II
LITERATURE REVIEW

2.1 GENERAL

Literature review is the basic requirement that provides necessary approach and helps in understanding the problems. The works on different aspects of Numerical modeling are described and summarized in this section. Extensive researches relevant to our study from different parts of the world will be studied and analyzed and due deliberation will be provided to explain our research in the most effective manner in the light of the following literature.

2.2 HYDRAULIC MODELING

Hydraulic model is defined as, any physical or numerical model for the simulation of flow processes, flow states and events, which concern problems of hydraulic engineering or technical hydromechanics. Also, a hydraulic model is a mathematical model of a fluid flow system and it's used to analyze hydraulic behavior.

Model in its widest sense is simplified representation of a subject, state or event (conceptual model, system model etc.) and similar model means in which all model parameters exhibit a certain relationship to the corresponding parameters in nature, which is determined by one or several model scales (Kobus, 1973). Modeling is mathematical or physical description of a physical process either using mathematical language or general logical frameworks or models.

Hydraulic modeling is used to evaluate important elements of free surface fluid flow. Generally, hydraulic modeling can refer to both numeric modeling (in which a simulation is performed on a computer), or physical modeling (where the physical flow geometry is scaled in such a way that it can be modeled in the laboratory). Numeric models are usually single, two or three-dimensional, whereas physical hydraulic models are always three-dimensional (Kobus, 1973).

Physical and numerical modeling tools have developed enormously during the last years. However several issues need still further developments, namely the physics and modeling of sediment transport, the wave-structure interaction analysis and loads determination, erosion and scour near coastal structures as well as medium to long term accurate simulation tools.

2.3 PHYSICAL MODELING

A physical model is a framework of ideas and concepts from which we interpret our observations and experimental results. A physical hydraulic model represents a real prototype and is used to find or confirm solutions for engineering problems. Differences between the model and prototype behaviour and results may be due to scale (similarity laws considered and incomplete reproduction of the forces involved), laboratory (model geometry–2D or 3D influences, reflections; flow or wave generation techniques turbulence intensity levels, linear wave theory approach; fluid properties etc.) or measurement (different equipments used in model and prototype – intrusive or not, probe sizes) effects. The estimation of these effects (qualitatively and quantitatively) affects the results and to know if they can be neglected is a challenge for physical modelers**.** They can also justify differences between physical and numerical models e.g. kinematic viscosity (Abraham, 1975).

The basic aspects of mechanical similarity (geometric - dimensions, kinematic – time, velocity, acceleration and discharge, and dynamic – forces: inertial, gravitational, viscous, surface tension, elastic, pressure) are well known, leading to the different numbers (similarities or laws) when considering the ratios of all the forces in relation to the inertial one (the most relevant in fluid mechanics) – Froude, Reynolds, Weber, Cauchy, Euler. It is also known that although a perfect similarity would need the same value of these numbers between the prototype and the model, this is in general not possible as some of these similarities are incompatible (when using the same fluid or considering the same environment – gravity). So the most relevant force(s) present in the prototype must be selected and if compatible the model must be build according to the related similarity.

The relevant forces for most coastal hydrodynamics problems are the gravitational forces, friction, and surface tension (Langhaar, 1951). Thus, the dimensionless products are combinations of the Froude, Reynolds, and Weber numbers. Neglected are compressibility and elasticity effects. Yet the use of the same fluid on both model and prototype prohibits simultaneously satisfying the Froude, Reynolds and Weber number scaling criteria and thus, most coastal models are run respecting Froude's similarity only, which implies assuming that gravitational effects are the most significant and that the viscosity and surface tension of water do not play significant roles.

2.4 NUMERICAL MODELING

Numerical models are mathematical models that use some sort of numerical time-stepping procedure to obtain the models behavior over time. The mathematical solution is represented by a generated table and/or graph. A numerical simulation is a calculation that is run on a computer following a program that implements a mathematical model for a physical system. Numerical simulations are required to study the behaviour of systems whose mathematical models are too complex to provide analytical solutions, as in most nonlinear systems.

Fluid motion is controlled by the basic principles of conservation of mass, energy and momentum, which form the basis of fluid mechanics and hydraulic engineering. Complex flow situations must be solved using empirical approximations and numerical models, which are based on derivations of the basic principles i.e backwater equation, Navier-Stokes equation etc (Henderson, 1966).

Computational fluid dynamics (CFD) can be defined as a branch of fluid mechanics that uses numerical methods and algorithms to solve and analyse problems involving fluid flows. The term CFD model is commonly used to refer to a high-order numerical model capable of solving complex flow situations with relatively few simplifications (eg single, double or three-dimensional, multi-fluid, compressible, thermodynamic effects etc.). In reality, all numerical models are CFD models (even a simple spreadsheet solution of the backwater equation). There are generally considered to be two methods of analyzing fluid motion: by describing the detailed flow pattern at every point in the flow field (small scale or differential analysis), or by examining a finite region and determining the gross effects of and on the region (finite or control-volume analysis). Since they are generally concerned with describing or determining the fluid properties within space, most numerical models adopt a control-volume approach.

Hydraulic models may be categorized by the spatial and temporal simplifications that the model employs. Each category has associated with it a number of fluid property and dynamic assumptions (Toombes, 2011).

2.5 NUMERICAL MODELING VERSUS PHYSICAL MODELING

Numerical models represent the real problem but with some simplifications. Thus, the modeler is forced to make a compromise between the details of the model and the prototype. An incorrectly designed model always provides wrong predictions, independently of the sophistication of the instrumentation and measuring methods. The cost of physical modeling is often more than that of numerical modeling, and less than that of major field experiments, but this depends on the exact nature of the problem being studied. Physical modeling has gathered new perspectives due to the development of new sophisticated equipment, allowing the measurement of variables in complex flows, which was previously impossible. New experimental techniques, automated data acquisition and analysis systems, rapid processing and increased data storage capabilities also provide useful information for the validation of numerical models (Mohammed, 2006).

With relation to numerical models it can be said that recent developments such as SPH and in computing capacity have made these tools more powerful than even before, leading to a better description of the complexity of the hydraulic phenomena (physical environment and borders as well as non-linear aspects of the equations used). From another perspective this tool is in general more attractive to researchers and practitioners.

To obtain theoretical solutions, simplifications of the physical environment (especially the boundaries) are needed as well as of the equations that govern the phenomena. As a result of that mathematical solutions may have lower quantitative value, and therefore could be more useful for qualitative or comparative analyses. The geometry can be reproduced with the desired detail but it is not enough to ensure a correct reproduction of the reality in the model as this can generate a behaviour sometimes different from the prototype. So calibration is needed. Physical modeling reproduces both linear and nonlinear aspects of the phenomena, avoiding the simplifications of the numerical modeling that simplifies not only the geometry but also fundamental equations.

Physical and numerical model input conditions can be controlled and systematically varied, whereas field studies have no such control. However, many problems in coastal engineering are not amenable to mathematical analysis because of the nonlinear character of the governing equations of motion, lack of information on wave breaking, turbulence or bottom friction, or numerous connected water channels.
Due to the quantitative deficiencies and limitations of predictive numerical models when applied to complex flows, the need for physical modeling still remains and investments in laboratory facilities, equipment and new techniques are more and more needed, highlighting the need for synergies between the various research tools, physical and numerical modeling included, not only because of the actual complexity

of the maritime hydraulics problems, but also to improve some design approaches (Hughes, 1993).

2.6 ONE-DIMENSIONAL NUMERICAL MODELING

The most widely used approach to modeling fluvial hydraulics has been 1D finite difference solutions of the full Saint-Venant Equations. The Saint-Venant Equations are based on conservation equations of mass and momentum for a control volume, as shown in differential form in Equations 2.1 and 2.2.

$$\partial A/\partial t + \partial Q/\partial x = 0 \qquad (2.1)$$

$$\partial Q/\partial t + \partial/\partial x\ (uQ) + gA(\partial h/\partial x - s\_0\ ) + gAs\_f\ = 0 \qquad (2.2)$$

Where Q is discharge, A is cross-sectional flow area, u is longitudinal flow velocity, h is Flow depth, S_o is bed slope, and S_f is friction slope. 1D solutions of the full Saint-Venant Equations are derived based on several assumptions: the flow is one-dimensional, the water level across the section is horizontal, the streamline curvature is small and vertical accelerations are negligible, the effects of boundary friction and turbulence can be accounted for using resistance laws analogous to those for steady flow conditions, and the average channel bed slope is small so the cosine of the angle can be replaced by unity (Cunge, 1980).

Widely available software such as MIKE11 and HEC-RAS use the general form of the section-averaged Navier-Stokes equations. HEC-RAS has a similar approach except Manning's roughness is used to calculate friction losses instead of the Chezy coefficient (USACE, 2010).

The unsteady equations are solved by HEC-RAS using a four-point implicit scheme which requires that spatial derivatives and functions are evaluated at an interior point $(n+\theta)\Delta t$ (USACE, 2010). Thus, values at the next time step are required for all terms in the general 1D equation. A system of simultaneous equations results from the implicit scheme. The effect of the implicit scheme allows information from anywhere within the reach to influence the solution. This discretization scheme requires much more computational effort than an explicit scheme, but it has improved numerical stability. Von Neuman stabilities analyses conducted by (Fread, 1993) found that the four point implicit scheme is unconditionally stable for $0.5<\theta<1.0$.

2.7 TWO-DIMENSIONAL NUMERICAL MODELING

In two-dimensional modeling, some of the physical constraints seen in a one-dimensional model can be overcome. Given that flow can be simulated in one or two-dimensions by using either a series of cross-sections or a continuous surface, the assumptions made in hydraulic modeling as well as the quality of the terrain data and the cross-sectional configuration for a one-dimensional model or mesh resolution for a two-dimensional model will have a large impact on the resulting inundation.

Two-dimensional hydraulic models are commonly used for modeling of floodplains, coastal and marine situations where the flow path is poorly defined. Two-dimensional models calculate water depths and velocities across a grid or mesh that defines the topographic information. Traditionally, the mesh has been a fixed-space rectilinear grid with the governing equations solved using implicit finite difference techniques. More recent models have allowed for a flexible mesh (typically consisting of triangles or quadrilaterals) solved using finite-element methods, which have significantly greater ability to handle complex geometries and boundaries at the expense of increased numerical complexity.

The numerical solution used by two-dimensional hydraulic models is usually based on the Saint Venant equations, which are derived from the depth-integrated conservation of mass and Navier Stokes equations. The Saint Venant equations are also commonly known as the shallow water equations, and are based on the assumption that the horizontal length scale is significantly greater than the vertical scale, implying that vertical velocities are negligible, vertical pressure gradients are hydrostatic and horizontal pressure gradients are due to displacement of the free surface.

Unlike the algorithms used by one-dimensional models, two-dimensional models can often model both subcritical and supercritical flow conditions. For example, MIKE 21 by DHI Software requires at least two grid cells in the direction of flow to correctly resolve transition from sub- to supercritical flow at a control such as a weir (McCowan, 2001).

2.8 HEC-RAS NUMERICAL MODELING

HEC-RAS (Hydrologic Engineering Center River Analysis System) is numerical software for flow river hydraulics calculations (Darshan, 2014). It was developed by the Hydrologic Engineering Center, a research group for the U.S. Army Corp of Engineers. This software contains three hydraulic components for flow analysis: (1) steady flow water surface profile computations; (2) unsteady flow simulation; and (3) movable boundary sediment transport computations (Hasani, 2013). A key element is that all the three components will use a common geometric data representation and common geometric and hydraulic computation routines (Amir, 2012). It is widely used in one-dimensional flow characteristics calculations including water surface profiles, energy grade line, water depth, velocity, wetted perimeter, in case of steady and unsteady river flow regimes (Henry, 2008; Robert, 2012). These computations are essential in the analysis of various problems, including the determination of the effect of hydraulic structures on the upstream and downstream channels; the estimation of flood plain; the analysis of the capacity of river for irrigation; the monitoring of the depth at any point in river; the choice of implantations sites of hydraulic structures (such as dams, pumping stations) the corrections of the rivers in order to avoid a possible overflow in the event of rising. These various applications deserve to give to the flow main parameters calculations a great interest (Karney, 2010). The steady flow component of HEC-RAS was used to perform flow parameters (such as water surface profiles, water surface elevation, energy grade line elevation and water flow velocity) of the River in order to analyze the hydraulic of the system. The basic data requirements for simulation are included geometric data, river system schematic, cross section geometry, reach lengths, Manning's roughness coefficient, contraction and expansion coefficients, steady flow data, boundary condition, flow regime (Harman, 2008).

2.8.1 Basic Equations of HEC-RAS

In HEC-RAS steady state simulation, water surface elevation and energy grade line of two adjacent cross sections are calculated by applying the standard step iterative method to one-dimensional energy equation (Goodell and Warren, 2006). The equation 2.3 is shown for two adjacent cross-sections (Nemati, 2011). Figure: 2.1

illustrates the main computing process based on solution of energy equation (Maghsoud, 2012).

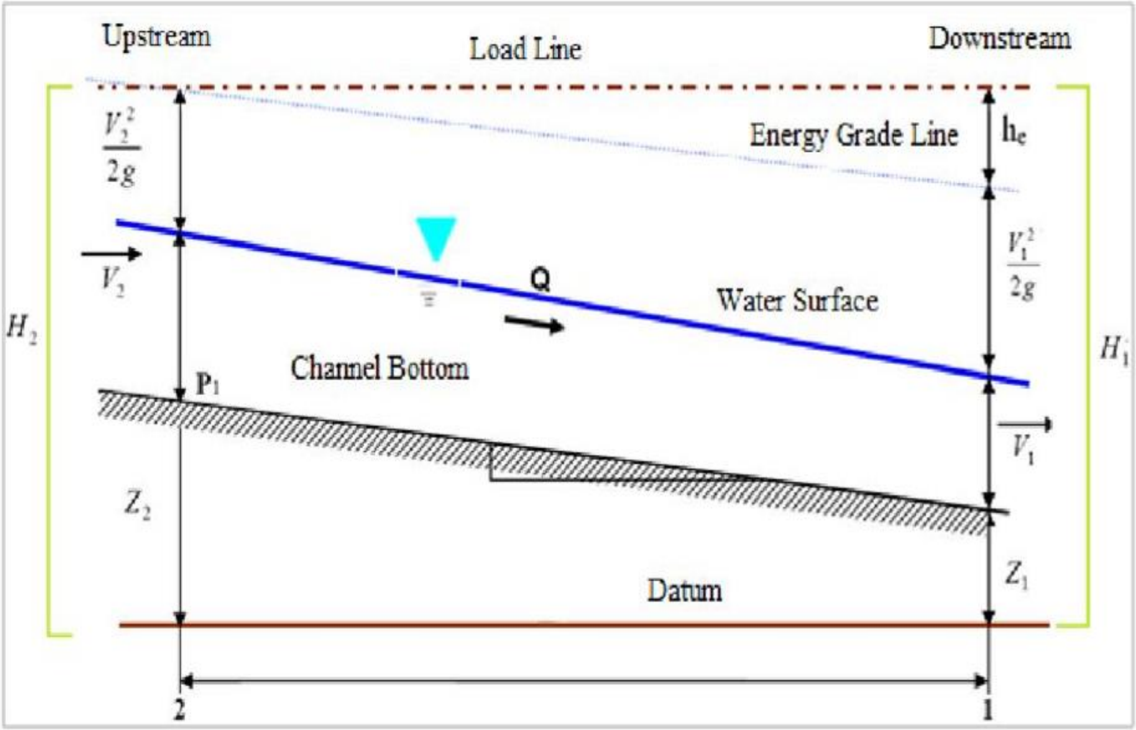

Fig. 2.1 Energy Equation between Two Sections by Maghsoud, 2012

$$Z_2 + Y_2 + \frac{\alpha_2 V_2^2}{2g} = Z_1 + Y_1 + \frac{\alpha_1 V_1^2}{2g} + h_e \qquad (2.3)$$

Where Z_1, Z_2 are elevations of the main channel inverts, Y_1, Y_2 are depths of water at cross-sections, V_1, V_2 averages velocities (total discharge/total flow area), α_1, α_2 are velocity weighting coefficients, g is gravitational acceleration, h_e is energy head loss. The energy head loss (h_e) estimation is given by the Manning's equation 2.4 which is considered to be empirical (Mazhar, 2010).

$$h_e = L\overline{S_f} + C\left|\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}\right| \qquad (2.4)$$

Where L is discharge weighted reach length, S_f is representative friction slope between two adjacent sections and C is an expansion or contraction loss coefficient. The representative friction slope using the average conveyance equation and the distance weighted reach length are defined in Equation 2.5 and Equation 2.6, respectively (Dragan, 2014).

$$\overline{S_f} = \left(\frac{Q_1 + Q_2}{K_1 + K_2} \right)^2 \tag{2.5}$$

$$L = \frac{L_{lob}\overline{Q_{lob}} + L_{ch}\overline{Q_{ch}} + L_{rob}\overline{Q_{rob}}}{\overline{Q_{lob}} + \overline{Q_{ch}} + \overline{Q_{rob}}} \tag{2.6}$$

Where K is conveyance, L_{lob} , L_{ch} , and L_{rob} are cross-section reach lengths for flow in the left over-bank, main channel, and right over-bank, respectively, and $\bar{Q}_{lob}$, $\bar{Q}_{ch}$, and $\bar{Q}_{rob}$ are arithmetic average of the flows between sections for the left over-bank, main channel, and right over-bank, respectively. To determine total conveyance and the velocity coefficient for a cross-section, HEC-RAS subdivides flow in the main channel from the over-banks. The Conveyance is calculated for each subdivision using Equations 2.7 and 2.8 (Dragan, 2014).

$$Q = KS_f^{1/2} \tag{2.7}$$

$$K = \frac{1.486}{n} AR^{2/3} \tag{2.8}$$

Where K is conveyance for the subdivision, n is Manning's roughness coefficient for the subdivision, A is flow area for the subdivision; R is hydraulic radius for each subdivision. The total conveyance for each subdivision is calculated as the sum of the conveyance from the left over-bank, main channel, and right over-bank. Solving these equations requires knowledge of the geometry of the stream, its roughness characteristics, the flow rate and boundary conditions.

