CHAPTER 5

STUDY OF EARLY FLOOD WARNING SYSTEM IN THE TAPI RIVER BASIN

5 STUDY OF EARLY FLOOD WARNING SYSTEM IN THE TAPI RIVER BASIN

Study of Early Flood Warning System in the Tapi River Basin discussed in this chapter by using different graphical techniques along with study river behavior, tidal effect, historical flood impact and rain fall contribution in flood.

Graphical (Equation) Method

Earlier storage equations were solved graphically (Now solved numerically). Graphical method is used for simplified representation of cross sectional geometry. This may be valid for large dam, where large flows are encountered and width to depth ratios are large. This method may not be applicable where channel geometry would be more critical.

5.1 Flood Warning

Studying River Behavior, Tidal Effect, Historical Flood Impact, and Rain fall Contribution in Flood are discussed here. As Ukai dam is major control point for Tapi river flood to save downstream Surat city. Addition to that, contribution for increasing large amount of flood inflows from upstream dams located in Upper Tapi Basin which accumulated ultimately at Ukai dam reservoir studied by graphical techniques. From this graphical technique remaining necessary rain gauge stations located to collect right information for flood routing. From Gidhare to Ukai in 7500 km^2 contribution is 38% compare to total inflow shows in Figure.No.5.2. The results of analysis shown in Figure.No.5.1 and 5.2.

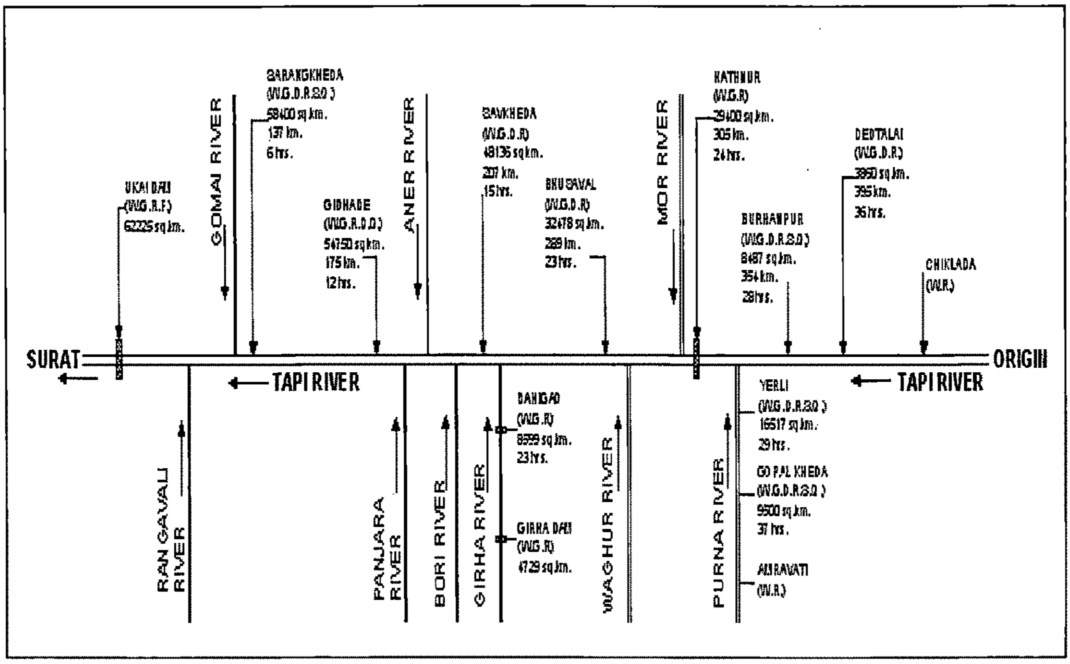


Fig.5.1 Line Diagram of River Tapi from Origin to Ukai Dam

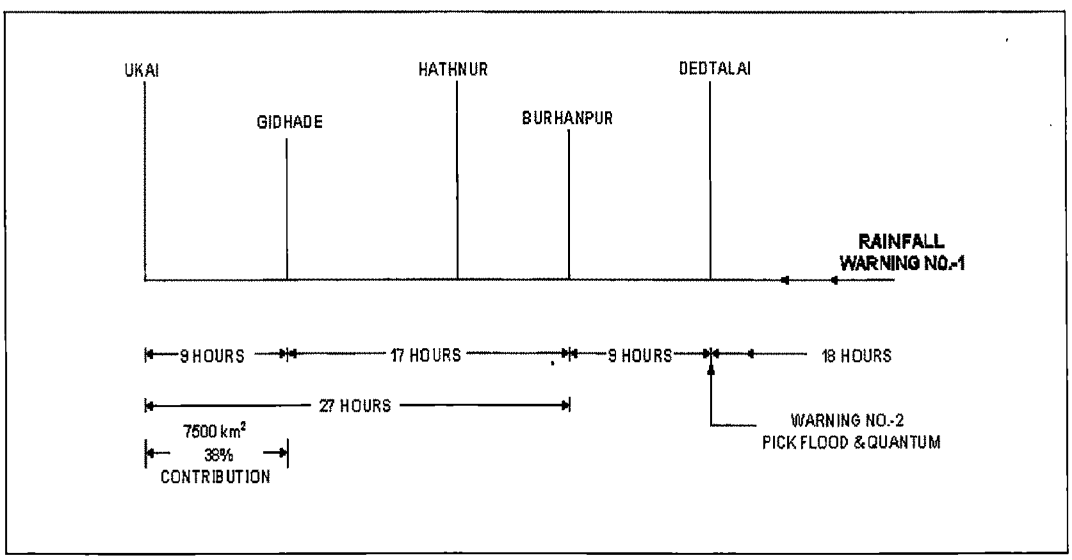


Fig.5.2 Rainfall Contribution in Flood

5.1.1. Hathnur Dam Located U/S from Ukai Dam

a) Criteria: Inflow forecasts for Hathnur are being issued as per the following criteria:

When reservoir level is

- From 209.0 m to 213.0 m – For a flood of peak discharge of 1000 cumecs and above

• Above 213.0 m – For a peak discharge of 250 cumecs and above

b) Procedure: Six hourly regular forecasts for Hathnur Dam are issued by Tapi Division, CWC, Surat based on the hydrometerological data of base station Burhanpur (on main river Tapi) and yearly on tributary Purna.

5.1.2 Ukai Dam

a) Criteria: Inflow forecasts for Ukai are being issued for a flood of peak discharge of 35,315 cusecs (1000 cumecs) and above irrespective of water level of reservoir.

b) Procedure: 12 hourly regular inflow forecasts for Ukai Dam are issued by Tapi Division, CWC, Surat based on the hydrometreological data of base station Gidhade and available condition of the intermediate catchment between Gidhade and Ukai.

These forecasts are monitored regularly and revised (if required) after 6 hours based on hydrometerological data of Sarangkheda and Ukai.

In addition to 12 hours regular forecasts and 6 hourly revised forecasts as stated above, advisory warning for expected high flood for Ukai Dam are also issued when the reservoir level is above 330.0 ft (100.59 m).

5.1.3 Surat

a) Criteria: Flood level forecast for Surat are being issued when it is expected that water level of Nehru Bridge, Surat may cross warning level – 8.50 m.

b) Procedure : Based on the release from Ukai Reservoir and hydrometerlogical data of downstream of Ukai Dam, flood level forecast for Surat City are formulated by using various correlation, G & D curve, Time lag curve etc. and issued to the user agency by about 6 to 8 hours in advance.

Based on the forecast received on real time, the user agencies take immediate action for regulation of reservoirs or for precautionary measures against the onslaught of coming floods to minimize the flood damages.

In addition to dissemination of forecast to the user agencies, messages are conveyed to newspapers also wherever and whenever required. Based on these forecast State Governments are arranging to broadcast the flood messages over Radio and TV to inform the public in time.

5.1.4 Improvements for Flood Forecasting

While rendering the flood forecasting services in Tapi Basin it is observed that some new wireless stations should be opened for the following two reaches in order to increase more warning time and accuracy of inflow forecast.

5.1.5 Origin to Dedtalai

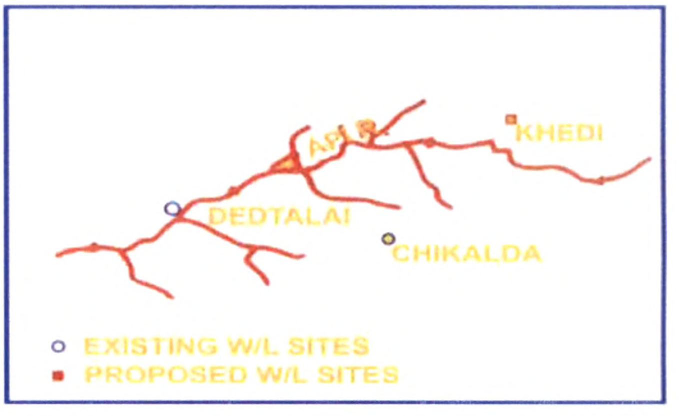
Catchment area in Sq.Km. at Dedtalai: 6660 (approximately)

The below Fig.No.5.3 shows outline of the catchment between origins to Dedtalai.

Wireless Station : Realtime Data Available

Chikaldhara – Rainfall reporting only

Dedtalai – Hourly gauge, daily discharge, and 3 hourly rainfall / daily rainfall



Source: Ukai Flood Control Cell

Fig.5.3 Outline of the catchment between Origins to Dedtalai

In order to increase the warning time it is proposed that one / two rainfall reporting stations should be opened near Khedi / Tiska / Bhaisdehi and a rainfall runoff relationship / Unit Hydrograph be developed at Dedtalai for predicting the peak flood at this site and in turn more warning time can be given to Ukai dam authorities.

