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Numerical Simulation of Backwater Effects by a Downstream Dam using HEC-RAS: A case of SunKoshi- Marin Diversion Headworks, Nepal

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Abstract

This study evaluates the scenario of the flood backwater impacts on upstream of the Sunkoshi-Marin headworks. The design flood and hydrological analysis were carried out based on the stream flow data from 1968 to 2015 of Khurkot station. Probabilistic method was used to estimate design flood discharge and check flood values for 1000 and 10,000 years return periods frequency. Estimated design floods and check floods were 12,328 and 15,630 m3/s discharge, respectively. Numerical simulation of backwater effects was carried out in three different cases- (i) headworks without affecting existing road (ii) headworks affecting existing road and (iii) headworks with inline structures using HEC-RAS. Water surface profiles estimation and backwater inundation map were generated for a 1000-year return period flood. In the case (i) scenario simulated upstream and downstream water surfaces were 478.10 m and 477.22 m respectively. In case (ii) those values were found 471.75m and 470.64 m respectively. Similarly, in case (iii) scenario upstream and downstream water surfaces were found at 475.79 m and 471.39 m respectively. The total inundated area including the river waterway was 340.89 ha with the extension up to 6 km in the Tamakoshi side and 8 km in the Sunkoshi side. The net inundation area excluding the river waterway was estimated at 216.92 ha. The inundated areas lie within three rural municipalities, namely; Sunkoshi, Khadadevi, and Manthali. Due to backwater inundation recommended length of the realigned section of the BP highway is about 1.3 km.

1. Introduction

Water dams act as effective and important tools for integrated water resource management and development. Multi-purpose dams including irrigation, navigation, hydropower, water supply, and flood control are applicable to worldwide rivers (Emamgholizadeh et al., 2018; ICOLD, 2017). Since many years researchers are interested on the analysis of backwater effects on upstream areas caused by extreme flooding due to the placement of a downstream dam in large rivers. The construction of a dam causes water surface elevation (WSE) thus influencing the upstream of the river. This phenomenon alters the hydraulic conditions with an increment of upstream river depth gradually. This bridges quasi-normal flow and standing

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water forming a smooth transition between them (CR and Thatikonda, 2020). This response would affect the river upstream exceeding several hundreds of kilometers of backwater zone in low-slope rivers (Te Chow, 1959).

Dams function as a barrier on rivers and form backwater conditions affecting upstream water surface profile (Liro, 2019; Maselli et al., 2018; Scott et al., 2012; Volke et al., 2019; Yang et al., 2017) Numerous studies have been carried out analyzing backwater effects and raising water level at upstream due to the construction of hydraulic structures like dams (Sheng, 2014).

The upstream flooding caused by the backwater effect depends upon various parameters viz. river morphology, geometry including flow and floodplain characteristics (Teo, 2010). The increment in water surface elevation in upstream regions imposes threats of submergence during flood affecting the longitudinal reach of the river. This study evaluates upstream inundation and backwater effects with the placement of a downstream headwork.

In comparison to 2-D and 3-D models, reduced topographic data and computational time requirement 1-D models are popularly used in generating flood inundation maps (Costabile et al., 2015; Macchione and Viggiani, 2004). However, the 2D unsteady hydraulic model is more applicable in understanding the backwater profile (Costabile et al., 2015; Dasallas et al., 2019; Patel et al., 2017; Scott et al., 2012).

Sunkoshi Marin Diversion Multipurpose Project (SMDMP) is proposed as a run-of-river basin diversion scheme planned and designed mainly to provide irrigation facilities in the Bagamti River Basin. The project aims to augment water at the head reaches of the Bagmati Irrigation Project by diverting river discharge from the Sunkoshi River into the Bagmati River via the Marin Khola, a major tributary of the Bagmati River. The head available as a result of diversion is used to generate 31 MW power by constructing a powerhouse at the toe of the hill on the left bank of Marin Khola. Diversion headworks are realized in the form of a barrage consisting of a low-head dam with the number of gates to control the river discharge. Barrage gated spillways are designed to pass design flood of 12,328 m^3/s corresponding to a 1000-year return period and a check flood of 15,630 m^3/s corresponding to a 10,000-year return period. The Barrage is provided with 6 Nos of radial gates with size 15 m wide and 16 m high.

The objective of this study is to compute the backwater levels of the Sunkoshi- Marin diversion headworks and its backwater effect on upstream areas. The scope of the research includes computation of maximum design flood for once in 1000 years and 10,000 years for observed maximum discharge considering successive floods, computation of backwater levels with and without the barrage with 1000 years and 10,000 years flood discharges and estimation the extent of a backwater on the upstream area.

2. Materials and methods

2.1. Study area

The study area lies at elevations between 390 m and 490 m above mean sea level in the Sindhuli District of Bagmati Province of Nepal. Geographically, the headworks and powerhouse lie between latitudes 270 20’ 38.64476” N and 270 15’ 31.5237” N and longitudes 850 59’ 03.90287” E and 850 52’ 29.99232” E. The study area with location map is shown in Figures 1 and 2.

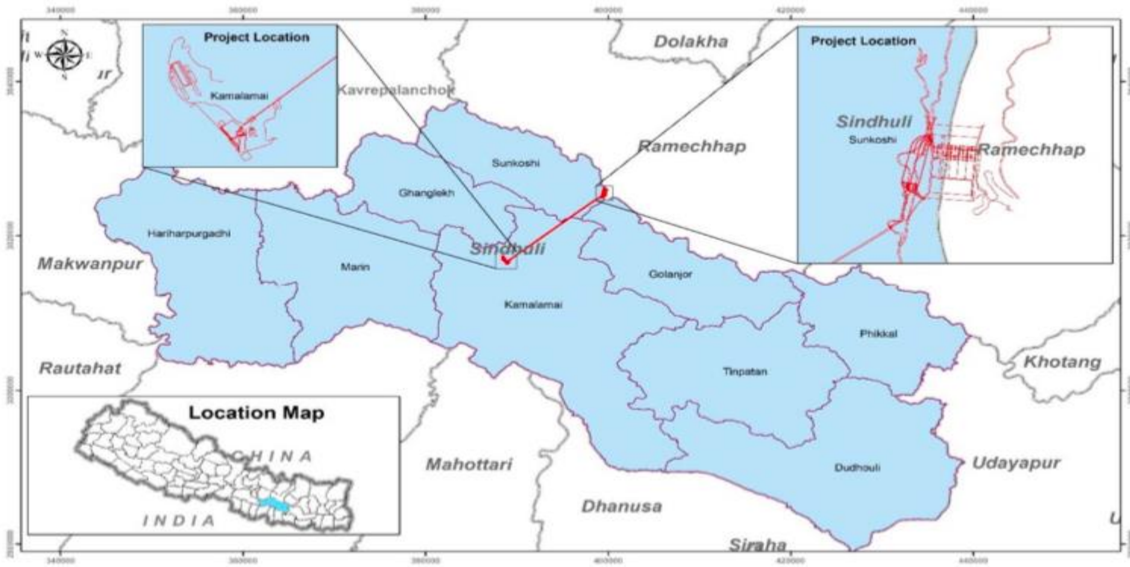

Figure 1. Location map.

Figure 2. Sunkoshi Marin Headworks at Khurkot.