2.9 REVIEW OF RELATED LITERATURE ACROSS THE GLOBE

Karthikeyan (2007) mentions methods to achieve the required head for maximum and minimum discharges respectively through hybrid model studies. Many hydraulic phenomena which occur in nature are too complex to be described by rigorous mathematical techniques alone and models are used as an alternative means of obtaining the information necessary to complete efficient and satisfactory design. Scale models permit visual observation of the flow and make it possible to obtain certain desired numerical data. The increasing use of mathematical techniques and computers during the past two decades have led to increasing use of hybrid models combining the advantages of both physical and mathematical model.

In addition, hydraulic performance of various components of the barrage was assessed and velocity profile in the Tailrace channel for maximum discharge was obtained. An undistorted physical model of a low head hydropower plant of scale 1:100 was constructed to achieve the designated head through model studies by designing the Tailrace and headrace channels. In order to minimize the number of modifications and thereby time consumption, design and analysis were carried out using HEC-RAS software and the results were implemented in the physical model. Rationale for deviations between the two results was analyzed and suitable modifications in the physical model were implemented and the required head for maximum and minimum discharge have been successfully achieved by designing a Tailrace channel from the outlet of the draft tube and showed that HEC-RAS can be productively used in designing the Tailrace channel of a low-head power plant to achieve the required head with acceptable standards of accuracy.

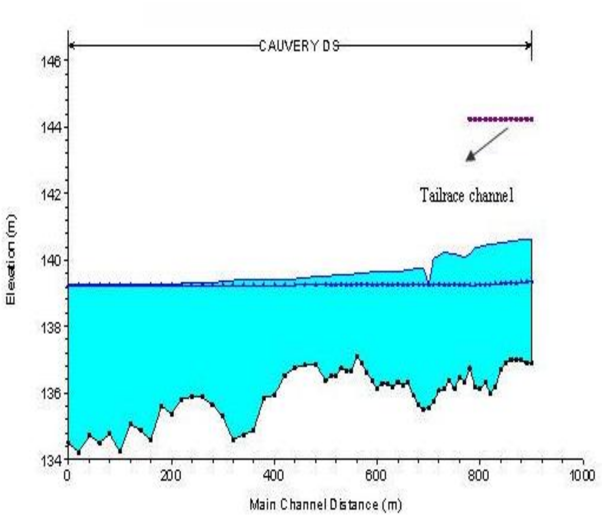

Fig. 2.2 Hydraulic Design of Tailrace Channel for a Low-Head Hydro Power Plant Using HEC-RAS software by M. Karthikeyan (2007)

In order to reduce the number of modifications and trail runs in the physical model, the computations were carried out using steady flow component of HEC-RAS (Hydrologic Engineering Center-River Analysis System), developed by the U.S. Army Corps of Engineers. The basic computational procedure is based on the solution of the one-dimensional energy equation. The physical model revealed the existence of low levels in patches below the outlet of the draft tube. These patches were made use for aligning and constructing the tailrace channel to minimize the quantum of excavation. Once the geometry, steady flow data and boundary conditions has been established, the model can be used to calculate the steady flow water surface which in turn provides the water level at the outlet of the draft tube for the corresponding discharge, from that the head available for that discharge can be found.

Traore (2015) analyzes the dynamic of the flow in Anambe river basin The flood plain of this river is selected by the Government of Senegal to develop irrigated rice cultivation in the fight against hunger. HEC-RAS model was used to calculate the main flow characteristics along the study reach to better understand the hydraulic behavior of this river. This allows identifying the high and low flow characteristics areas such as flow velocity, depths, slopes, surface, volume and their spatial evolution along the river reach. The large and narrow width areas have been identified. The analysis of the results show that the most of hydraulic parameters decreases from upstream to downstream. These results give a basis for reflection for decision makers to better understand the Anambe system and optimize water resources management o for an application to the irrigation. However, it is important to recognize the flow is supposed to be channeled while the natural flow is 3D.

Sutherland (2011) describes hydraulic studies have been traditionally undertaken with physical models, which reproduce flow phenomena at reduced scale with dynamic similarity. Numerical models are increasingly being used in place of physical models and rely on mathematical descriptions of complex turbulent processes and boundary conditions but can be cheap and versatile. Physical and numerical models both have their strengths and weaknesses and their merits must be compared to the benefits of theoretical analysis (desk studies) and measurements made in the field. It draws light on work undertaken within the research project Hydralab and subsequently in an IAHR working group on Composite Modeling. The strengths and weaknesses of physical and numerical models are analysed, along with selected case studies that focused on the methodologies used and their impact on the modeling approach. Some reflections on key elements in composite modeling are presented. Composite modeling is still in its infancy in the hydraulic community, but there is a growing trend towards using both numerical and physical models together.

Marchi (2016) have provided a detailed study a large number of hydropower plants in Brazil. Brazil currently has more than one hundred medium and large hydropower plants that, together with small hydropower plants, produce on average 91% of the total electricity in the country. To use most of the hydropower potential of a basin, hydropower plants are built normally in cascade. In some cases, the reservoir operation can cause elevation of Tailrace water level from upstream plants, the backwater effect. In this case, the original water level-discharge relationship isn't valid, but depends also on the downstream reservoir level. In the future, this effect tends to be even more intense and frequent as new hydropower plants and reservoirs are built. Study was to analyze the system with 143 reservoirs and three different hydrological scenarios with planning horizons of five years, corresponding to periods of dry, wet, and medium inflows in historical series. The backwater effect was significant, resulting in reduced generation by approximately 400 MW, or 0.6% to 0.8% of the total hydropower production. Furthermore, the effect is concentrated in a small number of large hydropower plants.

Ferrick (1985) defines the variation of tail water elevation with project discharge. This data is used to compute the generating head available at each discharge level. The tail water level elevation depends upon downstream channel geometry, project discharge and downstream back water effect. For a new project tail water rating curve are estimated from known water surface profile. For existing project a historical record of tail water elevation and discharge data is usually available to aid in development of tail water rating curve. When reservoir elevation is constant a head discharge curve may be developed directly from tail water rating curve.

2.10 REVIEW OF PREVIOUS STUDIES

2.10.1 GTZ-WAPDA Feasibility Studies

A feasibility study for small hydel stations on irrigation structures and more economical for installation of hydel power station at the level of project planning was carried out by GTZ-WAPDA team and conclusion of the study was:

- Power Plant with installed capacity of 270 MW (12 units of 22.5 MW each) can be constructed.
- Maximum discharge for 12 units is 3000 Cumecs.

However on serious observations raised by the Government of the Punjab Irrigation department, it was decided with the help of hydraulic model studies to construct power plant with 8 units.

2.10.2 Hydraulic Model Studies before Chashma Group of Consultants

- ✓ Comprehensive Sediment flow pattern at upstream of Barrage due to Power House construction in Laboratory of IRIN (Irrigation Research institute Nandipur) year 1985 & 1987.

Horizontal scale	=	1:100
Vertical Scale	=	1:50
Velocity scale 10 ft-sec	=	1.41 ft/sec
Time 1 minute	=	4.24 sec

- ✓ To address the concerns raised by the Government of the Punjab Irrigation department, study of pattern of silt entry into off-taking canals &

aggregation/degradation upstream of the barrage in Laboratory of IRIN (Irrigation Research institute Nandipur) year 1990.

2.10.3 Hydraulic Model Studies After Chashma Group of Consultants

- ✓ Comprehensive sediment model studies in order to optimize the number of bulb units in Laboratory of SOGREAH 1991.
- ✓ Velocity measurement along exist of the power channel; rip rap stability under load rejection, study of power house operation with fixed bed 1:30 Hydraulic model study in Laboratory of IRIN (Irrigation Research institute Nandipur) year 1995.
- ✓ Optimization of different design elements of power house with fixed bed 1:30 Hydraulic model study in Laboratory of IRIN (Irrigation Research institute Nandipur) year 1996.

2.10.4 Hydrographic Survey of Chashma Reservoir By Wapda

The hydrographic survey is carried out after every five years to access the capacity of Chashma Reservoir as per Standing Operating Procedure (SOP) of Chashma Barrage. The last survey 4th Hydrographic Survey of Chashma Reservoir was conducted by International Sedimentation Research Institute Pakistan (ISRIP), WAPDA in 2008 on the recommendations of Dams Safety Organization (DSO), WAPDA.

The 5th Hydrographic Survey of Chashma Reservoir was conducted by ISRIP in 2012 to check changes in the regime of reservoir after passing the exceptionally high flood of various discharges at Chashma Barrage in 2010.

The result of study is that the storage has increased due to exceptionally high flood in 2010. The comparison of capacities of 2008 and 2012 Survey shows that:

- At the maximum conservation level (El 649.00 feet) the gross capacity of the reservoir has been increased from 0.3207 MAF to 0.3482 MAF showing an increase of 8.6%.
- At dead storage level El 637.00 feet, the reservoir capacity has changed from 0.0574 MAF to 0.0595 MAF with an increase of 3.7%.
- The present live storage is 0.2887 MAF against the capacity calculated in 2008 of 0.2633 MAF indicating 9.6% increases in live storage due to exceptionally high flood in 2010.
- The Gross Storage Capacity Loss has been reduced from 63.1% in the 4th Hydrographic Survey (2008) to 60.0% in 5th Hydrographic Survey (2012).

The summary of table shown below gives clear idea about the storage lost due to sedimentation in terms of percentage (ISRIP, 2012).

Table 2.1 Loss in Storage Capacities of Chashma Reservoir in 1986-87

Year of Survey	Storage Capacity (MAF)			Loss Since Impounding (MAF)			Percentage Loss (%)		
	Gross	**Live**	**Dead**	**Gross**	**Live**	**Dead**	**Gross**	**Live**	**Dead**
Original	0.8700	0.7170	0.1530						
1971-72	0.7577	0.6308	0.1269	0.1123	0.0862	0.0261	12.9	12.0	17.1
1981-82	0.5530	0.4934	0.0596	0.3170	0.2236	0.0934	36.4	31.2	61.0
1986-87	0.4970	0.4348	0.0623	0.3730	0.2822	0.0907	42.9	39.4	59.3

Table 2.2 Loss in Storage Capacities of Chashma Reservoir in 2008

Year of Survey	Storage Capacity (MAF)			Loss Since Impounding (MAF)			Percentage Loss (%)		
	Gross	**Live**	**Dead**	**Gross**	**Live**	**Dead**	**Gross**	**Live**	**Dead**
Original	0.8700	0.7170	0.1530						
2008	0.3207	0.2633	0.0574	0.5493	0.4537	0.0956	63.1	63.3	62.5

Table 2.3 Loss in Storage Capacities of Chashma Reservoir in 2012

Year of Survey	Storage Capacity (MAF)			Loss Since Impounding (MAF)			Percentage Loss (%)		
	Gross	**Live**	**Dead**	**Gross**	**Live**	**Dead**	**Gross**	**Live**	**Dead**
Original	0.8700	0.7170	0.1530						
2012	0.3482	0.2887	0.0595	0.5218	0.4283	0.0935	60.0	59.7	61.1

Reference: ISRIP, 2008 & ISRIP, 2012.

2.10.5 Chashma Barrage Operation

Normal regulation is carried out with the pond level 195.7 m to feed the two canals to their design capacities, to control sediment entry into the canals and to control shoal formation upstream of the barrage.

Every year during the flood season from July to September when flood recedes to 4200 m^3/s the pond level is raised to its maximum limit of 197.8 m.

The stored water is released for irrigation during the period from October to February. High inflows during this period are stored to the maximum possible level for irrigation. Maintaining high pond levels for longer time is generally discouraged because besides subjecting the structures to undue strains, they induce the possibility of shoal formation. The barrage is capable of passing safely 11,300 m^3/s with water level at peak elevation of 197.8 m (passing higher discharge at this level result in damageable erosion on the downstream side) and 27,000 m^3/s at elevation of 195.7m. In an emergency, the pond level can be lowered from 197.8 m to 195.7 m in about 24 hours with an average discharge of 5,500 m^3/s plus the base river flow. The maximum pond level available for power generation is 197.8 m. In addition the head across the barrage has been restricted to 11.6 m to ensure safety of the barrage (WAPDA, 1985).

2.10.6 Headrace & Tailrace Design Approach

The design features are developed by empirical approach based on the results of the model test, stability consideration of the side slopes, and required approach based on the following:

- Geometry of the nose part, especially the Left Guide Bank which in the final shape becomes partly a component of the existing Right Guide Bank of the Chashma Barrage after its modification in the construction process.
- Slope profile of the flanking embankments, partly in fill, considering the stability in the event of rapid draw down.
- Rip-rap size required by maximum follow velocities and in the event of sudden load rejection and wave action.
- Flow velocities resulting from barrage operation viz-a-viz construction of the Headrace Channel, noting that in no case the Right Guide Bank of the barrage can be left without protection during the high flow season.

For the 100 years return period flood, a peak discharge of 25,000 m^3/s, and assuming that the cross-section are the same for 15,000 and 25,000 m^3/s, the flow velocity just near the Guide Bank will be 3.7 m/s. This value is adopted in the design.

The minimum bed levels observed near the existing Guide Bank during past years are 169 m (1991) and 176 m (1988). The upward/reverse slope provided in the bed in this region is from elevation EI 176 m to 186 m is IV:6H.

Table 2.4 Typical Section Design Features

Bed	Width	136 m
	Level	Varies from EI 187.60 to 185.00
	Lining	60 cm, Type I.
Sides	Slopes	IV:4H
	Lining	1.3 m rip-rap type 1 over geotextile upto EI. 192.00
		1.3 m rip-rap type 3 over geotextile above EI. 192.00

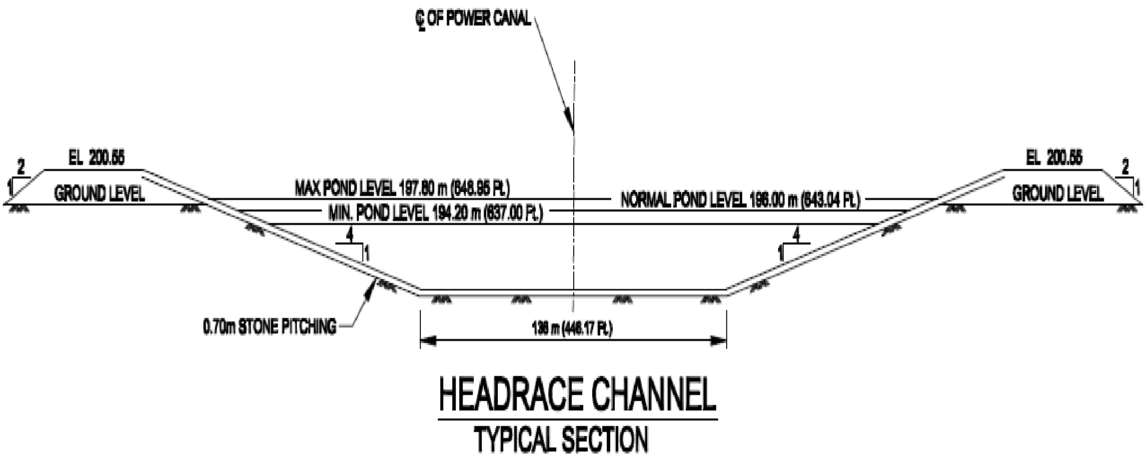

Fig. 2.3 Typical Section of Headrace Channel

The design was based on the survey carried out in 1995. However, in 1997 dramatic variation in the river flow pattern were observed in the upstream area of the bell-mouth. Sever cross currents and scours were observed. Developments, the geometry of the right embankment of Headrace were considerably modified. The return part of this embankment was lengthened by 350 m to arrest probability of reverse flows in the downstream area**.**

Tailrace Channel adopted bed width is based on the width of powerhouse structure. Various calculations were carried out to assess the hydraulic conditions downstream of the draft tube and check the location where the hydraulic jump forms resulting from full load rejection (discharge cut with the wicket gate, later by the draft tube gates). Head losses of the wicket gates corresponding to the total head with different discharges per turbine (200, 264 and 300 m^3/s) and initial discharge of the turbines 50, 100, 150, 200 & 264 m^3/s (CGC, 2001).

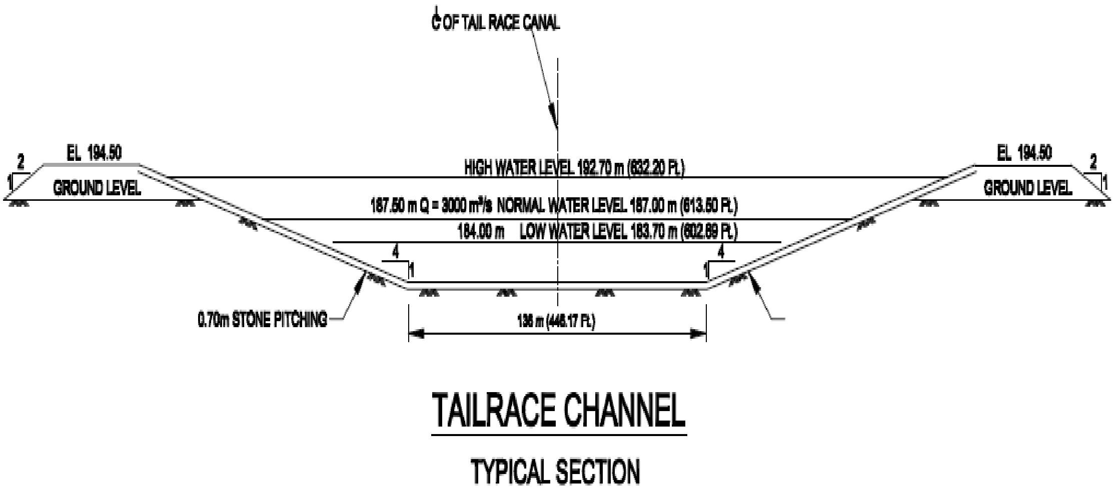

Fig. 2.4 Typical Section of Tailrace Channel

2.11 LITERATURE ON MANNING'S FORMULA

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{2.9}$$

Where 'V' is the mean velocity in fps, 'R' is the hydraulic radius in ft, 'S' is the slope of Channel and 'n' is the Manning's Coefficient of Roughness. The Manning formula has become the most widely used of all uniform-flow formulas for open channel flow computations (Chow, 1959).