5.1.6 Gidhade to Ukai

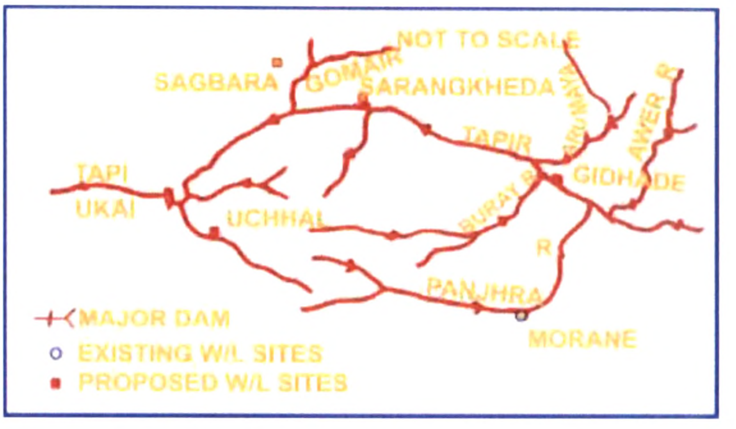
The below Fig.No.5.4 shows Ukai flood control cell.

Catchment area in Sq. Km.

At Gidhade 54750

At Sarangkheda 58400

At Ukai 62225



Source: Ukai Flood Control Cell

Fig.5.4 Outline of Catchment Area from Gidhade to Ukai

Wireless Station	**Real time data available**
Gidhade:	Hourly gauge, daily discharge and 3 hourly rainfall / daily rainfall
Ukai:	Hourly gauge and daily rainfall

During the flood occurred in September 1998, the flow reached in to Ukai was about 18% more than the volume of flood passed from Gidhade, The peak of flood at Ukai was about 38% higher than the peak at Gidhade. It shows that runoff due to the intermediate catchment of 7500 km^2 between Gidhade and Ukai plays a vital role (if, high rainfall is occurred over this catchment) in the assessment of correct inflow for Ukai dam. It shows that runoff due to the intermediate catchment of 7500 km^2 between Gidhade and Ukai plays a vital role (if, high rainfall is occurred over this catchment) in the assessment of correct inflow for Ukai dam. Till 1998, there was no arrangement for getting R.F./Gauge/discharge data for this intermediate catchment on real-time basis. Approximately same situation occurred at a time of flood in August 2006. In December 1998, wireless facility has been provided to existing site at Saragkheda, which is found useful in the assessment of runoff due to this catchment.

5.1.7 Flood Routing Studies

5.1.7.1 Method of Flood Routing

There are three broadly classified methods

- Calculus method.

- Graphical method.
- Trial and error method.
- Mass curve method using the computer model.

A number of flood routing studies have been carried out by mass curve method using the computer model along with all the inflow flood hydrograph taking different initial conditions and outflow patterns into consideration. The latest reservoir capacities at different elevations as obtained from the surveys conducted by the GERI in the year 2003 have been used. The spillway discharge capacities for different water levels in the reservoir and for different gate openings. The flood routing calculations are carried out by the mass curve method using the computer model. A time interval of 3 hours is considered in all the calculations. The studies are broadly brought into groups as detailed are as shown in results.

The results of analysis shown below.

- Considering present carrying capacity of the Tapi river of 11,323 Cumec (4 Lac Cusec), the various inflow floods have been routed at the constant outflow rate of 11,323 Cumec (4 Lac Cusec), considering the two initial reservoir levels viz; EL 103.33 m (339 ft) and 105.15 m (345 ft). The results of the studies indicate that in most of the events the reservoir elevation exceeds the HFL 106.99 m (351 ft).
- Flood routing is carried out by considering initial reservoir level at RL 103.33 m (339 ft) with outflow restrictions applied in increasing order ranging from 11,323 Cumec to 24,069 Cumec (4 Lac and 8.5 Lac Cusec), as the reservoir level raises. In this case the MWL works out below the present HFL of 106.99 m (351 ft) except for PMF.
- Similar type of studies as in B above, are carried out considering the initial reservoir EL 105.15 m (345 ft) and considering release ranging from 11,323 Cumec to 24,069 Cumec (4 Lac to 8.5 Lac Cusec) up to the RL 106.38 m (349 ft) and there after making outflow equal to inflow. In this case also the MWL works out below HFL 106.99 m (351 ft) except for the PMF. Flood routing studies are carried out considering the initial reservoir level at RL 103.33 m (339 ft) with the release policy as under:

Reservoir level in m (ft)	**Outflow in lac Cumecs (Cusecs)**
103.33 to 105.15 m (339 to 345 ft)	• 11,323 Cumec (4.0 Lac Cusec)
105.15 m (345 ft Onwards)	• 14,158 Cumec (5.0 Lac Cusec)
	• 16,990 Cumec (6.0 Lac Cusec)
	• 19,822 Cumec (7.0 Lac Cusec)

- 24,069 Cumec (8.5 Lac Cusec)

In these studies for hypothetical flood, with restricted release of 19,822 Cumecs (7 Lac Cusecs) and 24,069 Cumec (8.5 Lac Cusec), the MWL attained is below HFL. It is also within HFL for observed flood of 1970. In rest of the studies MWL crosses the HFL 106.99 m (351 ft).

Flood routing studies with advance pre-depletion of 9.0 hrs routing interval 1.00 hour, base flow of 991 Cumec to 2832 Cumec (35000 to 1 Lac Cusec) and initial reservoir level at FRL 105.15 m (345 ft) are carried out. In this case, three different restricted release policies were adopted and observed flood of 1959 and 1968 as well as design flood SPF and PMF and were routed. Except for PMF, the MWL attained for all the cases is below HFL. For PMF routing MWL crosses HFL 106.99 m (351 ft) for policy.

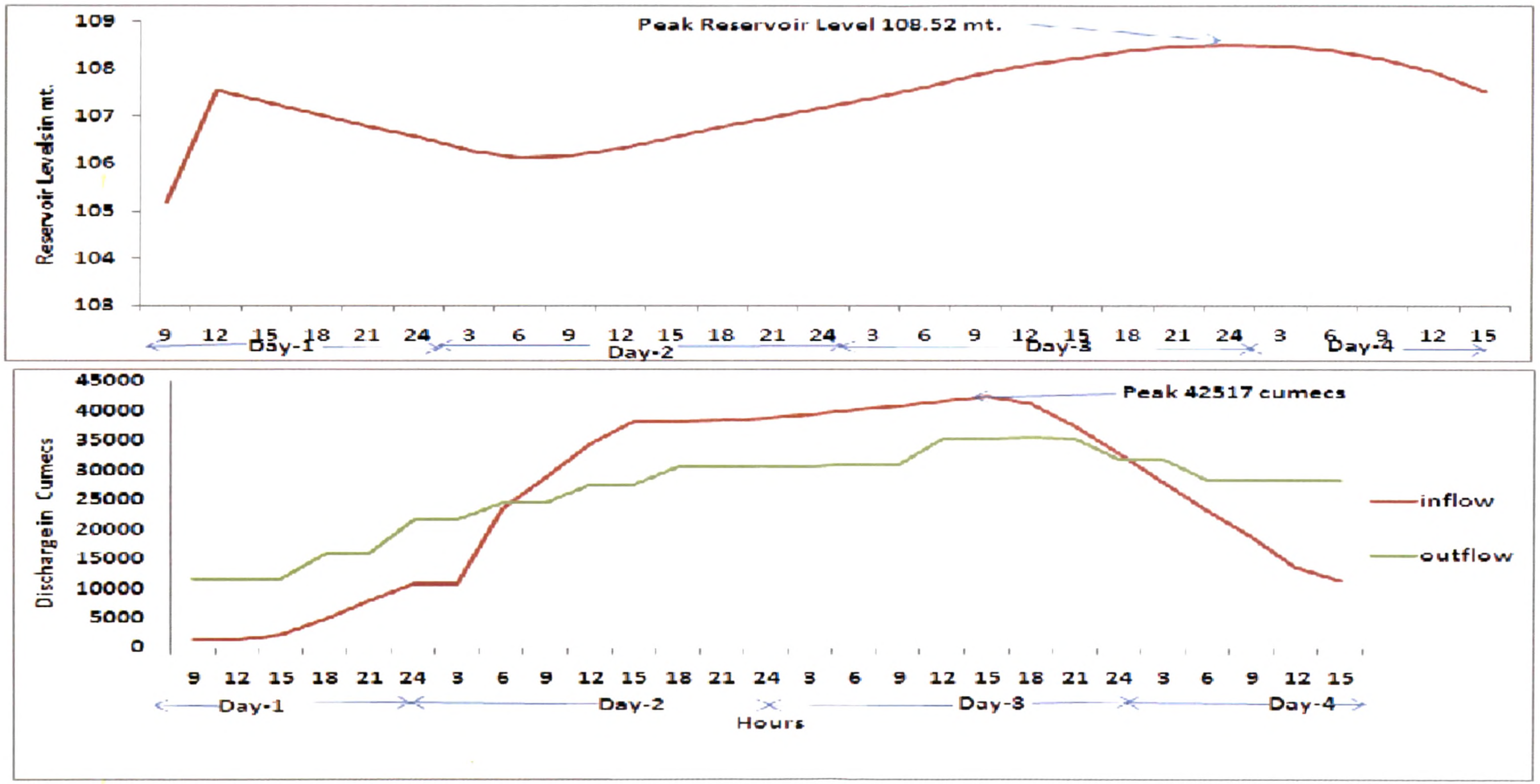
It is assumed that the volume of incoming flood predicted and conveyed to dam site by CWC in advance with constant updating of forecast at every 12 hours is available. The average rate of incoming inflow is worked out by deducting the capacity of reservoir between initial R.L. and HFL 106.99 m (351 ft) from the predicted flood volume considering the duration of flood or flood period. Flood routing studies are carried out only for historical floods of year 1959, 1968 and SPF.

Hypothetical flood event with the maximum flood peak of 33,980 Cumec (12 Lac Cusec) was routed with a constant restricted release of 14,158, 15,574, 16,282, 16,990, 18,406 and 19,822Cumec (5, 5.5, 5.75, 6, 6.5 and 7.0 Lac Cusec) at initial level of 105.15 m (345 ft). For 514,158 Cumec and 15,574 Cumec (5 and 5.5 Lac Cusec) constant releases, the MWL crosses HFL 351 ft (106.99 m). For 16,282 Cumec (5.75 Lac Cusec) release it is close to HFL whereas for 16,990 and 18,406 Cumec (6 and 6.5 Lac Cusec), the level reaches to 106.60 m (349.73 ft) and 105.90 m (347.43 ft) respectively. But in case of restricted outflow of magnitude 19,822 Cumec (7 Lac Cusec), the entire hydrograph can be routed successfully at FRL itself.

