सिंचाई खंड-द्वितीय, महराजगंज,
सिंचाई एवं जल संसाधन विभाग, उत्तर प्रदेश

महाव नाला में बाढ़ की समस्या के दीर्घकालिक सुरक्षा और समाधान के उपायों के लिए हाइड्रोलिक और गणितीय मॉडल अध्ययन

पर प्रतिवेदन

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जनवरी, 2023

Report

on

HYDRAULIC AND MATHEMATICAL MODEL STUDIES FOR LONG TERM SAFETY AND SOLUTION MEASURES FOR FLOOD PROBLEM IN MAHAV NALA

Study Proposed by

Irrigation Division-II, Mahrajganj
Irrigation and Water Resources Department,
Uttar Pradesh

NATIONAL INSTITUTE OF HYDROLOGY

ROORKEE – 247 667 UTTARAKHAND

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REPORT ON HYDRAULIC AND MATHEMATICAL MODEL STUDIES FOR LONG TERM SAFETY AND SOLUTION MEASURES FOR FLOOD PROBLEM IN MAHAV NALA

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I express gratefulness towards the officials of UPID for providing encouragement, help and support throughout the study. I am also thankful to the Divisional Forest Officer, Sohagibarwa, Mahrajganj for his support and help during the field visit as well as for providing necessary information regarding the project.

(Pankaj Mani)

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1 ABOUT THE PROJECT

National Institute of Hydrology, Roorkee received a proposal from the office of Chief Conservator of Forests Wildlife (East), Gonda to investigate the hydrological feature of the Mahav nala in Maharajganj district of Uttar Pradesh (Annexure-I). The Mahav nala is a left bank tributary of Bhagel nala which in turn meets the Rohini river. It is a small size river originating from low mountains in Nepal. Mostly flash flood is reported in this stream and the duration of flooding ranges from 4 hours to 48 hours depending upon the rainfall spell in the catchment. The entire catchment of the nala is located in Nepal. In the hills, the slope of the nala is very steep and which reduces drastically when enters into foothills (tarai region). The nala is highly meandering in nature. The heavy siltation in upper reaches has significantly reduced the flow depth. During flash flood, the bank overtopping and bank erosion is the recurrent problem in the region. The site visit to the site was carried out during June 2018 along with the officials of Forest Department and UP Irrigation Department. Based on the site visit, the salient hydrological features were reported. Three major observations were reported based on site visit; (i) in the initial reach of Indo-Nepal border, nala bed is lower than the country side terrain. Thus, the nala banks are strong with consolidated land mass. (ii) in the mid reach, it is observed that the nala bed is almost at the same level as that of country side terrain. This reach is more prone to overtopping and thus flooding. (iii) in the downstream reach, it is observed that though the country side terrain is higher than the nala bed, it is lower than the nala bank level and extensive terrain below the bank level on both side of the nala are available which was proposed to be used as detention basin to absorb the peak flood.

Further in April 2022, a letter is received from DFO Sohagibarwa wild life Division, Maharajganj, UP to propose suitable measures for flood mitigation in Mahav nala (Annexure-II). Moreover, another letter from Executive Engineer Irrigation Section-II, Maharajganj, Irrigation and Water Resources Department, UP (UPID) is received to carry out the hydraulic/ mathematical technical model studies for long term safety and solution measure for flood problem in Mahav nala (Annexure-III). Accordingly, a site visit was made on 23rd April 2022 along with the officials of UPID and Forest Department, Maharajganj. During the field visit/ meeting, it was proposed to improve the carrying capacity of the nala in its entire stretch; from international border to its confluence with the Bhaghel nala. Further, the design flood for the nala is to be estimated based on UPID historical data/ earlier reports/ standard flood estimation reports for the concerned region recommended by CWC/ State agencies etc. or standard methodologies. Subsequently, the nala section may be proposed to safely pass the estimated flood considering the available nala slope. A comprehensive study for flooding in Mahav nala using mathematical model is also proposed. Using the model study, the carrying capacity of the nala in existing condition would be evaluated; the critical locations would be identified. For the design flood, suitable section may be evaluated considering the flooding condition in Bhagel nala at the downstream end. The requirement for such studies like river cross-sections at closer intervals along with some observed hydrological data (river gauge and discharge data) to calibrate/ validate the mathematical model was informed to UPID.

In this background, the mathematical model study of Mahav nala is carried out with respect to the following details:

1. Extraction of existing drainage network and watershed boundary based on online Digital Elevation Model (DEM) and satellite data.
2. Design flood estimation based UPID report, CWC flood estimation report and other standard methods.
3. Development of river flow model for Mahav nala with existing river cross section and computation of safe carrying capacity of nala. Identification of the critical locations of overtopping.
4. Evaluating the river flow model for the nala section proposed by UPID. Computing the carrying capacity of the nala.
5. The design of nala section for the estimated design flood.

2 DESCRIPTION OF STUDY AREA

2.1 General

Mahav nala is the left bank tributary of the Bhaghel nala which in turn meets the Rohini river. It is a small size river originating from low mountains in Nepal. Mostly flash flood is observed in this stream and the duration of flooding ranges from 4 hours to 48 hours depending upon the rainfall spell in the catchment. The entire catchment of the nala is located in Nepal. In the hills, the slope of the nala is very steep and which reduces drastically when enters into foothills (tarai region). The soil of the lower Himalaya being highly loose and fragile, the nala brings heavy silt deposit with the flash flood. The siltation is very common in the tarai region (web reference 1, 2018). The nala is highly meandering in nature with poorly formed banks, the bank overtopping and bank erosion are the recurrent problem in the region. The river planiform as observed from Google Earth image shows that there are 142 bends in the 23 km stretch of nala, from Indo-Nepal border to its confluence with Baghel nala (Figure 2.1).

2.2 Data used in study

The following data are used in the study to derive various information:

1 On line DEM (SRTM data downloaded from https://earthexplorer.usgs.gov/) The DEM is used for extraction of drainage network, watershed boundary etc. However, the extracted drainage network is further modified using high resolution satellite images (Google Earth Image). Subsequently, the drainage characteristics are computed.

2 PMP Atlas for the Ganga River basin (downloaded from site-http://www.cwc.gov.in/mp-atlases). The PMP Atlas prepared jointly by Central Water Commission and India Meteorological Department, Government of India and is very useful in the assessment of design storm and for further assessment of design flood for any water resources development project.

3 River cross section data of Mahav nala (The surveyed section is provided by UPID and is given in Annexure-IV). Further, UPID has also provided the proposed river section for safe passage of the estimated design flood of about 60 m^3/s.

The longest flow path and watershed boundary of Mahav nala is extracted from online DEM (SRTM) is shown in Figure 2.2. The drainage map is superimposed over the ESRI topographical map in ARC GIS. The international border of India and Nepal can be clearly seen in this figure. The elevation of Mahav nala varies from about 1200 m to 100 m. The major part of catchment area of Mahav nala is lying in Nepal where elevation ranges from 1200 m to 110 m. Within Indian territory, the elevation ranges from 110-94 m. The catchment area of Mahav nala is computed as 218 km^2.

From the CWC PMP atlas four stations have been identified for which the extreme rainfall data has been used in computing the design storm. The location of the rainfall stations with respect to the Mahav nala watershed is shown in Figure 2.3. Out of four stations only Nautanwa is located in India while other three namely Parasi, Simari and Butwal are located in the higher reaches in Nepal. The Thiessen polygon weights are also shown in this figure.

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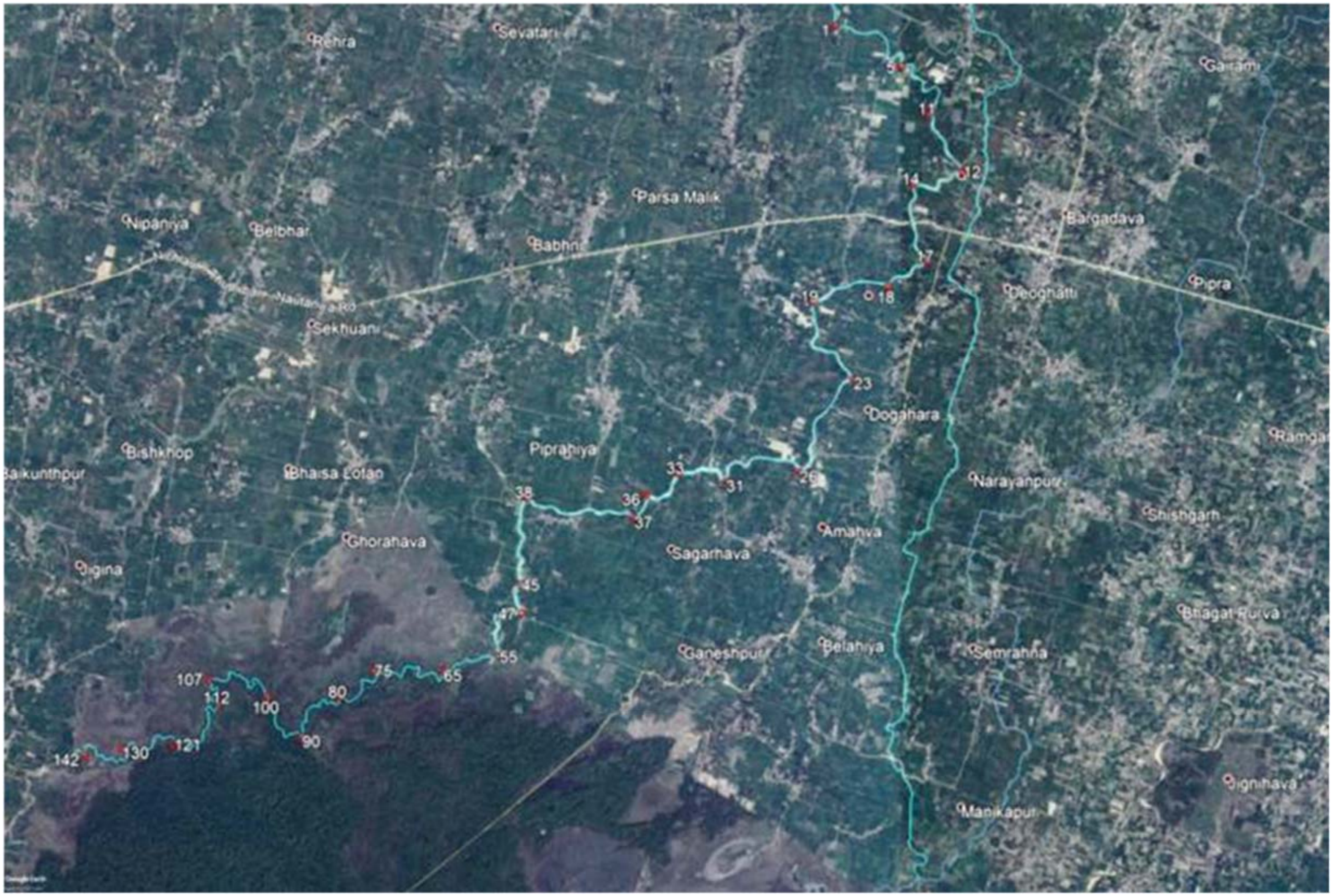

Figure 2.1: Meandering nature of Mahav nala in Google Earth image showing 142 bends in 23 km stretch.

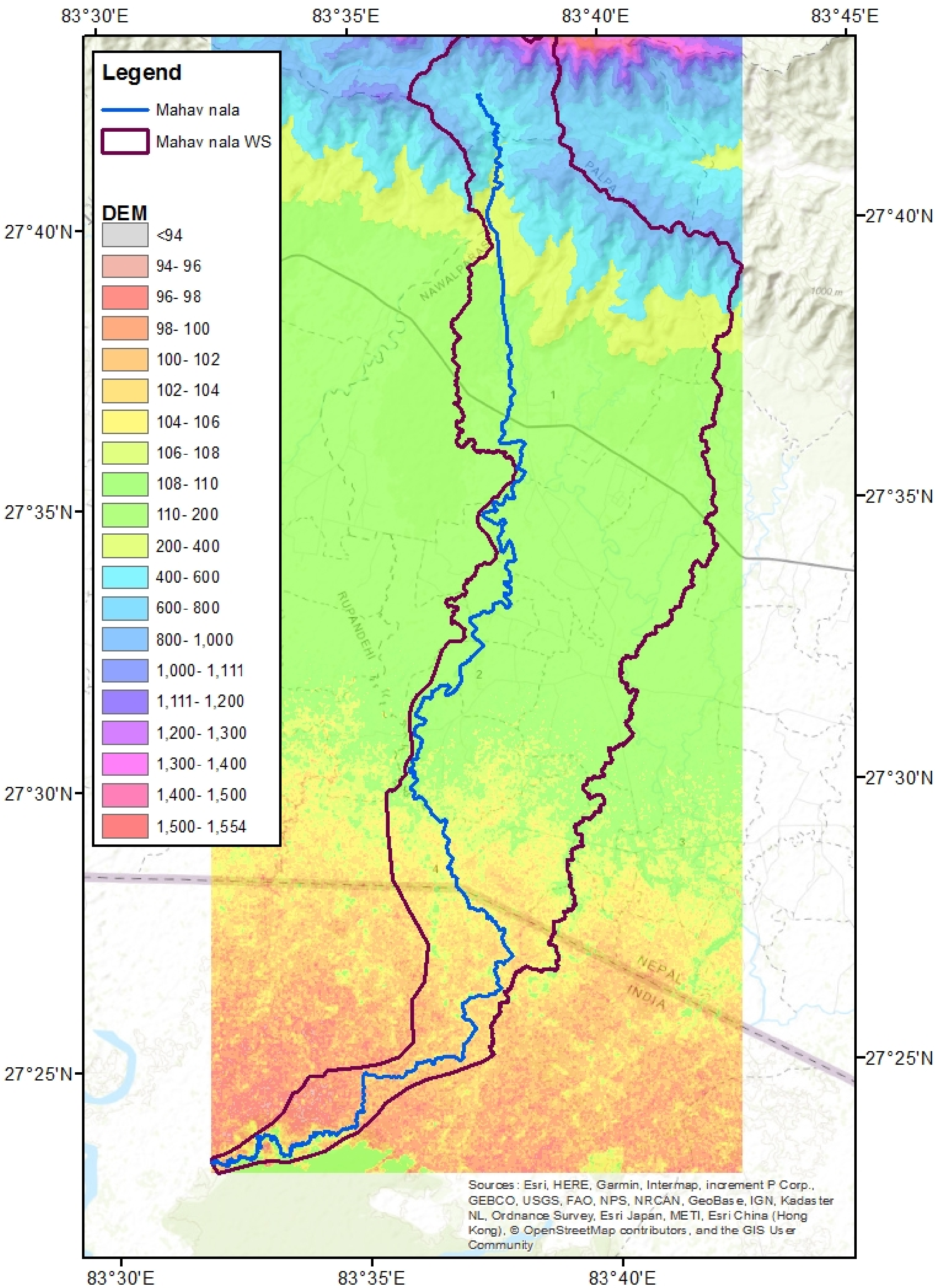

Figure 2.2: The Mahav nala and its watershed boundary.

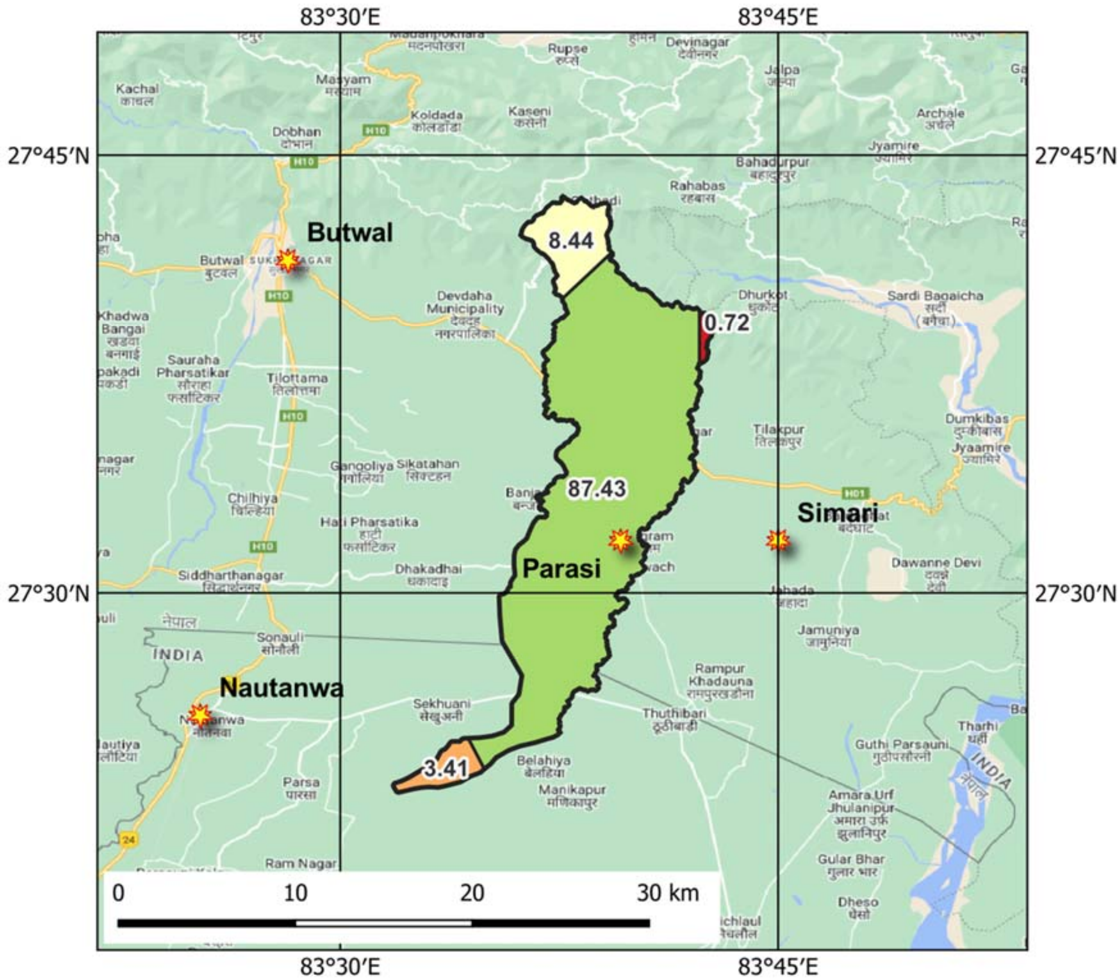

Figure 2.3: The rainfall stations used in computing design storm.