2.2. Catchment Area and Shape

The catchment area up to the headwork site is 10,155 km^2. It is pear-shaped having a dendritic drainage pattern, elongated towards the north, and is shown in Figure 3. It has elevation ranging from EL.355.0 m to about EL.7950 m. Out of the total area, 6761 km^2 lies in Nepal, and the remaining 3394 km^2 lies in China. The project area is divided into three parts on the basis of elevation variations namely as below 3000 m, (3000 -5000) m, and above 5000 m. The area below 3000 m is measured as 4983 km^2, between (3000-5000) m is 2881km^2 and above 5000 m is 2291km^2. Since some part of the catchment lies in the Himalayan regions, the river is perennial in nature and obtains continuous contribution of base flow during the dry season. The study conducted by WECS and DHM has categorized that the catchment area belongs to the Hydrological Region 1 and 3. The

Sunkoshi flows with an average river slope of about 1 in 500 at the headworks area of the project.

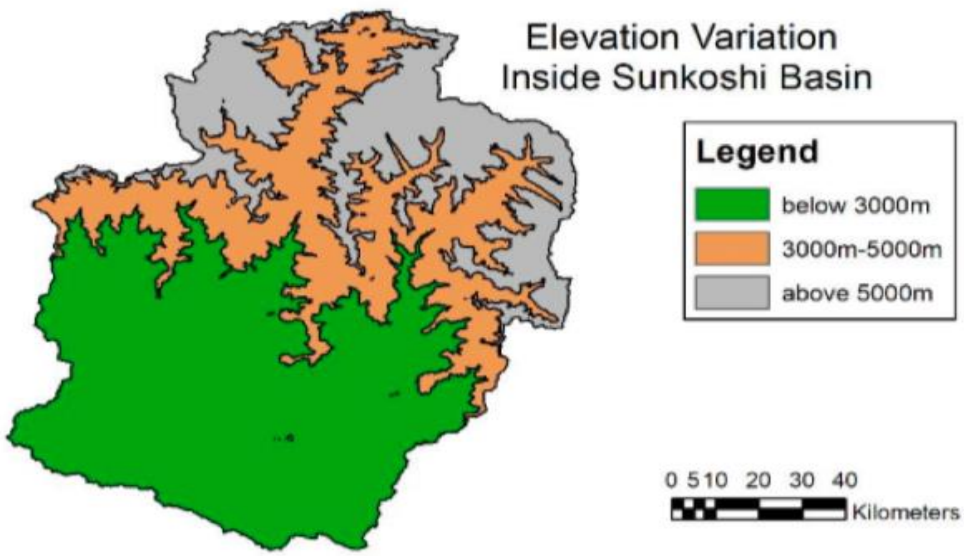

Figure 3. Catchment area.

2.3. Rating Curve for Measured Discharge

There is a gauging station at Khurkot (Station No. 652), 2.00 km downstream of the headworks site of Sunkoshi Marin Diversion Multipurpose Project. Daily average flows, and instantaneous yearly maximum and minimum flows with corresponding water levels are available from this station. For the hydrological analysis, the stream flow data from 1968 to 2015 of Khurkot station has been used. A rating curve was developed from measured data at the gauge station near the intake, which is presented in Figure 4.

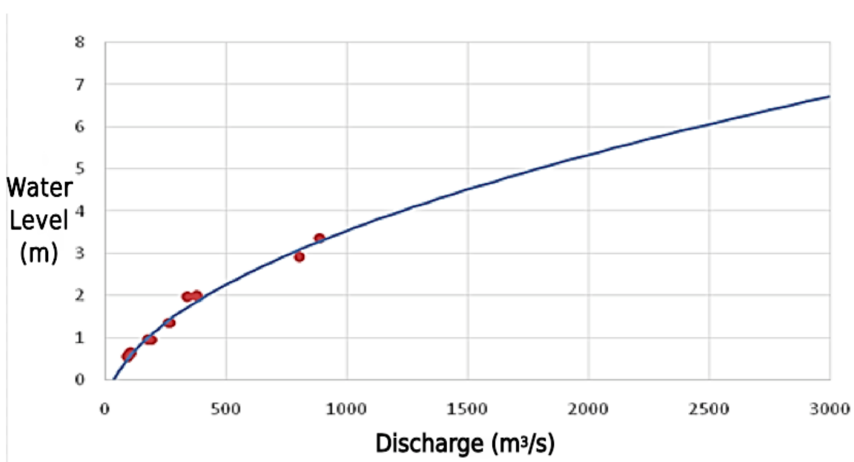

Figure 4. Rating Curve at Gauge Station Near to Proposed Intake.

2.4. Design Flood Estimation

The selection of the design flood estimation method depends upon the availability of data, the importance of structure, and the level of risk to be adhered. Design flood estimation may be carried out on the basis of either event-based or continuous simulation modeling methods (Beven, 2001; Reed and Robson, 1999). Commonly three event-based approaches (a) probabilistic, (b) deterministic, and (c) empirical methods are used at-site design flood estimation (Smithers, 2012; Van der Spuy and Rademeyer, 2016). In the case of availability of adequate length and quality of historical data probabilistic methods may be used for design flood estimation (Cordery and Pilgrim, 2000). Deterministic methods estimate flood event with a correlation of rainfall events lumping all heterogeneous catchment processes into a single process assuming the average condition of the catchment (Rahman et al., 2002). Empirical methods generally relate peak discharge to catchment size with considerations of physiographical and climatological indices (SANRAL, 2013). Hence, a statistical approach of flood frequency analysis is selected for deriving a design flood making use of the available long-term data set of Khurkot station. However, empirical and regional flood frequency analysis (WECS-DHM method) are also carried out for cross-checking and comparison purposes.

2.4.1. Peak Flood Estimation by Statistical Methods

Flood flow records are available at the Khurkot gauging station (2 km downstream from the intake site). The headworks site is the gauged station, so the first attempt for the estimation of design floods was flood frequency analysis from annual instantaneous floods from Khurkot station. To begin with this analysis, data were checked for consistency using computer software to identify the presence of trends and jumps if any. The result of the software is shown below (Table 1).

Table 1. Check for Data Consistency Using Statistical Tools.

Test	99% Confidence Interval	P-value	Remarks
Standard Normal Homogeneity Test	(0.483,0.509)	0.496	Null Hypothesis H_o cannot be rejected
Buishand's test	(0.251,0.273)	0.262	Null Hypothesis H_o cannot be rejected
Pettitt's test	(0.108,0.124)	0.116	Null Hypothesis H_o cannot be rejected

H_0: Homogeneous data
H_a: Date of change in the data

Analysis was done using the Log Pearson Type III distribution, Log Normal distribution, Pearson Type III distribution, Normal distribution and Gumbel distribution. The results obtained for the Sunkoshi Marin Diversion Multipurpose Project based on Statistical methods is enlisted (Table 2, Fig.5).

Table 2. Flood Estimates (m^3/s) in Khurkot Station under different Methods.