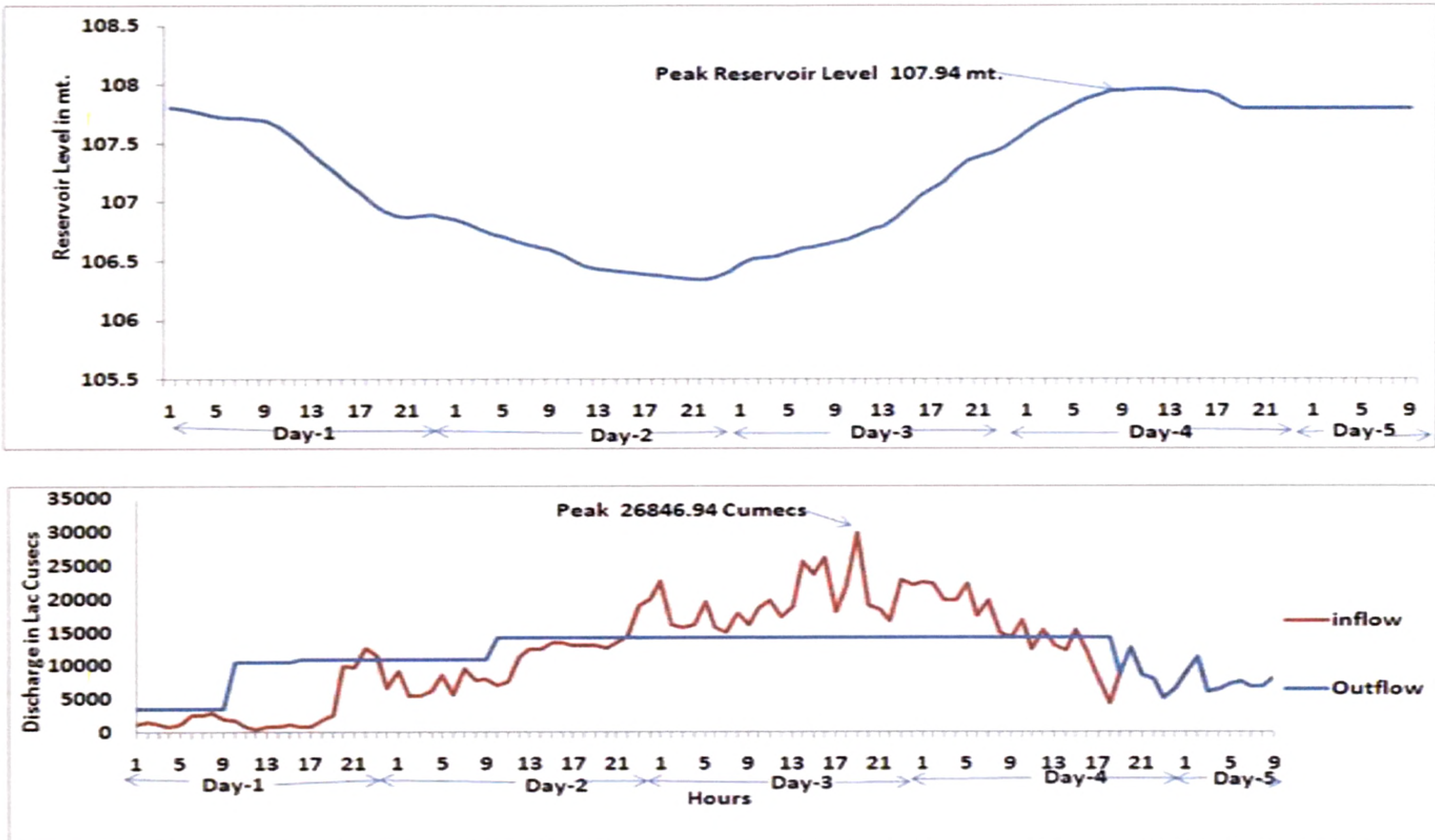
A detailed examination of all the routing studies narrated above showed that the study is crucial and important for working out the details of operation of Ukai reservoir. For normal floods, the outflow may be restricted starting from 11,323 Cumec (4 Lac Cusec) and going up to a maximum of 19,822 Cumec (7 Lac Cusec), without encroaching into the freeboard above the HFL 106.99 m (351 ft). Adopting a rule level of 103.63 m (340 ft) on September 15 is considered prudent, looking into the present downstream situation. In view of these, further routing studies were taken up considering initial reservoir level as 103.63 m (340 ft). The results of the routing studies are presented. The studies are carried out with advance depletion of 9.0 hrs and 12.0 hrs respectively. In all these studies, the MWL attained is below the HFL of 106.99 m (351 ft).

Routing of the river is essential to get additive discharge due to the rainfall of the intervening catchment between two forecasting stations. It can be done by Muskingum method for correction of the inflow to the downstream station. It is insisted that the correction of inflow should be done at Kakrapar Weir, Mandvi, Ghala and Kathor to achive higher accuracy. By viewing all this and using Thessian polygon method we can say that the existing rain gauge stations are inadequate. The no. of rain gauge stations required can be computed and new automatic rain gauge/river gauge station should be established.

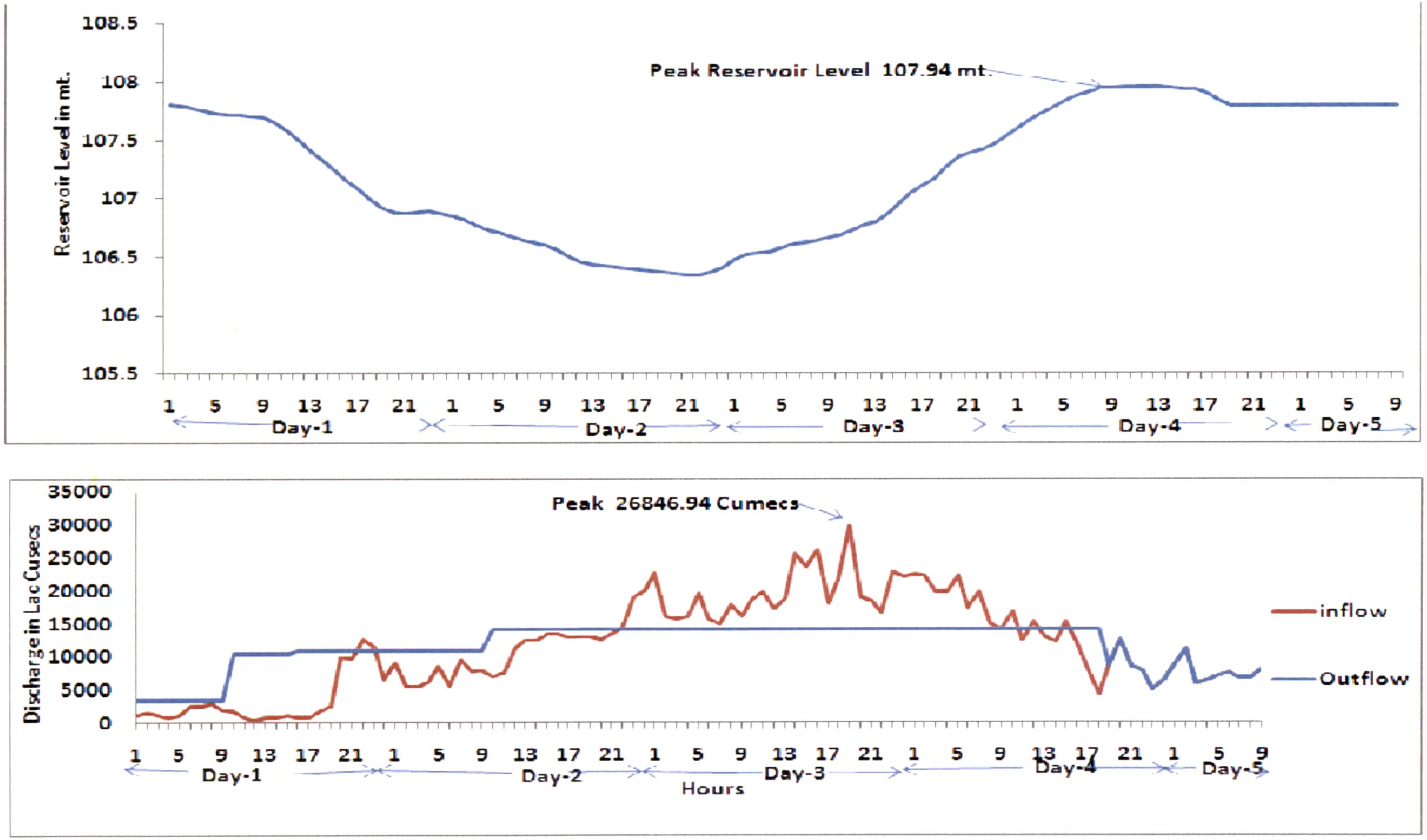
The Below Graph.No.5.1 shows inflow and outflow hydrograph-Ukai reservoir corresponding reservoir levels for the flood event-1968, Graph.No.5.2 shows inflow and outflow hydrograph-Ukai reservoir corresponding reservoir levels for the flood event-1998, Graph.No.5.3 shows inflow and outflow hydrograph-ukai reservoir corresponding reservoir levels for the flood event-PMF.



Graph.5.1 Inflow and Outflow Hydrograph-Ukai Reservoir Corresponding Reservoir Levels for the Flood Event-1968



Graph.5.2 Inflow and Outflow Hydrograph-Ukai Reservoir Corresponding Reservoir Levels for the Flood Event-1998



Graph.5.3 Inflow and Outflow Hydrograph-Ukai Reservoir Corresponding Reservoir Levels for the Flood Event-PMF

Inflow ordinates for various flood events for Tapi River interpreted and listed below Table. No. 5.1, Table.No.5.2 shows abstract results of various flood routing studies.