3 DESIGN OF DRAINS

The study is envisaged in two important components; (i) estimation of design storm/ flood and (ii) evaluation of existing nala section and design of adequate section for safe passage of design flood. The methodology for these two components would be discussed in this section.

3.1 Estimation of design storm and design flood

In this study, design flood has been estimated using T year return period rainfall and unit hydrograph. Rainfall of specified exceedance probability is estimated using PMP atlas for Ganga basin developed by CWC and IMD.

3.1.1 Data availability

For estimating the synthetic unit hydrograph on the basis of physiographic parameters of the catchment, Flood Estimation Report for Middle Ganga Plains subzone 1(f) (CWC, 1985) has been used. Further, the PMP Atlas for Ganga River Basin Including Yamuna, published by CWC (2015) is also available, in which 1-day maximum rainfall for various return periods are available. The time distribution coefficient to estimate the hourly rainfall is also available in the PMP Atlas.

3.1.2 Physiographic parameters

The delineated catchment area map of the Mahav Nala is shown in Figure 3.1. The catchments parameters viz. catchment area, longest flow path, centroidal longest flow path, equivalent stream slope of each sub catchment/sub basin as obtained from GIS processing are given in Table 3.1.

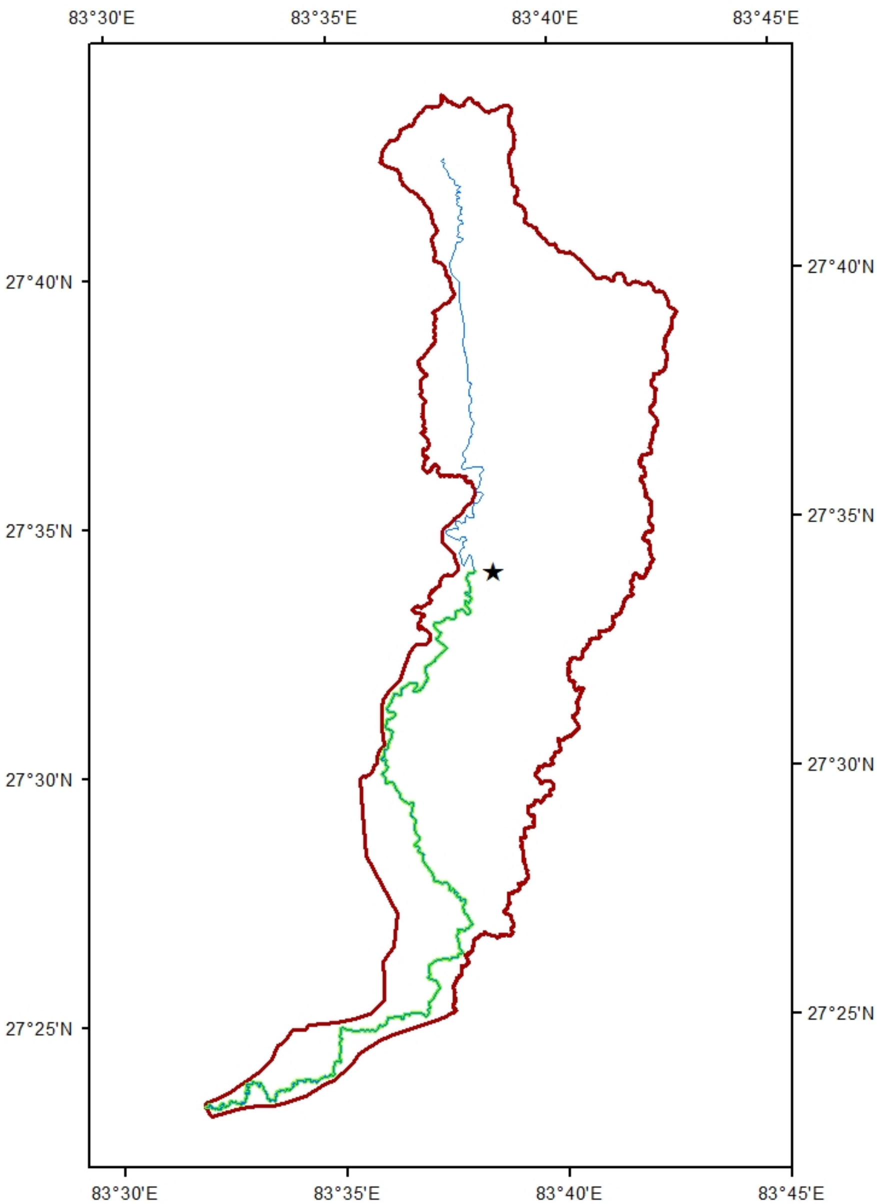

Figure 3.1: Catchments area map.

Table 3.1: Sub-basin parameters for Mahav Nala catchments

Catchment area (km^2)	L (km)	Lc (km)	Equivalent stream slope S (m/km)
218.5	66.34	42	0.524

3.1.3 Derivation of Synthetic Unit Hydrograph

The various parameters of the synthetic unit hydrograph are derived from the Flood Estimation Report (CWC, 1985). Detail description of parameter and their relationships are given in Table 3.2. As mentioned in the CWC (1985) report the relationships are for 6- hour unit hydrograph (6-h UH). The ordinates of 6-h SUH is given in Table 3.3 and plot of unit hydrographs is presented in Figure 3.2.

Table 3.2: Relationships for estimating 6-h UH parameters for the study area

Sl. No.	Parameters	Relationship	Value
1	Peak discharge of unit hydrograph per unit area of catchment (m^3/s/ km^2)	$q_p = 0.409 / (L/\sqrt{S})^{0.456}$	0.052
2	Time in hours from the centre of unit rain fall duration to the peak of unit hydrograph. (h)	$t_p = 1.217 / (q_p)^{1.034}$	25.9 (27)
3	Width of UH in hours at 50 percent of peak discharge. (h)	$W_{50} = 1.743 / (q_p)^{1.104}$	45.59
4	Width of UH in hours at 75 percent of peak discharge	$W_{75} = 0.902 / (q_p)^{1.108}$	23.87
5	Width of the rising limb of UH in hours at 50 percent peak discharge	$WR_{50} = 0.736 / (q_p)^{0.928}$	11.44
6	Width of the rising limb of UH in hours at 75 percent peak discharge	$WR_{75} = 0.478 / (q_p)^{0.902}$	6.88
7	Base width of unit hydrograph in hours	$TB = 16.432 / (t_p)^{0.646}$	138.15 (138)
8	Peak discharge of unit hydrograph in m^3/s	$Q_p = q_p \times A$	11.362

Where, L = Length of the main stream (km), S = Equivalent stream slope (m/km), A= Area of the catchment (km^2).

Table 3.3: SUH for the catchments of Mahav Nala

Time (hr)	Discharge (cumec)
0	0
6	1.053
12	3
18	6
24	10.3
30	11.362
36	10.6
42	9.45
48	8.3
54	7.2
60	6.3
66	5.5
72	4.75
78	3.95
84	3.3
90	2.7
96	2.15
102	1.7
108	1.3
114	0.95
120	0.7
126	0.4
132	0.2
138	0

3.1.4 Design Storm

The four available rain gauge stations are selected in and around the catchment area for obtaining design rainfall of various return periods. Thiessen polygon method is used to estimate catchment rainfall. The Thiessen polygon along with weights is shown in Figure 2.3. The design storm duration (TD) can be estimated using $1.1 \times t_p$ as recommended by CWC (1985). However, in this case the 1-day rainfall is used. The 1-day maximum rainfall of each station for various return periods are obtained from the PMP atlas. The estimated one-day storm depths for the catchment various return periods are given in Table 3.4. Areal Reduction Factor of 0.93 is used for converting point rainfall to areal rainfall.

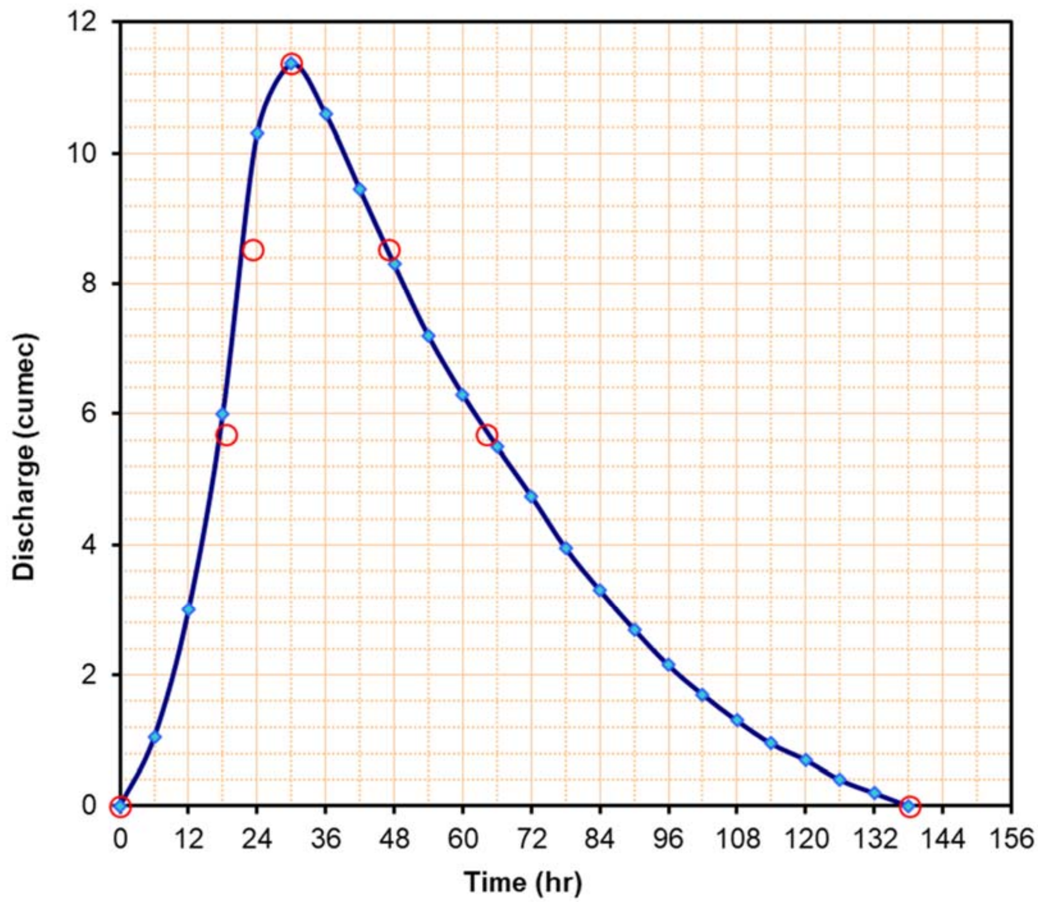

Figure 3.2: SUH for the Mahav Nala catchments

Table 3.4: Design Storm depths (mm) for various return periods

STN_Name	**Dist**	**Weight**	**Return Period (Year)**			
			R2.33	**R5**	**R10**	**R25**
Simari	Nawalparasi_NP	0.72	164	212	251	300
Nautanwa	Maharajganj_IN	3.41	160	197	228	267
Butwal	Rupandehi_NP	8.44	189	251	302	366
Parasi	Nawalparasi_NP	87.43	163	214	256	309
	Weighted Rain	100	**165.1**	**216.5**	**258.9**	**312.3**

The 1-day values are increased by 15% to convert the same into 24 hours value, upto a maximum value of 5 cm. The 24 hour rainfall has been converted into two 12 hourly rainfall bells by using the time distribution as provided in PMP Atlas. For hourly distribution of rainfall normalized distribution coefficient has been worked out for bell of 12 hours each using the hourly distribution coefficient. The hourly distribution coefficient of 24 hour rainfall and normalized distribution coefficient for 12 hour bell are given in Table 3.5.

Table 3.5: Hourly distribution coefficient of 24 hour rainfall for first 12 hour and Normalized distribution coefficient 12 hour bell

Time (hr)	Distribution coefficient for 24 hour rainfall (%)	Normalised Distribution coefficient for 12 hour bell (%)
1	16.6	21.28
2	26.3	33.72
3	33	42.31
4	39.5	50.64
5	44.2	56.66
6	48.7	62.43
7	52.8	67.69
8	58.1	74.48
9	63.4	81.27
10	69.7	89.35
11	74.4	95.38
12	78	100.00
13	81.4	
14	84	
15	85.8	
16	87.5	
17	89.3	
18	90.6	
19	93.4	
20	95.6	

Time (hr)	Distribution coefficient for 24 hour rainfall (%)	Normalised Distribution coefficient for 12 hour bell (%)
21	96.9	
22	98	
23	99.5	
24	100	

3.1.5 Loss rate and Base flow

A design loss rate of 0.3 cm/h as recommended in the FER is used in the study. Base flow / km^2 0.05 m^3/s per sq. km of catchment area as recommended by CWC (1985) report. The computed base flow is 10.93 m^3/s.

3.1.6 Critical sequencing of rainfall

Hourly distribution of PMP rainfall of each bell is given in Table 3.6 and Table 3.7. Critical sequencing of hourly effective PMP rainfall of each bell is given in Table 3.8. The reverse of critically sequenced effective rainfall has been used for convolution with ordinates of unit hydrograph to get Probable Maximum Flood Hydrograph.

Table 3.6: 12 Hourly bell distribution of rainfall for various return periods

Description	Rainfall value (cm)			
	2.33 Y	5 Y	10 Y	25 Y
1day aerial rainfall	15.35	20.14	24.08	29.05
24 hr aerial rainfall (with 15% clock hour correction restricted to maximum 50 mm)	17.66	23.16	27.69	33.40
Depth 1st 12 hr bell (78%)	13.77	18.06	21.60	26.05
Depth 2nd 12 hr bell (22%)	3.88	5.09	6.09	7.35

Table 3.7: Hourly distribution of rainfall

2.33 Year Return Period

Time	Dist coeff	Normalised Dist coeff	Cumulative rainfall depth		Incremental rainfall depth		Loss rate	Effective rainfall depth	
			1st 12 hr bell	2nd 12 hr bell	Incremental rainfall 1st bell	Incremental rainfall 2nd bell		Effective rainfall 1st bell	Effective rainfall 2nd bell
(hr)	(%)	(%)	(cm)	(cm)	(cm)	(cm)	cm/hr	(cm)	(cm)
6	48.7	62.43	8.6	2.43	8.60	2.43	0.3	8.30	2.13
12	78.0	100.00	13.77	3.88	5.17	1.45	0.3	4.87	1.15

5 Year Return Period

Time	Dist coeff	Normalised Dist coeff	Cumulative rainfall depth		Incremental rainfall depth		Loss rate	Effective rainfall depth	
			1st 12 hr bell	2nd 12 hr bell	Incremental rainfall 1st bell	Incremental rainfall 2nd bell		Effective rainfall 1st bell	Effective rainfall 2nd bell
(hr)	(%)	(%)	(cm)	(cm)	(cm)	(cm)	cm/hr	(cm)	(cm)
6	48.7	62.43	11.28	3.18	11.28	3.18	0.3	10.98	2.88
12	78.0	100.00	18.06	5.09	6.78	1.91	0.3	6.48	1.61

10 Year Return Period

Time	Dist coeff	Normalised Dist coeff	Cumulative rainfall depth		Incremental rainfall depth		Loss rate	Effective rainfall depth	
			1st 12 hr bell	2nd 12 hr bell	Incremental rainfall 1st bell	Incremental rainfall 2nd bell		Effective rainfall 1st bell	Effective rainfall 2nd bell
(hr)	(%)	(%)	(cm)	(cm)	(cm)	(cm)	cm/hr	(cm)	(cm)
6	48.7	62.43	13.48	3.8	13.48	3.8	0.3	13.18	3.50
12	78.0	100.00	21.6	6.09	8.12	2.29	0.3	7.82	1.99

25 Year Return Period

Time	Dist coeff	Normalised Dist coeff	Cumulative rainfall depth		Incremental rainfall depth		Loss rate	Effective rainfall depth	
			1st 12 hr bell	2nd 12 hr bell	Incremental rainfall 1st bell	Incremental rainfall 2nd bell		Effective rainfall 1st bell	Effective rainfall 2nd bell
(hr)	(%)	(%)	(cm)	(cm)	(cm)	(cm)	cm/hr	(cm)	(cm)
6	48.7	62.43	16.27	4.59	16.27	4.59	0.3	15.97	4.29
12	78.0	100.00	26.05	7.35	9.78	2.76	0.3	9.48	2.46

Table 3.8: Critical sequencing for effective hourly rainfall with respect to UH

2.33 Year Return Period

Time	UH ordinate	Critical sequence of hourly effective rainfall		Reversed sequence of hourly effective rainfall		Bell sequence used for convolution [B2-B1]
		1st 12 hr bell (B1)	2nd 12 hr bell (B2)	1st 12 hr bell (B1)	2nd 12 hr bell (B2)	
(hr)	(cumec)	(cm)	(cm)	(cm)	(cm)	(cm)
0	0					1.15
6	1.053					2.13
12	3	8.30	2.13	4.87	1.15	4.87
18	6	4.87	1.15	8.30	2.13	8.30
24	10.3					
30	11.362					
36	10.6					
42	9.45					
48	8.3					
54	7.2					
60	6.3					
66	5.5					
72	4.75					
78	3.95					
84	3.3					
90	2.7					
96	2.15					
102	1.7					
108	1.3					
114	0.95					
120	0.7					
126	0.4					
132	0.2					
138	0					

5 Year Return Period

Time	UH ordinate	Critical sequence of hourly effective rainfall		Reversed sequence of hourly effective rainfall		Bell sequence used for convolution [B2-B1]
		1st 12 hr bell (B1)	2nd 12 hr bell (B2)	1st 12 hr bell (B1)	2nd 12 hr bell (B2)	
(hr)	(cumec)	(cm)	(cm)	(cm)	(cm)	(cm)
0	0					1.61
6	1.053					2.88
12	3	10.98	2.88	6.48	1.61	6.48
18	6	6.48	1.61	10.98	2.88	10.98
24	10.3					
30	11.362					
36	10.6					

Time	UH ordinate	Critical sequence of hourly effective rainfall		Reversed sequence of hourly effective rainfall		Bell sequence used for convolution [B2-B1]
		1st 12 hr bell (B1)	2nd 12 hr bell (B2)	1st 12 hr bell (B1)	2nd 12 hr bell (B2)	
(hr)	(cumec)	(cm)	(cm)	(cm)	(cm)	(cm)
42	9.45					
48	8.3					
54	7.2					
60	6.3					
66	5.5					
72	4.75					
78	3.95					
84	3.3					
90	2.7					
96	2.15					
102	1.7					
108	1.3					
114	0.95					
120	0.7					
126	0.4					
132	0.2					
138	0					

10 Year Return Period

Time	UH ordinate	Critical sequence of hourly effective rainfall		Reversed sequence of hourly effective rainfall		Bell sequence used for convolution [B2-B1]
		1st 12 hr bell (B1)	2nd 12 hr bell (B2)	1st 12 hr bell (B1)	2nd 12 hr bell (B2)	
(hr)	(cumec)	(cm)	(cm)	(cm)	(cm)	(cm)
0	0					1.99
6	1.053					3.50
12	3	13.18	3.50	7.82	1.99	7.82
18	6	7.82	1.99	13.18	3.50	13.18
24	10.3					
30	11.362					
36	10.6					
42	9.45					
48	8.3					
54	7.2					
60	6.3					
66	5.5					
72	4.75					
78	3.95					
84	3.3					
90	2.7					
96	2.15					

Time	UH ordinate	Critical sequence of hourly effective rainfall		Reversed sequence of hourly effective rainfall		Bell sequence used for convolution [B2-B1]
		1st 12 hr bell (B1)	2nd 12 hr bell (B2)	1st 12 hr bell (B1)	2nd 12 hr bell (B2)	
(hr)	(cumec)	(cm)	(cm)	(cm)	(cm)	(cm)
102	1.7					
108	1.3					
114	0.95					
120	0.7					
126	0.4					
132	0.2					
138	0					

25 Year Return Period

Time	UH ordinate	Critical sequence of hourly effective rainfall		Reversed sequence of hourly effective rainfall		Bell sequence used for convolution [B2-B1]
		1st 12 hr bell (B1)	2nd 12 hr bell (B2)	1st 12 hr bell (B1)	2nd 12 hr bell (B2)	
(hr)	(cumec)	(cm)	(cm)	(cm)	(cm)	(cm)
0	0					2.46
6	1.053					4.29
12	3	15.97	4.29	9.48	2.46	9.48
18	6	9.48	2.46	15.97	4.29	15.97
24	10.3					
30	11.362					
36	10.6					
42	9.45					
48	8.3					
54	7.2					
60	6.3					
66	5.5					
72	4.75					
78	3.95					
84	3.3					
90	2.7					
96	2.15					
102	1.7					
108	1.3					
114	0.95					
120	0.7					
126	0.4					
132	0.2					
138	0					

3.1.7 Flood hydrograph

The reverse sequence of 6 hourly effective rainfall of various return periods has been convoluted with ordinates of the 6-h unit hydrograph to get the direct runoff hydrograph for corresponding return periods. The base flow contribution has been added to get the flood hydrograph. The estimated flood hydrographs are shown in Figure 3.3 and the ordinates are given in Table 3.9.