Return Period (Yr)	50	100	200	500	1000
Gumbel	8385	9273	10159	11327	12209
Log-Normal	8338	9241	10280	11380	12328
Log-Pearson Type III	8787	9911	11089	12741	14071
Normal	7507	7952	8359	8852	9198
Pearson Type III	8118	8909	9693	10722	11500

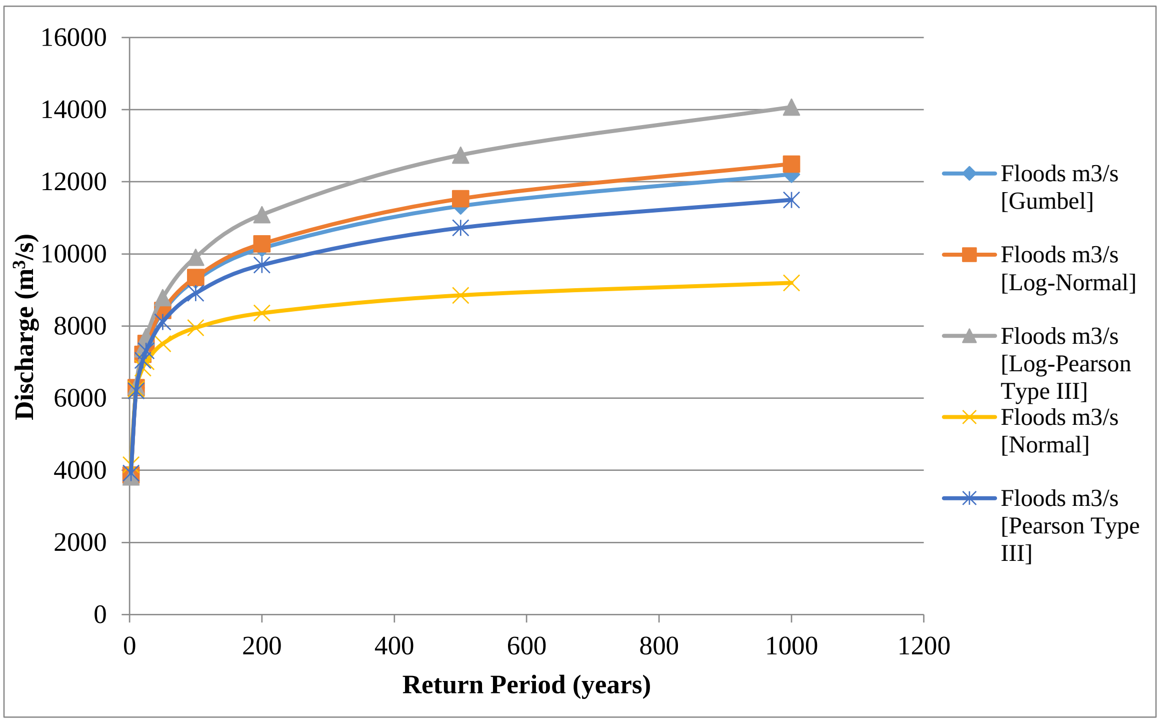

Figure 5. Flood Estimates (m^3/s) in Khurkot Station.

2.4.2. Peak Flood Estimation by Regional Methods

2.4.2.1. WECS/DHM 1990

This study is based on the flow records of DHM primary gauges. The method can be used for any ungauged point in Nepal and requires a catchment area below 5000 m and the average monsoon rainfall over the basin. It gives daily and instantaneous flood peaks for different return periods. It estimates the flood values at ungauged locations in general for the whole of Nepal but it may not give accurate results for particular basins. Flood results by this method were also obtained directly by the software called "HYDEST" (Table 3).

Table 3. Design Flood at Sunkoshi intake by WECS/ DHM.

Return Period (years)	Flood Discharge (m^3/s)	
	Daily	Instantaneous
50	5073	6887
100	5510	7586
200	5945	8291
500	6517	9231
1000	6951	9953
5000	7970	11679
10000	8415	12447

2.4.2.2. Peak Flood Estimation by Empirical Formula

Different empirical formulas Modified Dicken's, Fuller's and Horton's were used to estimate peak flood for various return periods (Table 4).

Table 4. Design Flood (m^3/s) in Khurkot Station under different Methods.

Methods	Return periods		
	50	100	500
Modified Dickens	6873	7449	8786
Fuller's Method	4550	5015	6093
Horton's Method	19079	22690	33929

2.5. Recommended Design Flood

For all of the above-mentioned distributions in statistical method, some goodness of fit tests was carried out for the peak instantaneous data obtained from the DHM. Chi-square test, Kolmogorov-Smirnov, and Anderson Darling tests were carried out. The outputs of goodness of fit tests are shown (Table 5).

Table 5. Goodness of fit test results.

Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Square	
	Statistic	Rank	Statistic	Rank	Statistic	Rank
Gumbel	0.216	4	3.692	4	4.089	4
Log Pearson Type III	0.113	2	0.479	2	2.689	3
Log Normal	0.101	1	0.405	1	2.549	2
Normal	0.147	3	1.119	3	1.010	1

The Log Normal distribution was found to be the best-fit distribution from tests. Based on international practices, guidelines, type and size of the structures, impounded volume, and preliminary assessment of the extent of likely damage in the event of worst failure, the frequency of design flood is considered equal to 1000 years. Assuming that the extreme events, mostly caused due to high precipitation will not coincide with the GLOF, the flood magnitude assessed without adding the GLOF event can be considered as a design flood. Hence, the design flood of 12,328 m^3/s corresponding to a period of return as 1,000 years with a check flood of 15,630 m^3/s corresponding to the period of return as 10,000 years is considered for the design of diversion structure. Flood discharges estimated by empirical and regional analysis are less than the discharges estimated by statistical methods. The empirical methods are generally applicable for ungauged stations and hence are used only for comparison purposes.

2.6. Methodology

The computation engine for the HEC-RAS program is based on the U.S. Army Corp of Engineers (USACE). The development of the HEC-RAS model starts with creating a HEC-RAS geometry file. The geometry is developed using the HEC-GeoRAS tool in ArcMap. HEC-GeoRAS processes geospatial data in ArcGIS using a graphical user interface (GUI). The prepared geometric data is then imported into HEC-RAS for processing and computations of water surface profiles.

The topographic information of the project location is obtained through GIS using the topographic map of a 1m contour interval prepared from topographic survey data, Finn map, and drone survey data. For the development of geometry, a total reach length of about 1.05 km is taken for the models of headworks area, 800 m upstream and 250 m downstream. Cross-sections are used that are developed from bathymetry survey and interpolated to have intervals ranging from 20 m to 25 m.

The Manning's coefficient (n) is chosen on the basis of the type of main channel and the characteristics of the banks along the course of the river as summarized by (Te Chow, 1959). For headworks area, the main channel of the Sunkoshi River can be characterized as clean, full stage with more stones and weeds for which the Manning's coefficient is selected as 0.03 for normal conditions and for streams with no vegetation in the channel and usually steep banks the n value is selected as 0.035. The model in HEC-RAS is calibrated using measured flow and water level at the same time. After calibration, the water surface profiles are computed for different discharges at different return periods.

The model was considered for the analysis at the headworks area with three conditions (i) without affecting the existing road (ii) affecting the existing road and (iii) inline structures. Steady state simulations for the model were carried out using a series of different discharge values.

3. Results and Discussions

3.1. Model I-A (Headworks area Without affecting existing road)

This model considers the headworks area without affecting the existing road. The model was run in subcritical steady-state simulation for a series of different discharge values at different return periods to obtain the water surface elevation at prominent sections. The below figure shows the headworks area with location (Figure 6). The water surface elevation in the channel along the reach for this model for different flood values is shown in Table 6.