Table.5.1 Inflow Ordinates for Various Flood Events

FLOOD 1959		FLOOD 1968		FLOOD 1970		FLOOD SPF		FLOOD PMF		HYPOTHETICAL FLOOD	
Hr.	Inflow (Lac Cusecs)	Hr.	Inflow (Lac Cusecs)	Hr.	Inflow (Lac Cusecs)	Hr.	Inflow (Lac Cusecs)	Hr.	Inflow (Lac Cusecs)	Hr.	Inflow (Lac Cusecs)
0	0.35	0	0.50	0	1.08	0	1.00	0	0.90	0	1.00
3	0.35	3	0.50	3	1.08	3	1.30	3	1.10	3	1.20
6	0.80	6	0.80	6	1.08	6	1.60	6	1.20	6	1.60
9	1.50	9	1.70	9	1.12	9	2.20	9	1.45	9	2.00
12	3.20	12	2.80	12	1.19	12	3.00	12	1.80	12	2.50
15	4.30	15	3.80	15	2.25	15	4.00	15	2.10	15	3.00
18	5.10	18	3.80	18	2.65	18	5.60	18	2.60	18	3.45
21	5.90	21	8.30	21	3.70	21	7.20	21	3.20	21	3.95
24	6.60	24	10.20	24	4.05	24	9.20	24	4.00	24	4.60
27	7.20	27	12.20	27	4.15	27	10.80	27	5.00	27	5.65
30	7.80	30	13.45	30	4.55	30	12.60	30	6.20	30	6.90
33	8.40	33	13.50	33	4.85	33	14.20	33	8.00	33	8.00
36	8.90	36	13.58	36	5.55	36	15.60	36	9.80	36	9.10
39	9.40	39	13.70	39	6.35	39	16.60	39	11.60	39	10.00
42	9.80	42	13.90	42	6.85	42	17.20	42	13.60	42	10.70
45	10.20	45	14.20	45	7.70	45	17.48	45	16.00	45	11.10
48	10.80	48	14.40	48	8.50	48	16.30	48	18.00	48	11.40
51	11.40	51	14.70	51	10.50	51	15.40	51	19.20	51	11.70
54	11.45	54	15.00	54	12.25	54	14.20	54	20.10	54	11.90
57	12.00	57	14.60	57	12.75	57	13.00	57	20.30	57	12.00
60	12.80	60	13.20	60	13.00	60	11.90	60	21.16	60	11.60

Table.5.1 Inflow Ordinates for Various Flood Events (Continue)											
FLOOD 1959		FLOOD 1968		FLOOD 1970		FLOOD SPF		FLOOD PMF		HYPOTHETICAL FLOOD	
Hr.	Inflow (Lac Cusecs)	Hr.	Inflow (Lac Cusecs)	Hr.	Inflow (Lac Cusecs)	Hr.	Inflow (Lac Cusecs)	Hr.	Inflow (Lac Cusecs)	Hr.	Inflow (Lac Cusecs)
63	13.16	63	11.60	63	13.14	63	10.60	63	19.80	63	10.70
66	12.80	66	9.90	66	13.05	66	9.60	66	17.60	66	9.80
69	12.20	69	8.20	69	12.50	69	8.60	69	15.80	69	8.90
72	11.50	72	6.60	72	11.80	72	7.50	72	14.20	72	7.80
75	10.50	75	4.80	75	11.10	75	6.60	75	12.40	75	6.80
78	9.60	78	4.00	78	10.25	78	5.70	78	10.60	78	5.70
81	8.70			81	9.35	81	5.00	81	9.40	81	5.00
84	7.70			84	8.70	84	4.40	84	8.20	84	4.60
87	6.60			87	7.75	87	3.80	87	7.40	87	4.00
90	5.40			90	6.40			90	6.40		
93	4.00			93	5.40			93	5.40		
								96	5.00		
								99	4.00		
250.41 Lac cusecs		243.93 Lac cusecs		224.64 Lac Cusecs		272.18 Lac cusecs		314.51 Lac cusecs		206.35 Lac cusecs	
270442.80 Mcft		263444.40 Mcft		242611.20 Mcft		293954.40 Mcft		339670.80 Mcft		222858.00 Mcft	
7658.02 Mcum		7459.84 Mcum		6869.92 Mcum		8323.78 Mcum		9618.32 Mcum		6310.58 Mcum	
6.21 Maft		6.05 Maft		5.57 Maft		6.75 Maft		7.92 Maft		5.12 Maft	

Table.5.2 Abstract Results of Various Flood Routing Studies

Crest RL	FRL	HFL/MWL	Freeboard	TOB
299.0 ft.	345.0 ft.	351.0 ft.	14.0 ft.	365.0 ft

Alt	Type of Study	Maximum Inflow (Lac Cusecs)	Maximum Out flow (Lac Cusecs)	Initial Reservoir Level (feet)	Minimum Reservoir Level Reached (feet)	Maximum Reservoir Level Reached (feet)
1	Hypothetical flood	12.00	4.0 (Restricted)	345	342.28	357.62
2	Observed flood of 1970	13.14	4.0 (Restricted)	345	341.21	358.81
3	Observed flood of 1959	13.16	4.0 (Restricted)	345	342.27	362.16
4	Observed flood of 1968	15.00	4.0 (Restricted)	345	342.17	363.84
5	S.P.F.	17.48	4.0 (Restricted)	345	342.83	365.85
6	P.M.F.	21.16	4.0 (Restricted)	345	341.27	370.25
7	Hypothetical flood	12.00	4.0 (Restricted)	339	336.20	354.40
8	Observed flood of 1970	13.10	4.0 (Restricted)	339	335.10	355.59
9	Observed flood of 1959	13.16	4.0 (Restricted)	339	336.19	358.94
10	Observed flood of 1968	15.00	4.0 (Restricted)	339	336.09	360.62
11	S.P.F.	17.48	4.0 (Restricted)	339	335.16	362.63
12	P.M.F.	21.16	4.0 (Restricted)	339	335.16	367.03
13	Hypothetical flood	12.00	4.0 (Restricted)	332.50	329.62	351.01
14	Observed flood of 1970	13.14	4.0 (Restricted)	330.15	326.12	351.00
15	Observed flood of 1959	13.16	4.0 (Restricted)	321	--	351.41
16	Observed flood of 1968	15.0	4.0 (Restricted)	321	--	353.14
17	S.P.F.	17.48	4.0 (Restricted)	321	--	354.78
18	P.M.F.	21.16	4.0	321	--	360.02
19	Observed flood of 1959	13.16	13.16	339	336.19	349.78
20	Observed flood of 1968	15.00	15.0	339	336.09	349.69
21	S.P.F.	17.48	16.0	339	336.77	350.81

Table 5.2 Abstract Results of Various Flood Routing Studies (Continue)						
Alt	**Type of Study**	**Maximum Inflow (Lac Cusecs)**	**Maximum Out flow (Lac Cusecs)**	**Initial Reservoir Level (ft.)**	**Minimum Reservoir Level Reached (ft.)**	**Maximum Reservoir Level Reached (ft.)**
22	P.M.F.	21.16	16.50	339	335.16	352.45
23	Observed flood of 1959	13.16	13.16	345	342.27	349.56
24	Observed flood of 1968	15.00	15.0	345	342.17	349.93
25	S.P.F.	17.48	16.10	345	342.83	350.78
26	P.M.F.	21.16	17.2	345	341.27	352.99
27	Hypothetical flood	12.00	5.00 (Restricted)	339	336.20	352.99
28	Hypothetical flood	12.00	6.00 (Restricted)	339	336.20	351.76
29	Hypothetical flood	12.00	7.00 (Restricted)	339	336.20	350.67
30	Hypothetical flood	12.00	8.50 (Restricted)	339	336.20	349.12
31	Observed flood of 1970	13.14	5.00 (Restricted)	339	335.10	354.28
32	Observed flood of 1970	13.14	6.00 (Restricted)	339	335.10	352.98
33	Observed flood of 1970	13.14	7.00 (Restricted)	339	335.10	351.80
34	Observed flood of 1970	13.14	8.50 (Restricted)	339	335.10	350.23
35	Observed flood of 1959	13.16	5.00 (Restricted)	339	336.19	357.04
36	Observed flood of 1959	13.16	6.00 (Restricted)	339	336.19	355.23
37	Observed flood of 1959	13.16	7.00 (Restricted)	339	336.19	353.55
38	Observed flood of 1959	13.16	8.50 (Restricted)	339	336.19	351.23
39	Observed flood of 1968	15.00	5.00 (Restricted)	339	336.09	358.88
40	Observed flood of 1968	15.00	6.00 (Restricted)	339	336.09	357.24
41	Observed flood of 1968	15.00	7.00 (Restricted)	339	336.09	355.68
42	Observed flood of 1968	15.00	8.50 (Restricted)	339	336.09	353.47
43	Design flood (SPF)	17.48	5.00 (Restricted)	339	336.77	360.59

Table 5.2 Abstract Results of Various Flood Routing Studies (Continue)						
Alt	**Type of Study**	**Maximum Inflow (Lac Cusecs)**	**Maximum Out flow (Lac Cusecs)**	**Initial Reservoir Level (ft.)**	**Minimum Reservoir Level Reached (ft.)**	**Maximum Reservoir Level Reached (ft.)**
44	Design flood (SPF)	17.48	6.00 (Restricted)	339	336.77	358.74
45	Design flood (SPF)	17.48	7.00 (Restricted)	339	336.77	357.04
46	Design flood (SPF)	17.48	8.50 (Restricted)	339	336.77	354.75
47	Check flood (PMF)	21.16	5.00 (Restricted)	339	335.16	364.91
48	Check flood (PMF)	21.16	6.00 (Restricted)	339	335.16	362.96
49	Check flood (PMF)	21.16	7.00 (Restricted)	339	335.16	361.16
50	Check flood (PMF)	21.16	8.50 (Restricted)	339	335.16	358.72
	Observed flood 1959					
51	Rel. policy (a)	13.16	12.92	345	340.02	350.17
52	Rel. policy (b)	13.16	12.60	345	338.37	349.14
53	Rel. policy (c)	13.16	13.16	345	338.37	349.14
	Observed flood 1968					
54	Rel. policy (a)	15.00	15.00	345	339.78	350.17
55	Rel. policy (b)	15.00	15.00	345	338.16	349.15
56	Rel. policy (c)	15.00	14.50	345	336.50	348.42
	Design flood (SPF)					
57	Rel. policy (a)	17.48	15.50	345	339.90	350.56
58	Rel. policy (b)	17.48	15.00	345	338.14	349.93
59	Rel. policy (c)	17.48	14.50	345	336.23	348.95
	Observed flood 1959					
60	Rel. policy (a)	21.16	16.00	345	337.58	352.60
61	Rel. policy (b)	21.16	16.00	345	335.05	351.60
62	Rel. policy (c)	21.16	15.57	345	332.30	350.76
63	Observed flood in 1959	13.16	7.0	345	336.37	350.30
64	Observed flood in 1968	15.00	8.0	345	334.75	350.97
65	SPF	17.48	8.0	345	334.14	351.44
66	Hypothetical flood	12.00	5.0	345	338.75	353.24
67	Hypothetical flood	12.00	5.5	345	337.55	351.48
68	Hypothetical flood	12.00	5.75	345	336.93	350.62
69	Hypothetical flood	12.00	6.00	345	336.29	349.73
70	Hypothetical flood	12.00	6.5	345	334.97	347.43