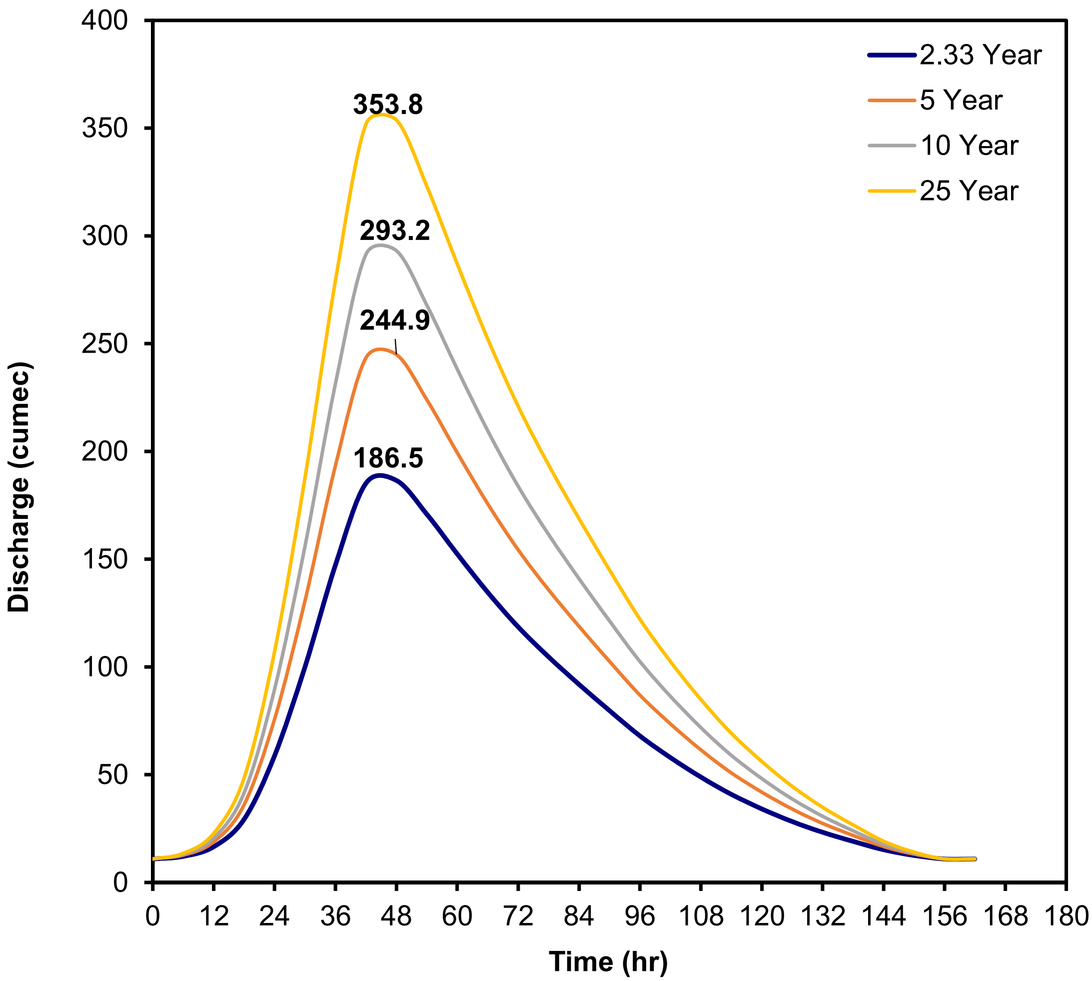

Figure 3.3: Design flood hydrographs for various return periods

Table 3.9: Flood hydrograph for various return periods

Time (hr)	Discharge (cumec)			
	2.33 Year	5 Year	10 Year	25 Year
0	10.9	10.9	10.9	10.9
6	12.1	12.6	13.0	13.5
12	16.6	18.8	20.6	22.8
18	29.3	36.0	41.6	48.5
24	58.9	75.8	89.8	107.3
30	100.1	130.7	156.0	187.9
36	147.3	193.3	231.4	279.2
42	185.2	243.4	291.4	351.8
48	**186.5**	**244.9**	**293.2**	**353.8**
54	170.9	224.0	267.9	323.1
60	152.4	199.3	238.1	286.9
66	134.6	175.7	209.6	252.3
72	118.5	154.3	183.8	220.9
78	104.7	135.8	161.5	193.8
84	91.9	118.8	141.0	168.9
90	79.7	102.5	121.3	145.0
96	68.0	86.9	102.5	122.2
102	58.0	73.6	86.4	102.6
108	48.9	61.5	71.9	84.9
114	40.9	50.8	59.0	69.3
120	34.2	41.9	48.2	56.2
126	28.3	34.0	38.7	44.7
132	23.3	27.4	30.7	34.9
138	19.1	21.8	24.0	26.8
144	15.2	16.6	17.8	19.2
150	12.6	13.1	13.6	14.1
156	10.9	10.9	10.9	10.9

The peak floods for the Mahav Nala are estimated as 186.5 m^3/s, 244.9 m^3/s, 293.2 m^3/s and 353.8 m^3/s for 2.33 Year, 5 Year, 10 Year and 25 Year return period respectively.

3.2 Modelling of Mahav Nala Flow

The river flow model for Mahav Nala from Indo-Nepal border to its confluence with Baghel nala (23 km stretch) is developed in MIKE 11. The modelling is carried out to evaluate the adequacy of the river section at different chainage. The maximum flood level at different chainage is computed and plotted against the bank top level to identify the critical locations where the water spills from the nala. Several setups of flow model are developed using existing river cross sections, proposed and design river cross sections. MIKE 11 is a versatile and modular engineering tool for modelling hydrodynamic conditions in rivers, lakes/reservoirs, irrigation canals and other inland water systems developed by DHI. It is a fully dynamic modelling tool for the detailed analysis, design, management and operation of both simple and complex river and channel systems (DHI, 2004).

3.2.1 Governing Equations

The governing equations in MIKE 11 are 1-D (one-dimensional) and shallow water type, which are the modifications of basic Saint-Venant equations. These are transformed to a set of implicit finite difference equations, and solved using double sweep algorithm (Abbot and Ionescu, 1967). The computational grid comprises of alternating Q and H_l points automatically generated by the model, on the basis of user requirements (Figure 3.4). Q points are always placed midway between neighbouring H_l points. H_l points are located at cross sections or at equidistant intervals, in between if the distance between cross-sections is greater than the maximum space interval, *dx* specified by the users (*dx*=200 m in the present setup).

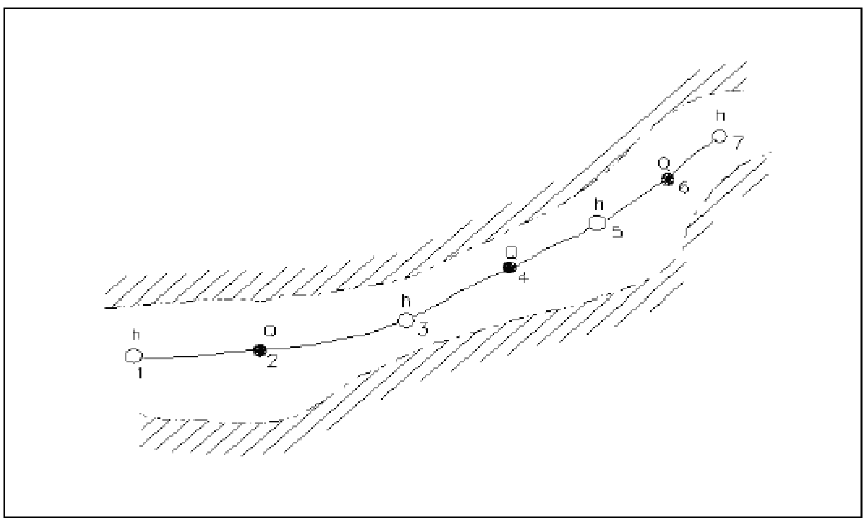

Figure 3.4 : MIKE 11 computational grids

3.2.2 MIKE 11 model setup

In the present study, the MIKE 11 model setup is prepared by defining following five input parameters:

1. Layout of drain (Alignment of Mahav nala extracted from online DEM and Google Earth images from Indo-Nepal border to its confluence with Baghel nala)

2. Cross section data definition (Three setups, (i) existing section obtained from UPID, (ii) proposed section obtained by UPID and (iii) design section based on estimated flood.
3. Defining hydrodynamic boundary conditions (Design flood is impinged at the upstream end while maximum observed flood level in Baghel is defined at the downstream end).
4. Setting the HD parameters (Manning's roughness as 0.0225)
5. Fixing the simulation parameters (time step of 5 sec is used while the flow is simulated for 8 for days)

The alignment of Mahav nala in Google Earth image and its replication in MIKE 11 is shown in Figure 3.5. The location of river section at every 200 m is also shown by black dots. The length of the Mahav Nala from Indo Nepal border (upstream end) to its confluence with Baghel is 23 km (downstream end). In model setup, the Mahav nala with Chainage 0 km at the upstream end and Chainage 23 km at the downstream end is defined. This definition of Chainage is in reverse order given in AutoCAD drawing by UPID (refer Figure 3.6, Figure 3.7 and Figure 3.8).

3.2.2.1 Cross section data definition

The topographical description of the area to be modelled is achieved through the specification of cross-sections of the nala, which lie approximately perpendicular to the direction of flow. Cross-sections are specified by a number of *x-z* co-ordinates where *x* is the transverse distance from a fixed point (often left bank top) and *z* is the corresponding bed elevation. The surveyed cross sections are available at an interval of 200 m and the same have been defined in the model. The cross section based on survey of existing nala and also based on proposed section have been defined at the same interval of 200 m. The *x-z* co-ordinates are entered as raw data in the cross-section editor. The raw data are then automatically processed into a form used in the hydrodynamic calculations, i.e. the hydraulic parameters; cross-sectional area, hydraulic radius and width are calculated for a number of elevations between a minimum and a maximum which are either determined automatically or may be user specified. The details of existing and proposed nala section is given in Annexure IV and the same is used in flow model.

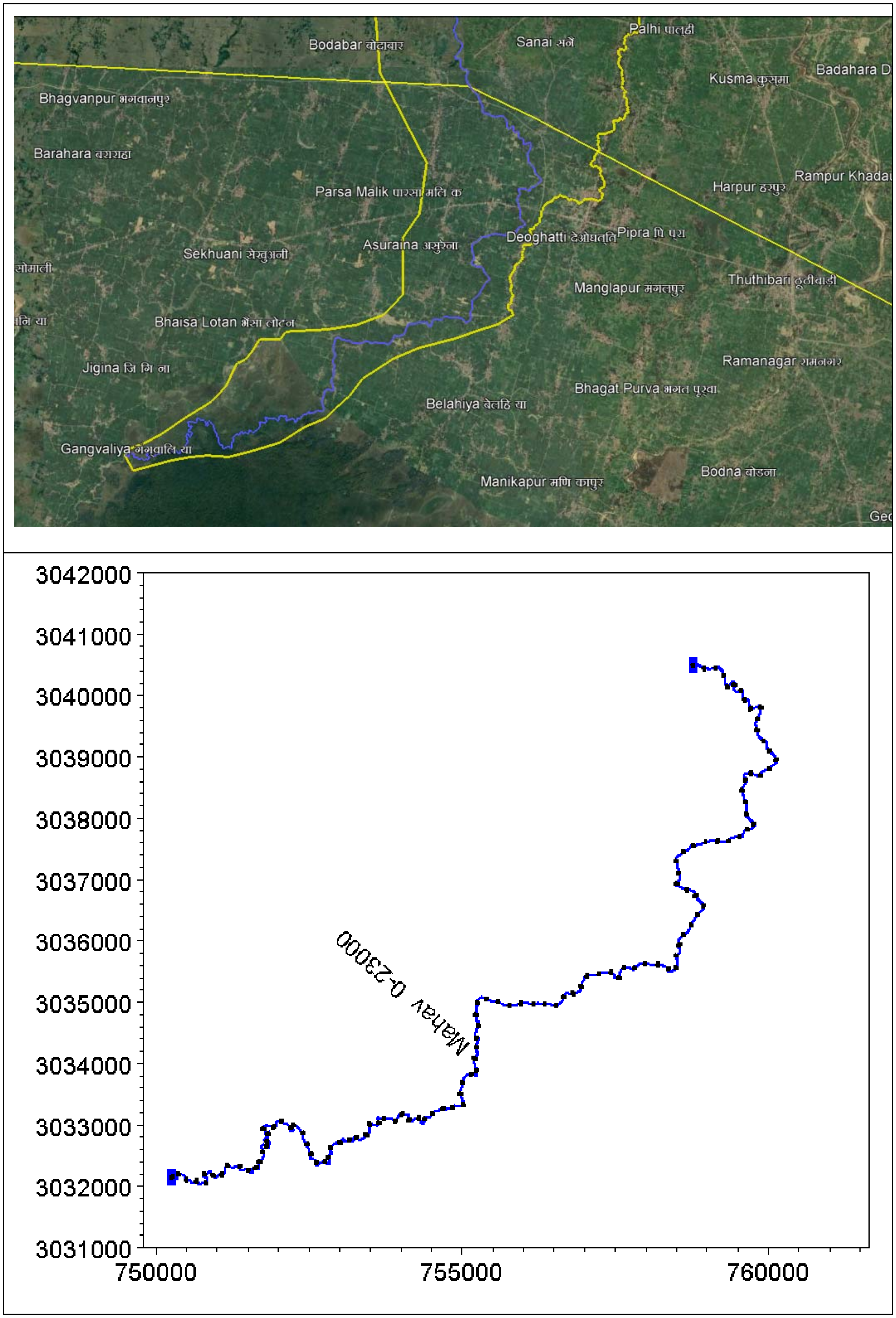

Figure 3.5: Mahav nala alignment in GE image and MIKE 11 flow model.