2.11.1 Literature on Manning's Roughness Coefficient 'N'

Hydraulic roughness is the measure of the amount of frictional resistance water experiences when passing over land and channel features. One roughness coefficient is Manning's n value. Manning's n is used extensively around the world to predict the degree of roughness in channels. Flow velocity is strongly dependent on the resistance to flow. An increase in this n value will cause a decrease in the velocity of water flowing across a surface. In order for proper determination of the roughness coefficient, four general approaches are given:

(A) To understand the factors that affect the value of n and thus to acquire a basic knowledge of the problem and narrow the wide range of guesswork. (B) To consult a table of typical n values for channels of various types. (C) To examine and become acquainted with the appearance of some typical channels whose roughness coefficients are known. (D) To determine the value of n by an analytical procedure based on the theoretical velocity distribution in the channel cross section and on the data of either velocity or roughness measurement (Chow, 1959).

2.11.2 Factors Affecting Manning's Roughness Coefficient

It is not uncommon for engineers to think of a channel as having a single value of n for all occasions. In reality, the value of n is highly variable and depends on a number of factors. In selecting a proper value of n for various design conditions, a basic knowledge of these factors should be found very useful. The factors that exert the greatest influence upon the coefficient of roughness in both artificial and natural channels are given below.

i) Surface Roughness

ii) Vegetation

iii) Channel Irregularity

iv) Channel Alignment.

v) Silting and Scouring

vi) Obstruction

vii) Size and Shape of Channel

viii) Stage and Discharge

ix) Seasonal Change

2.12 SUMMARY

In this Chapter, literature pertaining to the hydraulic modeling, numerical modeling & physical modeling, comparison of numerical modeling with physical modeling is described in detail. Furthermore, literature on one dimensional, two dimensional flow, HEC-RAS numerical modeling and basic equation also have been reviewed. Similarly, extensive researches from different parts of the world have been studied and analyzed and utmost relevance with our research has been established. Works like Hydrographic survey of Chashma reservoir, design aspects of headrace & tailrace Channel and operation of Chashma Hydropower Project have been presented in this chapter for ease of explanation. Due deliberation has been given to explain our research in the most effective manner in the light of the presented literature.

Chapter III
METHODOLOGY

3.1 GENERAL

In the research work specific steps were followed to achieve the objectives of the research. The complete procedure about research work has been briefed in this chapter. In the flow chart it is explained from data collection, data analysis and different steps for Tailrace modeling of Chashma Hydropower Project using HEC RAS.

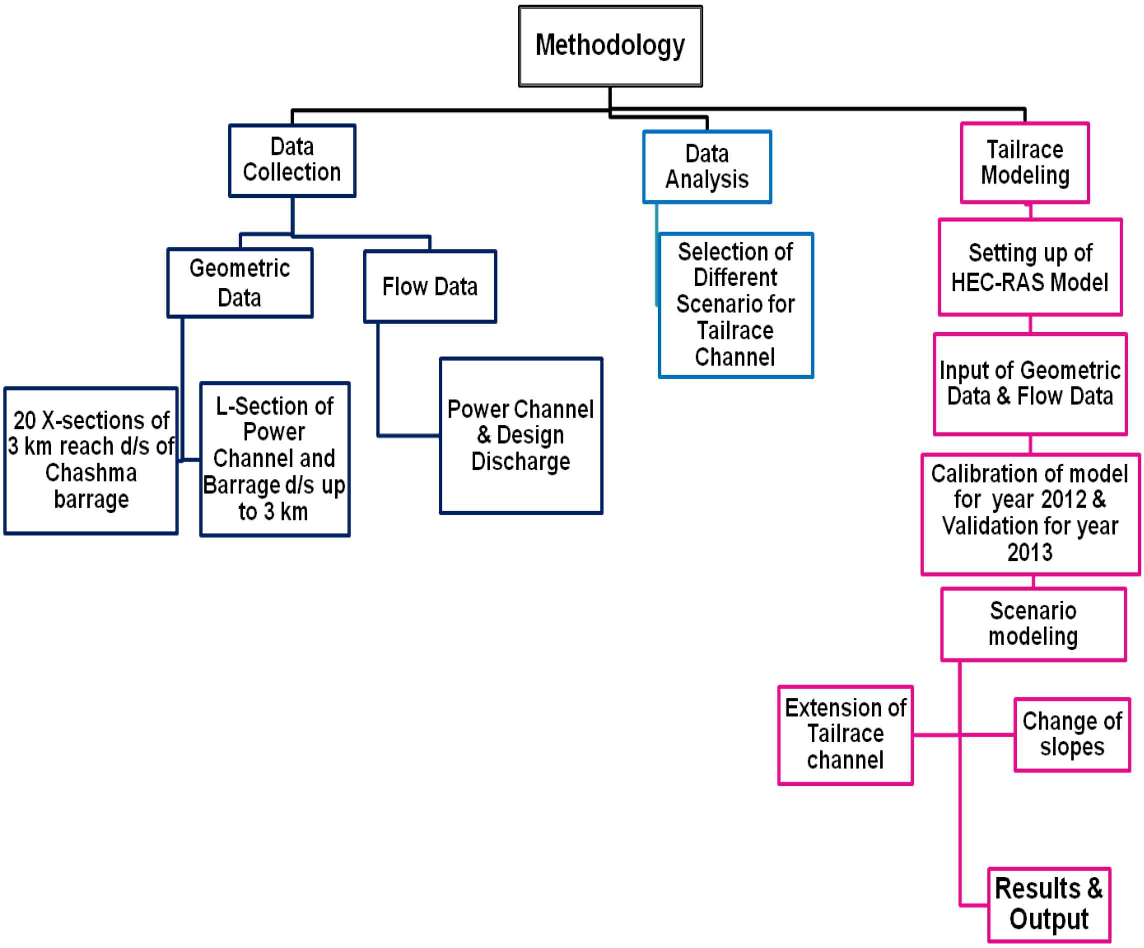

Fig. 3.1 Research Methodology Flow Chart

3.2 DATA COLLECTION

The important and suitable data was required to achieve the research objectives, and data collection from an appropriate source was fundamental component of the research work. Data was collected from different organizations of WAPDA that includes Chashma Barrage project office, Survey & Hydrology (S&H) Chashma and ISRIP respectively. The data was in raw, loose and in hard copy shape which was arranged in proper shape, converted in regular Excel sheets for the purpose of calculations and then brought to graphical shape for analysis and use for the computer model. Data includes geometric data (cross-sections and L-sections) and flow data (maximum flood of barrage and design discharge of power channel), the details of the data is as under.

3.2.1 Geometric Data

To carry out the study, the following field survey data was required. The survey was conducted by the ISRIP, WAPDA in January 2014;

- Geometric data includes x-sections of 3km reach d/s from Chashma barrage, twenty cross-sections of Indus River at interval of 150 meters (492 ft) for 3km reach starting immediately from downstream of Chashma Hydropower Station.
- Topographic strip survey of downstream for 3 km reach starting immediately from downstream of Chashma Hydropower Station and extending 40 meters beyond either side guide bank.
- River L-Section for 3km reach starting immediately from downstream of barrage.

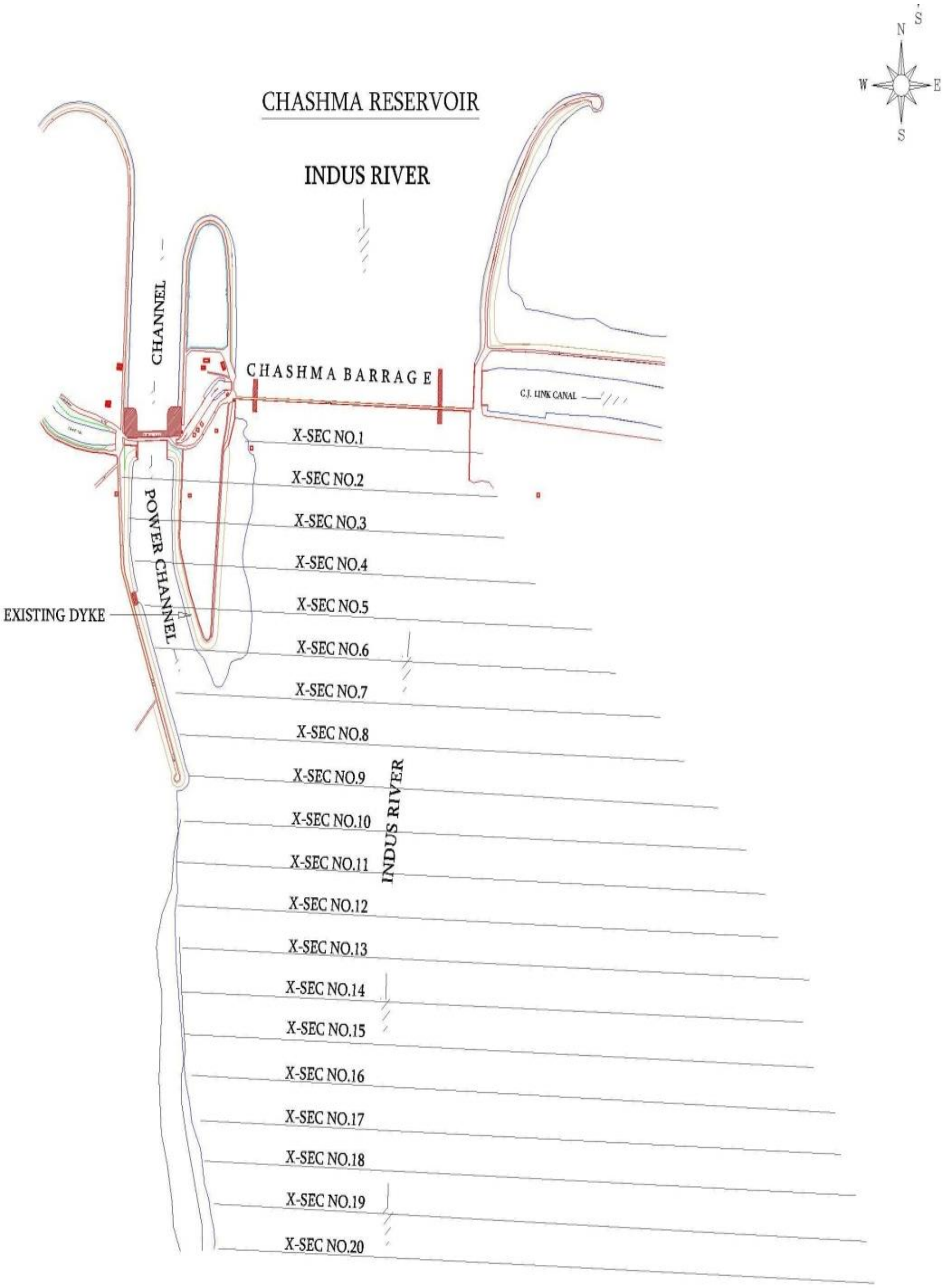

Fig. 3.2 Layout Survey plan of Indus River reach d/s of Chashma barrage

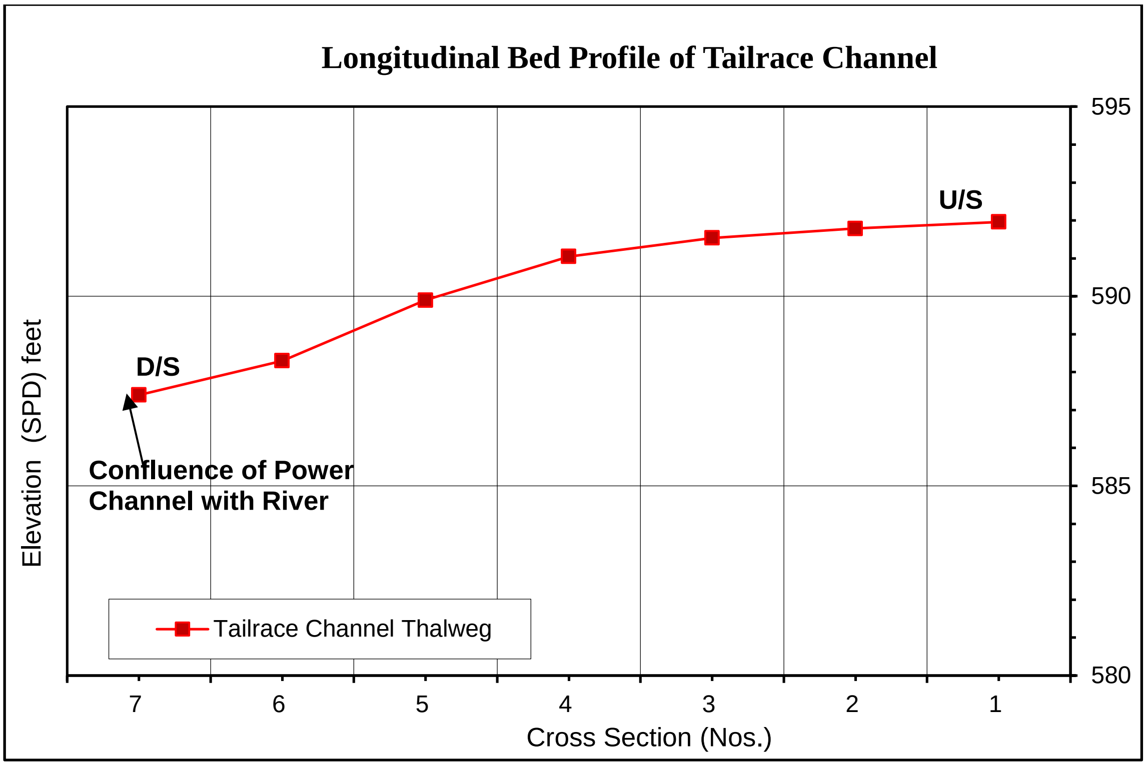

Fig. 3.3 Longitudinal Bed Profile of Tailrace Channel

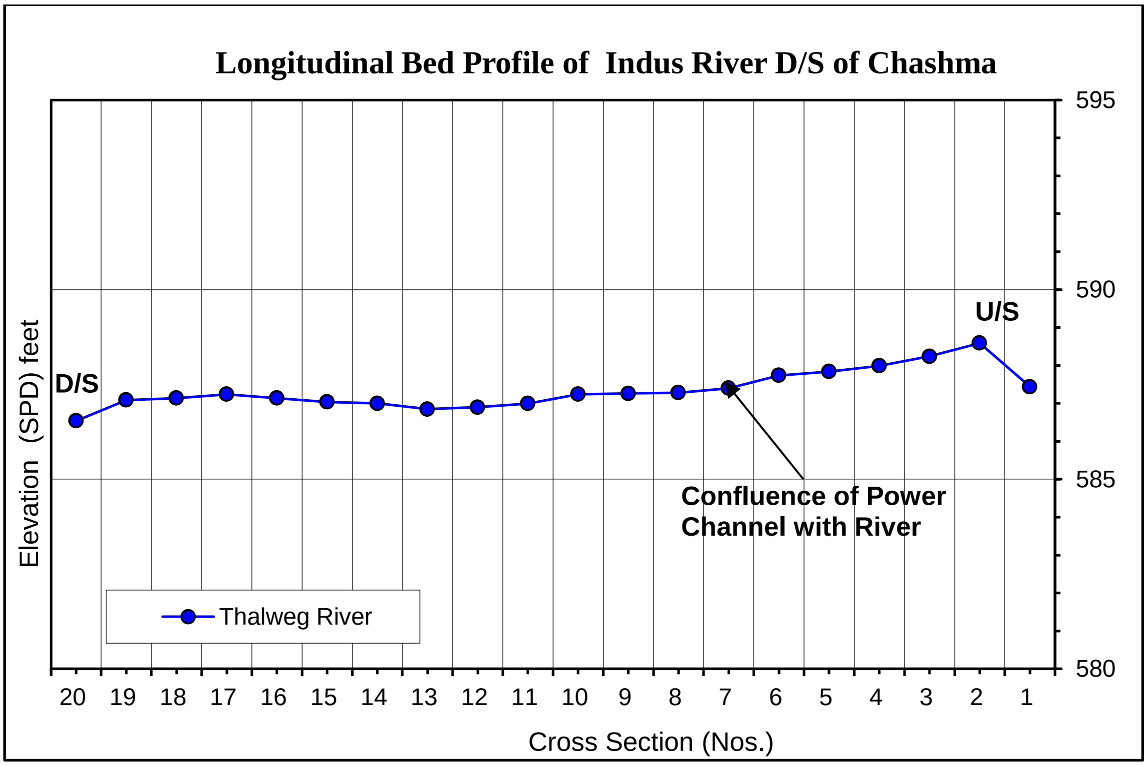

Fig. 3.4 Longitudinal Bed Profile of Indus River

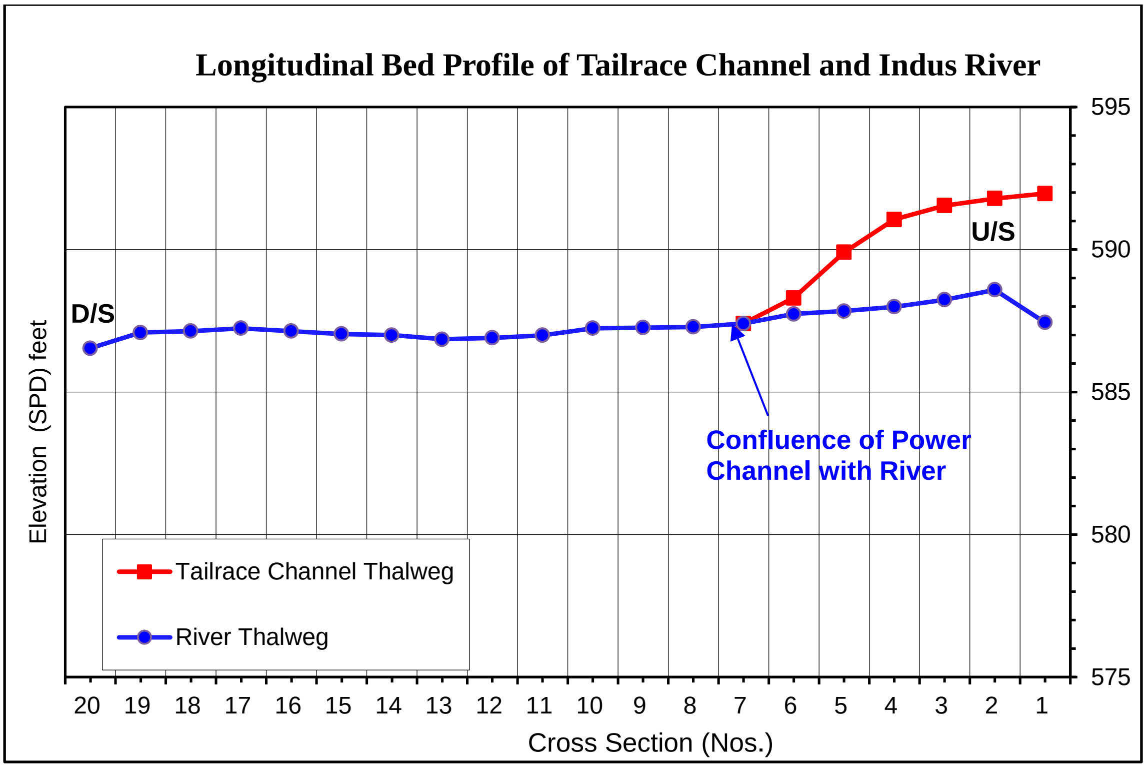

Fig. 3.5 Longitudinal Bed Profile of Tailrace Channel & Indus River

3.2.2 Flow Data

The daily discharge data of Chashma Barrage was obtained from office of Survey & Hydrology (S&H), Wapda Chashma. The data of Chashma barrage inflows & outflow with respect to their upstream and downstream gauges level were collected from 1971 to 2013 years to check the flow pattern. Also, design discharge for Tailrace channel & Maximum discharge passes downstream of the barrage was also considered for study.