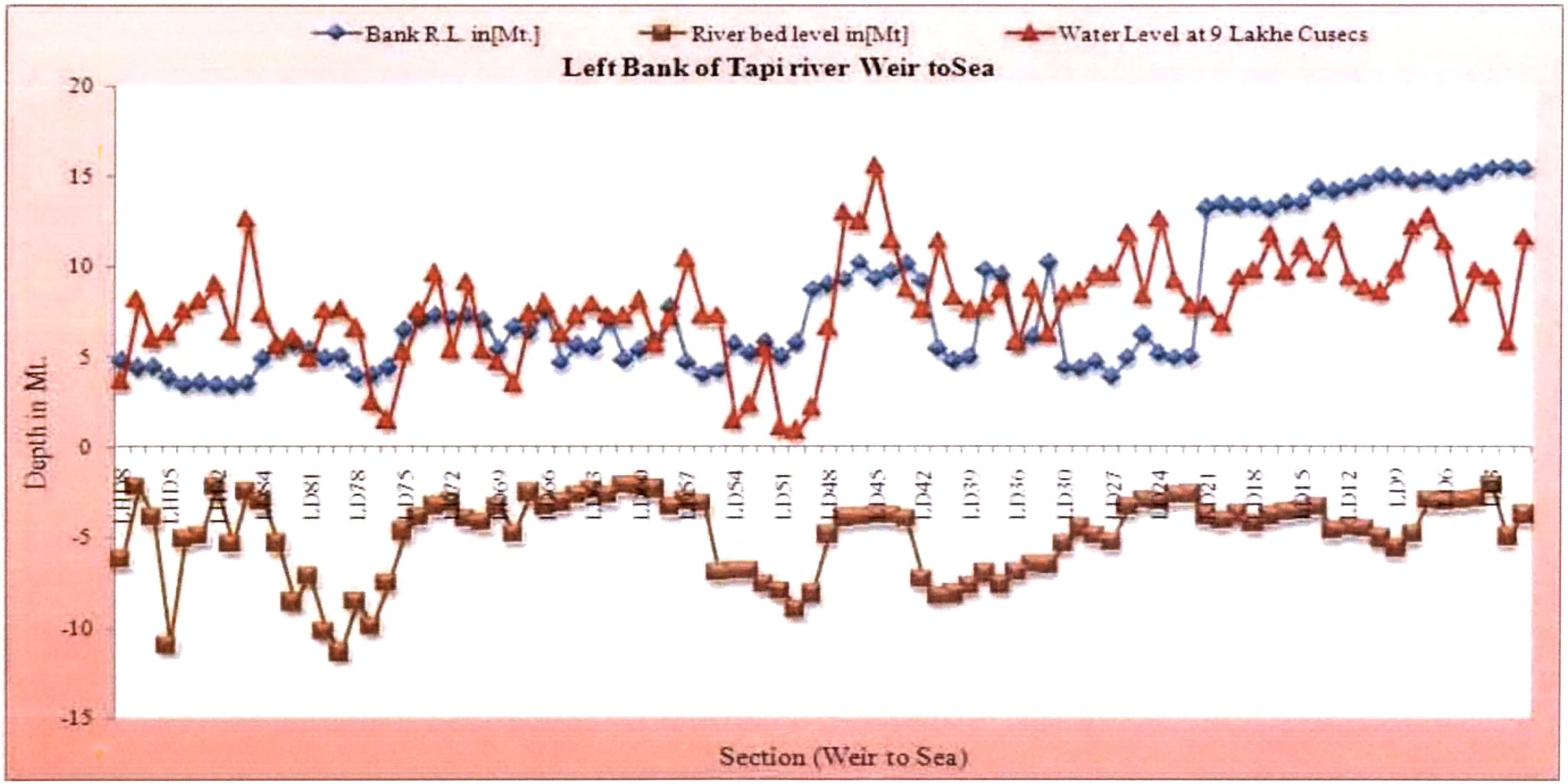
Table 5.2 Abstract Results of Various Flood Routing Studies (Continue)						
Alt	**Type of Study**	**Maximum Inflow (Lac Cusecs)**	**Maximum Out flow (Lac Cusecs)**	**Initial Reservoir Level (ft.)**	**Minimum Reservoir Level Reached (ft.)**	**Maximum Reservoir Level Reached (ft.)**
71	Hypothetical flood	12.00	7.0	345	333.60	345.02
72	Hypothetical flood	12.00	4.90	340	333.84	350.91
73	Observed flood in 1959	13.16	6.10	340	333.01	350.93
74	Observed flood in 1968	15.00	7.00	340	331.25	350.93
75	SPF	17.48	7.25	340	330.49	351.18
76	Hypothetical flood	12.00	4.75	340	333.28	350.98
77	Observed flood in 1959	13.16	5.90	340	332.03	350.99
78	Observed flood in 1968	15.00	6.75	340	330.16	350.95
79	SPF	17.48	6.95	340	329.66	351.41

5.2 Preparation of River Cross Section with Respect to Caring Capacity while River Flow through Surat City

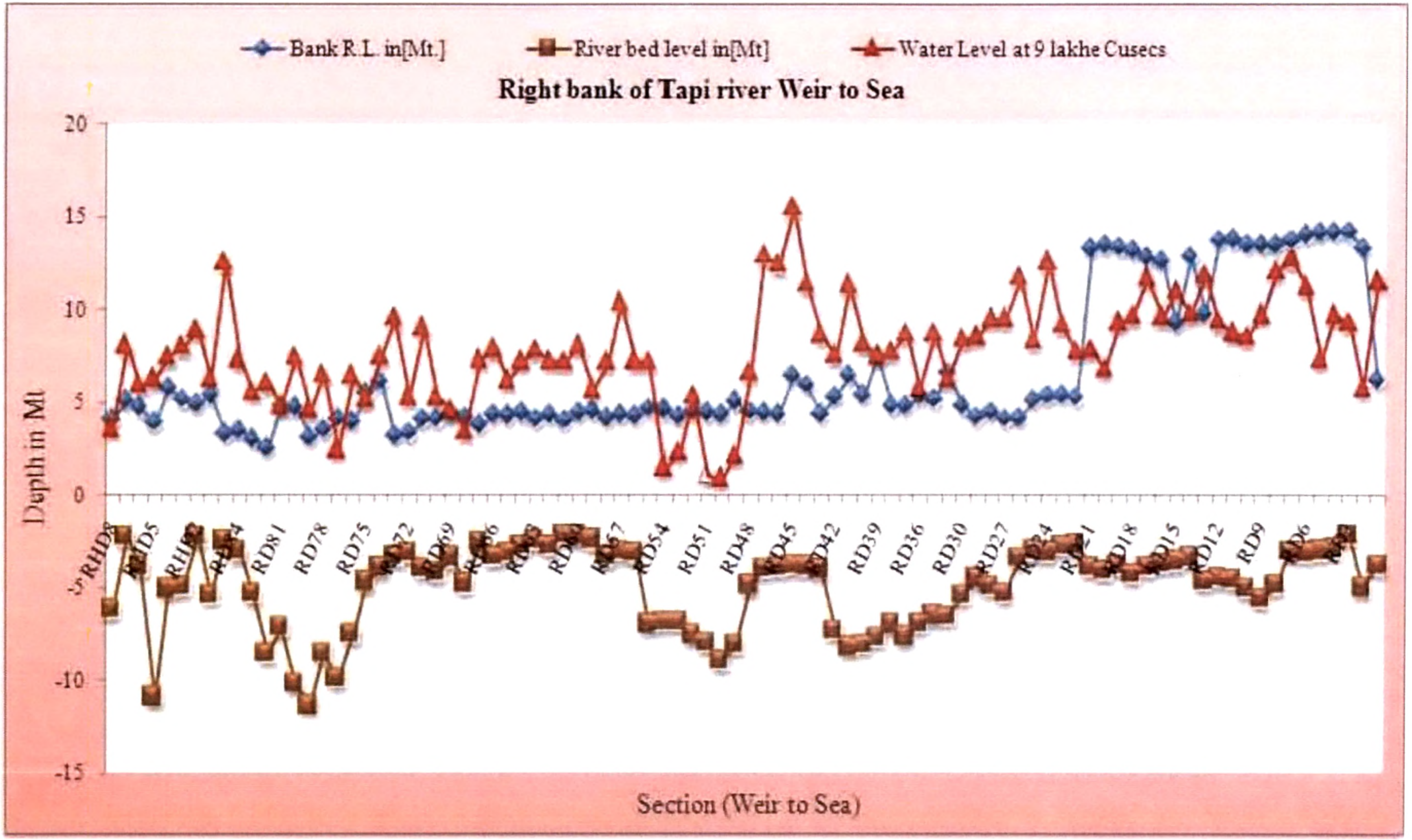
For the present work hydrological data on Tapi river have been used and major flood event discharge viz. 10000 m^3/s (3.5 Lac Cusecs), 15000 m^3/s (5.25 Lac Cusecs), 20000 m^3/s (7.0 Lac Cusecs), 25771.1 m^3/s (9.1 Lac Cusecs), 30000 m^3/s (10.60 Lac Cusecs), 35000 m^3/s (12.36 Lac Cusecs) and 40000 m^3/s (14.12 Lac Cusecs) observed at the Tapi river are taken in to account to perform the hydrological analysis in HEC-RAS software. The geometric data have been edited. The editing consists of cross section and Manning coefficient. Cross section extracted from DEM data has been corrected, particularly surrounding the river and where as the flooded area has been calculated for different discharge using river model results. Different water depths on different land use have a different impact in term of damage and cost. The distribution of water depth can be calculated by subtracting grid maps of water surface and Terrain. The calculation of flooded area per water depth class is done by flood depth map and following this the model of Surat city showing hydraulic features.