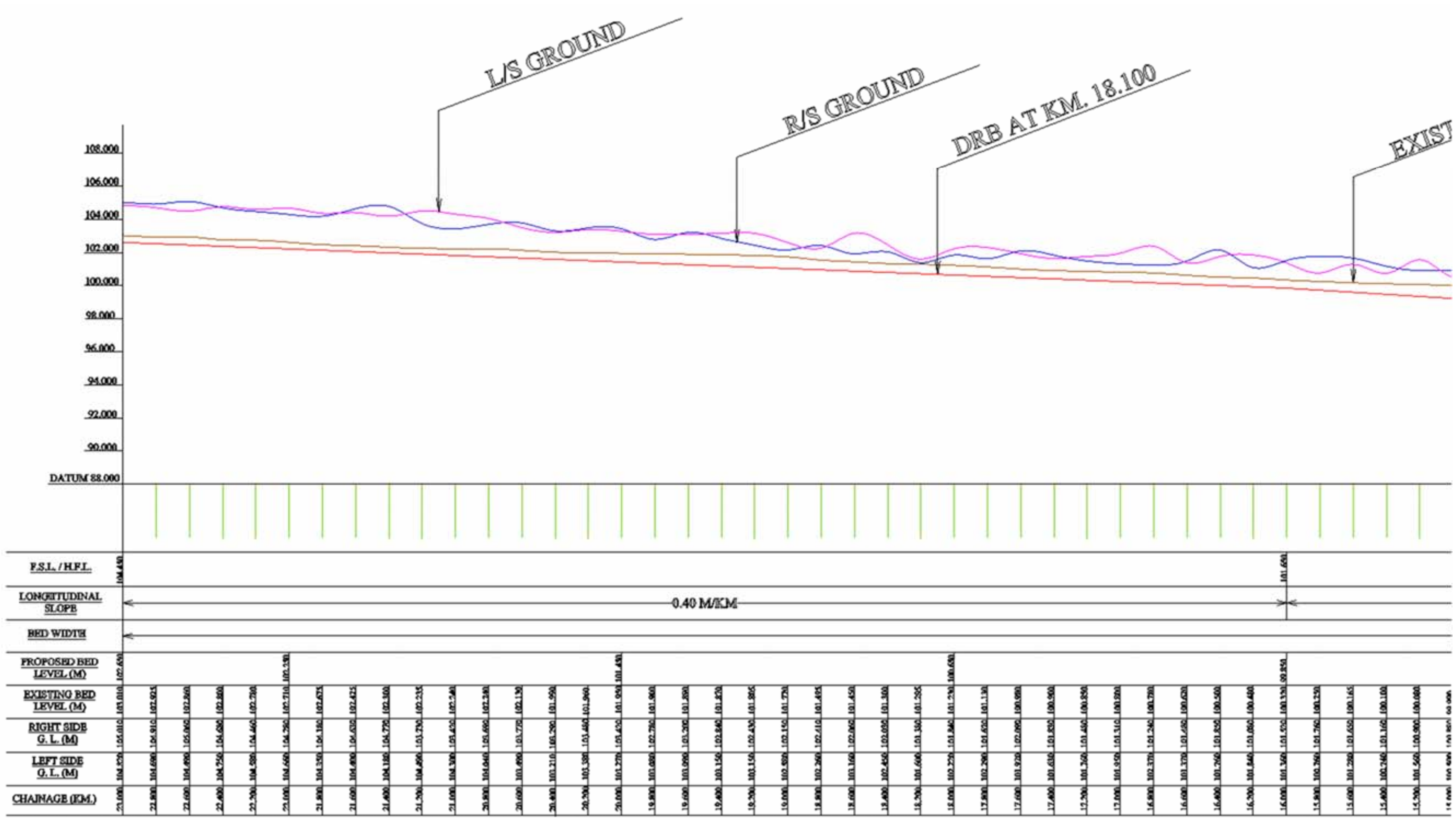

Figure 3.6: Survey details of Mahav nala provided by UPID (Stretch-1)

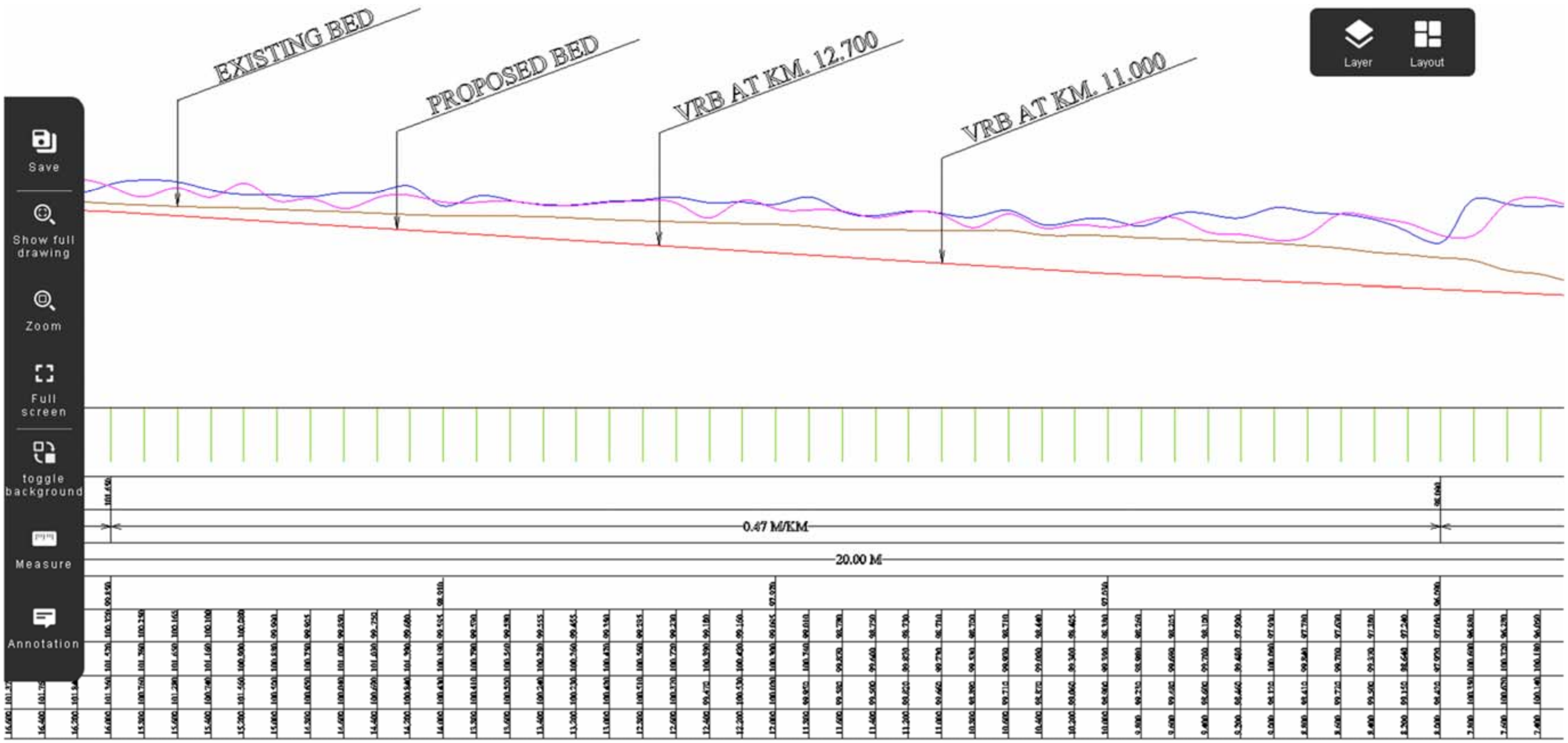

Figure 3.7: Survey details of Mahav nala provided by UPID (Stretch-2)

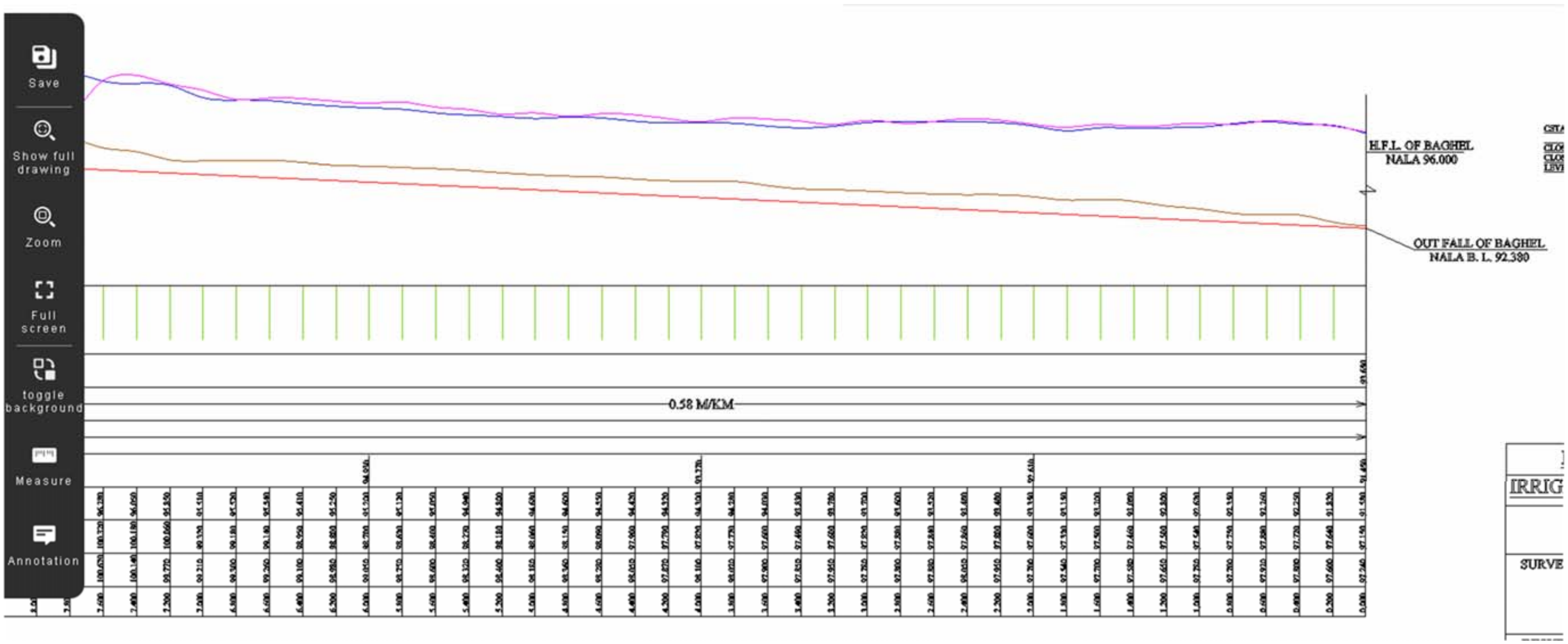

Figure 3.8: Survey details of Mahav nala provided by UPID (Stretch-3)

3.2.2.2 Hydrodynamic boundary condition definition

Boundary conditions are required at all model boundaries, i.e. upstream and downstream ends of model branch. In the present setup the boundary condition at the upstream end is defined for the design discharge value while the downstream boundary condition is defined for constant water level. For design condition, the downstream boundary condition is the observed HFL in Bhagel river i.e. RL 96 m (refer Figure 3.8).

3.2.2.3 Setting the Hydro dynamic (HD) parameters

The HD parameters, in the present study, include the initial conditions of water level and discharge, friction coefficient (*n*) and output parameters options. Initial conditions are required to avoid the dry bed conditions. The ‘n’ value (0.0225) for the drain has been defined as suggested by UPID in their calculation sheet, as shown in Figure 3.9. However, the sensitivity analysis of ‘n’ for computing maximum flood level at few Chainages has also been carried out during the analysis.

DESIGN DATA OF MAHAV NALA

Chainage in Km.			Catchment Area M (km^2)	Discharge factor (C)	Discharge $Q = CM^{3/4}$ (Cumec)	Rugosity factor 'N'	Bed Slope 'S'	Silt factor (m)	Bed Width 'B' (M)	Water Depth 'D' (M)	Side Slope	X-Area $A = (B+D/2)*D$	$V_0 = 0.546*m*D^{0.64}$	$P = B+2.236*D$	$R = A/P$	$V = (Qm^2/140)^{1/6}$	V/V0
0.00	To	8.00	525.65	0.50	54.89	0.0225	0.00058	0.90	20.00	2.20	1/2:1	46.42	0.81	24.92	1.86	0.83	1.02
8.00	To	16.00	484.85	0.50	51.66	0.0225	0.00047	0.90	20.00	2.00	1/2:1	42.00	0.77	24.47	1.72	0.82	1.06
16.00	To	23.00	446.05	0.50	48.53	0.0225	0.00040	0.90	20.00	1.80	1/2:1	37.62	0.72	24.02	1.57	0.81	1.13

Figure 3.9: Calculation of design cross section by UPID.

This estimate is in line with the recommendations specified in standard publications namely; Table-6.3 of IRC:SP:50-2013, (Guidelines on Urban Drainage by Indian Road Congress), Table -3 of BIS 7112-2002 and Table-1 of USGS guidelines (USGS, 1989).

3.2.2.4 Fixing the simulation parameters

Before running the model simulation, control parameters such as simulation period, simulation time step, data to be stored and storage time have to be specified. There exists a versatile relationship between the time step and the computational distance (*dx)* to define the Courant number given below, which is widely considered to choose the time step for the model simulation.

$$\text{Courant number } (C_R) = \frac{\Delta t\left(V + \sqrt{gy}\right)}{\Delta x} \tag{1}$$

Where, Δt = time step, V= mean flow velocity (m/s), y = water level (m) and Δx = *dx*-max. If there is a large value of *dx,* the time step should be chosen so small that the C_R value should be low. Low value of C_R is needed to avoid instability during the model simulation. In the present setup the time step is kept very low as 5 seconds. The simulation has been performed for 8. days (192 hours) period starting from 11/18/2022 8:00:00 AM to 11/26/2022 8:00:00 AM, although the design hyetograph is of 156 hours duration. This selection of start date is randomly selected and does not affect the results.

3.3 Design of Drains

Due to the dynamic nature of open channel flow, it is difficult to model and/or analyze the flow as it occurs in the natural environment. Therefore, a number of assumptions are made to enable and simplify analysis. These assumptions are considered reasonable and valid in the context of the analysis and design procedures described in this manual. The assumptions made are:

- Steady Flow;
- Uniform Flow;
- Velocity is averaged over the whole cross section; and
- Flow is non-turbulent.

3.3.1 Fundamental Equations

The most basic equation in the analysis of open channel flow defines the relationship between flow rate, velocity and the cross-sectional area of flow and is represented by the following formula:

$$Q = V \times A \tag{2}$$

Where: Q = flow rate (m^3/s); V = average velocity (m/s); and A = cross-sectional area of flow (m^2).

This equation also forms the basis of the theory behind the Continuity Equation. The theory allows simple analysis over changes in the channel irrespective of cross section, slope or roughness. The theory assumes no addition to or subtraction from the flow between the 2 sections being considered (i.e. $Q_1 = Q_2$).

The Continuity Equation is:

$$Q_1 = V_1 \times A_1 = V_2 \times A_2 = Q_2 \tag{3}$$

The formula most commonly used for the calculation of steady, uniform flow in open channels is Manning's Equation. This equation is used to determine the velocity of flow at a specific point in the channel, and therefore the variables in the equation must be representative of the point being assessed.

Manning's Equation is: $V = \frac{1}{n} R^{2/3} S^{1/2}$ (4)

Where: V = average velocity (m/s); R = hydraulic radius (m); S = slope of energy line (m/m); and n = Manning's roughness coefficient.

The hydraulic radius R is given by: $R = \frac{A}{P}$ (5)

Where: A = cross-sectional area of flow (m^2); P = wetted perimeter (m).

In determining the hydraulic radius, the wetted perimeter is defined as the length of line (normal to the flow) where the water touches the surface of the ground (channel) or the perimeter of flow less the surface (exposed to the atmosphere) length as shown in Figure 3.10.

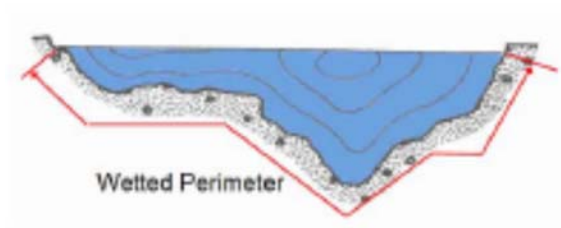

Figure 3.10: Wetted perimeter in natural channel

For natural channels the slope of the energy line, S, is almost impossible to determine. Therefore, a suitable estimation of S is required. The slope of the water (flow) surface, S_w, could be used, however this is also difficult to determine. The only easily available slope that can be used to estimate S is the channel bed, S_o, provided that the channel and the bed slope about the point being assessed is reasonably uniform.

In applying Manning's Equation, particularly to natural channels, the greatest difficulty lies in the determination of the roughness coefficient n. For the natural and artificial channels values of n can be obtained from Table-6.3 of IRC:SP:50-2013, (Guidelines on Urban Drainage by Indian Road Congress), Table -3 of BIS 7112-2002 and Table-1 of USGS guidelines (USGS, 1989).

3.3.2 Design Methodology

The nala cross-section that should convey the design flow down a slope producing a velocity which does not create scour or silting. In the study natural drains in fine sand soil has been considered based on particle size analysis. The design considerations are detailed in the following sections.

3.3.2.1 General Considerations

The velocity of flow in open channels is a key aspect. Channel slope is an important factor in flow velocity. Generally, flow in channels is intermittent, and the channel must be constructed to allow all storm water to drain away (no ponding). In Table 3, BIS 7784:1993 specifies the maximum velocity criteria for various types of channels. The guideline specifies the non-eroding maximum flow velocity for silty channel in the range of 0.7 -1 m/s. The channel side slopes should be based on the slope stability of the material. Typically, this will be in the range 1:1 to 1.5:1 (section 4.2 of BIS 7112-2002). The design steps are discussed as below:

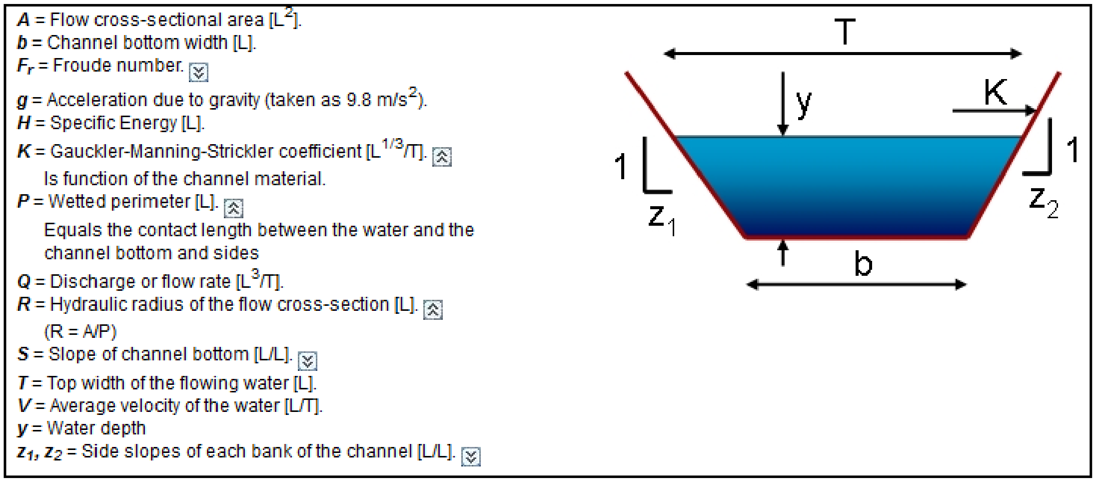

Figure 3.11: Flow hydraulics attributes used for design of channel

For the design of channels the design steps includes:

(a) Determine design discharge (Q), bed slope (S_o), Manning's n-value and channel side slopes Z_1 and Z_2.

$$A = \frac{y}{2}(b+T),\ \ P = b + y\left(\sqrt{1+z_1^2} + \sqrt{1+z_2^2}\right), T = b + y(z_1 + z_2) \tag{6}$$

$$F_r = V\sqrt{\frac{T}{gA\cos\theta}} \text{ and } \theta = \tan^{-1} S_o \tag{7}$$

Combining Manning's and Continuity Equations:

$$Q = \frac{AR^{2/3}S_f^{1/2}}{n} \tag{8}$$

S_o is used to approximate S_f (Friction Slope)

Calculate the channel geometry and add freeboard. Calculate Froude Number for channel section / flow and together with velocity and depth of flow; check that conditions are acceptable where the designed channel discharges into an existing channel etc.