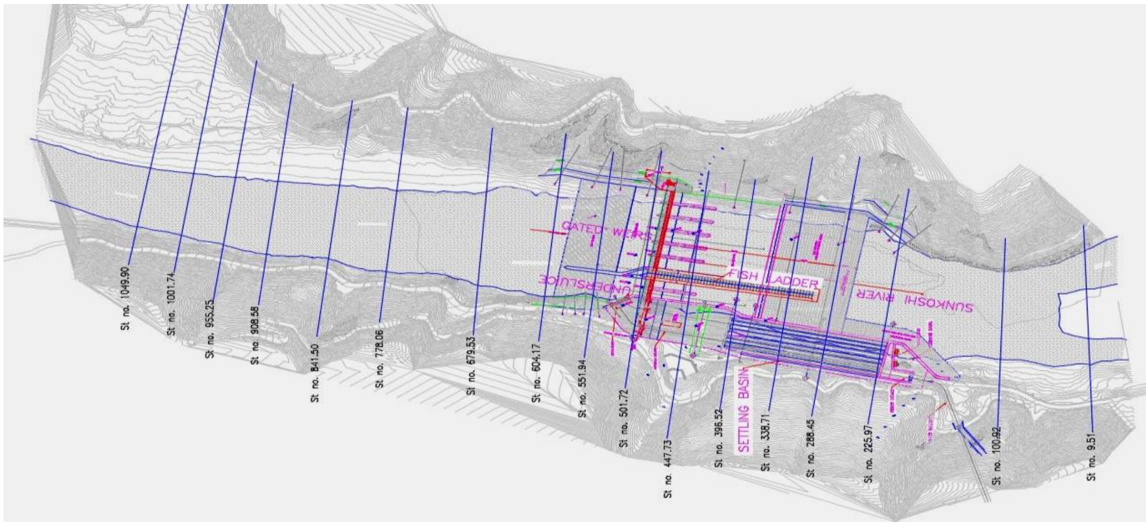

Figure 6. Layout of Model I-A.

Table 6. Water surface elevation at u/s and d/s end for Model I-A.

Profile	Discharge (m^3/s)	Return Period (Years)	Water Surface Elevation (m)	
			U/S End (River station 1049.895)	D/S End (River station 267.62)
PF 1	139.77	Measured Discharge	459.33	458.66
PF 2	8338	50	473.91	473.26
PF 3	9241	100	474.92	474.21
PF 4	11380	500	477.16	476.33
PF 5	12328	1000	478.10	477.22
PF 6	15630	10000	481.21	480.21

The rating curve of Model I-A is shown in Figure 7. Likewise, the cross-section profiles and the L profile at the downstream end of the Stilling Basin location (River station no. 267.62) are shown in Figure 8 and Figure 9 respectively.

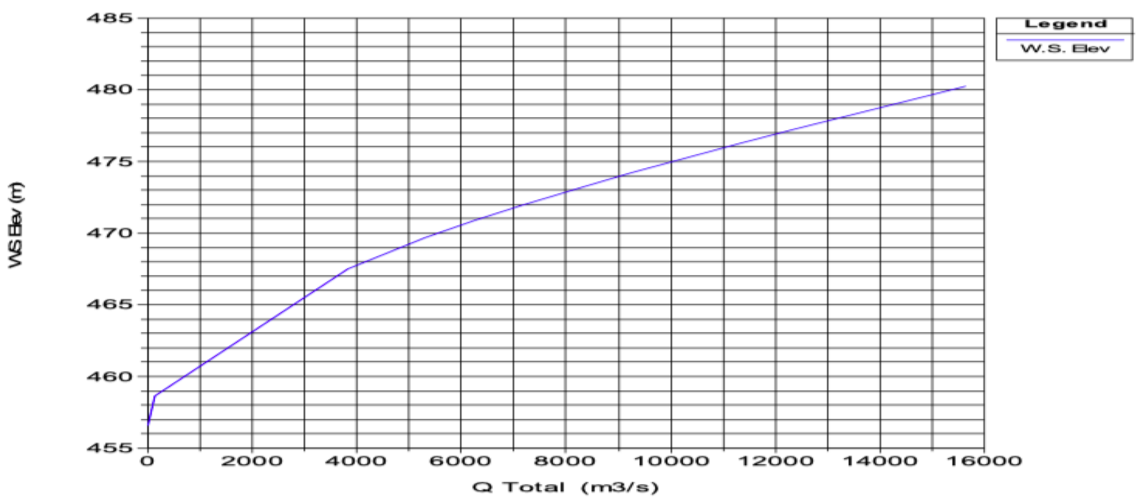

Figure 7. Rating curve at end of stilling basin in model I-A.

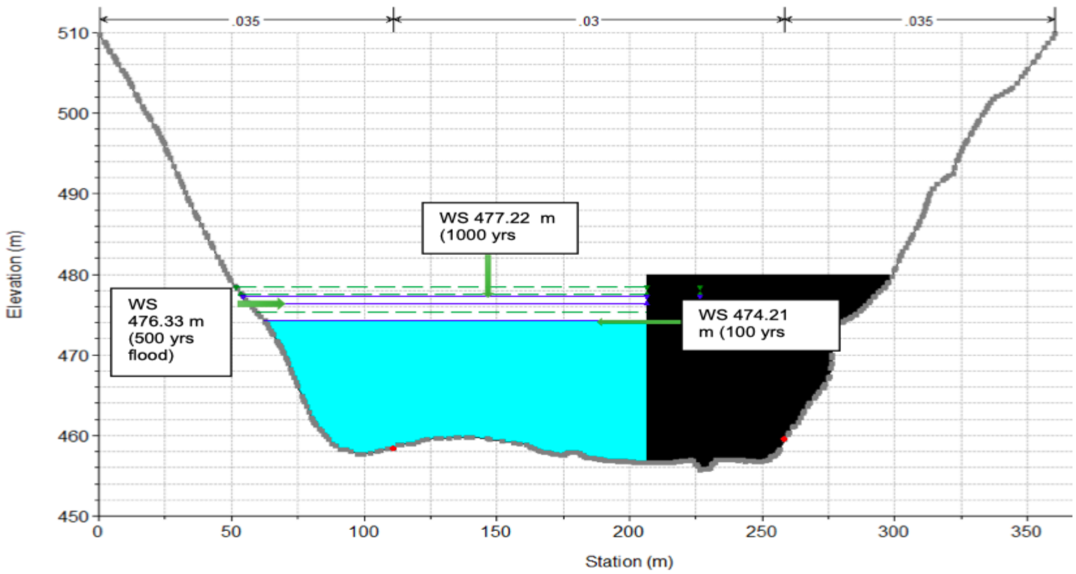

Figure 8. Cross-section at end of stilling basin in model I-A.