Compare to the left bank of river Tapi, right bank is very low in height as shown in Graph.No. 5.4 and 5.5. Even at same chainger the difference in the height of bank is significant. The graph for right bank, shown in Graph.5.5 clearly indicates that most of the bank levels are lower than water surface level at 25775 m^3/s (9.1 Lac Cusecs). During the flood of 2006 the water spilled from the section RD20 to RD47 near Adajan, RD55 to RD72 near Bhator village and at other section shown in the graph.

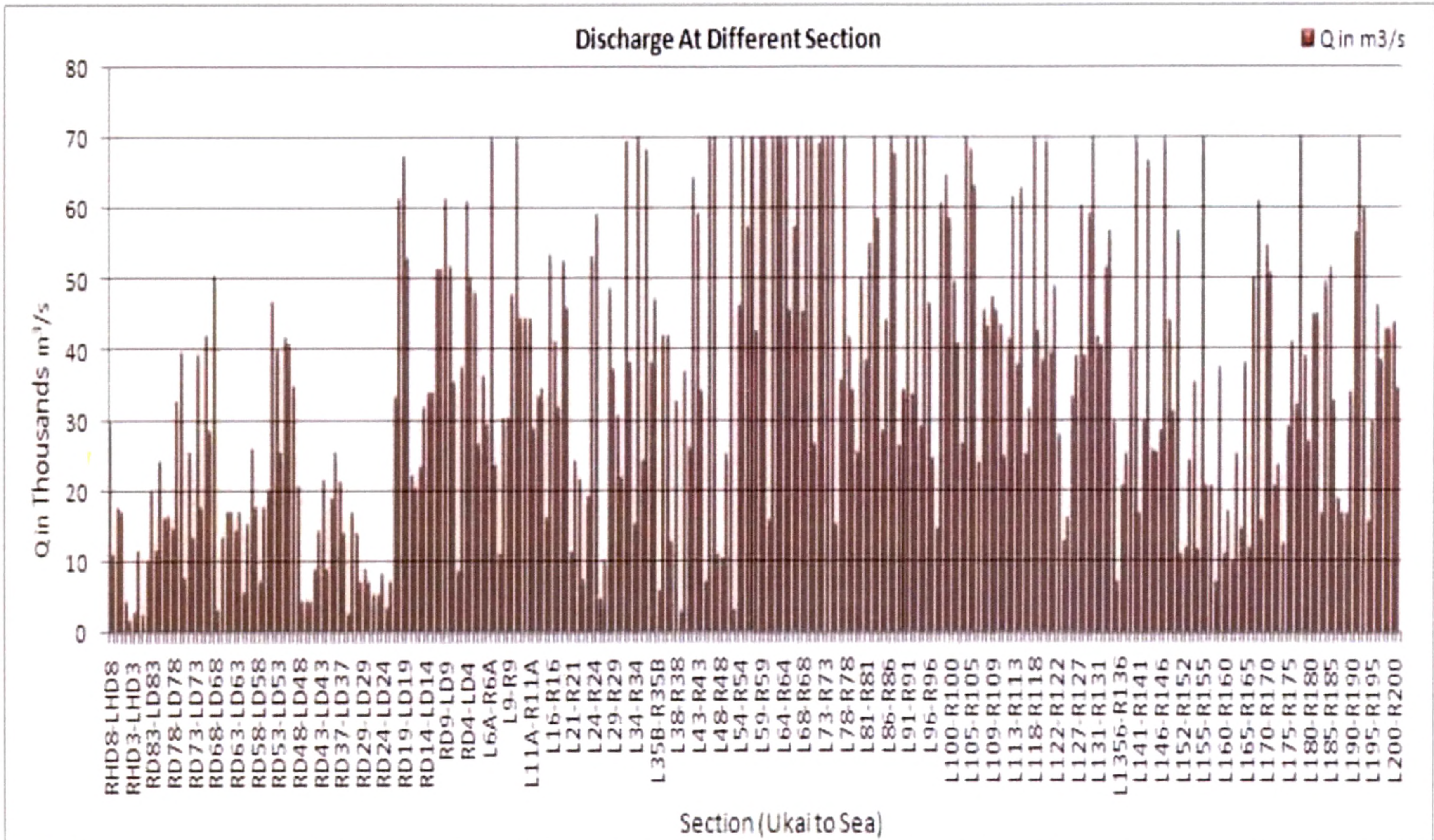
Software developed in Microsoft Excel for finding out maximum carrying capacity of the river gives very reliable results shown in Graph.No.5.6. The river hydraulic data are very useful for analysis to find narrowing of the Tapi River. 40 years back it could carry 28310 m^3/s (10 Lac Cusecs) of water. It has been reduced to 9910 m^3/s (3.5 Lac Cusecs).



Graph.5.4 Water Level & Bank Level of Left Bank at Various Section



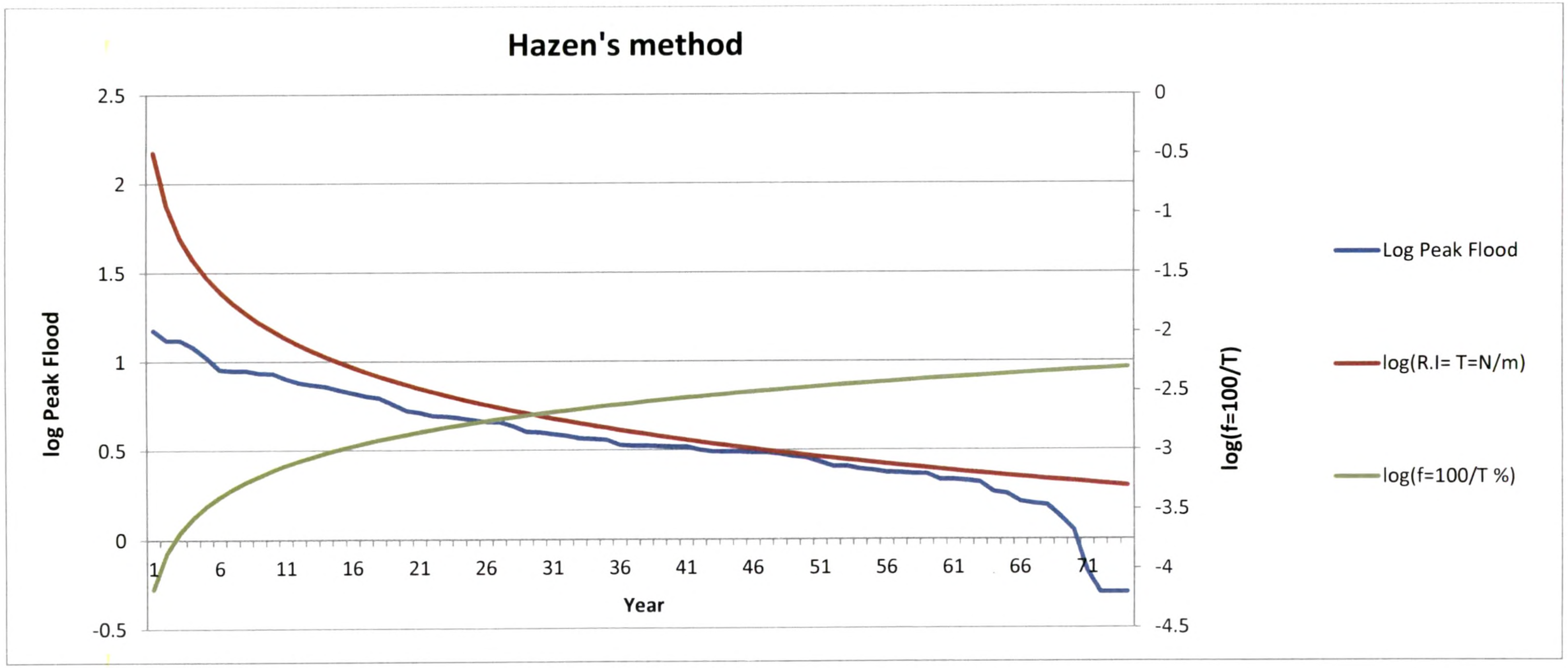
Graph.5.5 Water Level & Bank Level of Right Bank at Various Section