4 ANALYSIS AND RESULTS

4.1 Particle Size Analysis of nala bed Material

The soil samples from nala bed were collected during the field visit on

4.2 Estimation of Design Flood

The design discharge estimate obtained from UPID is 64 m^3/s for Mahav nala while the estimated design discharge corresponding to Average Annual Flood (return period of 2.33 years) computed by NIH is about 187 m^3/s.

4.3 Flow simulation in Mahav nala with existing cross section

The flow model is developed with existing cross section and proposed cross sections separately. The available flow depth (difference between top of bank level and nala bed level) varies from 0.19 m (at AutoCAD Chainage 18.2 km and MIKE 11 Chainage 4800 m) to 5.28 m (at AutoCAD Chainage 0.2 km and MIKE 11 Chainage 22800 m) for existing cross section The longitudinal profile of Mahav nala with the existing cross sections is shown in Figure 4.1. The figure shows that the available flood depth in upper reach of Mahav nala (up to MIKE 11 Chainage 15000) is smaller, in the range of 0.19-1.76 m compared to lower reach where flow depth varies in the range of 2.77 – 5.28 m. The width of nala varies from 7.77 m to 18.87 m with the average of about 11.8 m. The available top width is wider in upper reach compared to lower reach.

The flow model is simulated for design flood of 64 m^3/s with the existing cross section. The maximum water surface profile for this flood case is shown in Figure 4.2. The figure shows that the flood level exceeds excessively at all the sections. In subsequent simulations, the inflow flood (in place of design flood of 64 m3/s) is reduced gradually to lower magnitude of 25 m^3/s, 10 m^3/s and 5 m^3/s to evaluate the adequacy of existing cross section to pass these flows without spilling. The maximum water surface profiles for these floods of 25 m^3/s, 10 m^3/s and 5 m^3/s are shown in Figure 4.3, Figure 4.4 and Figure 4.5, respectively. These figures shows that the Mahav nala overtops at almost all chainage in upper reach for inflow of 25 m^3/s. Even for small inflow of 10 m^3/s it spills in the upper reach at most of the locations. The reason for spilling in upper reach is significant reduction in flow depth due to silting in this reach. The smaller nala cross section in the lower reaches causes flow congestion and thus velocity drop which in turn is responsible for siltation in upper reach. Thus, even if the profile of left and right banks in upper reach is maintained, the carrying capacity of the Mahav nala with existing cross section is limited to about 10 m^3/s only.

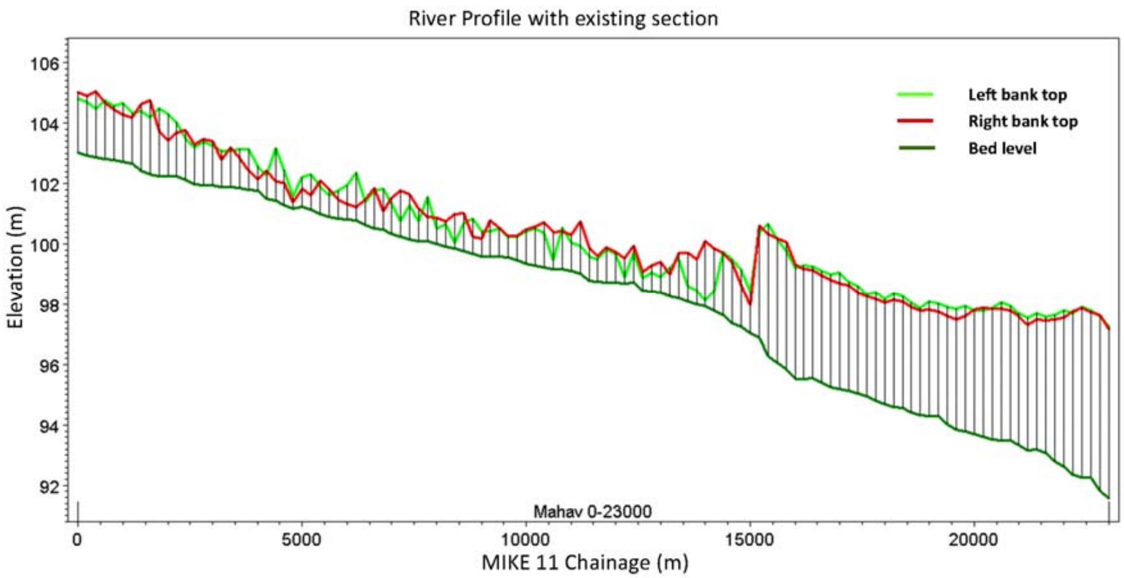

Figure 4.1: Longitudinal profile of Mahav nala with existing cross section.

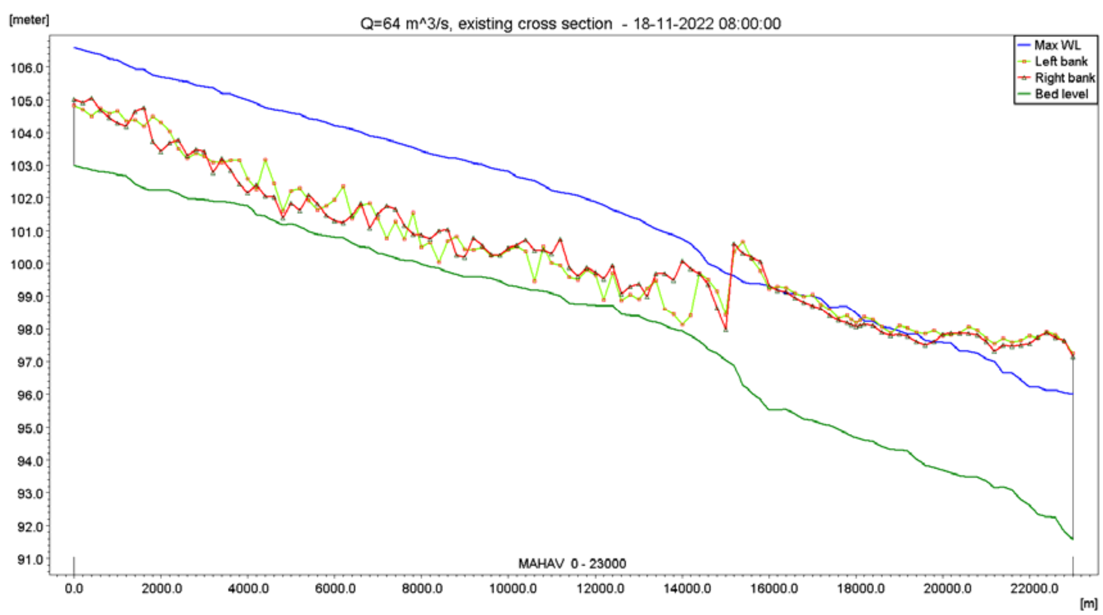

Figure 4.2: Maximum flood level profile for Q=64 m3/ s with existing cross section.

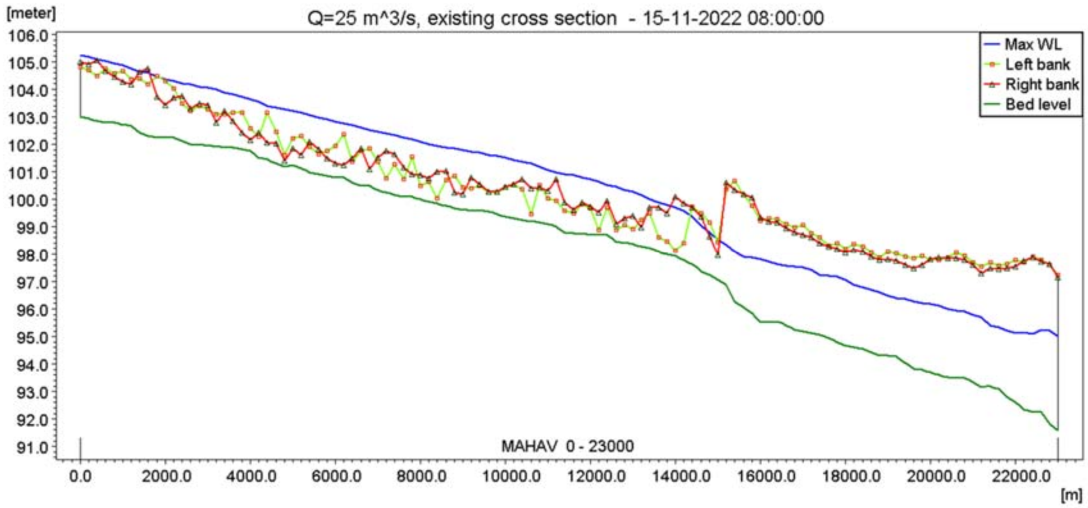

Figure 4.3: Maximum flood level profile for Q=25 m3/ s with existing cross section.

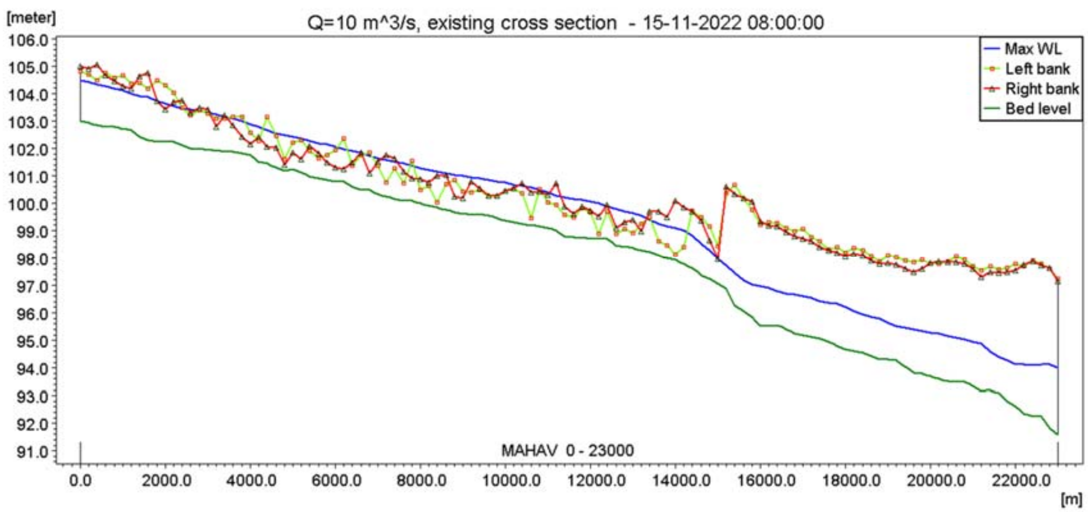

Figure 4.4: Maximum flood level profile for Q=10 m3/ s with existing cross section.

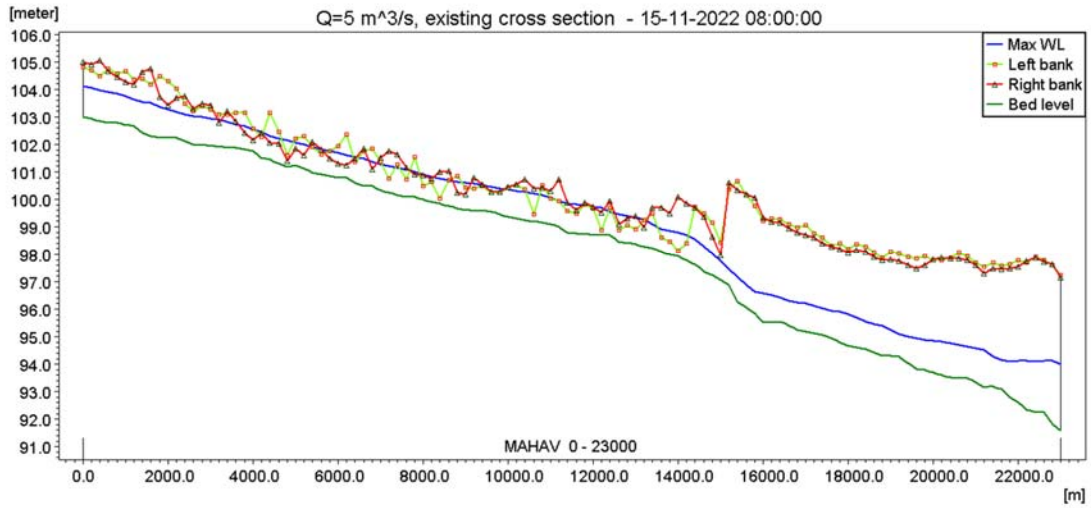

Figure 4.5: Maximum flood level profile for Q=5 m3/ s with existing cross section.

4.3.1 Identification of flow congestion location

As discussed above, the carrying capacity of the Mahav nala is very limited (of the order of 10m^3/s), however sufficient nala depth for flow is available downstream. To identify the location of congestion, a time series of inflow hydrograph with peak of 10 m^3/s is simulated in the flow model setup with existing cross section and the flow depth and velocity pattern at various chainage is studied. The plot of flow depth pattern and velocity pattern are shown in Figure 4.6 and Figure 4.7, respectively. The time series of inflow hydrograph with peak value of 10 m^3/s is developed considering the shape of design flood discussed in section 3.1.7. The inflow hydrograph is shown by green line and plotted on secondary Y axis in Figure 4.6 and Figure 4.7. On primary Y axis, the flood depth is plotted in Figure 4.6. This figure shows that as the flood moves downstream, the flow depth is decreasing from Ch. 10000 m to 15200 m. However, suddenly at Ch. 15400 m, the flow depth increases. This is due to flow congestion at this location. The velocity plot in Figure 4.7 also shows the sudden fall of flow velocity at Ch. 15200 m which is again due to congestion. Hence, the flow congestion in Mahav nala starts at Chainage 15200 m.

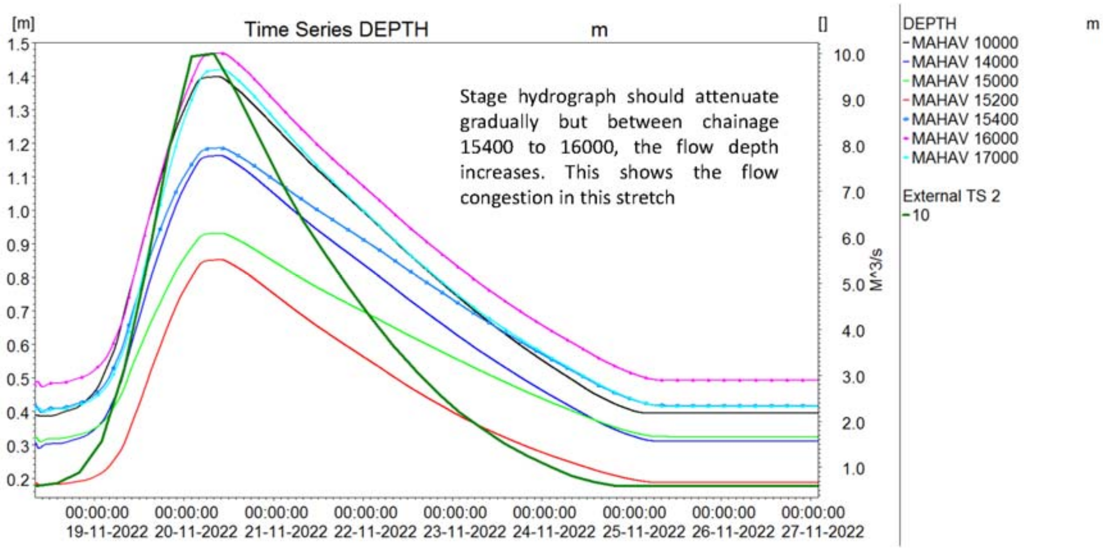

Figure 4.6: Flow depth pattern at various chainage due to time series inflow hydrograph with peak flood of 10 m3/s.

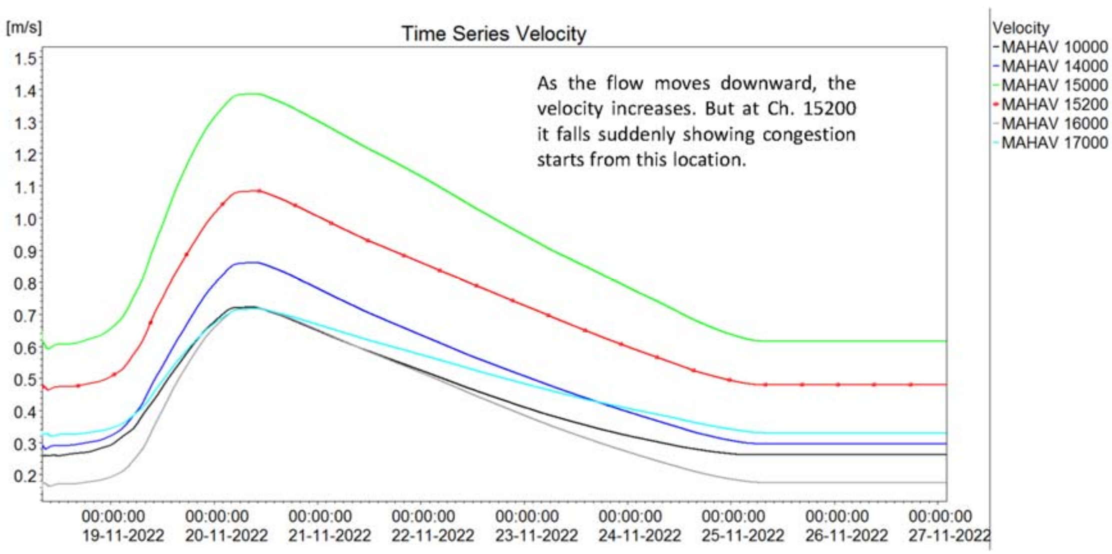

Figure 4.7: Flow velocity pattern at various chainage due to time series inflow hydrograph with peak flood of 10 m3/s.

4.4 Sensitivity of Manning's roughness

In absence of observed hydrological data, Manning's n has been used based on UPID suggestions and other standard literatures as discussed in section 3.2.2.3. However, being the only calibration parameter for the flow model, the sensitivity of adopted 'n' values are very much recommended. To carry out the sensitivity analysis flow model with existing cross section and inflow condition of 10 m^3/s is simulated with varying Manning's roughness value of 0.0215, 0.0225 and 0.0235. The time series of water level variation at selected cross section locations are plotted and compared to visualize the effect of Manning' s 'n'. The time series plot at MIKE 11 Ch. 600 m and 1500 m are shown in Figure 4.8 and Figure 4.9, respectively. The figure shows that with the increase in Manning's 'n', the maximum flood level increase at both the sections. Similarly, with the decrease in 'n' value, the maximum flood level decreases. However, the increase or decrease is very small, of the order of 0.025 m with the variation of about 4% change in the value of 'n'. Hence, the adopted 'n' value may be considered as adequate to represent the flow roughness in Mahav nala.