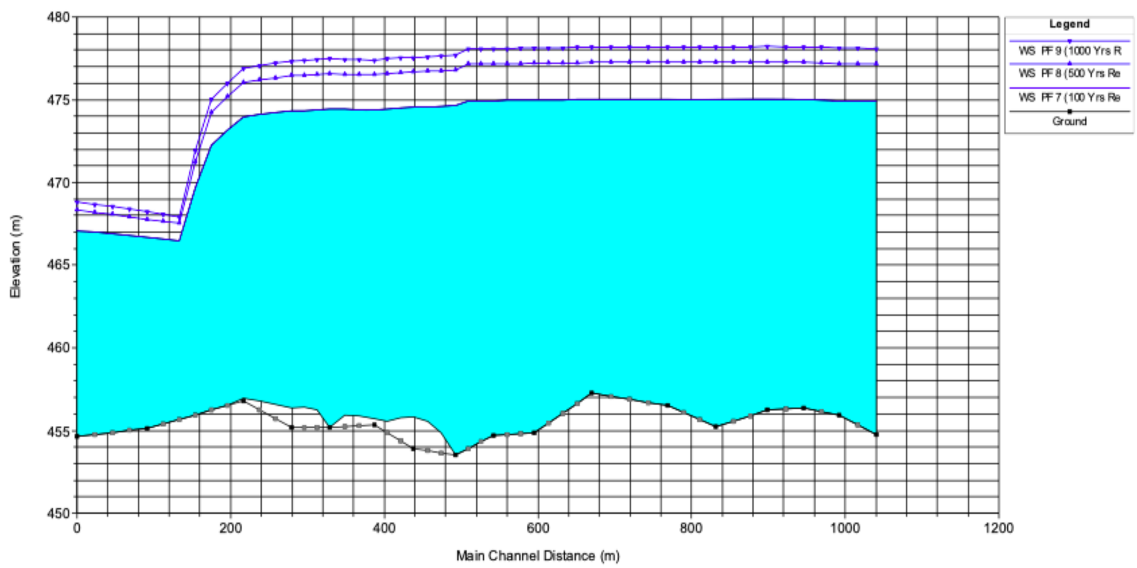

Figure 9. L - Profile of the model I-A.

3.2. Model I-B (Affecting existing road)

This model considers the headworks area affecting existing road. The model was run in steady-state simulation to obtain the water levels for different discharge values. Figure 10 shows the layout of the Model I-B.

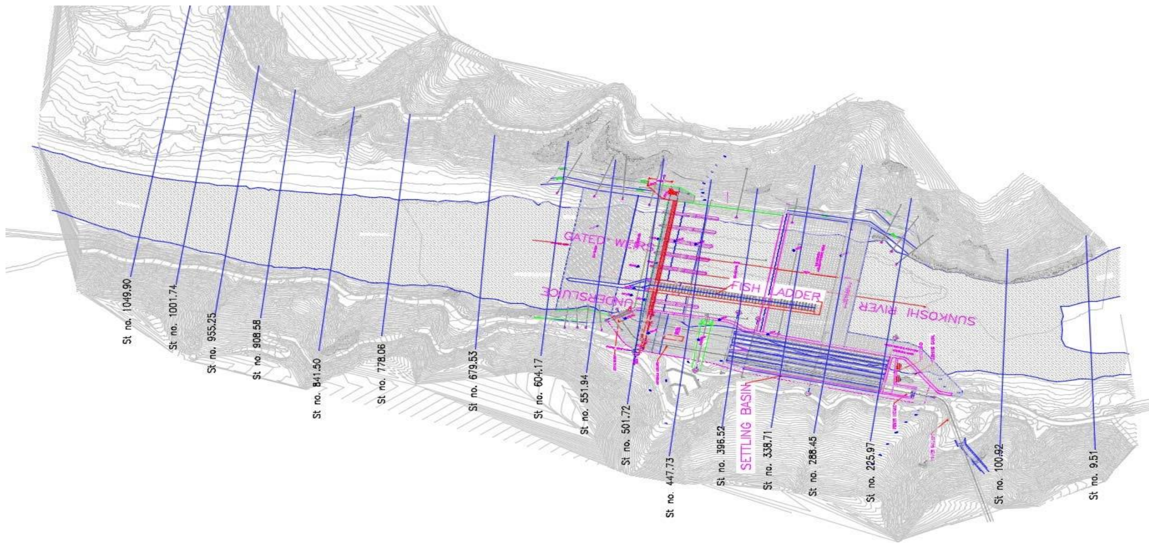

Figure 10. Layout Model I-B.

The water surface elevation in the channel along the reach for this model for different flood values is shown in Table 7. The rating curve of Model I-B is shown in Figure 11. Likewise, the cross-section profiles and the L-profile at the end of the Stilling Basin location are shown in Figure 12 and Figure 13 respectively.

Table 7. Water surface elevation at u/s and d/s end for Model I-B.

Profile	Discharge (m^3/s)	Return Period (Years)	Water Surface Elevation (m)	
			U/S End (River station 1049.895)	D/S End (River station 267.62)
PF 1	139.77	Measured Discharge	459.12	458.24
PF 2	8338	50	469.11	468.16
PF 3	9241	100	469.72	468.76
PF 4	11380	500	471.14	470.09
PF 5	12328	1000	471.75	470.64
PF 6	15630	10000	473.78	472.40

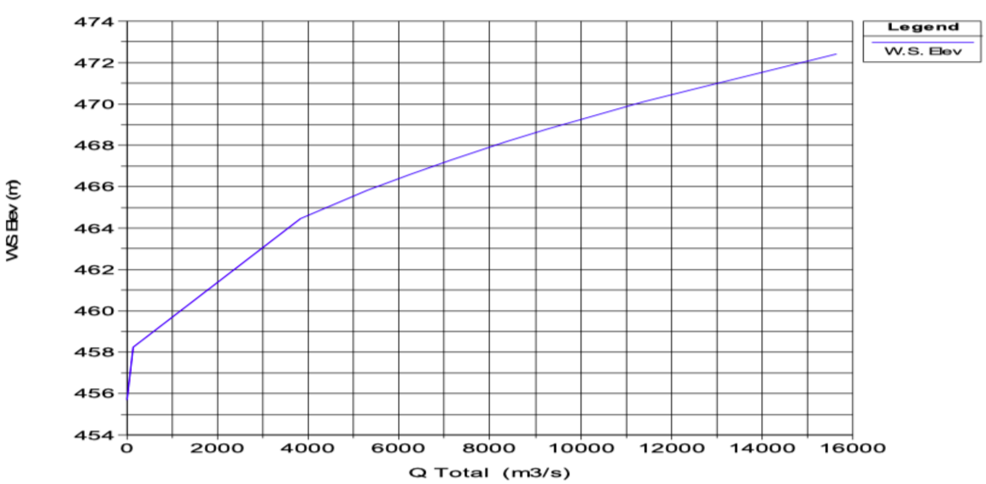

Figure 11. Rating curve at end of stilling basin in model I-B.

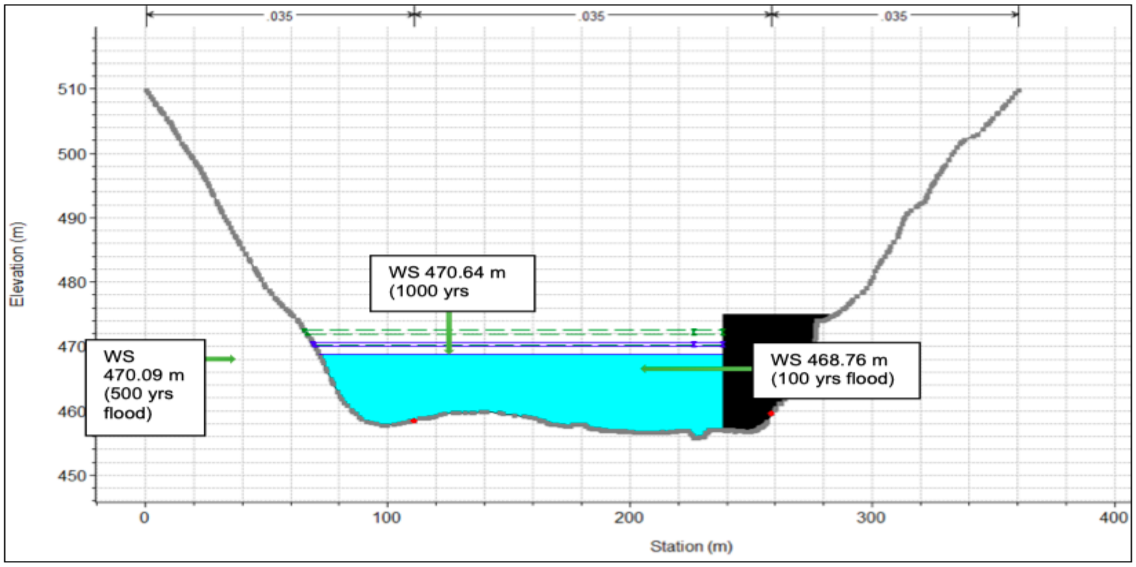

Figure 12. Cross-section at end of stilling basin end in Model I-B.