3.3 SETTING UP OF HEC-RAS MODEL

In the light of geometric and flow data, different scenarios of Chashma Tailrace Channel were studied and finalized. After finalization of scenarios, it is important to develop HEC-RAS model according to each scenario. Details of setting up of HEC-RAS model is as under:

3.3.1 In Put Data for Hec-Ras Model

It comprises of cross-sections data with downstream reach lengths, downstream boundary conditions, flow data & Manning's n values.

3.3.2 River System Schematic

The schematic defined in the HEC-RAS shows the simplified plan view and locations of the cross sections. The setting up of the model was carried out by considering the downstream of Chashma barrage and Tailrace channel.

3.3.3 Geometric Data

The basic geometry data consists of establishing the connectivity of the river system (River System Schematic); cross-sections data with downstream reach lengths, stream junctions information and Manning's n values.

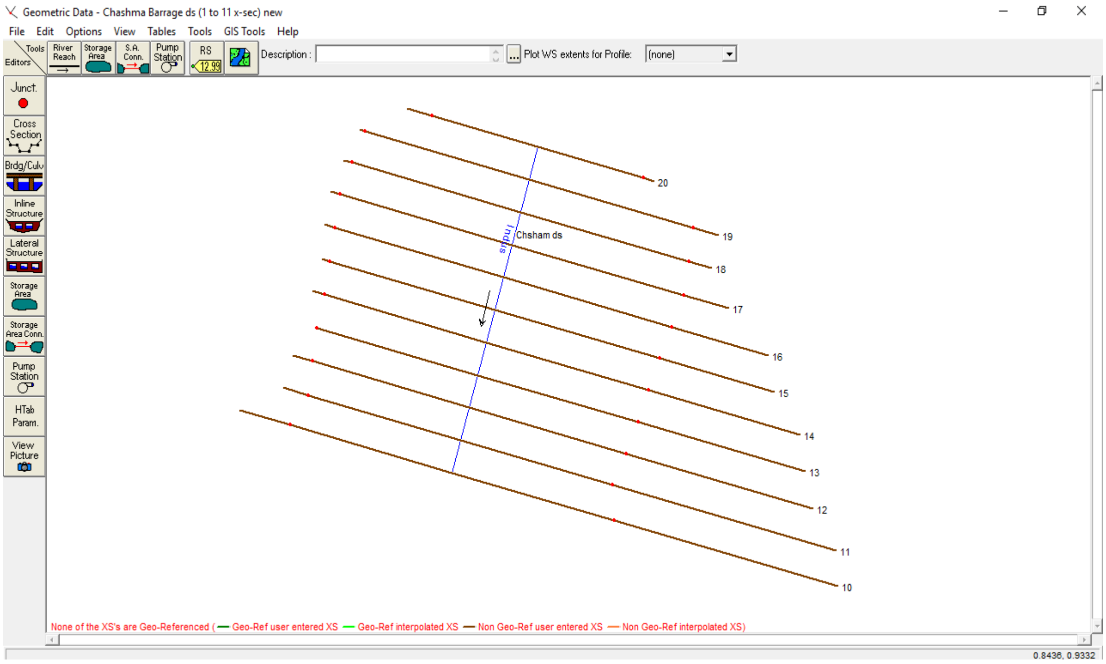

Fig. 3.6 Schematic Diagram Showing Indus River X-Sections in HEC-RAS

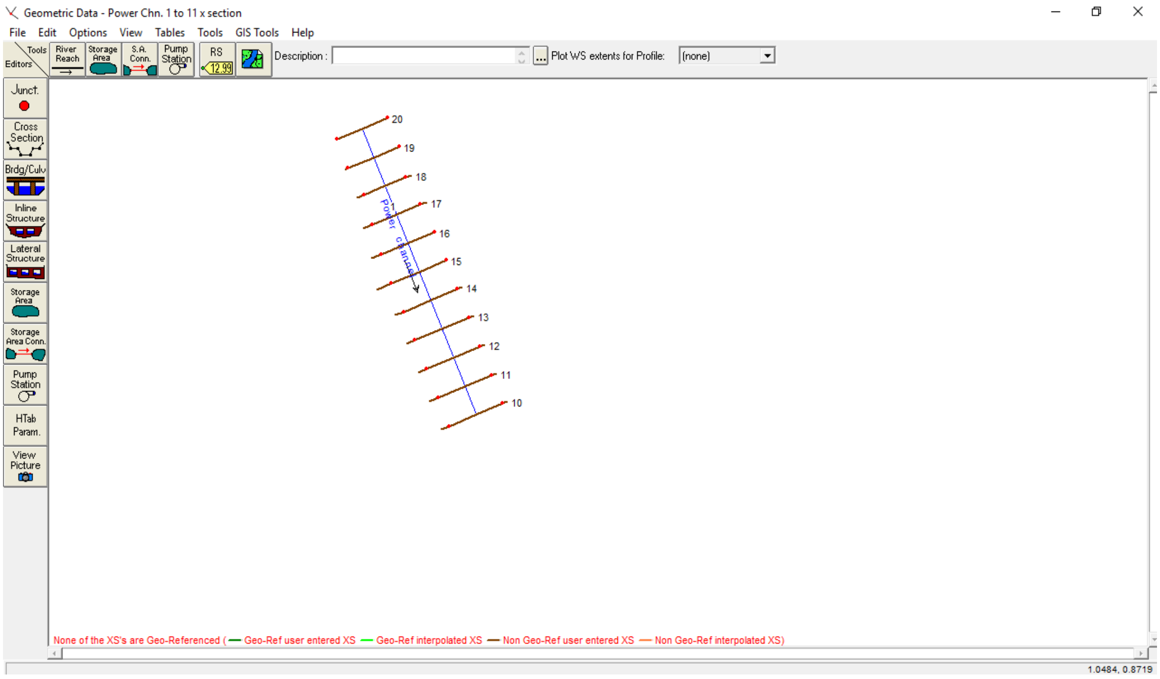

Fig. 3.7 Schematic Diagram showing Tailrace X-Sections in HEC-RAS Model

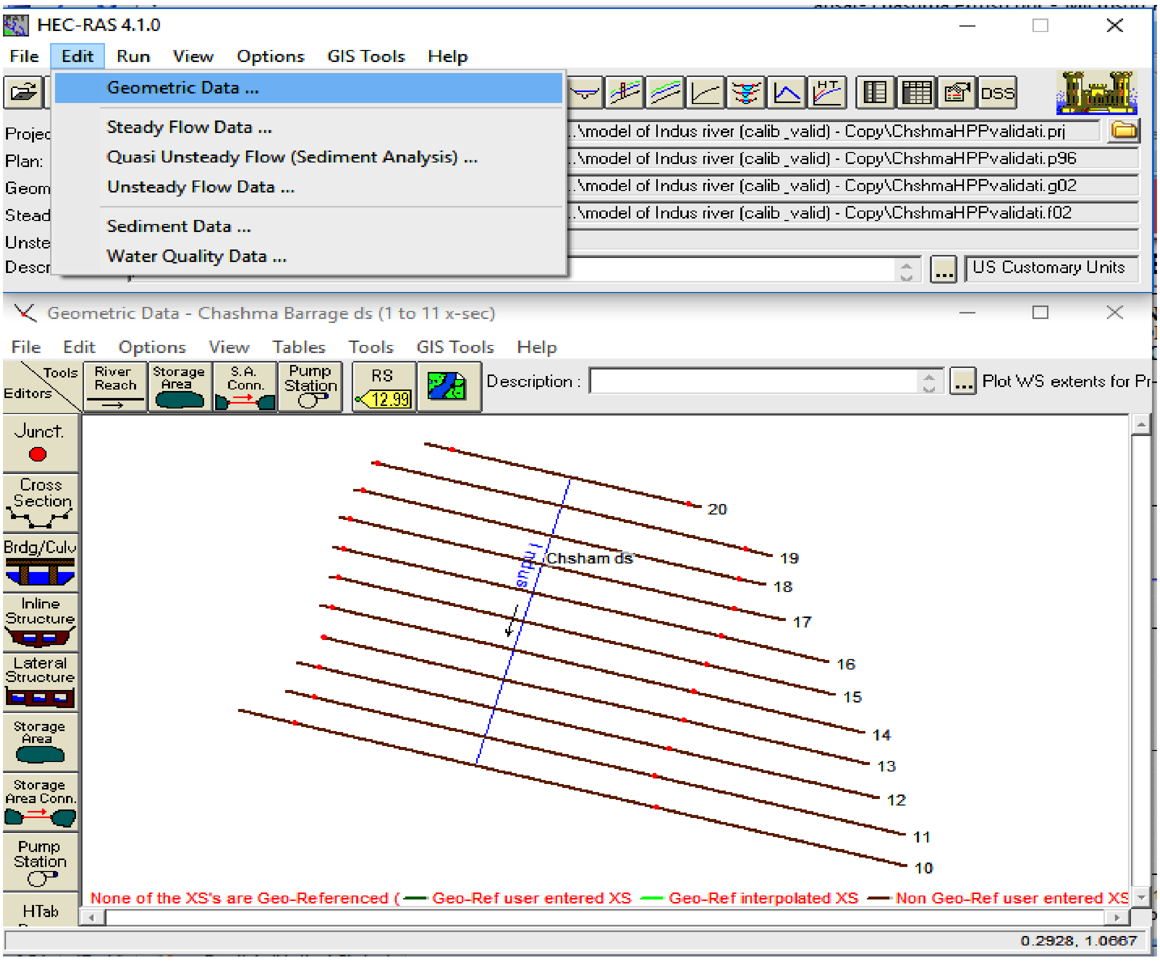

Fig. 3.8 Cross-Section Data in HEC-RAS

3.3.4 Cross-Sections

Boundary geometry for the analysis of flow in the river was specified in terms of ground surface profiles (cross sections) and the measured distance between these (reach lengths at each cross-section). The cross sectional data of Tailrace Channel was entered in HEC-RAS by the cross sectional data editor. The data entered into the cross section data editor comprises river station information, elevation and demarcation of main channel bank station. Downstream reach lengths (i.e., the distance up to the next downstream cross section.) for main channel, left over bank and right over bank and Manning’s roughness coefficient (both vertical and horizontal variation of n values were considered).

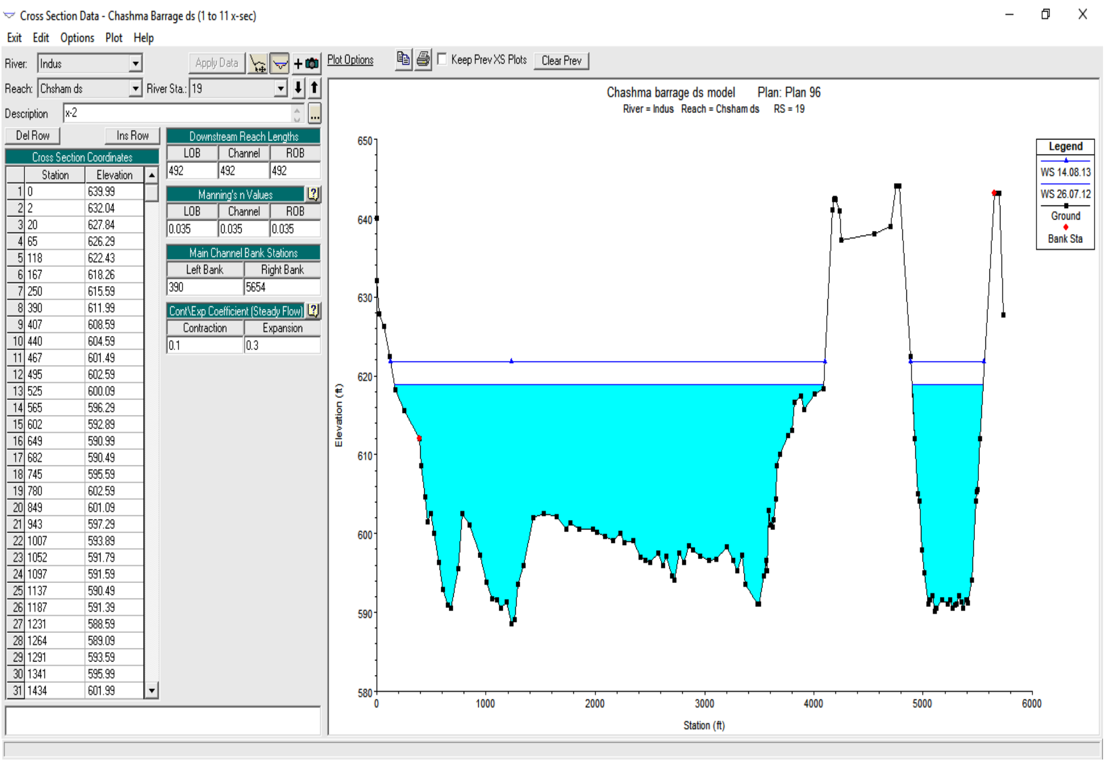

Fig. 3.9 Longitudinal Profile by HEC RAS

3.3.5 Cross Section Locations

The HEC-RAS model requires cross sections data with downstream reach length. In the software, there is geometry data editor which is used to incorporate the cross sectional data of the specified reach in the model. Along with the cross section data, other information like reach length that is distance between two consecutive cross sections, left and right over bank, Manning' s ‘n’ value for all cross sections were entered in data editor.

3.3.6 Boundary Conditions

Like all flow models have few conditions to control the parameters of the model, which are known as boundary conditions. There are different options in HEC-RAS such as boundary conditions like upstream boundary conditions and downstream boundary conditions. Slope, Normal depth, Stage time series and Rating curve may be provided as the downstream boundary condition. In sub critical flow regime; boundary conditions are only necessary at the downstream ends of the river. There are four types of boundary conditions in HEC RAS and model was calculated by using normal depth as downstream boundary condition. Downstream boundary condition normal depth was prescribed as downstream boundary condition with friction slope equal to the river general bed slop at the downstream end.

3.3.7 Flow Data

The model was run for the discharges given in Table: 3.1.

Table 3.1 Design & Maximum Discharge of Chashma Hydel Power Project

Description	Discharge (Cusecs)	Remarks
Discharge for Tailrace channel	84,755	Design discharge
Discharge for Indus river downstream of the barrage	5,64,897	Flood discharge

3.3.8 Model Calibration

After setting up of HEC-RAS model, calibration was done by comparing HEC-RAS model simulation results and existing/observed data. Model calibration is a process of optimizing or systematically adjusting model parameter values to get a set of parameters which provides the best estimate of the observed values. The parameters that are adjusted to calibrate the model are called “calibrating parameters”. The observed difference in model’s output and existing level was adjusted by changing the model parameters such as bed slope, cross section & manning’s coefficient etc. The computer model was first calibrated for the maximum recorded flood of Indus River in 2012 that was 3,87,844 Cusecs. The barrage downstream water level observed by the project office was 619.55 ft and model calibrating parameters i.e bed slope, cross section & Manning’s coefficients were adjusted to get the close value of tailrace water level from HEC-RAS.

3.3.9 Model Validation

Validation is the process of determining the degree to which a simulation model and its associated data are an accurate representation of the observed value from the perspective of the intended uses of the model. Validation provides confidence in the modeling results when calibrated model is used for simulating outside the measured period or when model is used for predicting for future change scenarios. After calibration, validation process was carried out to check and verify the model output if it operated effectively and performed with acceptable capacity. This was done by carrying out trial analysis and comparison of observed data sets and model outputs obtained under known conditions. For validation, model calibrating parameters were not altered and given discharge of 5,64,897 Cusecs (recorded flood of Indus River in 2013) with known barrage downstream water level was used. The result obtained by this discharge is compared by the observed data collected by the project office i.e 622.87 ft.