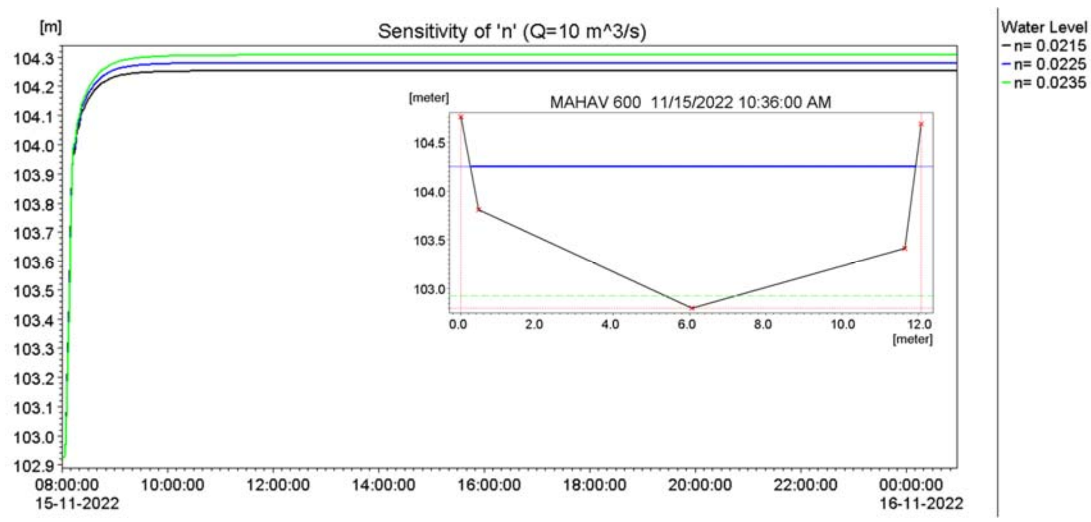

Figure 4.8: Sensitivity of Manning's n at Ch 600 m.

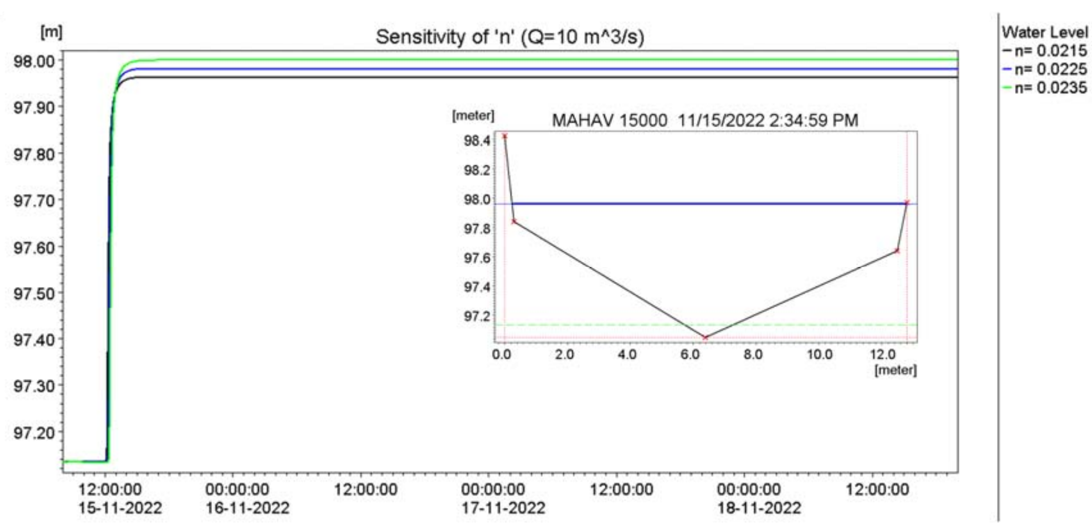

Figure 4.9: Sensitivity of Manning's n at Ch 1500 m.

4.5 Flow simulation in Mahav nala with proposed section

Hence, the nala section is to be modified for increasing its carrying capacity. The carrying capacity can be improved by increasing the nala section (flow depth and flow width) or improving the longitudinal slope of the nala.

4.5.1 Mahav nala section proposed by UPID

UPID has proposed to increase the flow depth (of the order of 2 m) and flow width (bed width of 20 m) throughout the stretch of Mahav nala. The proposed section details is shown in Figure 3.9. The bed level at various chainage is also modified as given in Annexure-IV. Using the data for proposed river section at various chainage of Mahav nala, the flow model is again developed.

4.5.2 Carrying capacity of Mahav nala with proposed section

The updated flow model is simulated for design flood of 64 m^3/s. The maximum water surface profile for this case is shown in Figure 4.10. The figure shows that even for modified cross section, the Mahav nala spills at most of the locations in the upper reach. The carrying capacity of the nala at various chainage is computed based on lowest bank level (lower of top level of left or right bank) and other parameters for proposed section. The carrying capacity of the Mahav nala at various chainage is shown in Table 4.1. The chainage at which the carrying capacity is smaller than 64 m^3/s is highlighted in red colour. The highlighted sections are the critical locations of spilling for this design flood. The table shows that even with the proposed section, the carrying capacity could not be improved sufficiently to safely pass the estimated design flood of 64 m^3/s. The table shows that in the upper reach (from Ch.0 to 15 km), the carrying capacity is lower than 64 m^3/s at several locations. The reason may be attributed to smaller flow depth in upper reach due to siltation. Although, the carrying capacity of nala is significantly higher in the lower reach due to higher flow depth (downstream of Ch. 15 km), the discontinuity of conveyance throughout the nala stretch reduces the overall conveyance efficiency of the nala. This, the proposed section is also inadequate for the UPID estimated design flood of 64 m^3/s.

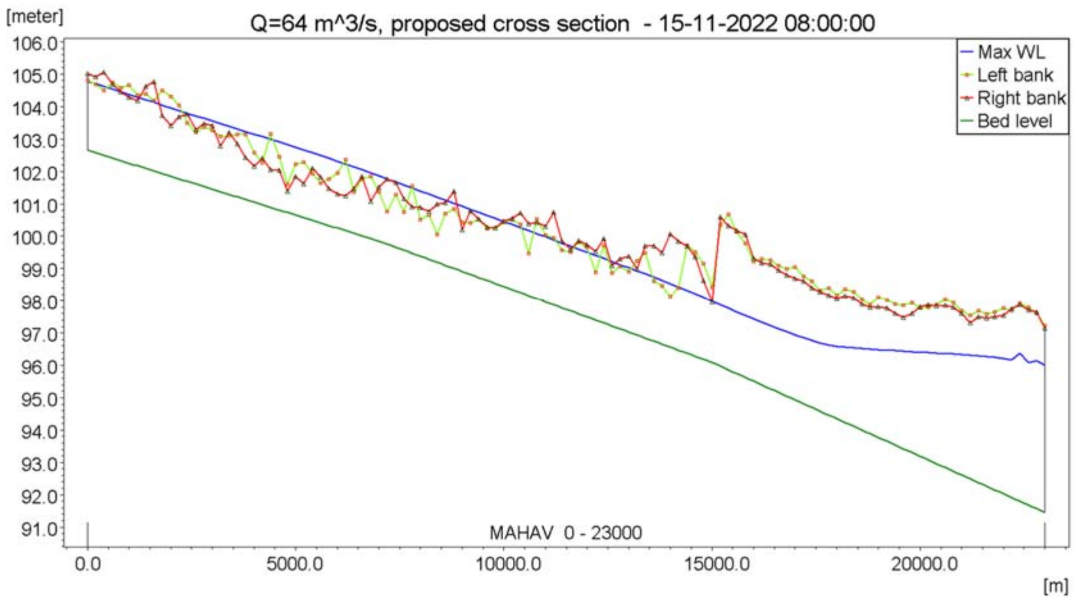

Figure 4.10: Maximum flood level profile for Q=64 m3/ s with proposed cross section.

Table 4.1: Carrying capacity if Mahav nala with proposed section at various chainage.

MIKE 11 Ch. (m)	lowest bank Level (m)	CS Area (m2)	Top width 9m)	Carrying capacity (m^3/s)
0	104.82	45.754	22.17	74.25052
200	104.69	44.647	22.12	71.35803
400	104.49	42	22	64.61668
600	104.68	47.976	22.27	80.18177
800	104.46	44.868	22.13	71.9326
1000	104.28	42.66	22.03	66.27535
1200	104.18	42.22	22.01	65.16759
1400	104.4	48.868	22.31	82.6086
1600	104.18	45.754	22.17	74.25052
1800	104.11	45.94	21.99	74.82802
2000	103.86	42.172	21.79	65.15485
2200	103.69	40.243	21.92	60.28111
2400	103.49	37.62	21.8	54.0184
2600	103.21	33.28	21.6	44.23252
2800	103.38	38.711	21.85	56.59257
3000	103.27	38.056	21.82	55.04199
3200	102.78	29.194	21.41	35.7081
3400	103.09	37.62	21.8	54.0184
3600	102.84	33.928	21.63	45.64782
3800	102.79	34.545	21.48	47.0744

MIKE 11 Ch. (m)	lowest bank Level (m)	CS Area (m2)	Top width 9m)	Carrying capacity (m^3/s)
4000	102.15	22.605	21.1	23.47695
4200	102.26	26.632	21.29	30.72214
4400	102.61	35.804	21.445	49.9765
4600	102.03	25.144	21.22	27.95909
4800	101.38	13.211	20.65	9.689804
5000	101.84	24.508	21.19	26.80821
5200	101.955	28.631	21.218	34.63705
5400	101.92	29.622	21.43	36.56933
5600	101.63	25.144	21.22	27.95909
5800	101.48	23.661	21.15	25.30505
6000	101.63	28.527	21.22	34.42672
6200	101.805	33.957	21.353	45.8534
6400	101.37	26.419	21.28	30.32081
6600	101.76	36.531	21.75	51.49499
6800	101.46	31.734	21.34	40.99387
7000	101.36	31.34	21.51	40.09953
7200	101.26	31.149	21.254	39.80726
7400	101.28	33.669	21.618	45.07946
7600	100.74	24.127	21.172	26.12734
7800	101.23	36.635	21.591	51.79
8000	100.5	23.027	21.12	24.20137
8200	100.65	28.21	21.364	33.75994
8400	100.52	27.384	21.088	32.24607
8600	100.69	33.107	21.592	43.85828
8800	100.84	38.405	21.836	55.86671
9000	100.19	26.419	21.28	30.32081
9200	100.41	33.15	21.594	43.95173
9400	100.5	37.141	21.778	52.90183
9600	100.24	33.539	21.612	44.7964
9800	100.23	35.358	21.696	48.8271
10000	100.42	41.56	21.98	63.52084
10200	100.51	45.621	22.164	73.90084
10400	100.37	44.603	22.118	71.24336
10600	99.93	36.957	21.542	52.58289
10800	100.42	49.895	22.356	85.4376
11000	100.02	43.101	22.05	67.39102
11200	100.345	52.389	22.272	92.55284
11400	99.58	37.576	21.798	53.91649
11600	99.5	37.882	21.812	54.63158
11800	99.82	46.998	22.226	77.54807
12000	99.66	45.533	22.16	73.66811
12200	99.21	37.682	21.644	54.21807
12400	99.71	50.835	22.398	88.05617

MIKE 11 Ch. (m)	lowest bank Level (m)	CS Area (m2)	Top width 9m)	Carrying capacity (m^3/s)
12600	98.87	34.405	21.652	46.69756
12800	99.06	40.594	21.936	61.13801
13000	98.9	39.148	21.87	57.63637
13200	98.98	42.969	22.044	67.05548
13400	99.48	56.282	22.638	103.796
13600	99.15	50.849	22.127	88.28333
13800	98.97	48.937	22.061	82.95627
14000	99.59	64.65	22.295	131.0295
14200	99.125	56.587	22.302	105.0288
14400	99.7	72.098	23.328	154.536
14600	99.37	66.62	23.092	136.1483
14800	98.64	52.136	22.456	91.72783
15000	97.97	39.367	21.88	58.16131
15200	99.809	84.06	23.835	197.4743
15400	99.769	85.856	23.911	204.2375
15600	99.053	71.701	23.311	153.1775
15800	99.77	91.466	24.144	225.8653
16000	99.21	80.845	23.7	185.573
16200	99.18	82.887	23.786	193.1026
16400	99.14	84.698	23.862	199.8669
16600	98.95	82.934	23.788	193.2793
16800	98.8	82.126	23.754	190.2854
17000	98.7	82.506	23.77	191.6918
17200	98.62	83.363	23.806	194.8727
17400	98.4	80.892	23.702	185.7466
17600	98.27	80.561	23.688	184.5329
17800	98.18	81.177	23.714	186.7897
18000	98.06	81.082	23.71	186.4417
18200	98.15	85.988	23.916	204.7358
18400	98.09	87.328	23.972	209.8399
18600	97.9	85.557	23.898	203.1071
18800	97.79	85.701	23.904	203.6493
19000	97.82	89.201	24.05	217.0421
19200	97.77	90.791	24.116	223.2207
19400	97.6	89.49	24.062	218.1597
19600	97.49	89.634	24.068	218.7195
19800	97.6	95.099	24.294	240.2689
20000	97.78	102.334	24.59	269.8564
20200	97.8	105.688	24.726	283.9669
20400	97.84	109.557	24.882	300.5514
20600	97.405	101.677	24.563	267.1192
20800	97.303	102.021	24.577	268.5502
21000	97.07	99.146	24.46	256.672

MIKE 11 Ch. (m)	lowest bank Level (m)	CS Area (m2)	Top width 9m)	Carrying capacity (m^3/s)
21200	97.32	108.165	24.826	294.549
21400	97.041	104.132	24.663	277.3901
21600	96.922	104.045	24.66	277.0268
21800	96.968	108.072	24.822	294.149
22000	97.068	113.438	25.038	317.506
22200	97.027	115.325	25.113	325.8622
22400	97.161	121.641	25.363	354.3604
22600	97.042	121.552	25.36	353.9543
22800	96.887	120.577	25.321	349.5014
23000	96.523	114.315	25.073	321.3803

4.5.3 Safe carrying capacity of proposed section

The model is further simulated with smaller inflow of 50 $m^3/$ to evaluate the safe carrying capacity. The maximum water surface profile for this flooding scenario is shown in Figure 4.11. The figure shows that even for this reduced inflow, the water spills at several locations in the upper reach. However, these spilling can be tactfully controlled if the profile of left and right banks are maintained. The maximum flood computed for this flooding condition is compared with the lowest bank level to evaluate the spilling of flood water at specified location. The chainage locations where spilling is estimated is shown in Table 4.2. The locations where spilling occurs may be modified for its bank level to accommodate the maximum flood level computed for this flooding case. The table also shows the depth of overtopping at various chainage and it specifies the magnitude of modification needed for that section. With such modification, the carrying capacity of the Mahav nala with proposed section can be improved to 50 m^3/s.

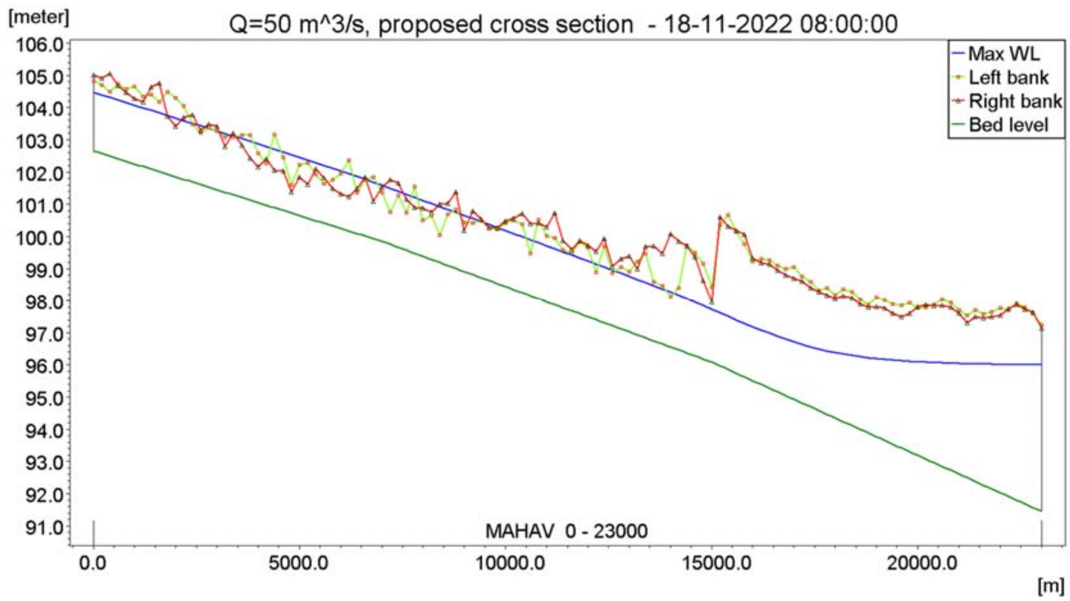

Figure 4.11: Maximum flood level profile for Q=50 m3/ s with proposed cross section.