Graph.5.6 Discharge Carrying Capacity at Various Cross Sections

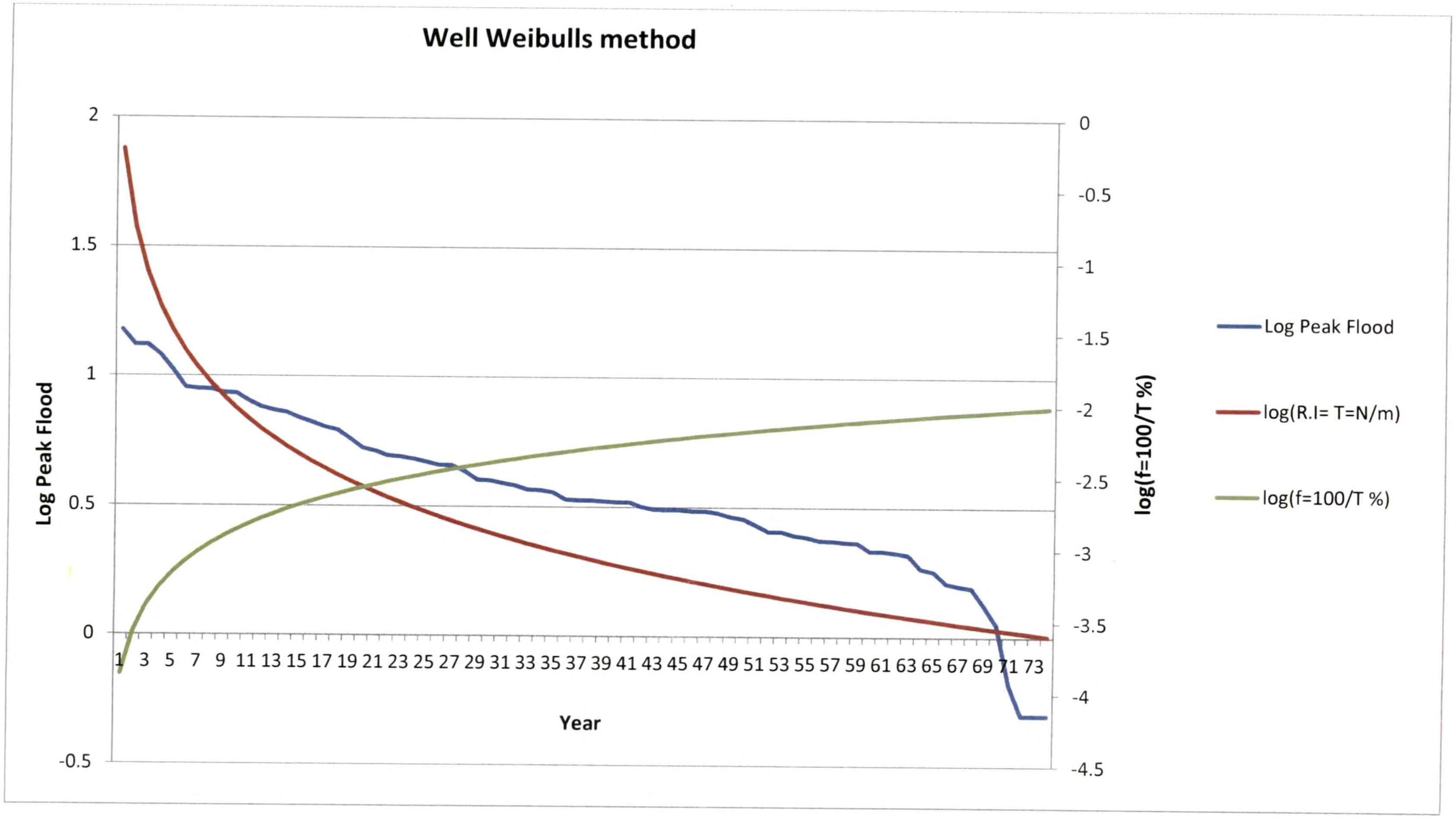
5.3 Determination of Peak Flood Frequency Probability by California Method

Probability analysis for occurrence of peak flood calculated by three methods 1st one name Hazen's statically method, 2nd one Well Weibulls method and 3rd one California method. Probability analysis for occurrence of peak flood calculated by Hazen's statically method and log graph plotted for Tapi river-Surat. The peak flood data used 73 years (from 1939-2012) for this research study. Peak flood will occur after fourty years according to this method. Respective equations used in each method for statistical analysis shown in Table.No.5.3. The result of analysis shown in Graph.No.5.7.



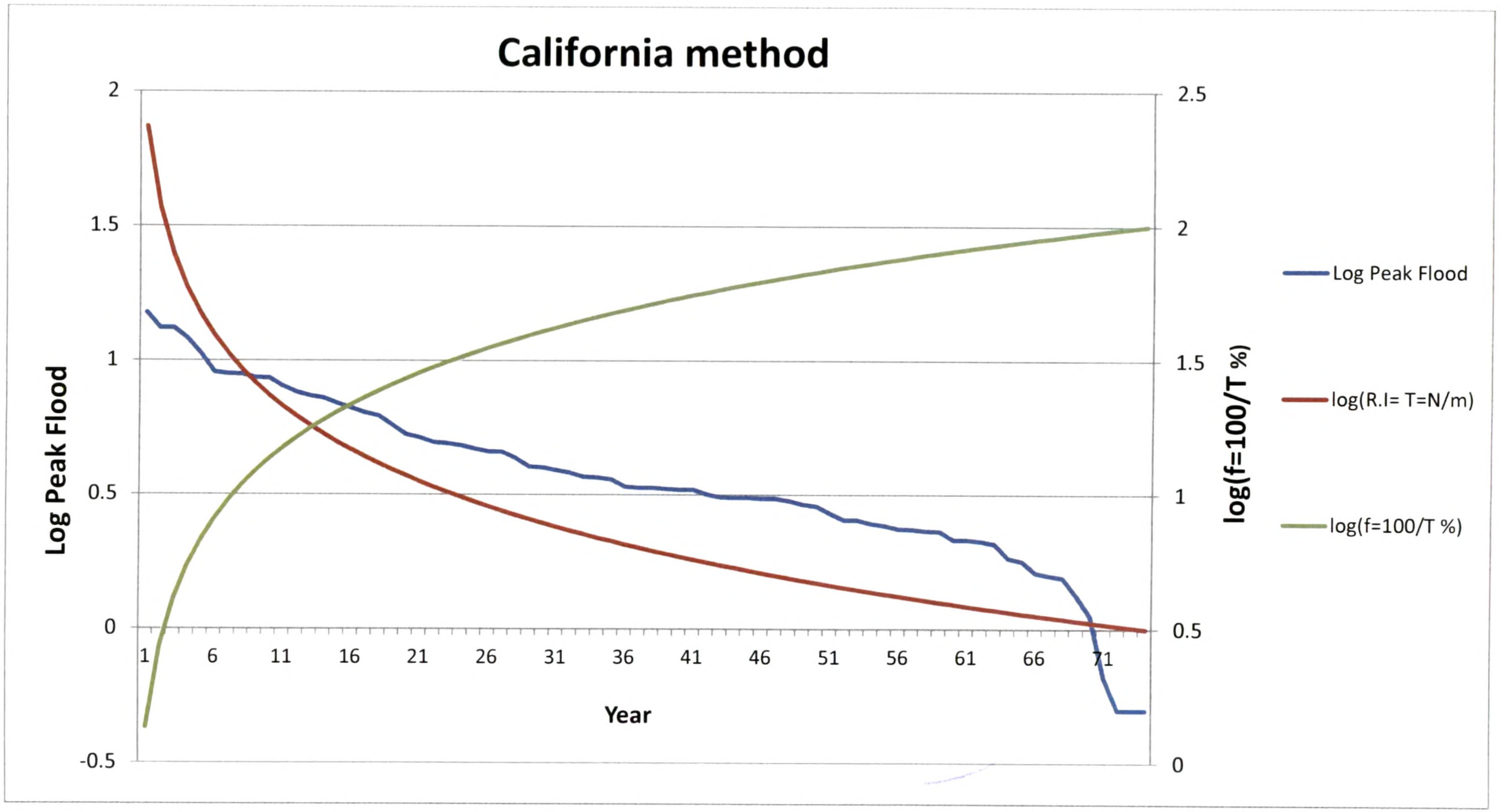
Graph.5.7 Flood Probability by Hazen's Method

Probability analysis for occurrence of peak flood calculated by Well Weibulls statically method and log graph plotted for Tapi river-Surat. Peak flood will occur after fourteen years according to this method. The result of analysis shown in Graph.No.5.8 and Table.No 5.3.



Graph.5.8. Flood Probability by Well Weibulls Method

Probability analysis for occurrence of peak flood calculated by California statically method and log graph plotted for Tapi river-Surat. Peak flood will occur after every eight years according to this method best suitable for Indian climate condition. The result of analysis shown in Graph.No.5.9 and Table. No 5.3.



Graph.5.9. Flood Probability by California Method

SMT. HANSA MEHTA LIBRARY

Table.5.3 Probability Analysis by California Method, Hazen's Method and Weibulls Method

Sr. No.	YEARS	PEAK FLOOD IN lacs cusecs	P.F in descending order	Rank (m)	P.F in descending order	California Method		P.F in descending order	Hazen's Method		P.F in descending order	Well Weibulls Method	
						R.I= T=N/m	f=100/T %		T=n/(m*0.5)	f=1/t*100		T=(N+1)/m	f=1/T*100
1	1939	5.150	15.000	1.000	15.000	15.000	6.667	15.000	30.000	3.333	15.000	16.000	0.001
2	1940	2.430	13.160	2.000	13.160	6.580	15.198	13.160	13.160	7.599	13.160	7.080	0.001
3	1941	4.810	13.140	3.000	13.140	4.380	22.831	13.140	8.760	11.416	13.140	4.713	0.002
4	1942	7.580	12.050	4.000	12.050	3.013	33.195	12.050	6.025	16.598	12.050	3.263	0.003
5	1943	1.790	10.530	5.000	10.530	2.106	47.483	10.530	4.212	23.742	10.530	2.306	0.004
6	1944	9.000	9.000	6.000	9.000	1.500	66.667	9.000	3.000	33.333	9.000	1.667	0.006
7	1945	7.220	8.880	7.000	8.880	1.269	78.829	8.880	2.537	39.414	8.880	1.411	0.007
8	1946	3.000	8.870	8.000	8.870	1.109	90.192	8.870	2.218	45.096	8.870	1.234	0.008
9	1947	2.910	8.580	9.000	8.580	0.953	104.895	8.580	1.907	52.448	8.580	1.064	0.009
10	1948	2.550	8.560	10.000	8.560	0.856	116.822	8.560	1.712	58.411	8.560	0.956	0.010
11	1949	6.620	7.990	11.000	7.990	0.726	137.672	7.990	1.453	68.836	7.990	0.817	0.012
12	1950	3.980	7.580	12.000	7.580	0.632	158.311	7.580	1.263	79.156	7.580	0.715	0.014
13	1951	1.620	7.360	13.000	7.360	0.566	176.630	7.360	1.132	88.315	7.360	0.643	0.016
14	1952	1.120	7.220	14.000	7.220	0.516	193.906	7.220	1.031	96.953	7.220	0.587	0.017
15	1953	3.600	6.890	15.000	6.890	0.459	217.707	6.890	0.919	108.853	6.890	0.526	0.019
16	1954	6.890	6.620	16.000	6.620	0.414	241.692	6.620	0.828	120.846	6.620	0.476	0.021
17	1955	2.360	6.370	17.000	6.370	0.375	266.876	6.370	0.749	133.438	6.370	0.434	0.023
18	1956	3.060	6.200	18.000	6.200	0.344	290.323	6.200	0.689	145.161	6.200	0.400	0.025
19	1957	1.580	5.730	19.000	5.730	0.302	331.588	5.730	0.603	165.794	5.730	0.354	0.028
20	1958	6.200	5.290	20.000	5.290	0.265	378.072	5.290	0.529	189.036	5.290	0.315	0.032
21	1959	13.160	5.150	21.000	5.150	0.245	407.767	5.150	0.490	203.883	5.150	0.293	0.034
22	1960	2.550	4.940	22.000	4.940	0.225	445.344	4.940	0.449	222.672	4.940	0.270	0.037
23	1961	7.360	4.900	23.000	4.900	0.213	469.388	4.900	0.426	234.694	4.900	0.257	0.039
24	1962	7.990	4.810	24.000	4.810	0.200	498.960	4.810	0.401	249.480	4.810	0.242	0.041