3.4 SCENARIO MODELLING

The different scenarios were developed in order to check the variation of head for Tailrace Channel of Chashma Hydropower Project. Scenarios comprise different types of channel lining materials and extension of Tailrace channel.

Following Scenarios were developed using HEC-RAS model.

1. Scenario–1 (Change of slope & Manning's value 'n' for Tailrace Channel)

2. Scenario–2 (Extension of Tailrace Channel/Divide Wall)

3.4.1 Scenario–1(Change of Slope & Manning's 'n' value for Tailrace Channel)

The Scenario-1 consists of numerical modeling of Tailrace channel of Chashma Hydropower Project using different types of materials i.e grouted stone lined channel and concrete lined channel. Furthermore, slope of the tailrace channel was also changed as shown in Figure: 3.10. The hydraulic behavior of different types of channel lining was studied using HEC-RAS model by changing the Manning's value 'n' according to type of lining. Following proposals were developed for modeling of Tailrace channel:

1. PROPOSAL - 1 (Existing Power Channel)

2. PROPOSAL - 2 (Proposed Grouted Stone Lined Power Channel)

3. PROPOSAL - 3 (Proposed Concrete Lined Power Channel)

3.4.1.1 Proposal - 1 (Existing Power Channel)

The 1st proposal for scenario-1; model was run with discharge of 84,755 cusecs for the existing geometry of the x-sections of Tailrace channel with downstream s = 0.00182927 is used for model simulation.

3.4.1.2 Proposal - 2 (Proposed Grouted Stone Lined Power Channel)

The 2nd proposal for scenario-1; model was run with discharge of 84,755 cusecs for change of slope of Tailrace channel with Manning's value n = 0.025 & s = 0.0012804 for grouted stone lined channel is used for model simulation.

3.4.1.3 Proposal - 3 (Proposed Concrete Lined Power Channel)

The 3rd proposal for scenario-l; model was run with discharge of 84,755 Cusecs for change of slope of Tailrace channel with Manning's value n = 0.018 & s = 0.0012804 for concrete lined channel is used for model simulation..

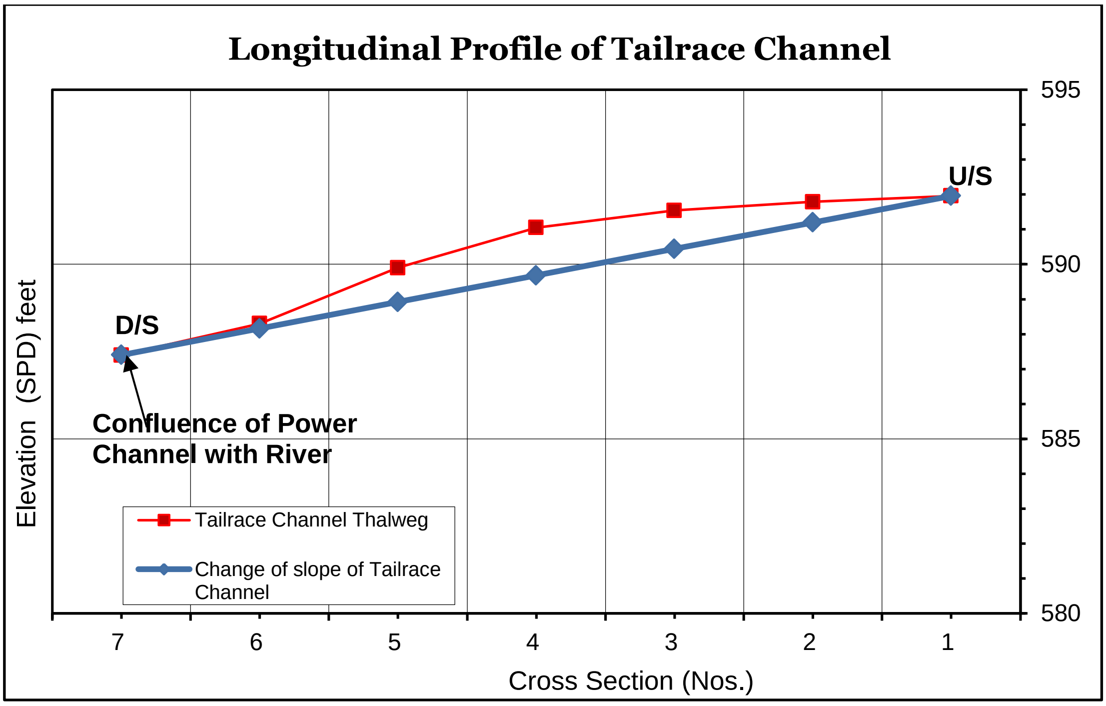

Fig. 3.10 Longitudinal Bed Profile of Existing & changed slope of Tailrace

3.4.2 Scenario - 2 (Extension of Tailrace Channel/Divide Wall)

The Scenario-2 involves numerical modeling for Tailrace channel of Chashma Hydropower Project by extending divide wall further 600m. Extension of Tailrace channel/divide wall was also shown in Figure: 3.11. The model scenarios were simulated using the HEC-RAS model.

Following proposals were developed for modeling:

1. PROPOSAL - 1 (Extension of Tailrace Channel/divide wall upto 11th x-Section)
2. PROPOSAL - 2 (Model of Indus River upto 11th x-Section d/s of Barrage)

3.4.2.1 Proposal - 1 (Extension of Tailrace Channel upto 11th x-Section)

The 1st proposal for scenario-2; the model was run by extending the Tailrace channel from 7th x-section to 11th x-section with downstream slope of Indus River, discharge of 84,755 cusecs and total length of extension was 600 m. By doing this, it is considered that confluence point of power channel and river is extended 600m from 7th cross section to 11th cross section due to extension of divide wall. This scenario was simulated using the HEC-RAS model.

3.4.2.2 Proposal - 2 (Model of Indus River upto 11th x-Section d/s of Barrage)

The 2nd proposal for scenario-2; Indus River model was run by extending the geometry of the model upto 11th cross section, with flood discharges of 2,29,140 Cusecs, 3,87,844 Cusecs & 5,64,897 Cusecs and total length of extension was 600m. This scenario was simulated using the HEC-RAS model. Figure: 3.11 below shows the layout plan of extended Tailrace Channel/divide wall upto 11th cross section.

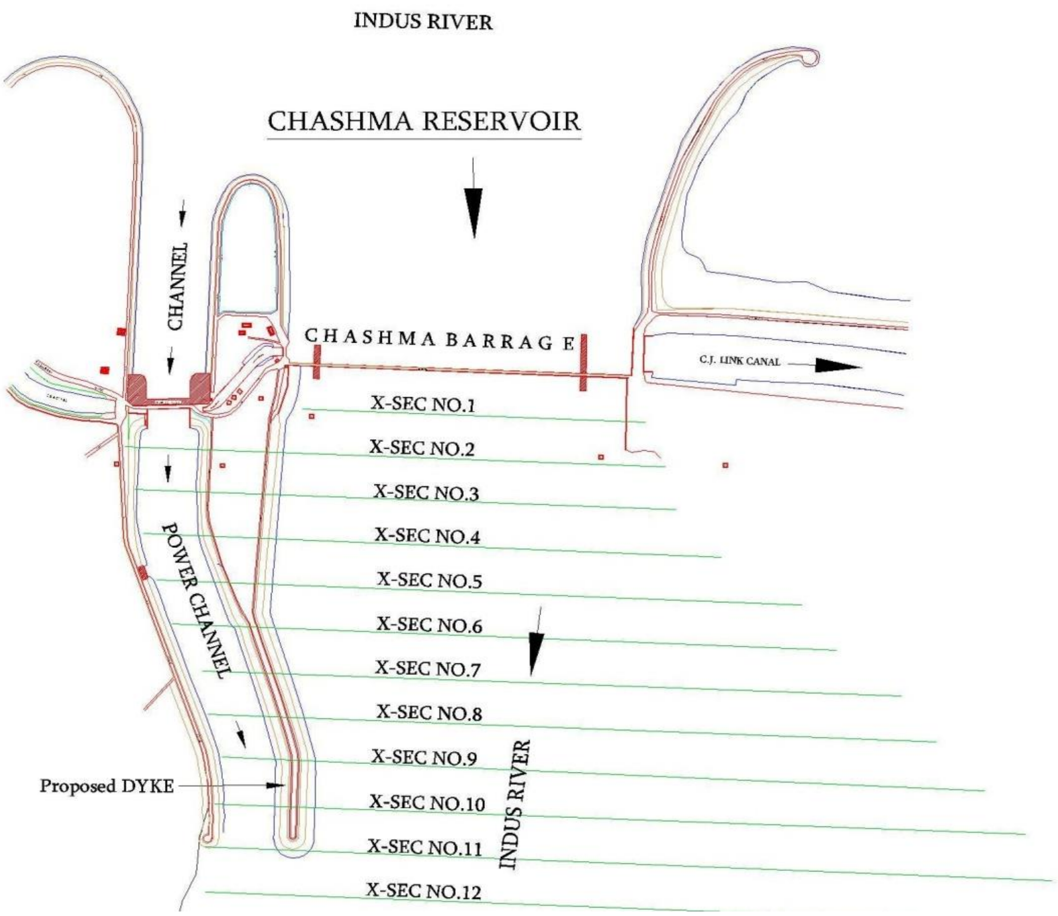

Fig. 3.11 Layout Survey Plan of Extended Tailrace Channel

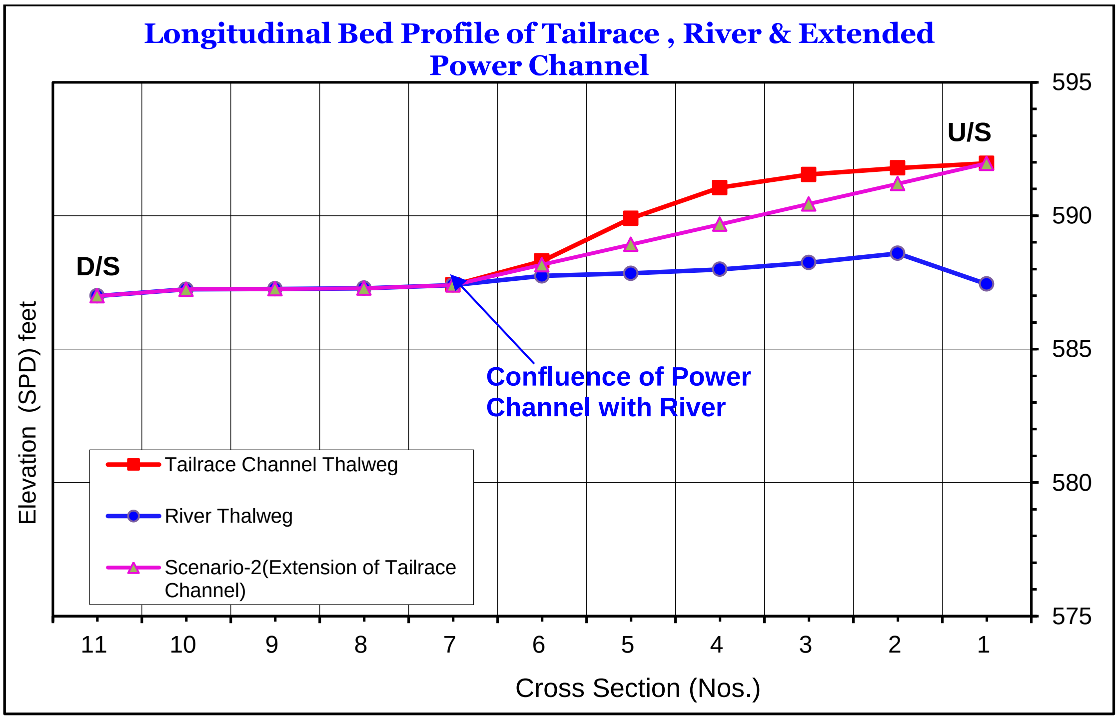

Fig. 3.12 L-section of River, Existing & Extended Tailrace Channel

Figure 3.12 show that Tailrace channel slope is changed from 1 to 7th x-section; also Tailrace channel length is extended further 600 m i.e 7th x-section to 11th x-section. The outputs of the models are discussed in next Chapter–IV (Results and Discussions).

3.5 SUMMARY

The complete methodology adopted during the research work has been explained in this chapter. A comprehensive list of activities including collection of data (geometric and flow data), selection of different scenarios for the Tailrace channel & HEC-RAS modeling has been explained in detail with relevant figures and literature. In the end setting up of HEC-RAS model, calibration for the year 2012, validation for the year 2013 and scenario modeling & parameters taken in the model setup have also been described in details for better understanding.

Chapter IV
RESULTS AND DISCUSSION

4.1 INTRODUCTION

Significance of results and discussion are based on properly compiled results of research report. HEC-RAS model was run according to each scenario, as discussed in previous chapter-III. The output of model in the form of results is prepared in the chapter Results and Discussions.

4.2 SIMULATION OF HEC-RAS MODEL

After input all requisite data, steady flow simulation was performed using HEC-RAS. The Steady flow analysis was carried out after the model was calibrated and validated.

4.3 MODEL CALIBRATION FOR YEAR 2011

Model was calibrated for the recorded flood of Indus River in 2012 i.e 3,87,844 Cusecs. The water level observed by the project office was 619.55 ft, whereas, water level computed from HEC-RAS model is 619.17 ft. The results for the model calibration are shown in Table 4.1.

4.4 MODEL VALIDATION FOR YEAR 2012

The calibrated model was used for validation of the year 2013. Recorded flood of Indus River 5,64,897 Cusecs with known barrage water level is used for validation of the model. The water level obtained by the HEC-RAS model i.e 622.32 ft which was compared with the water level observed by the project office i.e 622.87 ft.

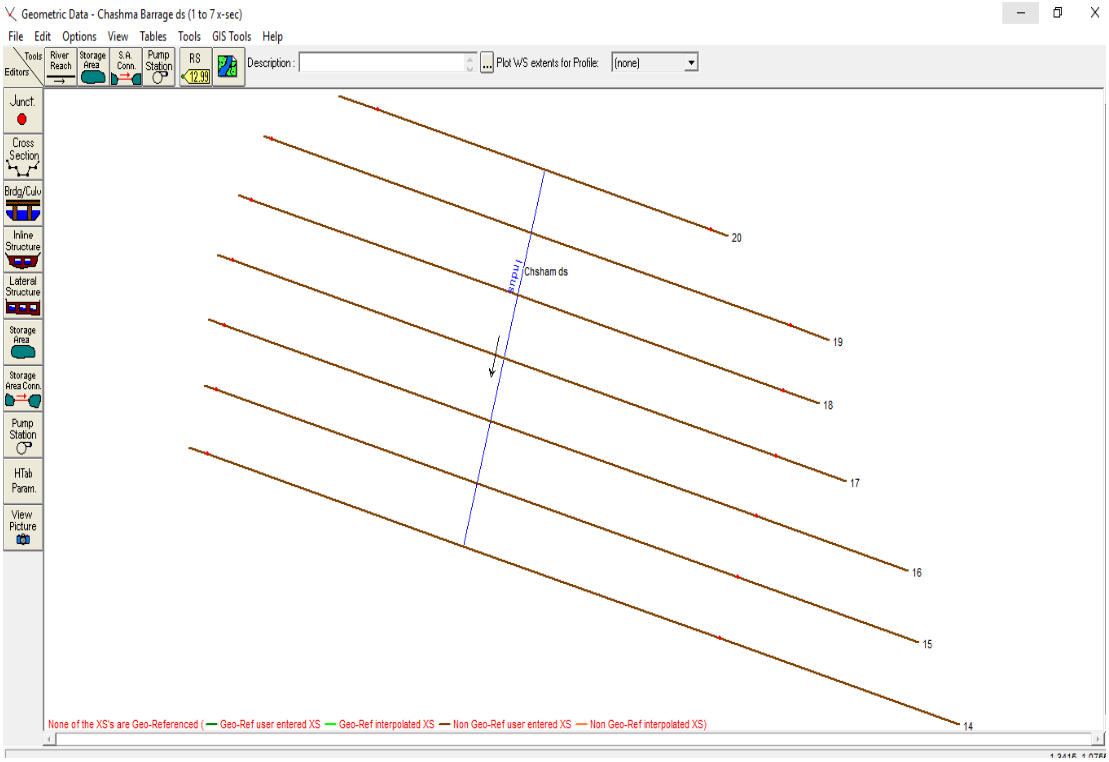

Fig. 4.1 Layout of Indus River Model

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: Plan 99 River: Indus Reach: Chsham

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Chsham ds	20	26.07.12	387844.00	587.44	619.17		619.50	0.000166	4.62	85040.09	3725.85	0.16
Chsham ds	20	14.08.13	564897.00	587.44	622.32		622.86	0.000234	5.93	96769.95	3738.69	0.20
Chsham ds	19	26.07.12	387844.00	588.59	619.10		619.41	0.000204	4.45	87783.16	4585.61	0.18
Chsham ds	19	14.08.13	564897.00	588.59	622.24		622.72	0.000266	5.58	02286.00	4659.71	0.21
Chsham ds	18	26.07.12	387844.00	588.24	619.07		619.30	0.000139	3.90	00265.30	4828.22	0.15
Chsham ds	18	14.08.13	564897.00	588.24	622.20		622.58	0.000189	4.95	15477.50	4885.41	0.17
Chsham ds	17	26.07.12	387844.00	587.99	618.99		619.23	0.000160	3.98	99721.41	5422.41	0.16
Chsham ds	17	14.08.13	564897.00	587.99	622.10		622.48	0.000208	4.97	16700.80	5487.91	0.18
Chsham ds	16	26.07.12	387844.00	587.84	618.96		619.14	0.000120	3.53	15982.20	6433.24	0.14
Chsham ds	16	14.08.13	564897.00	587.84	622.08		622.36	0.000155	4.37	36184.40	6504.23	0.16
Chsham ds	15	26.07.12	387844.00	587.74	618.92		619.08	0.000110	3.33	23427.90	7018.03	0.13
Chsham ds	15	14.08.13	564897.00	587.74	622.03		622.28	0.000140	4.11	45391.90	7062.42	0.15
Chsham ds	14	26.07.12	387844.00	587.40	618.50	609.29	618.95	0.000500	5.45	74446.51	7509.21	0.26
Chsham ds	14	14.08.13	564897.00	587.40	621.56	611.95	622.13	0.000501	6.22	97442.59	7524.50	0.27

Fig. 4.2 HEC- RAS Model Output Table of Indus River Model

Figure 4.2 shows HEC-RAS model Output for the recorded flood of 3,87,844 Cusecs & 5,64,897 Cusecs, dated 26.07.12 & 14.08.13 with water levels as 619.17 ft & 622.32 ft respectively at river station-20.