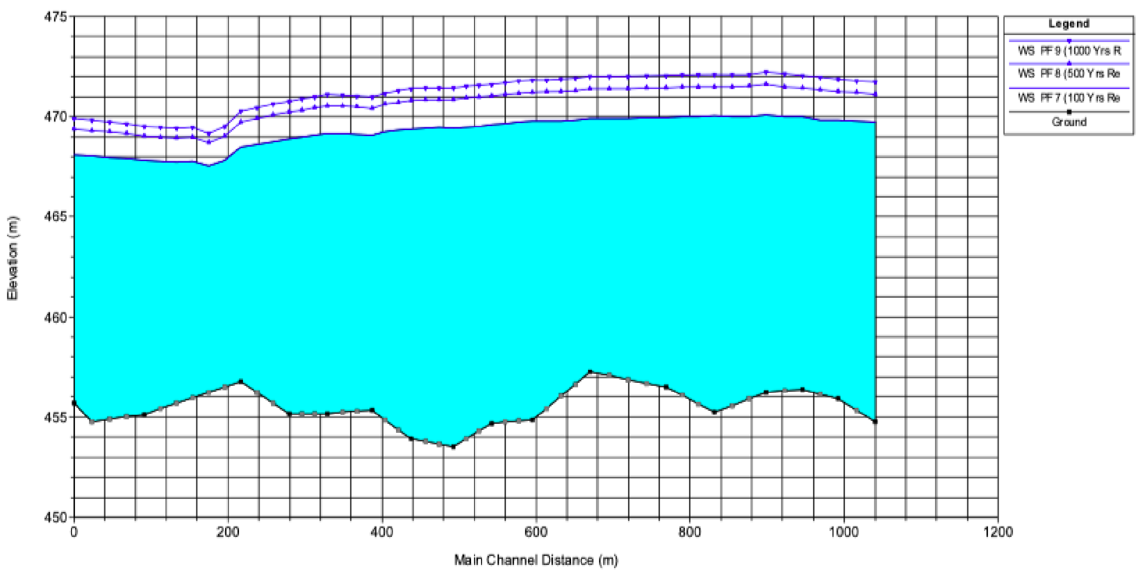

Figure 13. L Profile of the Model I-B.

3.3. Model II (Headworks Area with Inline Structure)

In this model, a gated barrage and under-sluice are added as inline structures. The model is run in subcritical steady-state simulation for a series of different discharge values at different return periods to obtain the water surface elevation at prominent sections. The head works with a gated barrage and under-sluice with its location shown below (Figure 14). The perspective view of the model in HEC-RAS is shown in Figure 15.

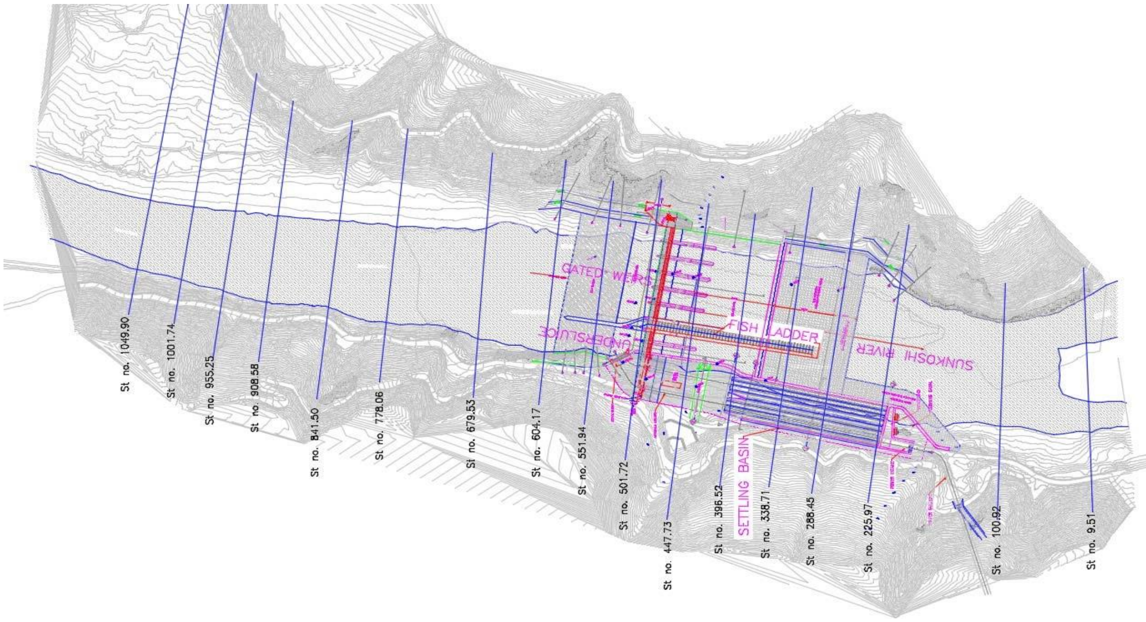

Figure 14. Layout of Model II.

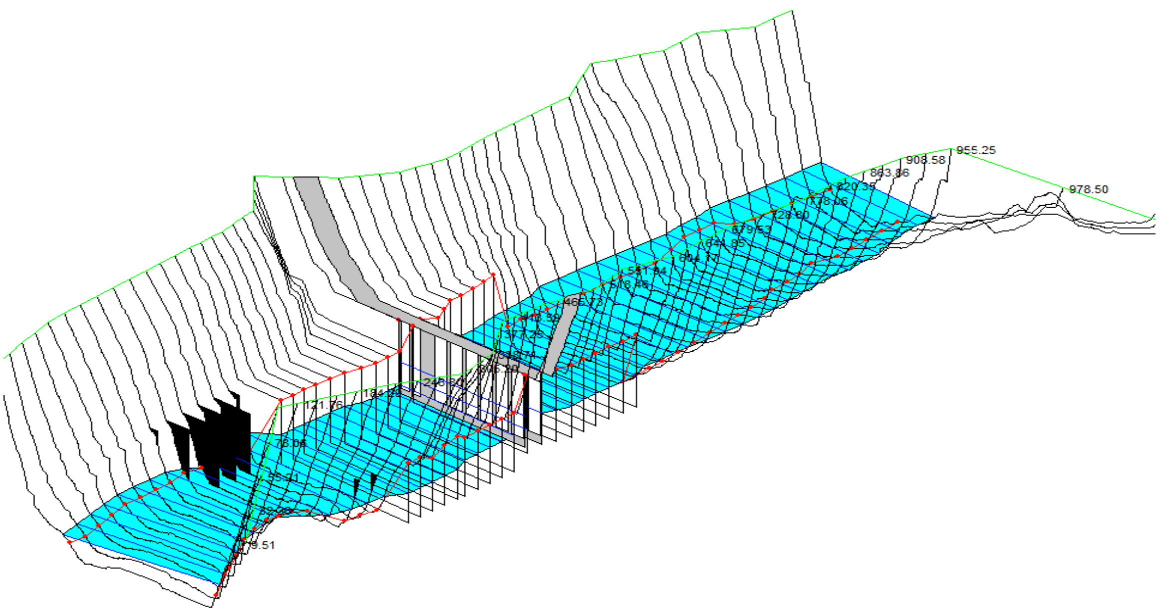

Figure 15. Perspective view of Model II in HECRAS.