Table 4.2:Chainage locations of spilling

MIKE 11 Ch. (m)	Lowest bank ground level (m)	Q50 with proposed section		Over topping by (m)
		Maximum FL (m)	Spilling	
0	104.82	104.47	No	0
200	104.69	104.39	No	0
400	104.49	104.31	No	0
600	104.68	104.23	No	0
800	104.46	104.15	No	0
1000	104.28	104.07	No	0
1200	104.18	103.99	No	0
1400	104.4	103.91	No	0
1600	104.18	103.83	No	0
1800	104.11	103.75	No	0
2000	103.86	103.67	No	0
2200	103.69	103.59	No	0
2400	103.49	103.51	Yes	0.022
2600	103.21	103.43	Yes	0.222
2800	103.38	103.35	No	0
3000	103.27	103.27	Yes	0.001
3200	102.78	103.19	Yes	0.411
3400	103.09	103.11	Yes	0.02
3600	102.84	103.03	Yes	0.19
3800	102.79	102.95	Yes	0.159
4000	102.15	102.87	Yes	0.717
4200	102.26	102.79	Yes	0.527
4400	102.61	102.71	Yes	0.095
4600	102.03	102.62	Yes	0.594
4800	101.38	102.54	Yes	1.159
5000	101.84	102.46	Yes	0.618
5200	101.955	102.38	Yes	0.42
5400	101.92	102.29	Yes	0.373
5600	101.63	102.21	Yes	0.579
5800	101.48	102.12	Yes	0.644
6000	101.63	102.04	Yes	0.408
6200	101.805	101.95	Yes	0.147
6400	101.37	101.86	Yes	0.494
6600	101.76	101.78	Yes	0.015
6800	101.46	101.68	Yes	0.223
7000	101.36	101.59	Yes	0.229

MIKE 11 Ch. (m)	Lowest bank ground level (m)	Q50 with proposed section		Over topping by (m)
		Maximum FL (m)	Spilling	
7200	101.26	101.50	Yes	0.235
7400	101.28	101.40	Yes	0.121
7600	100.74	101.31	Yes	0.567
7800	101.23	101.21	No	0
8000	100.5	101.12	Yes	0.618
8200	100.65	101.03	Yes	0.375
8400	100.52	100.93	Yes	0.41
8600	100.69	100.84	Yes	0.147
8800	100.84	100.74	No	0
9000	100.19	100.65	Yes	0.458
9200	100.41	100.56	Yes	0.145
9400	100.5	100.46	No	0
9600	100.24	100.37	Yes	0.127
9800	100.23	100.27	Yes	0.043
10000	100.42	100.18	No	0
10200	100.51	100.08	No	0
10400	100.37	99.99	No	0
10600	99.93	99.90	No	0
10800	100.42	99.80	No	0
11000	100.02	99.71	No	0
11200	100.345	99.61	No	0
11400	99.58	99.52	No	0
11600	99.5	99.43	No	0
11800	99.82	99.33	No	0
12000	99.66	99.24	No	0
12200	99.21	99.14	No	0
12400	99.71	99.05	No	0
12600	98.87	98.95	Yes	0.08
12800	99.06	98.85	No	0
13000	98.9	98.76	No	0
13200	98.98	98.66	No	0
13400	99.48	98.57	No	0
13600	99.15	98.47	No	0
13800	98.97	98.37	No	0
14000	99.59	98.27	No	0
14200	99.125	98.17	No	0
14400	99.7	98.06	No	0
14600	99.37	97.96	No	0
14800	98.64	97.85	No	0

MIKE 11 Ch. (m)	Lowest bank ground level (m)	Q50 with proposed section		Over topping by (m)
		Maximum FL (m)	Spilling	
15000	97.97	97.74	No	0
15200	99.809	97.62	No	0
15400	99.769	97.51	No	0
15600	99.053	97.40	No	0
15800	99.77	97.30	No	0
16000	99.21	97.19	No	0
16200	99.18	97.09	No	0
16400	99.14	96.99	No	0
16600	98.95	96.89	No	0
16800	98.8	96.80	No	0
17000	98.7	96.72	No	0
17200	98.62	96.64	No	0
17400	98.4	96.57	No	0
17600	98.27	96.50	No	0
17800	98.18	96.44	No	0
18000	98.06	96.39	No	0
18200	98.15	96.34	No	0
18400	98.09	96.30	No	0
18600	97.9	96.26	No	0
18800	97.79	96.23	No	0
19000	97.82	96.20	No	0
19200	97.77	96.17	No	0
19400	97.6	96.15	No	0
19600	97.49	96.13	No	0
19800	97.6	96.11	No	0
20000	97.78	96.10	No	0
20200	97.8	96.09	No	0
20400	97.84	96.07	No	0
20600	97.405	96.06	No	0
20800	97.303	96.06	No	0
21000	97.07	96.05	No	0
21200	97.32	96.04	No	0
21400	97.041	96.03	No	0
21600	96.922	96.03	No	0
21800	96.968	96.02	No	0
22000	97.068	96.02	No	0
22200	97.027	96.01	No	0
22400	97.161	96.01	No	0
22600	97.042	96.01	No	0

MIKE 11 Ch. (m)	Lowest bank ground level (m)	Q50 with proposed section		Overtopping by (m)
		Maximum FL (m)	Spilling	
22800	96.887	96.00	No	0

MIKE 11 Ch. (m)	Lowest bank ground level (m)	Q50 with proposed section		Overtopping by (m)
		Maximum FL (m)	Spilling	
23000	96.523			0

4.6 Design of Mahav nala for design flood estimate

Based on PMP atlas, the most recurrent flood (T=2.33 years) for Mahav nala is computed as 187 m^3/s. The section of Mahav nala is designed for this estimated design flood. As mentioned earlier, various characterises of river section affecting the channel carrying capacity are the size and shape of river and its longitudinal slope. The size and shape of the trapezoidal channel section is further specified through its flow depth, side slope and bed width.

The flow analysis with the proposed cross section (having average flow depth of 2 m) shows that the safe carrying capacity that can be achieved is of the order of 50 m^3/s. Further, the longitudinal profile of Mahav nala with proposed cross section shows that in the lower reach the average flow depth greater than 4 m is available which in turn, should, develop higher conveyance in this reach. However, due to discontinuity in the conveyance of nala throughout the stretch, the higher conveyance potential in the downstream reach of the nala remains unutilized. This is primarily due to constriction of nala section in the downstream reach (Chainage 18 km onwards in the forested area) of Mahav nala. To effectively discharge the design flood of Mahav nala to Bhaghel stream without spilling to the adjacent villages, the adequate nala cross section and nala slope are required throughout the stretch. For this the nala bed depth and bed width throughout the Mahav nala needs to be modified. However, the nala profile at the outfall location (confluence with Bhaghel) is to be kept unaltered. The modified bed profile of the Mahav nala is shown in Figure 4.12. The bed level of the nala at the India Nepal border is proposed to lower down by approximately 2 m so that overall depth of 4 m can be achieved. Such lowering of the bed level will also affect the longitudinal profile of Mahav nala and its longitudinal slope would be reduced from 0.000488 m/m to 0.000401 m/m. To estimate the suitable section, the carrying capacity of the Mahav nala is evaluated for evaluated for various combination of nala section (refer Figure 3.11). The computed carrying capacity of section is shown in Table 4.3. In the table trapezoidal section is considered with similar side slope for left and right bank. The side slope of 0.5 and 1 have been evaluated. Nala section at

SN 2 represents the proposed section by UPID for which the carrying capacity of 62 To enhance the carrying capacity, the bed width may also be increased; if it is increased by twice, the discharge also increases almost two times (see SN1 with SN3 and SN 2 with SN 4). Other possibility is to increase the flow depth as proposed earlier. Increasing the flow depth by additional 2 m, the carrying capacity is increased by about 3 times (compare SN1 with SN 5 and SN 2 with SN6). However, with increasing the flow depth to 4 m the archived carrying capacity for section is 180 m^3/s only while the estimated design discharge is 187 m^3/s. To achieve the safely pass the estimate design discharge, the bed width needs to be increased to 22 m. This combination of shape and size of nala section will have safe carrying capacity of 198 m^3/s (see SN 8). It is suggested to adopt this section for improving the carrying capacity of Mahav nala.

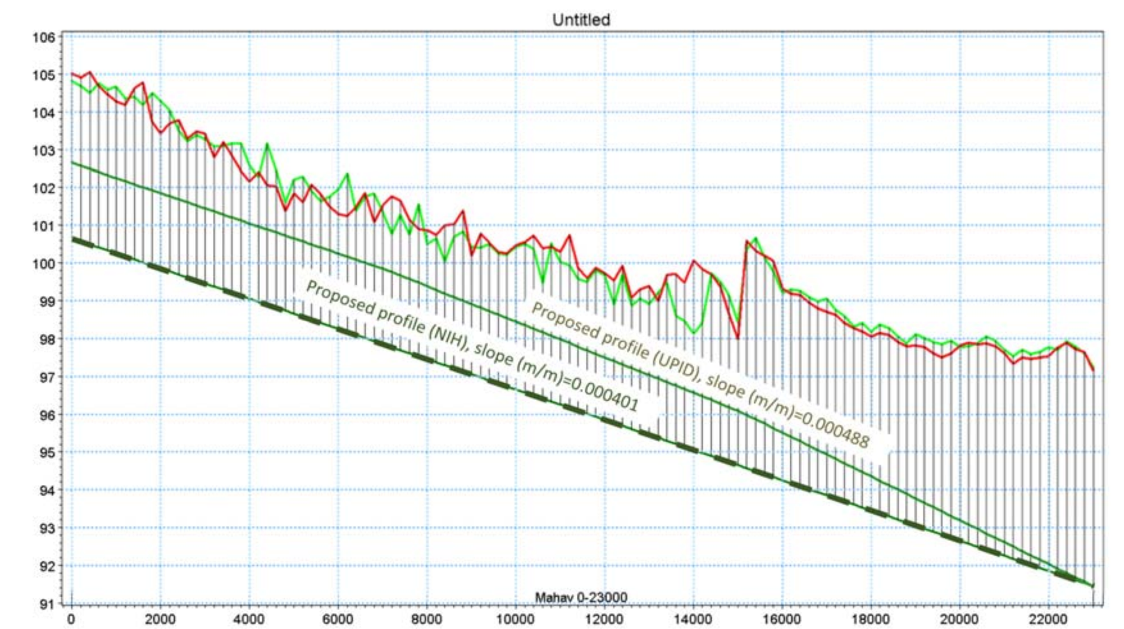

Figure 4.12: The modified river bed profile of Mahav nala to improve availability of flow depth

Table 4.3: Estimation of carrying capacity of section with varying shape and size.

SN	Z1	Z2	B (m)	y (m)	T (m)	A (m^2)	P (m)	Slope (m/m)	R (m)	V (m/s)	Q (m3/s)	Remarks
1	0.5	0.5	20	2	22	42	24.472	0.000488	1.7162	1.41	59	Nala profile proposed by UPID
2	1	1	20	2	24	44	25.657	0.000488	1.7149	1.41	62	
3	0.5	0.5	40	2	42	82	44.472	0.000488	1.8439	1.48	121	
4	1	1	40	2	44	84	45.657	0.000488	1.8398	1.47	124	
5	0.5	0.5	20	4	24	88	28.944	0.000401	3.0403	1.87	164	De sig n Nal

6	1	1	20	4	28	96	31.314	0.000401	3.0657	1.88	180	
7	0.5	0.5	22	4	26	96	30.944	0.000401	3.1024	1.89	182	
8	1	1	22	4	30	104	33.314	0.000401	3.1218	1.90	198	

4.6.1 Details of the revised cross section.

Based on the adopted parameters for the section and proposed nala bed profile (Figure 4.12), the section at every chainage is constructed considering the top of left and right bank level. The nala section at various chainage in tabular and graphical form are given in Annexure V.

4.6.2 Flow simulation in Mahav nala with the revised cross section.

The flow model is run with the revised cross section. The channel section with flow depth of 4 m and bed width of 22 m is used while the longitudinal profile shown in Figure 4.12 is used. The model is run for inflow flood of 187 m^3/s (~ estimated design flood of 186.5 187 m^3/s) while the downstream boundary condition is 96 m, i.e., observed HFL in Bhaghel river. The water surface profile for this flooding scenario is shown in Figure 4.13. With the revised section, the profile of left and right river bank would be maintained to prevent the spilling at any of the intermediate locations shown in this figure. The carrying capacity of revised section at various chainage is give in Table 4.4.

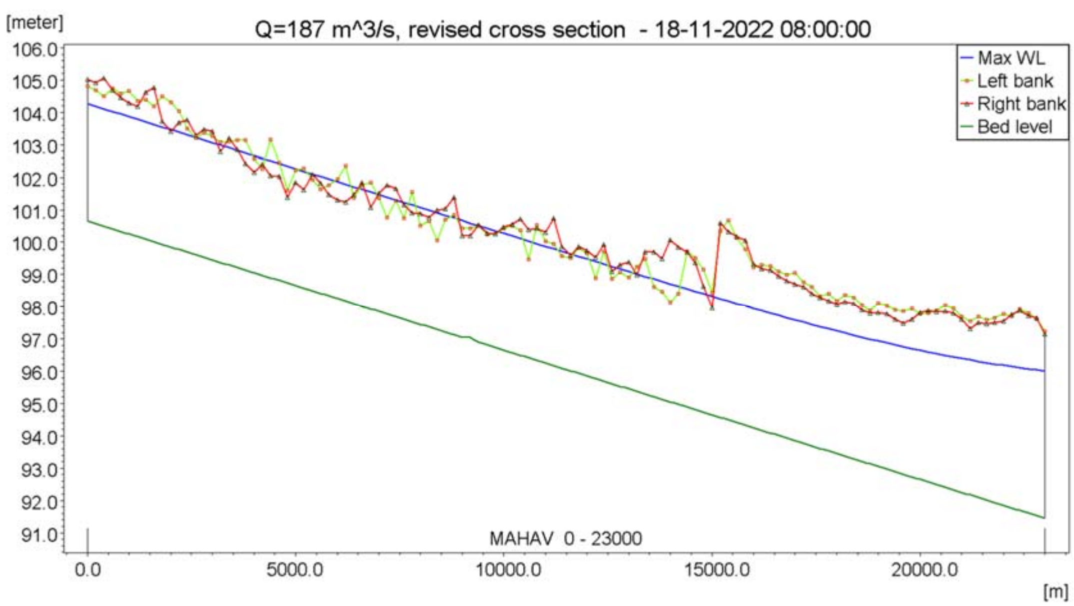

Figure 4.13: Maximum flood level profile for Q=187 m3/ s with revised cross section.

Table 4.4: Carrying capacity of Mahav nala with revised section at various chainage

Chainage (m)	Bed Level (m)	Ground Level of Lowest Bank (m)	Cross Sectional Area (m^2)	Top Width (m)	Carrying Capacity (m^3)/s
0	100.65	104.82	109.13	30.34	240.99
200	100.57	104.69	107.61	30.24	235.85
400	100.57	104.49	104.00	30.00	223.72
600	100.41	104.68	112.17	30.54	251.43
800	100.33	104.46	107.92	30.26	236.87
1000	100.25	104.28	104.90	30.06	226.72
1200	100.17	104.18	104.30	30.02	224.72
1400	100.09	104.40	113.40	30.62	255.67
1600	100.01	104.18	109.13	30.34	240.99
1800	99.93	104.11	109.36	29.98	242.03
2000	99.85	103.86	104.20	29.58	224.71
2200	99.77	103.69	101.61	29.84	215.80
2400	99.69	103.49	98.04	29.60	204.17
2600	99.61	103.21	92.16	29.20	185.47
2800	99.53	103.38	99.52	29.70	208.98
3000	99.45	103.27	98.63	29.64	206.09
3200	99.37	102.78	86.65	28.82	168.49
3400	99.29	103.09	98.04	29.60	204.17
3600	99.21	102.84	93.04	29.26	188.22
3800	99.13	102.79	93.85	28.96	190.99
4000	99.05	102.15	77.81	28.20	142.40
4200	98.97	102.26	83.20	28.58	158.15
4400	98.89	102.61	95.53	28.89	196.57
4600	98.81	102.03	81.21	28.44	152.26
4800	98.73	101.38	65.32	27.30	108.15
5000	98.65	101.84	80.36	28.38	149.77
5200	98.57	101.96	85.87	28.44	166.31
5400	98.49	101.92	87.23	28.86	170.24
5600	98.41	101.63	81.21	28.44	152.26
5800	98.33	101.48	79.22	28.30	146.47
6000	98.25	101.63	85.73	28.44	165.87
6200	98.17	101.81	93.02	28.71	188.65
6400	98.09	101.37	82.92	28.56	157.30
6600	98.01	101.76	96.56	29.50	199.42
6800	97.93	101.46	90.05	28.68	179.12
7000	97.85	101.36	89.54	29.02	177.33

Chainage (m)	Bed Level (m)	Ground Level of Lowest Bank (m)	Cross Sectional Area (m^2)	Top Width (m)	Carrying Capacity (m^3)/s
7200	97.77	101.26	88.84	28.48	175.53
7400	97.69	101.28	91.87	29.18	184.56
7600	97.61	100.74	78.66	28.26	144.84
7800	97.53	101.23	95.04	29.07	194.72
8000	97.45	100.50	76.40	28.10	138.38
8200	97.37	100.65	82.92	28.56	157.30
8400	97.29	100.52	81.38	27.98	153.07
8600	97.21	100.69	88.67	28.96	174.66
8800	97.13	100.84	95.38	29.42	195.65
9000	97.05	100.19	78.94	28.28	145.65
9200	97.05	100.19	78.94	28.28	145.65
9400	96.89	100.50	92.45	29.22	186.39
9600	96.81	100.24	87.23	28.86	170.24
9800	96.73	100.23	89.25	29.00	176.44
10000	96.65	100.42	97.15	29.54	201.32
10200	96.57	100.51	102.20	29.88	217.77
10400	96.49	100.37	100.41	29.76	211.89
10600	96.41	99.93	89.73	28.58	178.21
10800	96.33	100.42	106.71	30.18	232.79
11000	96.25	100.02	97.15	29.54	201.32
11200	96.17	100.35	109.20	29.96	241.51
11400	96.09	99.58	88.96	28.98	175.55
11600	96.01	99.50	88.96	28.98	175.55
11800	95.93	99.82	100.71	29.78	212.86
12000	95.85	99.66	98.34	29.62	205.13
12200	95.77	99.21	87.46	28.56	171.12
12400	95.69	99.71	104.60	30.04	225.72
12600	95.61	98.87	82.35	28.52	155.61
12800	95.53	99.06	90.12	29.06	179.13
13000	95.45	98.90	87.80	28.90	172.00
13200	95.37	98.98	92.45	29.22	186.39
13400	95.29	99.48	109.74	30.38	243.06
13600	95.21	99.15	102.05	29.33	217.74
13800	95.13	98.97	99.10	29.17	208.00
14000	95.05	99.59	119.41	29.61	279.95
14200	94.97	99.13	108.42	29.60	239.37
14400	94.89	99.70	128.96	31.62	311.48
14600	94.81	99.37	121.11	31.12	282.92
14800	94.73	98.64	101.31	29.82	214.82
15000	94.65	97.97	84.06	28.64	160.70
15200	94.57	100.35	160.57	33.56	434.90
15400	94.49	100.32	162.25	33.66	441.81