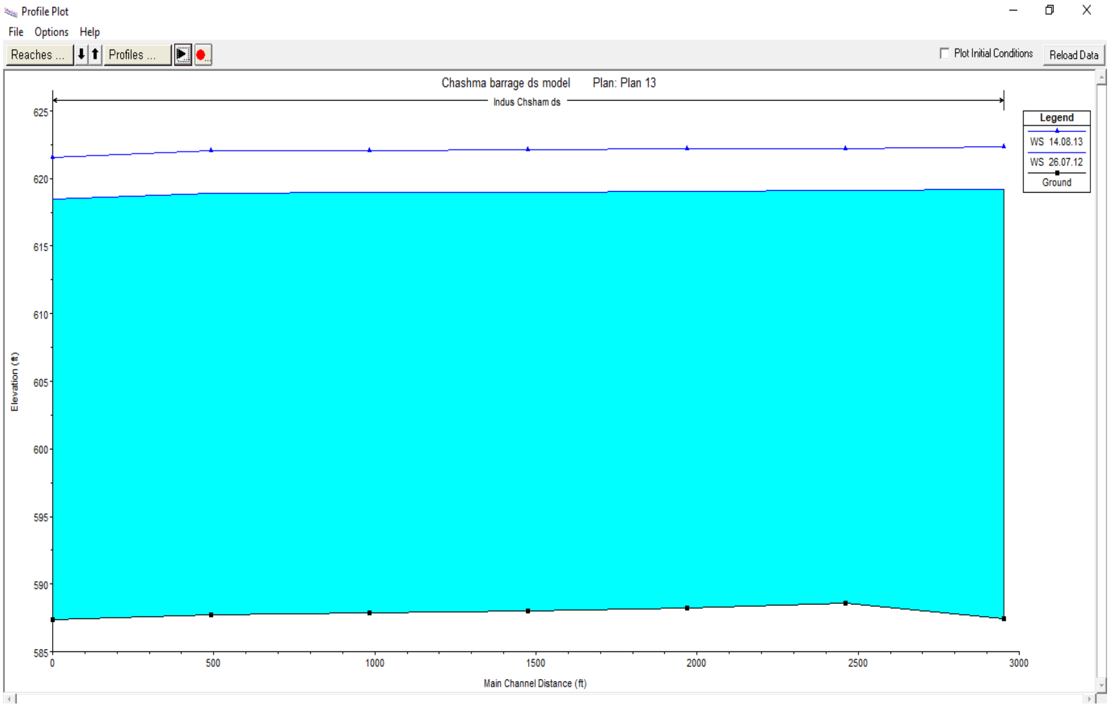

Fig. 4.3 Longitudinal section of Indus River d/s Chashma Barrage

Figure 4.3 shows HEC-RAS model longitudinal section of Indus River downstream of Chashma barrage. The summary of the results for model validation is given in Table 4.1.

Table 4.1 Summary of the results for Model Calibration & Validation

Reach	**River Station**	**Profile**	**Q (Cusecs)**	**W. Surface Elevation from HEC-RAS (ft)**	**W. Surface Elevation Supplied by Project office (ft)**	**Difference (ft)**
1.	20	26.07.12	3,87,844	619.17	619.55	0.38
2.	20	14.08.13	5,64,897	622.32	622.87	0.55

Table 4.1 is the summary of results for model calibration & validation. The table shows the difference between the water level provided by the Project Office and water level computed from the HEC-RAS. Difference of 0.38 ft for model calibration and 0.55 ft for model validation & calibration is shown in last column of the Table 4.1.

4.5 RESULTS OF SCENARIO MODELLING

The different scenarios were developed in order to check the variation of head for Chashma Hydropower Project as discussed in methodology Chapter-III. The results of the analysis of different scenarios are explained below using HEC-RAS model.

4.5.1 Scenario–1(Change of Slope & Manning's value 'n' for Tailrace Channel)

The Scenario-1 consists of HEC-RAS modeling of Tailrace channel of Chashma Hydropower project using different types of materials i.e stone lined channel and concrete lined Channel. The results of the 1st scenario of Tailrace channel are given below.

4.5.1.1 Proposal – 1 (Existing Power Channel)

HEC-RAS Model for Scenario-1, Proposal-1; model was run for existing x-sections of Tailrace channel with downstream slope = 0.00182927. The design discharge 84,755 Cusecs was used for simulation, calibration and validation of model with downstream boundary condition of normal depth. The L-section of Tailrace channel as shown in Figure: 4.3 reveals that water level of Tailrace channel is 611.20 ft.

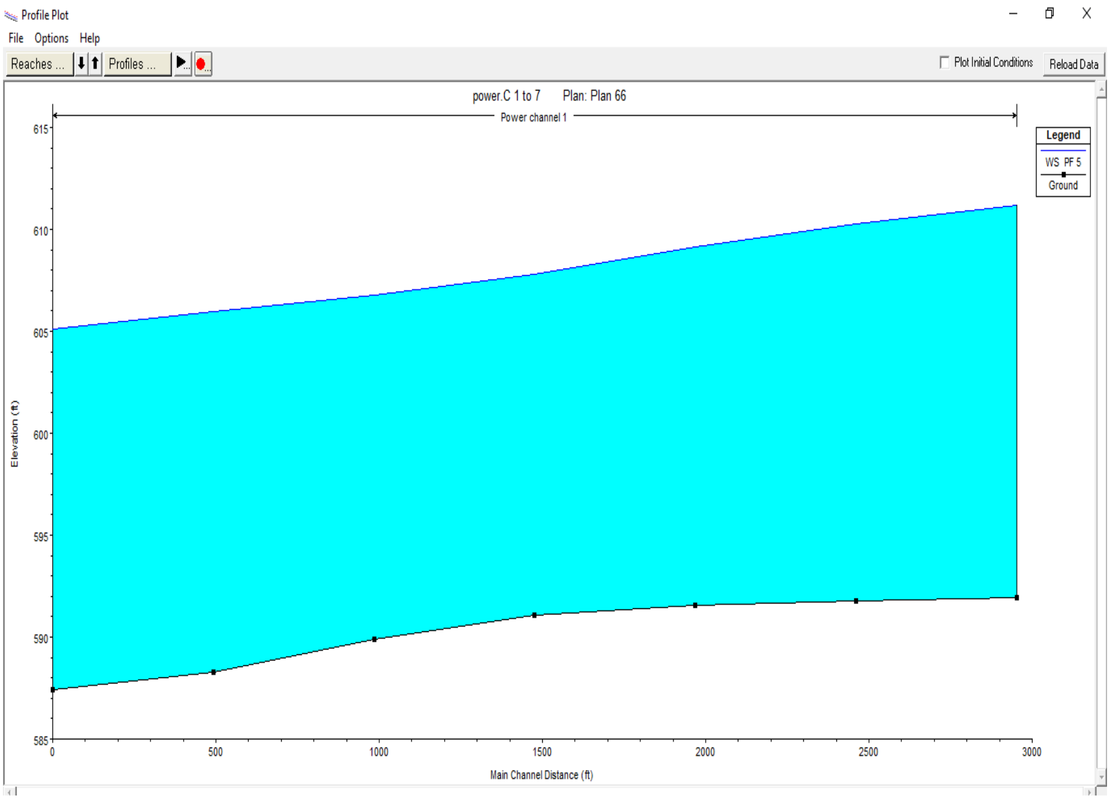

Fig. 4.4 Longitudinal Section of Existing Tailrace Channel

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: Plan 66 River: Power channel Reach: 1 Profile: PF

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	20	PF 5	84755.50	591.96	611.20		613.19	0.001331	11.33	7482.78	461.94	0.50
1	19	PF 5	84755.50	591.79	610.28		612.47	0.001534	11.87	7138.26	458.19	0.53
1	18	PF 5	84755.50	591.54	609.16		611.61	0.001846	12.57	6740.65	457.44	0.58
1	17	PF 5	84755.50	591.05	607.82		610.58	0.002237	13.34	6353.47	456.86	0.63
1	16	PF 5	84755.50	589.90	606.75		609.48	0.002194	13.26	6391.30	456.91	0.62
1	15	PF 5	84755.50	588.30	605.98		608.41	0.001822	12.52	6768.24	457.49	0.57
1	14	PF 5	84755.50	587.40	605.08	600.41	607.52	0.001820	12.52	6770.26	457.49	0.57

Fig. 4.5 HEC- RAS Output Table of Existing Tailrace Channel

Figure 4.5 HEC-RAS model output table of Scenario-1, Proposal-1 shows that water level at river station-20 which is start of the Tailrace channel computed as 611.20 ft and water level at river station-14 which is end of the Tailrace channel computed as 605.08 ft. The summary of the results for Proposal-1 is shown in Table 4.2.

4.5.1.2 Proposal-2 (Proposed Grouted Stone Lined Power Channel)

HEC-RAS Model for Scenario-1, Proposal-2; for this proposal the model was prepared with changed slope of Tailrace channel s = 0.0012804 & Manning's value n = 0.025 (for grouted stone lined channel). The design discharge 84,755 Cusecs was used for simulation, calibration and validation of model with downstream boundary condition of normal depth. The L-section of Tailrace channel as shown in Figure 4.5 reveals that water level of Tailrace channel is 609.45 ft.

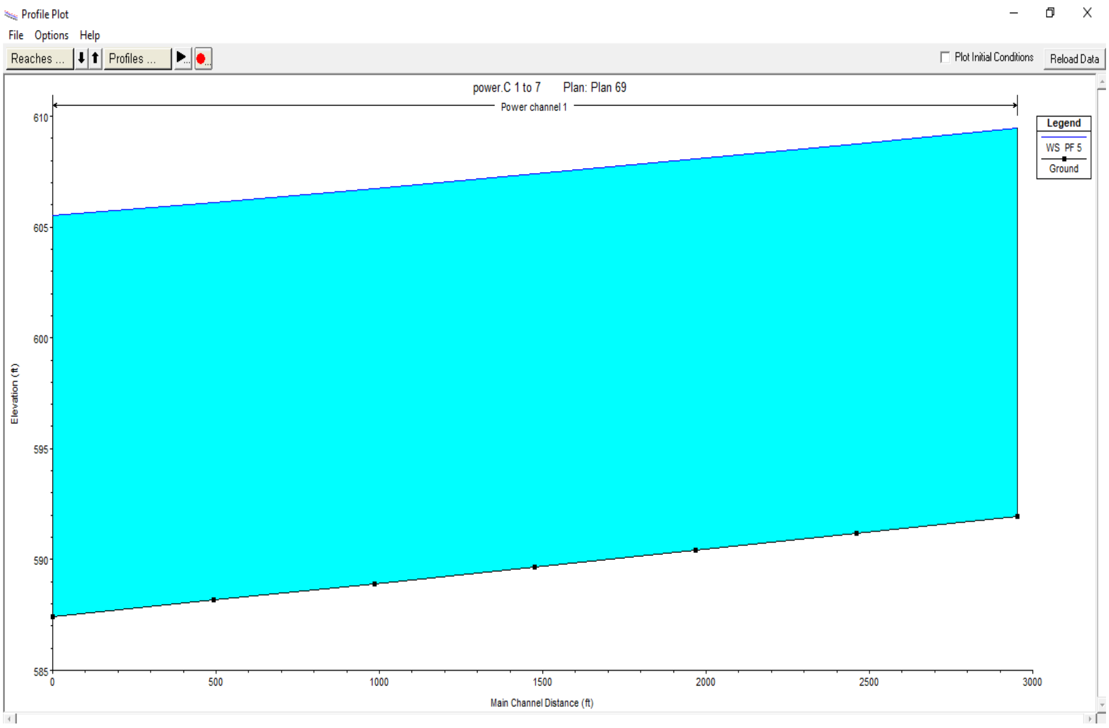

Fig. 4.6 Longitudinal Section of Proposed Grouted Stone Lined Tailrace Channel

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: Plan 69 River: Power channel Reach: 1 Profile: PF 5

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
1	20	PF 5	84755.50	591.96	609.45		612.22	0.001465	13.35	6348.25	434.45	0.62
1	19	PF 5	84755.50	591.20	608.75		611.50	0.001446	13.30	6374.16	434.51	0.61
1	18	PF 5	84755.50	590.44	608.06		610.78	0.001423	13.23	6406.07	434.57	0.61
1	17	PF 5	84755.50	589.68	607.39		610.08	0.001395	13.15	6445.15	434.64	0.60
1	16	PF 5	84755.50	588.92	606.74		609.39	0.001362	13.05	6492.61	434.71	0.60
1	15	PF 5	84755.50	588.16	606.11		608.71	0.001324	12.94	6549.73	434.79	0.59
1	14	PF 5	84755.50	587.40	605.51	600.77	608.06	0.001280	12.81	6617.86	434.89	0.58

Fig. 4.7 HEC-RAS Output Table of Proposed Grouted Stone Lined Channel

Figure 4.7 HEC-RAS model output table of Scenario-1, Proposal-2 shows that water level at river station-20 which is start of the Tailrace channel computed as 609.45 ft and water level at river station-14 which is end of the Tailrace channel computed as 605.51 ft. The summary of the results for Proposal-2 is shown in Table 4.2.

4.5.1.3 Proposal-3 (Proposed Concrete Lined Power Channel)

HEC-RAS Model for Scenario-1, Proposal-3; for this proposal the model was prepared with slope of Tailrace channel s = 0.0012804 & Manning's value n= 0.018 (for Concrete lined channel). The design discharge 84,755 Cusecs was used for simulation, calibration and validation of model with downstream boundary condition of normal depth.

The L-section of Tailrace channel as shown in Figure: 4.7 reveals that water level of Tailrace channel is 607.95 ft.

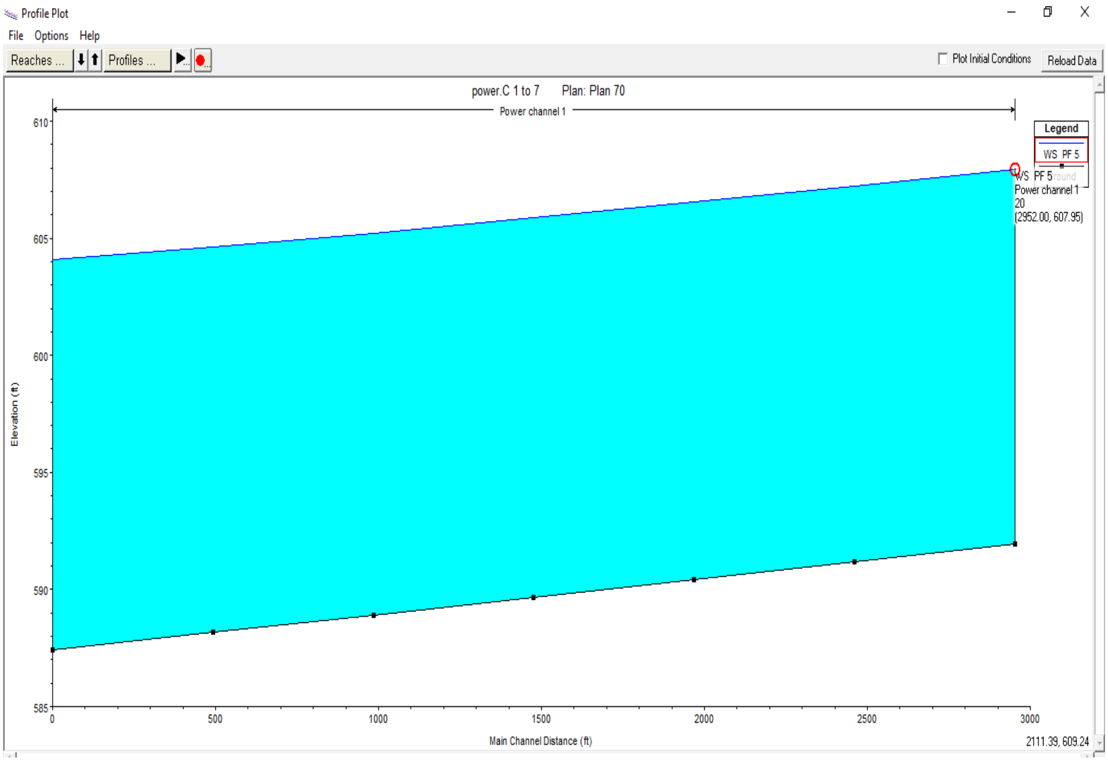

Fig. 4.8 Longitudinal Section of Proposed Concrete Lined Tailrace Channel

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: Plan 70 River: Power channel Reach: 1 Profile: PF 5

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	20	PF 5	84755.50	591.96	607.95	606.55	612.71	0.001508	17.51	4839.63	368.26	0.85
1	19	PF 5	84755.50	591.20	607.23		611.96	0.001492	17.45	4855.95	368.32	0.85
1	18	PF 5	84755.50	590.44	606.53		611.22	0.001469	17.37	4879.02	368.40	0.84
1	17	PF 5	84755.50	589.68	605.86		610.49	0.001439	17.26	4911.34	368.49	0.83
1	16	PF 5	84755.50	588.92	605.22		609.76	0.001398	17.10	4955.42	368.55	0.82
1	15	PF 5	84755.50	588.16	604.62		609.06	0.001345	16.90	5014.05	368.63	0.81
1	14	PF 5	84755.50	587.40	604.07	601.99	608.37	0.001282	16.65	5089.77	368.73	0.79

Fig. 4.9 HEC-RAS Output Table of Proposed Concrete Lined Tailrace Channel

Figure 4.9 Model Output table of Scenario-1, Proposal-3 shows that water level at river station-20 which is start of the Tailrace channel computed as 607.95ft and water level at river station-14 which is end of the Tailrace channel computed as 604.07 ft. The summary of the results for Proposal-3 is shown in Table 4.2.