The water surface elevation in the channel along the reach for this model for different flood values is shown in Table 8. The rating curve of Model III is shown in Figure 16. Likewise, the cross-section profiles and the L profile at the upstream and downstream end of the Stilling Basin location (St. 267.62) are shown in Figure 17 and Figure 18 respectively.

Table 8. Water surface elevation at u/s and d/s end for Model II.

Profile	Discharge (m^3/s)	Return Period (Years)	Water Surface Elevation (m)	
			U/S End (River station 1049.895)	D/S End (River station 267.62)
PF 1	139.77	Measured Discharge	460.63	458.2
PF 2	8338	50	472.08	468.61
PF 3	9241	100	472.96	469.28
PF 4	12741	500	474.95	470.77
PF 5	14071	1000	475.79	471.39
PF 6	15630	10000	478.59	473.43

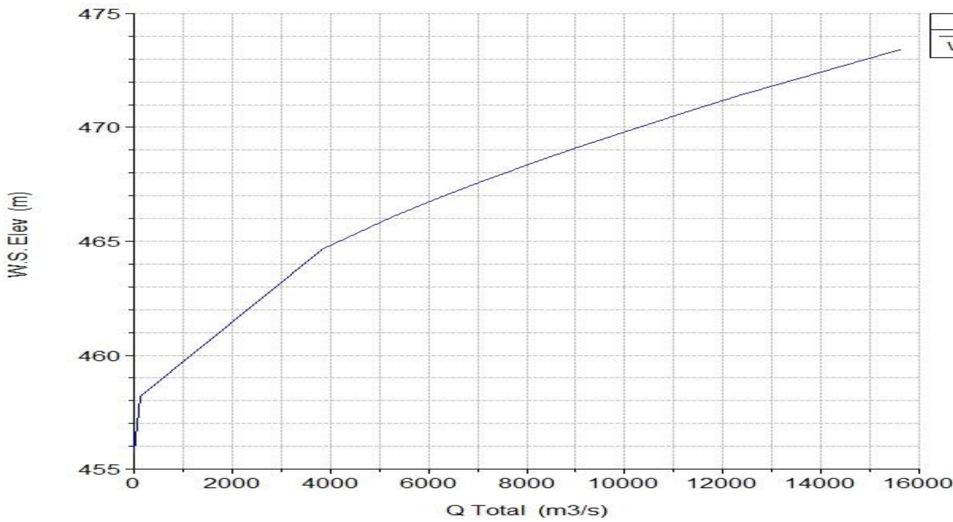

Figure 16. Rating Curve at End of Stilling Basin in Model II (St. 267.62).

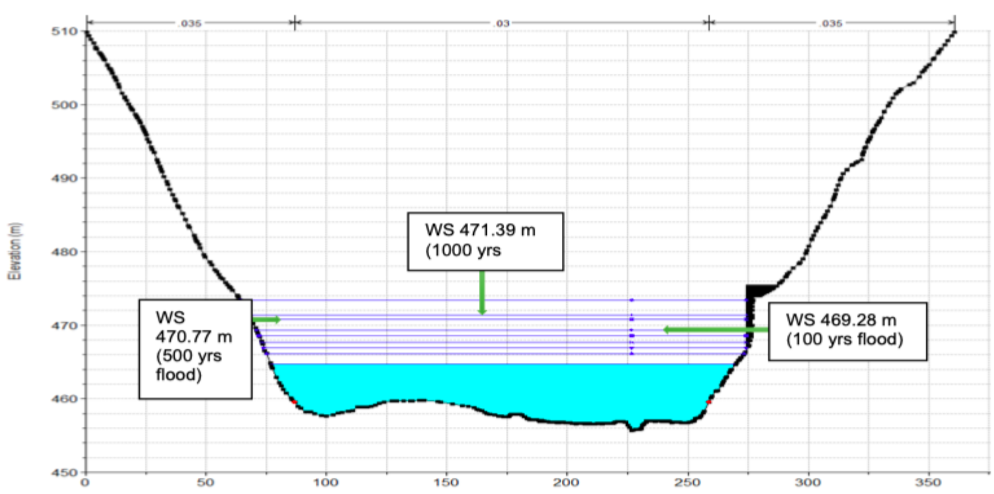

Figure 17. Cross-section at end of stilling basin end in Model II (St 267.62).

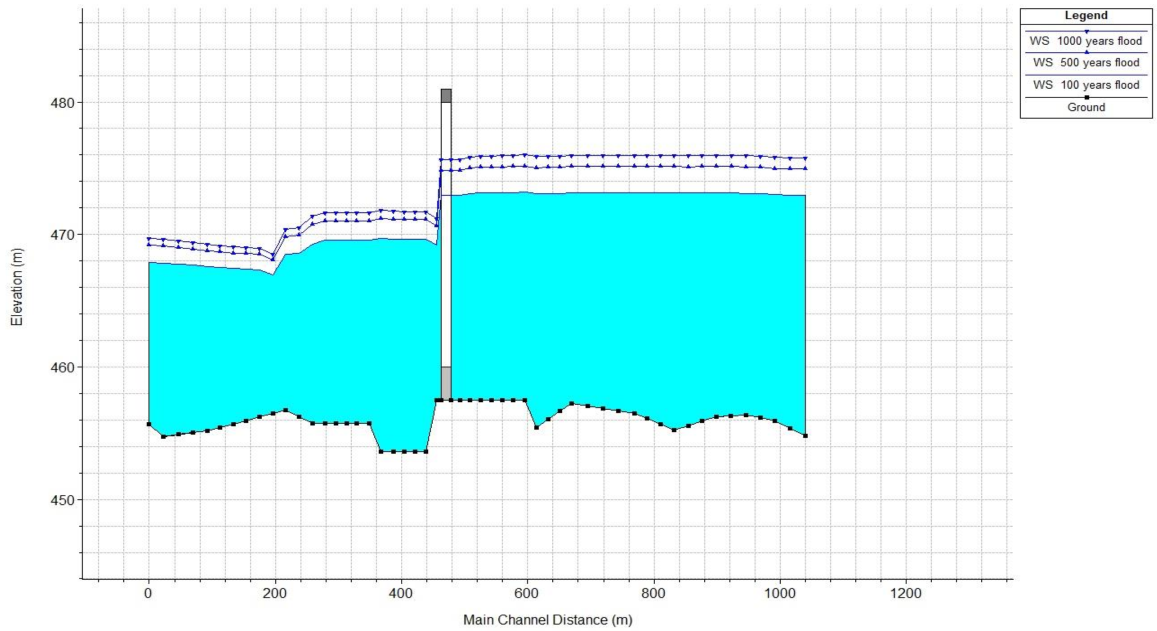

Figure 18. L Profile of the Model II.