Chainage (m)	Bed Level (m)	Ground Level of Lowest Bank (m)	Cross Sectional Area (m^2)	Top Width (m)	Carrying Capacity (m^3)/s
15600	94.41	100.14	158.89	33.46	428.05
15800	94.33	99.77	149.27	32.88	389.35
16000	94.25	99.21	133.72	31.92	329.26
16200	94.17	99.18	135.32	32.02	335.29
16400	94.09	99.14	136.60	32.10	340.15
16600	94.01	98.95	133.08	31.88	326.86
16800	93.93	98.80	130.86	31.74	318.54
17000	93.85	98.70	130.22	31.70	316.18
17200	93.77	98.62	130.22	31.70	316.18
17400	93.69	98.40	125.80	31.42	299.90
17600	93.61	98.27	124.24	31.32	294.19
17800	93.53	98.18	123.92	31.30	293.05
18000	93.45	98.06	122.67	31.22	288.52
18200	93.37	98.15	128.01	31.56	307.99
18400	93.29	98.09	128.64	31.60	310.32
18600	93.21	97.90	125.18	31.38	297.61
18800	93.13	97.79	124.24	31.32	294.19
19000	93.05	97.82	127.69	31.54	306.82
19200	92.97	97.77	128.64	31.60	310.32
19400	92.89	97.60	125.80	31.42	299.90
19600	92.81	97.49	124.86	31.36	296.46
19800	92.73	97.60	130.86	31.74	318.54
20000	92.65	97.78	139.18	32.26	349.98
20200	92.57	97.80	142.41	32.46	362.46
20400	92.49	97.84	146.32	32.70	377.71
20600	92.41	97.86	149.60	32.90	390.66
20800	92.33	97.80	150.26	32.94	393.27
21000	92.25	97.60	146.32	32.70	377.71
21200	92.17	97.32	139.82	32.30	352.46
21400	92.09	97.50	148.29	32.82	385.45
21600	92.01	97.46	149.60	32.90	390.66
21800	91.93	97.50	153.57	33.14	406.48
22000	91.85	97.54	157.56	33.38	422.60
22200	91.77	97.70	165.63	33.86	455.78
22400	91.69	97.88	174.50	34.38	493.13
22600	91.61	97.72	171.75	34.22	481.48
22800	91.53	97.60	170.39	34.14	475.71
23000	91.45	97.15	157.89	33.40	423.96

4.6.3 Evaluating the carrying capacity of Nala with partial modification in cross section

The Mahav nala after Ch. 15 km is passing through the forest area. The flow model is simulated with the partial modification in the river cross section. The cross sections from Ch. 0 km to Ch. 15 km are modified for flow depth and river bed width as discussed in section 4.6.1. The river cross section from Chainage 15 km to 23 km are kept unchanged as mentioned in section 2.2. The flow model is simulated for inflow condition of 187 m^3/s while the downstream boundary condition is maintained at RL 96 m (HFL at Bhagel river. The maximum water surface profile for this case is plotted as shown in Figure 4.14. The figure shows a considerable spilling of flood water in almost entire reach of Mahav nala. Although the initial reach (up to 15 km) of Mahav nala is widened and deepen the continuity of the flow is obstructed due to insufficient cross-sectional area in later reach (d/s of 15 km). Further, the spilling in later reach shows that the existing cross section area in this reach is again insufficient to carry the design discharge of 187 m3/s. Hence, it may be concluded that the modification of river cross section is required throughout the reach of Mahav nala to safely carry the design discharge.

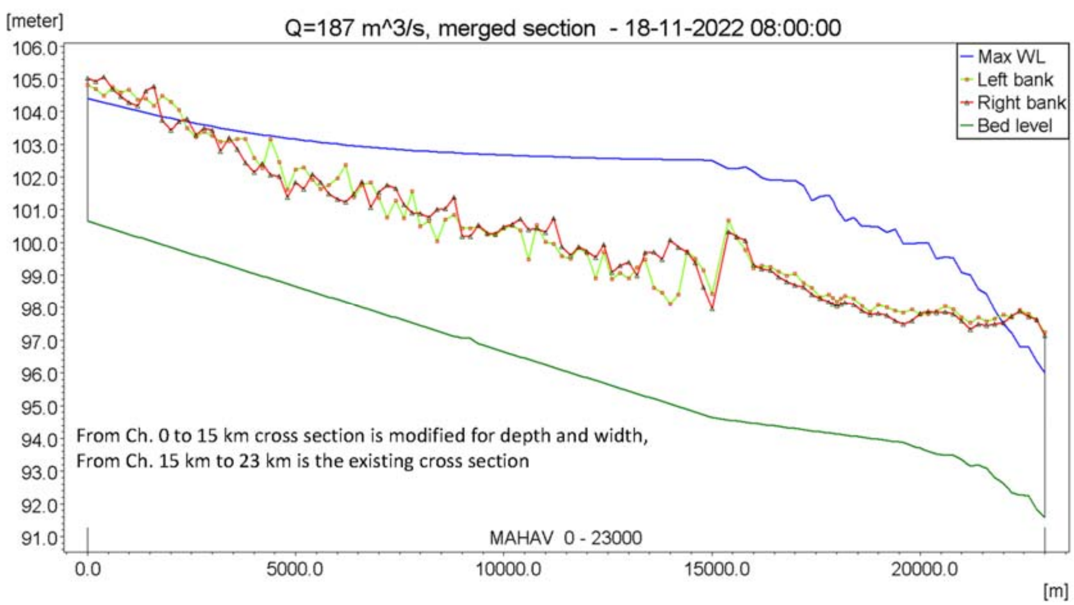

Figure 4.14: Maximum flood level profile for Q=187 m3/ s with partially revised cross section.

4.6.4 Evaluation of nala cross section for non-silting condition.

The sediment sample from nala bed level was tested in lab for particle size distribution. The particle size distribution curve is shown in Figure 4.15. The figure shows that about 78 % of soil having grain size diameter is about 0.219 mm and only 9 % soil grain have grain diameter smaller than 0.075 mm. Hence, the soi may be classified as fine sandy soil. d_{50} for the sample is estimated as 0.232 mm. Further, the non-silting velocity for a given particle size sediment can be obtained from the experimental results developed by Earle (2019). The relationship between the particle size and non-silting velocity is shown in Figure 4.16. The figure shows that for d_{50}= 0.232 mm, the non-silting velocity should be more than 0.17 m/s.

The flow model is simulated for AAR (design flood hydrograph for T= 2.33; Figure 3.3). The velocity time series computed at various section is shown in Figure 4.17. The figure shows the time series of the design flood hydrograph as well as the velocity developed in the nala section at its various chainage. It shows that the minimum velocity developed for the lowest flood at the rising and falling limb of design flood is more than 0.17 m/s. Further, the model is evaluated for steady inflow of 5 m^3/s and 10 m^3/s and the corresponding velocity (minimum velocity) in the nala at any section is computed as 0.67 m/s and 0.51 m/s, respectively.

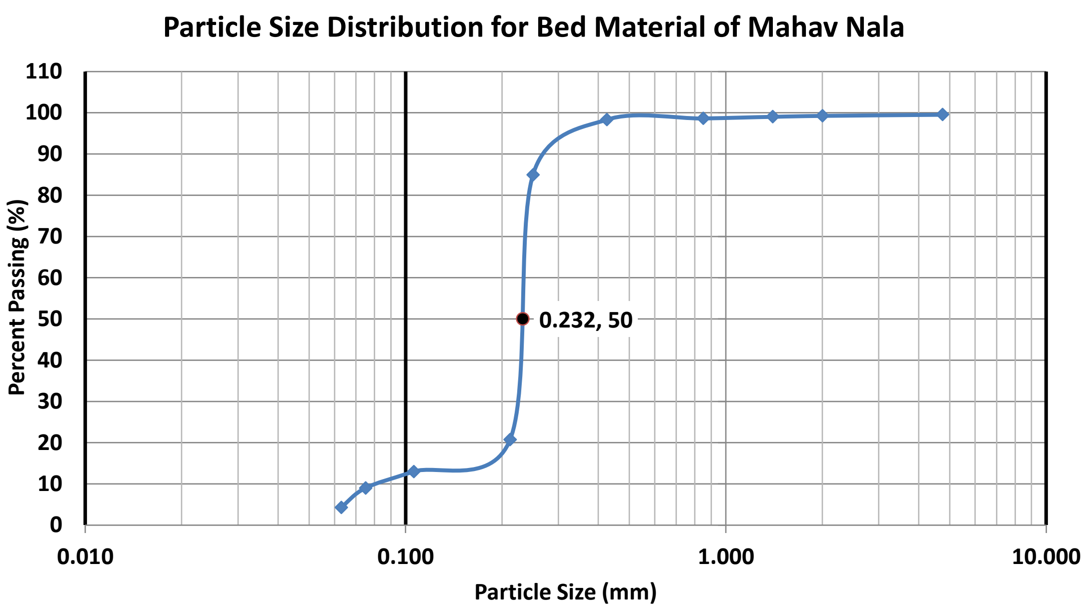

Figure 4.15: Particle size distribution curve for the nala bed sample.

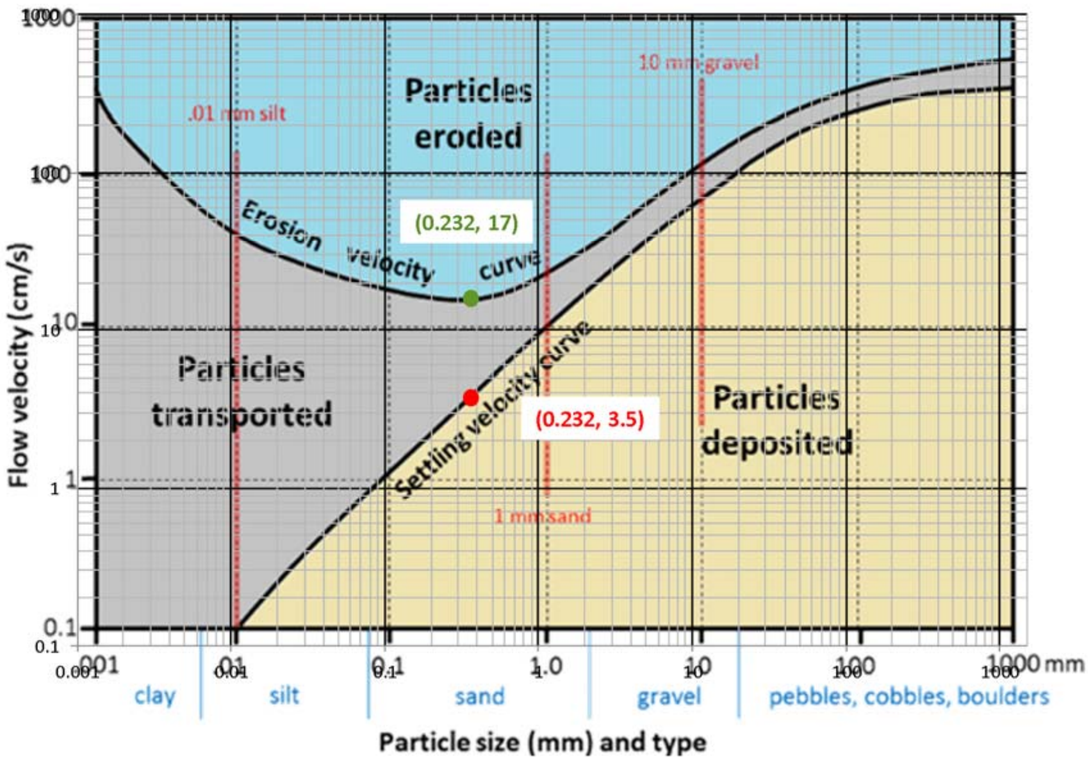

Figure 4.16: non silting velocity for various particle size.

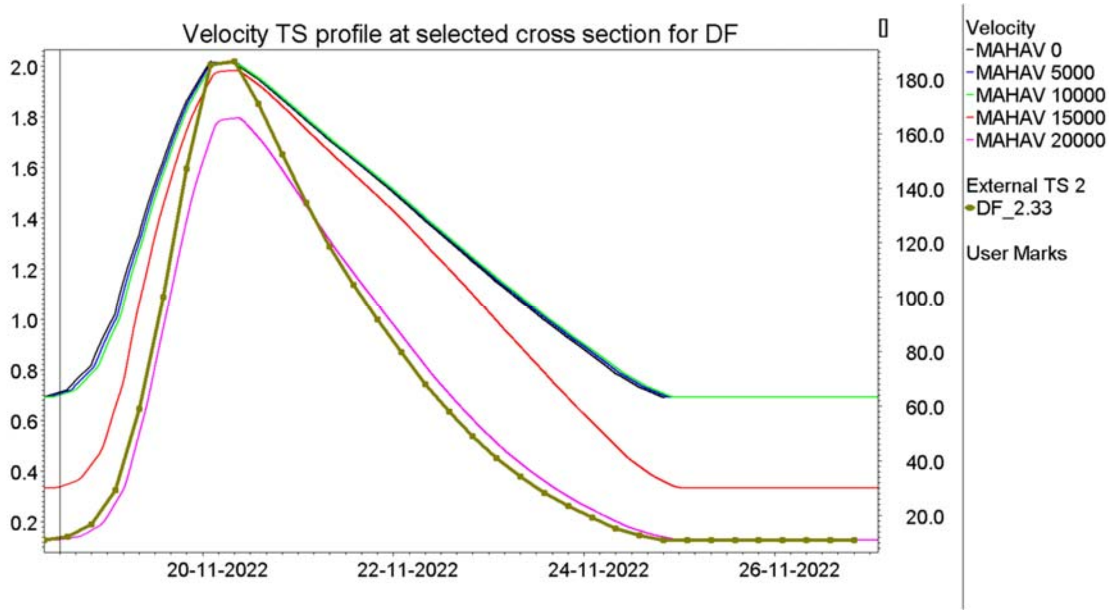

Figure 4.17: Time series of velocity computed for DF T=2.33 year at various section.

5 CONCLUSIONS & RECOMMENDATIONS

5.1 Conclusions

Based on the study, following conclusions can be drawn:

1. Using the SRTM DEM data the drainage line and watershed boundary for Mahav Nala has been extracted. The computed catchment area of Mahav nala is 218.5 km^2 while the length of longest flow path is computed as 66.34 km. The length of Mahav nala from watershed centroidal to outfall location is computed as 42 km. The equivalent slope of Mahav nala from its origin to outfall location is computed as 0.524 m/ km.
2. The peak floods for the Mahav Nala are estimated as 186.5 m^3/s, 244.9 m^3/s, 293.2 m^3/s and 353.8 m^3/s for return period of 2.33 Year, 5 Year, 10 Year and 25 Year return period, respectively.
3. The carrying capacity of Mahav nala with existing cross sections are computed of the order of 10m^3/s, although the maximum flood level at several chainage spills due to irregular bank profile. The flow congestion in Mahav nala starts at Chainage 15200 m.
4. The proposed section by UPID has been evaluated through flow model and it was found that the safe carrying capacity of section is about 50 m^3/s provided that the bank profiles are maintained smoothly.
5. The design section of Mahav nala is computed for 187 m^3/s which is estimated design flood for T=2.33 year (which are the estimates of the most recurrent flood) as follows:
    a. Flow depth = 4m,
    b. Side slope = 1:1
    c. Bottom width = 22 m

    The detailed profile of modified cross section at various chainage is reported in tabular and graphical form. The safe carrying capacity of the modified section is also tabulated in the report.
6. The sieve analysis of nala bed material is carried out for particle size analysis. The d_{50} of the sample is estimated as 0.0232 mm. The sample falls in fine sandy soil

classification. The non-silting flow velocity for such type of soil has been obtained from literature. The non-silting flow velocity is about 0.17 m/s. The flow velocity corresponding to lower values of design flood are found to be more than 0.17 m/s with the revised cross section.

5.2 Recommendations

Based on the study, the following recommendations are made for long term flood mitigation in the Mahav nala:

1. The trapezoidal nala section with base width of 22 m, side slope of 1:1 and flow depth of 4 m throughout the Mahav nala stretch, from Indo-Nepal border to its confluence with Bhaghel nala is needed to safely pass the design flood in Mahav nala without any spilling. The detailed profile of recommended nala section at every chainage is given in the report.
2. One hydrological observation site for collection of water level, discharge and silt data on daily basis should be established and data for at least 5 years should be generated.
3. Although the section has been evaluated for non-silting velocity, the silting in the nala should be observed regularly. If silting is excessive, the recommended nala profile (cross section and longitudinal slope) should be maintained by periodic removal of silt from the nala bed.

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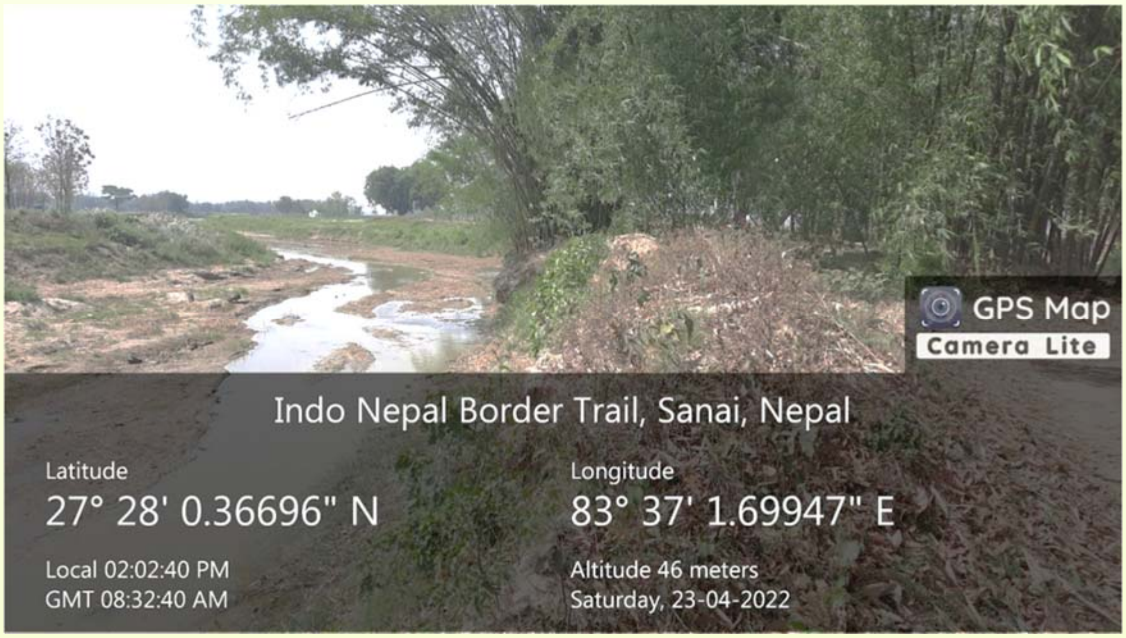
Annexure

Files attached separately



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