4.5.1.4 Results of Scenario-1

The results of Scenario-1, proposal 1, 2 & 3 shows considerable decrease in the Tailrace channel water levels i.e. 609.45 ft (Grouted stone lined) and 607.95ft (Concrete lined) from 611.20 ft. Results reveal that with the decrease in water levels of Tailrace channel there is gain in the net Head. Summary of these results are shown in Table 4.2.

Table 4.2 Summary of the results for Scenario-1 (Gain in Net Head)

Sr. No.	River Station No.	Discharge (Q)	Scenario – 1 (Proposals)	W. Surface Elevation from HEC-RAS	Difference of Proposal 1 with 2 & 3	Gain Net Head	
		(Cusecs)		(ft)	(ft)	(ft)	(m)
1.	20	84,755	**Proposal -1 (Existing Power Channel)**	611.20	-	-	-
2.			**Proposal -2 (Proposed Grouted Stone lined Power Channel)**	609.45	611.20 - 609.45 = 1.75	1.75	0.53
3.			**Proposal -3 (Proposed Concrete lined Power Channel)**	607.95	611.20 - 607.95 = 3.25	3.25	0.99

Table 4.2 shows that proposal of changing Tailrace channel into grouted stone lined the gain in the net head is 1.75 ft (0.53 m) and changing Tailrace channel into concrete lined the gain in the net head is 3.25 ft (0.99 m).

4.5.2 Scenario–2 (Extension of Tailrace Channel/Divide wall)

In this scenario 1^{st} model was prepared by extending the Tailrace channel/divide wall further 600m upto 11^{th} cross section of Indus River. Similarly Indus model was prepared with same length of extension, upto 11^{th} cross section and run with flood discharges. The water levels of extended Tailrace model and extended Indus River model were computed using HEC-RAS model and difference/comparison of model water level was done. Following are the results of different Proposals for extension of Tailrace channel and Indus River.

4.5.2.1 Proposal-1(Extension of Tailrace Channel/Divide wall upto 11^{th} x-section)

The 1st proposal for scenario-2; the model was prepared by extending the Tailrace channel from 7^{th} to 11^{th} cross section with downstream slope of Indus River, discharge of 84,755 cusecs and total length of extension was 600 m. This scenario was simulated using the HEC-RAS model.

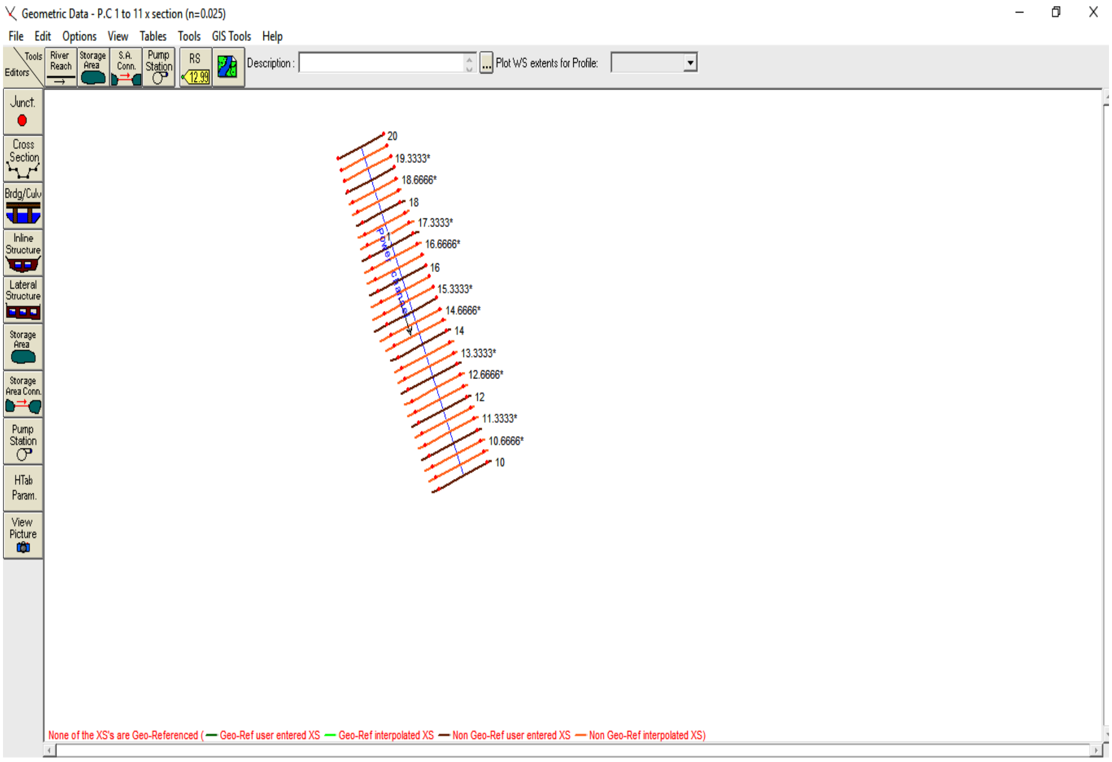

Fig. 4.10 Layout of Extended Tailrace Channel

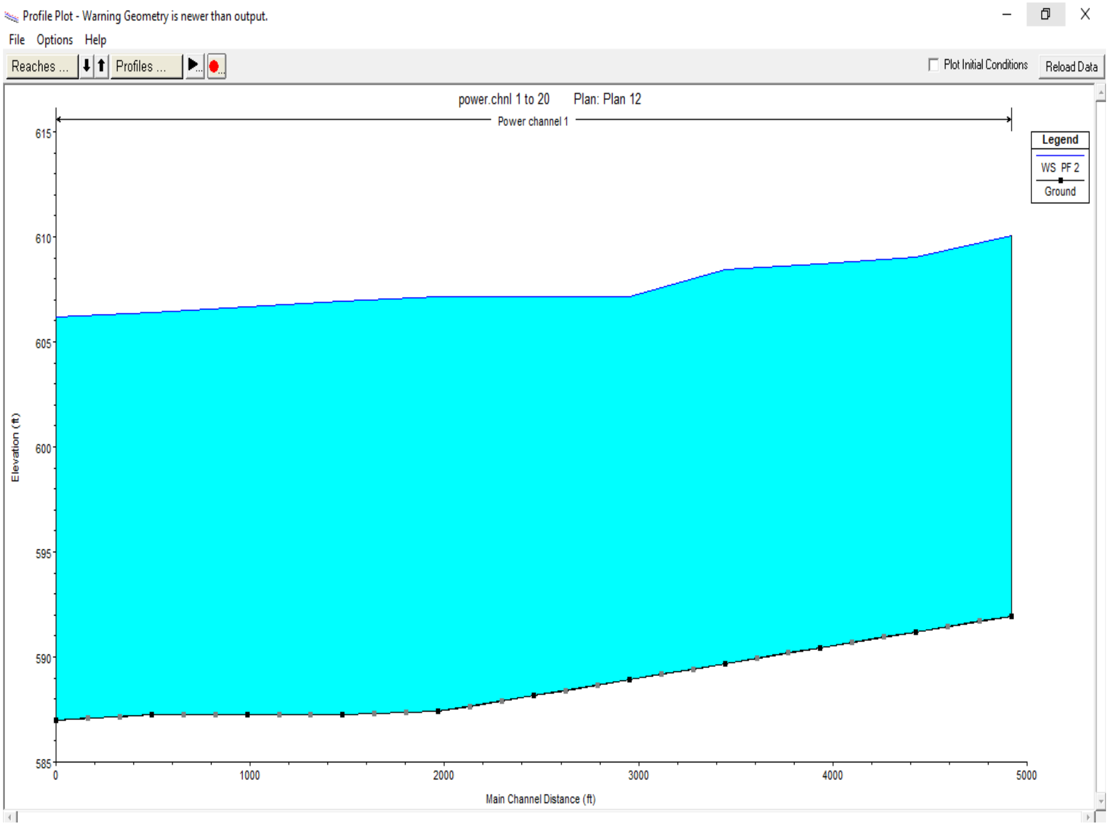

Fig. 4.11 Longitudinal Section of Extended Tailrace Channel

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: Plan 12 River: Power channel Reach: 1 Profile: PF 2

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
1	20	PF 2	84755.00	591.96	610.76		611.93	0.000761	8.70	9742.11	601.39	0.38
1	19.6666*	PF 2	84755.00	591.71	610.31		611.76	0.001049	9.68	8756.71	587.67	0.44
1	19.3333*	PF 2	84755.00	591.45	609.94		611.56	0.001211	10.21	8299.81	572.26	0.47
1	19	PF 2	84755.00	591.20	609.78		611.35	0.001137	10.06	8428.67	566.36	0.46
1	18.6666*	PF 2	84755.00	590.95	608.90		611.06	0.001826	11.79	7186.77	543.43	0.57
1	18.3333*	PF 2	84755.00	590.69	608.59		610.74	0.001994	11.77	7199.31	583.53	0.59
1	18	PF 2	84755.00	590.44	608.93		610.28	0.000970	9.34	9078.95	603.47	0.42
1	17.6666*	PF 2	84755.00	590.19	608.61		610.10	0.001140	9.78	8664.31	607.65	0.46
1	17.3333*	PF 2	84755.00	589.93	608.52		609.89	0.000976	9.40	9016.81	593.60	0.43
1	17	PF 2	84755.00	589.68	608.55		609.68	0.000749	8.55	9912.90	611.84	0.37
1	16.6666*	PF 2	84755.00	589.43	608.21		609.52	0.000970	9.20	9207.65	624.03	0.42
1	16.3333*	PF 2	84755.00	589.17	607.77		609.32	0.001236	9.99	8487.36	612.44	0.47
1	16	PF 2	84755.00	588.92	607.28		609.07	0.001542	10.73	7897.16	603.98	0.52
1	15.6666*	PF 2	84755.00	588.67	607.16		608.79	0.001356	10.25	8268.52	616.32	0.49
1	15.3333*	PF 2	84755.00	588.41	607.14		608.51	0.001099	9.41	9010.68	653.27	0.45
1	15	PF 2	84755.00	588.16	607.19		608.27	0.000820	8.34	10163.40	707.63	0.39
1	14.6666*	PF 2	84755.00	587.91	607.07		608.13	0.000791	8.27	10247.49	702.79	0.38
1	14.3333*	PF 2	84755.00	587.65	607.08		607.95	0.000708	7.49	11312.00	830.34	0.36
1	14	PF 2	84755.00	587.40	607.15		607.79	0.000444	6.45	13142.49	850.36	0.29
1	13.6666*	PF 2	84755.00	587.36	607.07		607.72	0.000447	6.46	13111.71	850.04	0.29
1	13.3333*	PF 2	84755.00	587.32	606.99		607.64	0.000451	6.48	13080.38	849.71	0.29
1	13	PF 2	84755.00	587.28	606.91		607.57	0.000454	6.50	13048.55	849.37	0.29
1	12.6666*	PF 2	84755.00	587.27	606.83		607.49	0.000461	6.53	12988.02	848.74	0.29
1	12.3333*	PF 2	84755.00	587.27	606.75		607.42	0.000469	6.56	12917.03	847.99	0.30
1	12	PF 2	84755.00	587.26	606.67		607.34	0.000476	6.59	12854.12	847.33	0.30
1	11.6666*	PF 2	84755.00	587.25	606.58		607.26	0.000483	6.63	12790.06	846.66	0.30
1	11.3333*	PF 2	84755.00	587.25	606.49		607.18	0.000492	6.67	12715.43	845.88	0.30
1	11	PF 2	84755.00	587.24	606.40		607.10	0.000501	6.70	12648.74	845.17	0.31
1	10.6666*	PF 2	84755.00	587.16	606.32		607.02	0.000501	6.70	12646.75	845.15	0.31
1	10.3333*	PF 2	84755.00	587.07	606.24		606.94	0.000500	6.70	12653.55	845.22	0.31
1	10	PF 2	84755.00	586.99	606.16	597.36	606.85	0.000500	6.70	12651.84	845.21	0.31

Fig. 4.12 HEC-RAS Output Table of Extended Tailrace Channel

Figure 4.12 HEC-RAS Model Output table of Scenario-2, Proposal-1 shows that water level at river station-14 (7^{tt} x-section) computed as 607.15 ft and water level at river station-10 (11^{th} x-section) which is the end of extended Tailrace channel model computed as 606.16 ft. Summary of the water levels for scenario-2, Proposal-1 is shown in Table 4.3.

Table 4.3 Summary of the results for Scenario-2, Proposal-1

Discharge (Cusecs)	**Location**	**Water Surface Elevation (ft)**
84,755	Water level at 7^{th} x-section of Tailrace Channel.	607.15
	Water level at 11th x-section of downstream of barrage.	606.16

4.5.2.2 Proposal -2 (Model of Indus River upto 11^{th} X-Section d/s of Barrage)

The 2nd proposal for scenario-2; Indus River model was run by extending the geometry of the model upto 11^{th} cross section, with flood discharges of 2,29,140 Cusecs, 3,87,844 Cusecs & 5,64,897 Cusecs and total length of extension was 600m.

This scenario was simulated using the HEC-RAS model. The Figures 4.13 & 4.14 showing HEC-RAS model layout & L-Section of extended Indus River model upto 11th x-section downstream of Chashma Barrage.

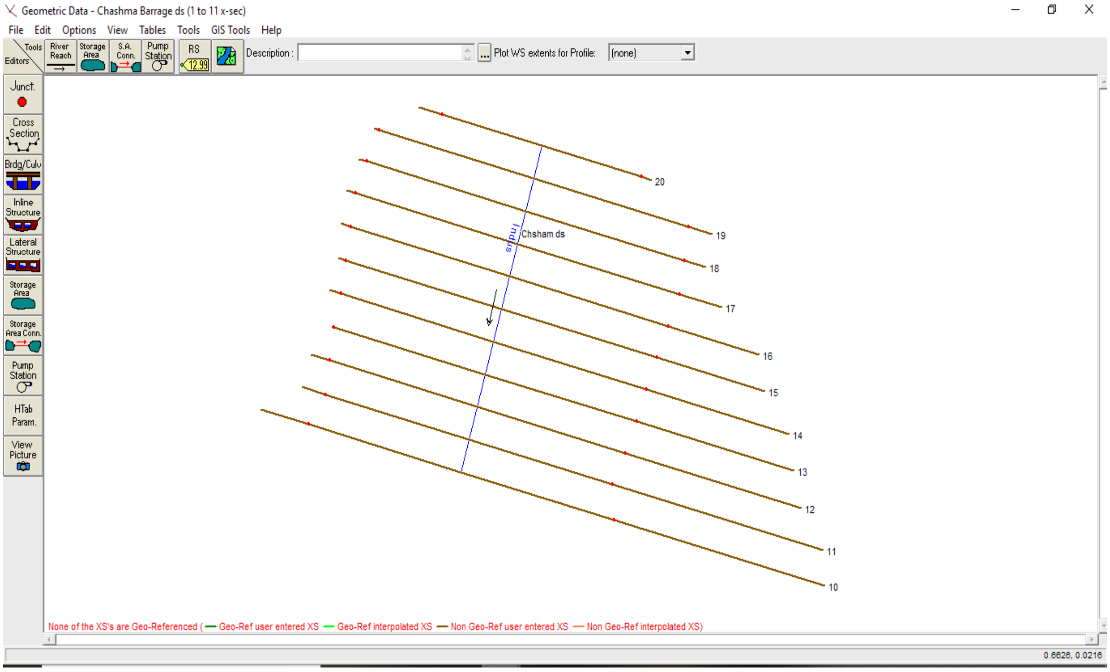

Fig. 4.13 Layout of Extended Indus River Model (1 to 11 X-Section)

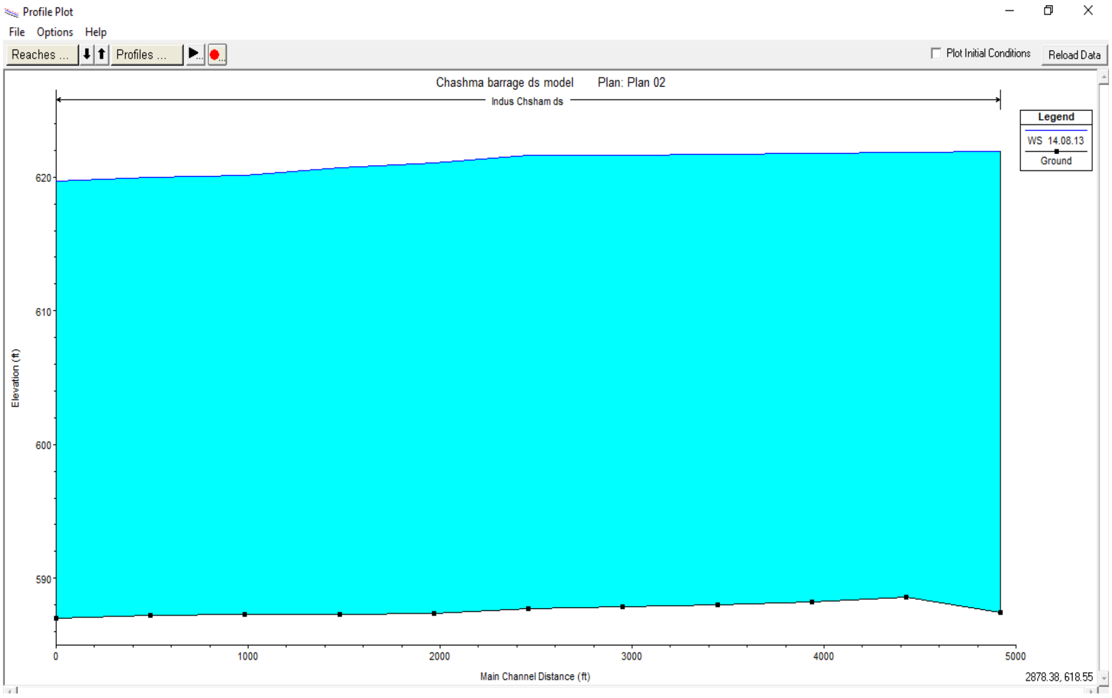

Fig. 4.14 Longitudinal Section of Extended Indus River Model (1 to 11 X-Section)