3.4. Backwater Inundation Mapping

The proposed headworks of the Sunkoshi-Marin diversion will create the afflux of 4.0m considering the high flood level of 12,328 m^3/s (1000 years return period). The flood level will be 478 m corresponding to 1000 years return period. The total inundation area including the river waterway is 340.89 ha with the extension up to 6 km in the Tamakoshi side and 8 km in the Sunkoshi side. The net inundation area excluding the river waterway will be 216.92 ha. The inundated areas lie within three rural municipalities, namely; Sunkoshi, Khadadevi and Manthali (Figures 19, 20).

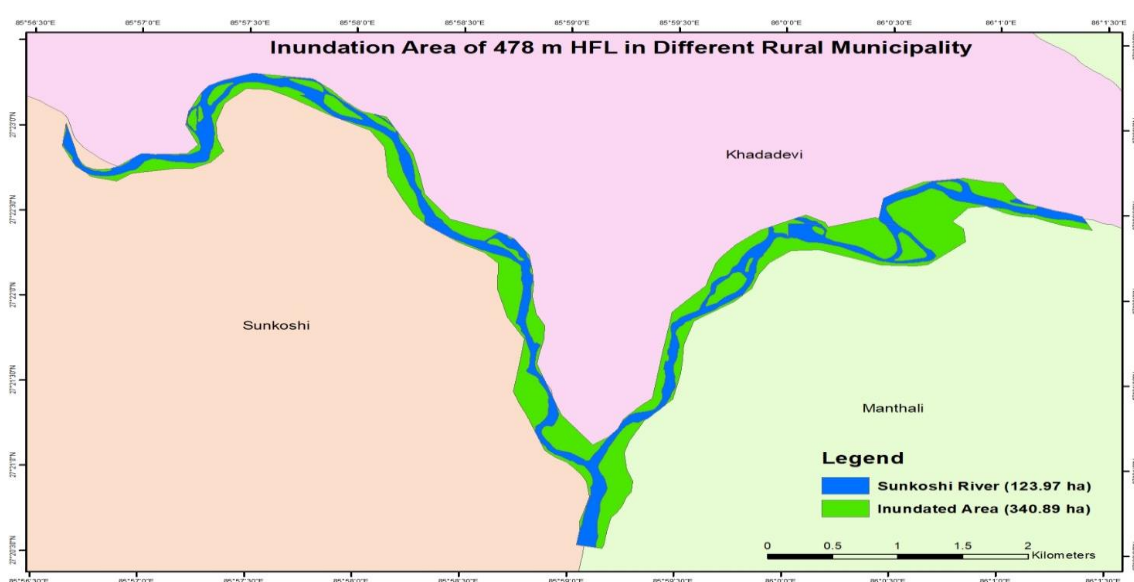

Figure 19. Inundation area of 478m HFL in different rural municipalities.

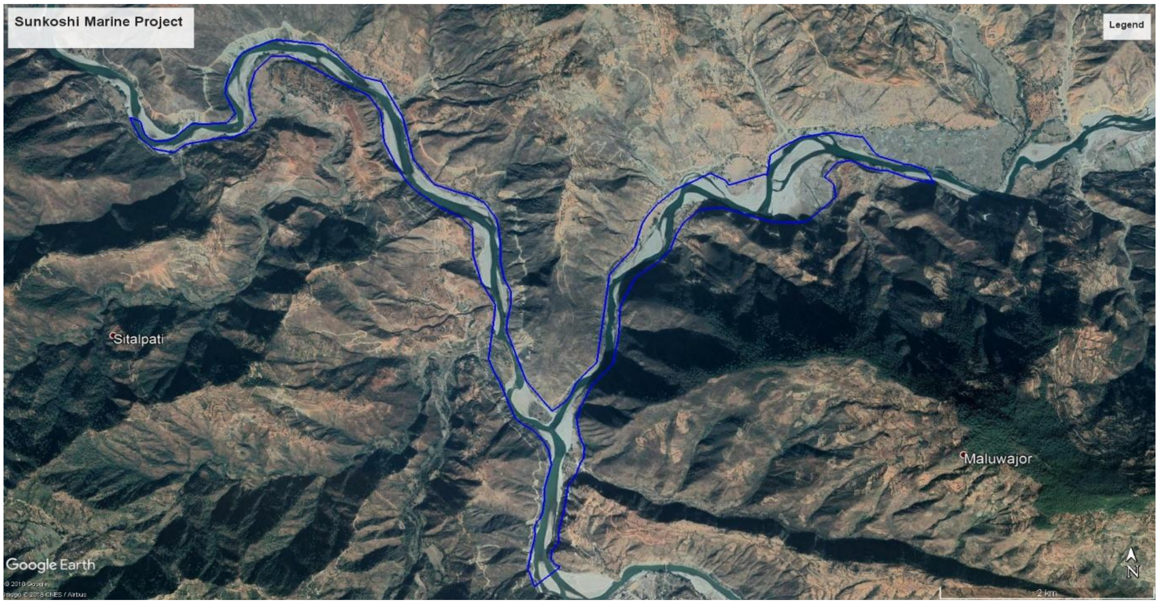

Figure 20. Inundation area of 478.0 m HFL in Google Earth map.

4. Conclusions

The numerical simulation of backwater effects by a downstream dam in the SunKoshi-Marin Diversion Headworks was carried out in three different cases (i) headworks without affecting existing road (ii) headworks affecting existing road and (iii) headworks with inline structures. Design flood was estimated by statistical methods and compared with regional methods and empirical formulas. Based on goodness of fit test parameters, the results of the Log-Normal distribution method were adopted for design flood. The headworks were designed for a 1000-year return period flood (12,328 m^3/s) with check flood of 10000 years (15,630 m^3/s). Water surface profiles were obtained for various frequencies of return periods 50, 100, 500, 1000, and 10000 years for all cases. For the HEC-RAS water surface profiles simulation upstream and downstream ends were taken River station (1049.895m) and (267.62m) respectively. In case (i) head works without affecting existing road scenarios simulated upstream and downstream water surface for design flood of 1000 years return periods were 478.10 m and 477.22 m respectively. Likewise in case (ii) head works affecting existing road scenarios simulated upstream and downstream water surface for design flood of 1000 years return periods were 471.75m and 470.64 m respectively. Moreover, in case (iii) head works with inline structures scenario simulated upstream and downstream water surface for design flood of 1000 years return periods were 475.79 m and 471.39 m respectively.

The total inundated area including the river waterway is 340.89 ha with the extension up to 6 km in the Tamakoshi side and 8 km in the Sunkoshi side. The net inundation area excluding the river waterway will be 216.92 ha. The inundated areas lie within three rural municipalities, namely; Sunkoshi, Khadadevi, and Manthali. The stretch of 271m of the BP highway falls within the inundated area. Accomodation of the intake, desander and other structures requires another stretch of 413 m. Hence, a total stretch of 684 m of the highway has to be realigned. The length of the realigned section of the highway is about 1.3 kms. Looking at the possibility of occurrence of flood for different return periods, there is a possibility of trade off in choosing these return periods for acquiring land for the implementation of the project.

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Conflicts of Interest

The author declares no conflict of interest.

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