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: Plan 99 River: Indus Reach: Chs

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Chsham ds	20	25.07.13	229140.00	587.44	615.26		615.43	0.000104	3.26	70545.66	3657.79	0.13
Chsham ds	20	26.07.12	387844.00	587.44	618.91		619.24	0.000172	4.67	84040.80	3724.75	0.17
Chsham ds	20	14.08.13	564897.00	587.44	621.92		622.48	0.000245	6.02	95289.06	3737.07	0.20
Chsham ds	19	25.07.13	229140.00	588.59	615.20		615.37	0.000130	3.25	70702.13	4170.29	0.14
Chsham ds	19	26.07.12	387844.00	588.59	618.83		619.14	0.000213	4.51	86533.23	4579.17	0.18
Chsham ds	19	14.08.13	564897.00	588.59	621.83		622.33	0.000283	5.68	00406.60	4650.17	0.21
Chsham ds	18	25.07.13	229140.00	588.24	615.18		615.30	0.000089	2.81	81968.45	4464.89	0.11
Chsham ds	18	26.07.12	387844.00	588.24	618.79		619.03	0.000145	3.95	98939.88	4823.20	0.15
Chsham ds	18	14.08.13	564897.00	588.24	621.79		622.18	0.000199	5.03	13492.00	4877.98	0.18
Chsham ds	17	25.07.13	229140.00	587.99	615.12		615.25	0.000110	2.93	79301.22	5017.17	0.13
Chsham ds	17	26.07.12	387844.00	587.99	618.71		618.96	0.000168	4.04	98207.06	5419.04	0.16
Chsham ds	17	14.08.13	564897.00	587.99	621.69		622.08	0.000221	5.06	14432.90	5472.92	0.19
Chsham ds	16	25.07.13	229140.00	587.84	615.09		615.20	0.000086	2.64	91320.55	6309.81	0.11
Chsham ds	16	26.07.12	387844.00	587.84	618.68		618.87	0.000126	3.59	14171.80	6426.84	0.14
Chsham ds	16	14.08.13	564897.00	587.84	621.66		621.96	0.000165	4.46	33472.60	6494.75	0.16
Chsham ds	15	25.07.13	229140.00	587.74	615.06		615.15	0.000079	2.51	96676.69	6848.52	0.11
Chsham ds	15	26.07.12	387844.00	587.74	618.63		618.80	0.000115	3.38	21436.20	7005.15	0.13
Chsham ds	15	14.08.13	564897.00	587.74	621.61		621.87	0.000149	4.19	42417.60	7059.00	0.15
Chsham ds	14	25.07.13	229140.00	587.40	614.74		615.05	0.000518	4.49	51119.10	5225.87	0.25
Chsham ds	14	26.07.12	387844.00	587.40	618.18		618.66	0.000544	5.60	72092.33	7507.65	0.27
Chsham ds	14	14.08.13	564897.00	587.40	621.09		621.71	0.000557	6.44	93925.19	7522.17	0.28
Chsham ds	13	25.07.13	229140.00	587.28	614.40		614.76	0.000655	4.84	47539.36	5180.35	0.28
Chsham ds	13	26.07.12	387844.00	587.28	617.82		618.36	0.000657	5.95	67358.15	7051.27	0.29
Chsham ds	13	14.08.13	564897.00	587.28	620.71		621.40	0.000661	6.82	88859.47	7681.71	0.30
Chsham ds	12	25.07.13	229140.00	587.26	613.80		614.32	0.001190	5.78	39779.88	5185.68	0.36
Chsham ds	12	26.07.12	387844.00	587.26	617.25		617.95	0.001001	6.76	58308.98	5789.88	0.35
Chsham ds	12	14.08.13	564897.00	587.26	620.11		620.99	0.000951	7.66	79020.36	8033.83	0.36
Chsham ds	11	25.07.13	229140.00	587.24	613.46		613.83	0.000717	4.94	49097.88	6967.95	0.29
Chsham ds	11	26.07.12	387844.00	587.24	617.06		617.50	0.000589	5.60	76204.72	8145.70	0.28
Chsham ds	11	14.08.13	564897.00	587.24	620.00		620.54	0.000554	6.24	01066.80	8532.96	0.28
Chsham ds	10	25.07.13	229140.00	586.99	613.22	602.10	613.52	0.000500	4.41	52265.12	5340.20	0.25
Chsham ds	10	26.07.12	387844.00	586.99	616.79	606.96	617.24	0.000501	5.40	75416.25	8301.45	0.26
Chsham ds	10	14.08.13	564897.00	586.99	619.73	610.33	620.28	0.000500	6.15	01160.40	8918.98	0.27

Froude number for the main channel.

Fig. 4.15 HEC-RAS Output Table of Extended Indus River upto 11th X-section

The result of HEC-RAS model Scenario-2, Proposal-2; reveals the decrease in the water level with the extension of Tailrace channel of Chashma Hydropower Project. As shown in Figure: 4.15 that discharge of 2,29,140 Cusecs, computes water level at river station-14 (7th x-section) as 614.74 ft and water level at river station-10 (11th x-section) which is the end of extended model for the Indus River model computed as 613.22 ft. The net difference of water levels is 1.52 ft, as shown in Table: 4.4. Also, discharge of 3,87,844 Cusecs gives water level at river station-14 (7^{th} x-section) as 618.18 ft and water level at river station-10 (11^{th} x-section) which is the end of extended model for the Indus River model computed as 616.79 ft. The net difference of decrease in water levels is 1.39 ft. Furthermore, discharge of 5,64,897 Cusecs reveals water level at river station-14 (7^{th} x-section) as 621.09 ft and water level at river station-10 (11^{th} x-section) which is end of the Indus River model computed as 619.73 ft. The net difference of water levels is 1.36 ft, as shown in Table 4.4.

Table 4.4 Summary of the results for Scenario-2, Proposal-2

Discharge (Cusecs)	**Location**	**Water Surface Elevation (ft)**	**Difference**
2,29,140	Water level at 7^{th} cross section start of Barrage	614.74	1.52
	Water level at 11^{th} cross section of downstream of barrage	613.22	
3,87,844	Water level at 7^{th} cross section start of Barrage	618.18	1.39
	Water level at 11^{th} cross section of downstream of barrage	616.79	
5,64,897	Water level at 7^{th} cross section start of Barrage	621.09	1.36
	Water level at 11^{th} cross section of downstream of barrage	619.73	

4.5.2.3 Summary of Results for Scenario-2, Proposal 1 & 2

The comparison of water surface profiles for proposal 1 & 2, of scenario-2, indicates that with discharge of 84,755 Cusecs, water level in the Tailrace channel after extension comes out as 606.16 (Refer Table: 4.3). Whereas, water level with flood discharges of 2,29,140 Cusecs, 3,87,844 Cusecs and 5,64,897 Cusecs are computed as 613.22 ft, 616.79 ft & 619.73 ft respectively (Refer Table: 4.4). The water levels computed by the HEC-RAS model with flood discharges are high as compare to Tailrace channel water levels, resulting/causing in heading up of water in Tailrace. Figure 4.16 below shows the longitudinal bed profile of river & proposed extended Tailrace Channel with respective water levels comes out by HEC-RAS modeling.

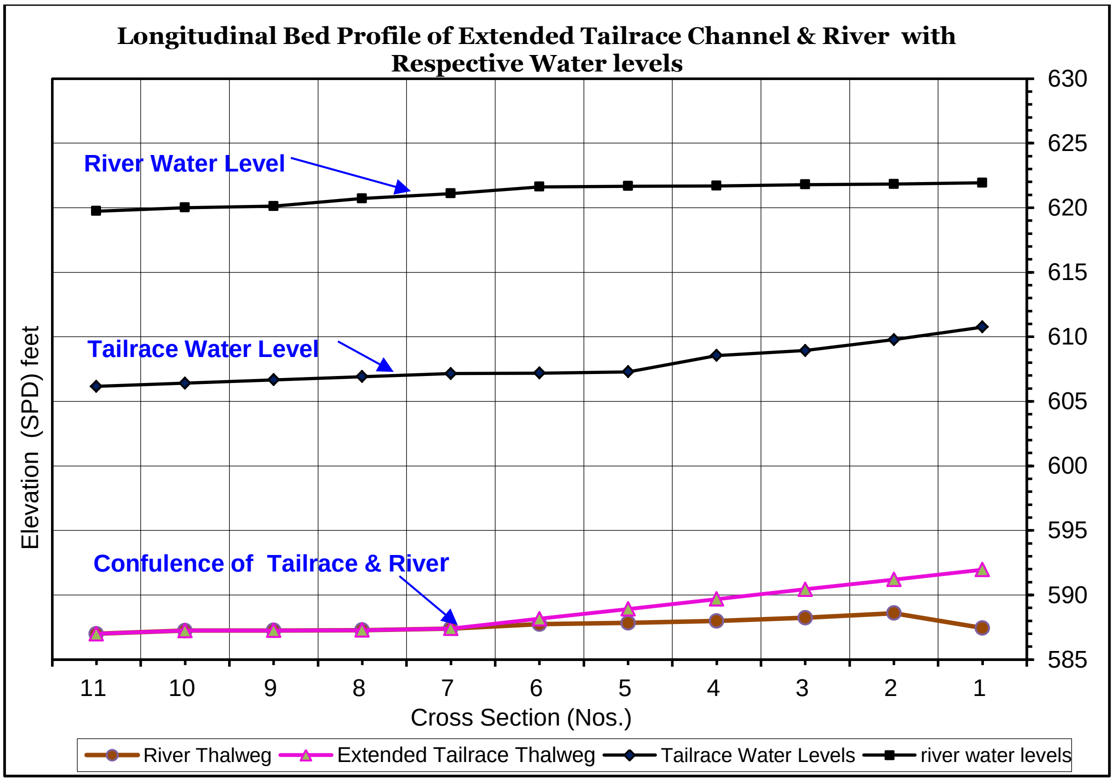

Fig. 4.16 Graph showing L-section of Tailrace & River with respective Water levels

4.6 COMPARISON OF WATER LEVEL WITH & WITHOUT TAILRACE EXTENSION

As discussed in previous section- 4.5.2.3 that after simulation of model for extension of Tailrace channel, the water levels correspond to the values of flood discharges which are also maintained in Tailrace channel. The water levels comes out using HEC-RAS modeling for extension of Tailrace channel are compared with the water levels provided by the Chashma Project Office without extension in the Tailrace Channel. Table 4.5 provides the difference between Project Office water levels without extension and water levels with extension of Tailrace channel.

Table 4.5 Comparison of Tailrace water levels with and without extension

Location	**Discharge (Q)**	**Tailrace Water Surface Elevation**		**Difference in Water Surface Elevations.**	
		Without extension	**With extension**		
	Cusecs	**(ft)**	**(ft)**	**(ft)**	**(m)**
End of extended Tailrace Channel	2,29,140	615.30	613.22	2.08	0.63
	3,87,844	618.56	616.79	1.77	0.54
	5,64,897	621.03	619.73	1.30	0.40

Table: 4.5 shows decrease in the Tailrace water level by 0.63 m (2.08 ft), 0.54 m (1.77 ft) and 0.40 m (1.30 ft), with extension of Tailrace channel.

4.7 COMPARISON OF GAIN NET HEAD WITH & WITHOUT TAILRACE EXTENSION

As discussed in previous section that water levels in Tailrace channel are decreased with the extension of Tailrace channel as shown in Table: 4.5. This decrease in water level results in an increase in the net head. The corresponding net head values with their flood discharges computed by the HEC-RAS model are compared with existing Tailrace channel net head values without extension of Tailrace channel. Table 4.6 gives the summary of net head gain before and after extension of Tailrace channel.

Table 4.6 Summary of Gain in Net Head before & after Extension of Tailrace

Sr. No.	Discharge (Q)	Power House Head Water Level	Tailwater Level provided by Project Office before Extension		Existing Net Head	Tailwater Level by HEC-RAS Model After Extension		Net Head After Extension	Gain Net Head	
	(Cusecs)	(m)	(m)	(ft)	(m)	(m)	(ft)	(m)	(m)	(ft)
1	2,29,140	194.35	187.53	615.30	6.70	186.90	613.22	7.45	0.63	2.08
2	3,87,844	194.46	188.53	618.56	5.92	187.99	616.79	6.47	0.54	1.77
3	5,64,897	194.83	189.28	621.03	5.55	188.88	619.73	5.95	0.40	1.30

Table 4.6 showing existing net head of Tailrace channel before extension is 6.70 m, 5.92 m & 5.55 m, with discharges of 2,29,140 Cusecs, 3,87,844 Cusecs and 5,64,897 Cusecs respectively. Similarly, the net head values computed after extension in the Tailrace Channel using HEC-RAS model of Tailrace channel comes out as 7.45 m, 6.47 m & 5.95m with same said discharges, and gain in net Head is 0.63 m, 0.54 m and 0.40 m, as shown in Table 4.6.

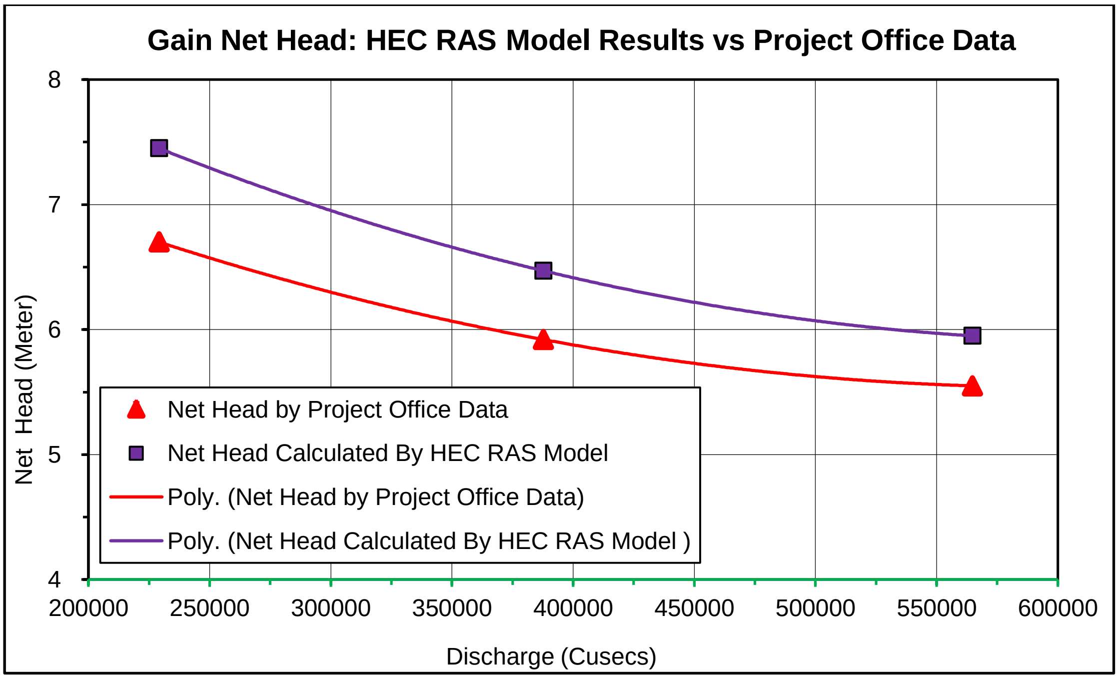

Fig. 4.17 Graph Showing Gain in Net Head after Extension of Tailrace

Figure 4.16 showing correlation between existing (provided by project office) and proposed gain in net head (calculated by HEC-RAS model) with increase in discharge and it shows linearly proportional trend with a steady decrease in net head with increasing discharge. Flood discharges of 2,29,140 Cusecs, 3,87,844 Cusecs and 5,64,897 Cusecs indicate gain in net head of 0.63 m, 0.54 m & 0.40 m respectively.

Chapter V
CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSION

Following conclusions are drawn from the study:

- The Existing Tailrace Channel when changed into Grouted **Stone Lined** Channel, it reveals 1.75 ft (0.53 m) gain in net Head and a subsequent increase in power generation of 10.68 MW.

- The Existing Tailrace channel when changed into **Concrete Lined** Channel, it reveals 3.25 ft (0.99 m) gain in net Head and a subsequent increase in power generation of 19.84 MW.

- Extension of the Tailrace Channel of 600 m with Flood discharges of 2,29,140, 3,87,844 and 5,64,897 Cusecs shows a respective gain in net head of 2.08 ft (0.63m), 1.77 ft (0.54m) & 1.30 ft (0.40m). This also depicts a corresponding increase in Power generation of 12.69 MW, 10.81 MW and 7.94 MW respectively.

5.2 RECOMMENDATIONS

- Existing Tailrace Channel may be transformed to a more hydraulically efficient Concrete Lined Channel as it shows marked improvement in the gain in net Head and subsequent increase in Power Generation.

- Tailrace Channel/Divide wall may also be extended as it shows considerable gain in net Head during high flows resulting increase in Power Generation.

- Economic and financial analysis needs to be carried out for the change in slope & extension of Tailrace Channel to check the economic viability.

- Physical hydraulic modelling regarding change in slope, Concrete lining & extension of Tailrace Channel may also be done for comparison with the results of this study.

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