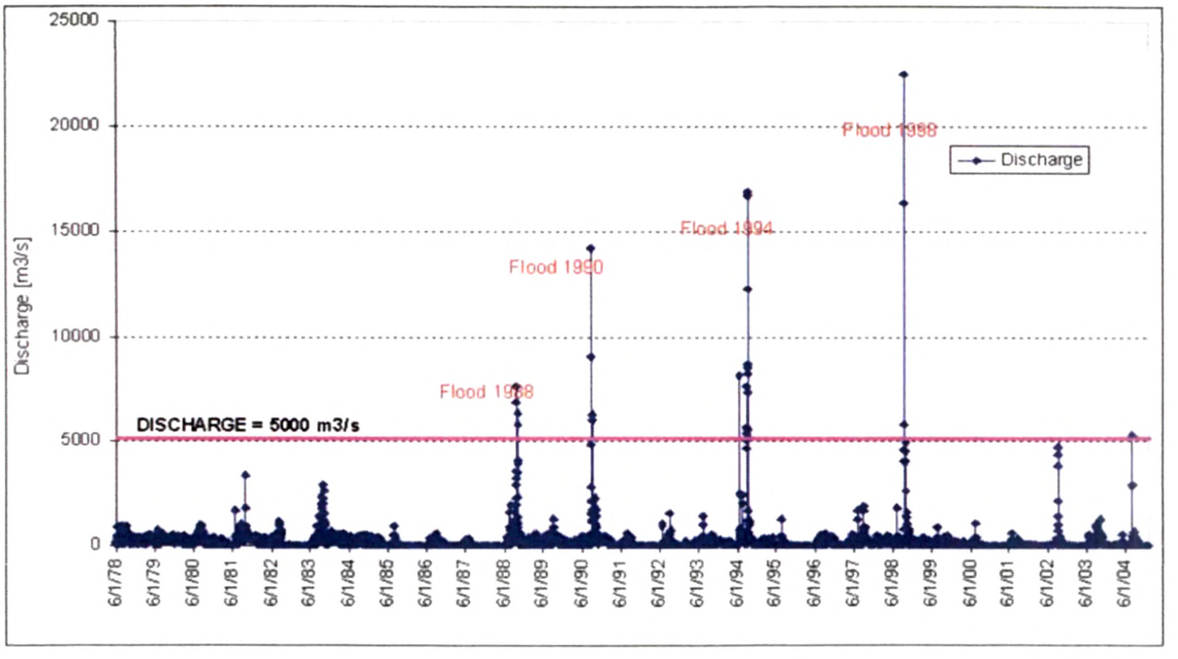
25	1963	2.700	4.680	25.000	4.680	0.187	534.188	4.680	0.374	267.094	4.680	0.227	0.044
26	1964	2.150	4.560	26.000	4.560	0.175	570.175	4.560	0.351	285.088	4.560	0.214	0.047
27	1965	1.550	4.550	27.000	4.550	0.169	593.407	4.550	0.337	296.703	4.550	0.206	0.049
28	1966	3.660	4.320	28.000	4.320	0.154	648.148	4.320	0.309	324.074	4.320	0.190	0.053
29	1967	4.550	4.010	29.000	4.010	0.138	723.192	4.010	0.277	361.596	4.010	0.173	0.058
30	1968	15.000	3.980	30.000	3.980	0.133	753.769	3.980	0.265	376.884	3.980	0.166	0.060
31	1969	8.560	3.890	31.000	3.890	0.125	796.915	3.890	0.251	398.458	3.890	0.158	0.063
32	1970	13.140	3.810	32.000	3.810	0.119	839.895	3.810	0.238	419.948	3.810	0.150	0.067
33	1971	0.660	3.680	33.000	3.680	0.112	896.739	3.680	0.223	448.370	3.680	0.142	0.071
34	1972	2.470	3.660	34.000	3.660	0.108	928.962	3.660	0.215	464.481	3.660	0.137	0.073
35	1973	5.290	3.600	35.000	3.600	0.103	972.222	3.600	0.206	486.111	3.600	0.131	0.076
36	1974	3.060	3.350	36.000	3.350	0.093	1074.627	3.350	0.186	537.313	3.350	0.121	0.083
37	1975	4.560	3.350	37.000	3.350	0.091	1104.478	3.350	0.181	552.239	3.350	0.118	0.085

Table.5.3 Probability Analysis by California Method, Hazen's Method and Well Wall's Method (Continue)

Sr. No.	YEARS	PEAK FLOOD IN lacs cusecs	P.F in descending order	Rank (m)	P.F in descending order	California Method		P.F in descending order	Hazen's Method		P.F in descending order	Well Weibulls Method	
						R.I= T=N/m	f=100/T %		T=n/(m*0.5)	f=1/t*100		T=(N+1)/m	f=1/T*100
38	1976	3.810	3.320	38.000	3.320	0.087	1144.578	3.320	0.175	572.289	3.320	0.114	0.088
39	1977	3.090	3.300	39.000	3.300	0.085	1181.818	3.300	0.169	590.909	3.300	0.110	0.091
40	1978	8.880	3.300	40.000	3.300	0.083	1212.121	3.300	0.165	606.061	3.300	0.108	0.093
41	1979	8.580	3.170	41.000	3.170	0.077	1293.375	3.170	0.155	646.688	3.170	0.102	0.098
42	1980	3.170	3.100	42.000	3.100	0.074	1354.839	3.100	0.148	677.419	3.100	0.098	0.102
43	1981	5.730	3.090	43.000	3.090	0.072	1391.586	3.090	0.144	695.793	3.090	0.095	0.105
44	1982	1.330	3.090	44.000	3.090	0.070	1423.948	3.090	0.140	711.974	3.090	0.093	0.108
45	1983	3.400	3.060	45.000	3.060	0.068	1470.588	3.060	0.136	735.294	3.060	0.090	0.111
46	1984	0.500	3.060	46.000	3.060	0.067	1503.268	3.060	0.133	751.634	3.060	0.088	0.113
47	1985	0.500	3.000	47.000	3.000	0.064	1566.667	3.000	0.128	783.333	3.000	0.085	0.118
48	1986	2.860	2.910	48.000	2.910	0.061	1649.485	2.910	0.121	824.742	2.910	0.081	0.123

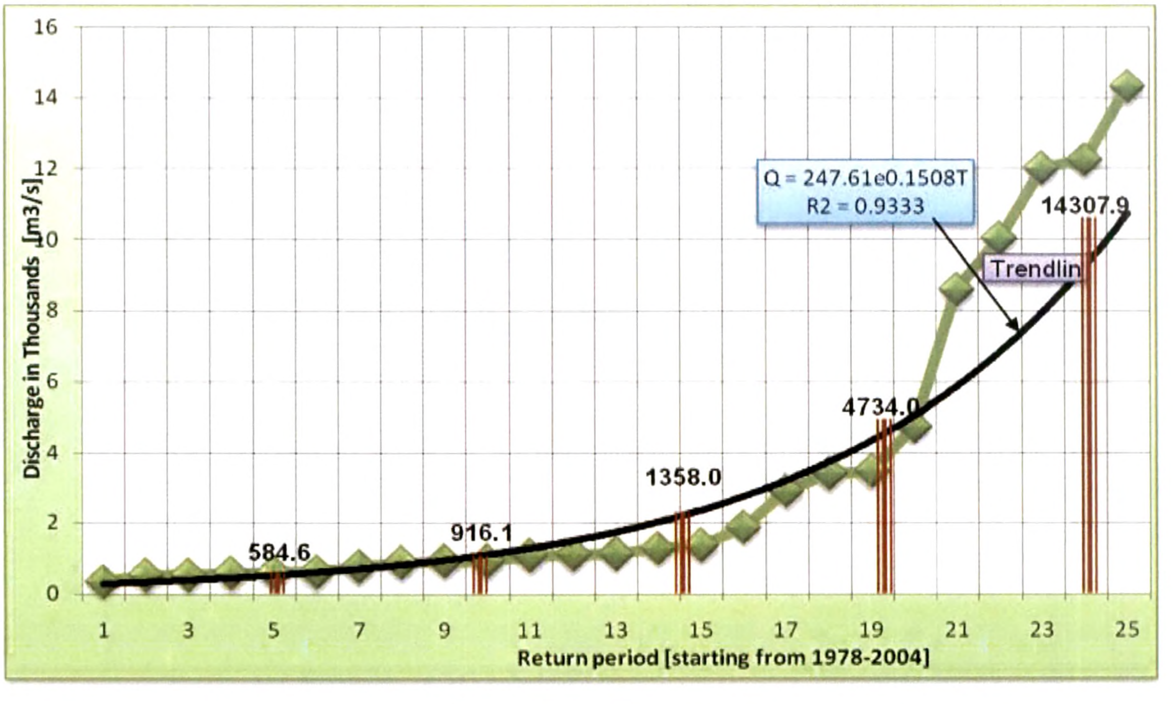
49	1987	0.500	2.860	49.000	2.860	0.058	1713.287	2.860	0.117	856.643	2.860	0.079	0.127
50	1988	3.300	2.700	50.000	2.700	0.054	1851.852	2.700	0.108	925.926	2.700	0.074	0.135
51	1989	3.100	2.550	51.000	2.550	0.050	2000.000	2.550	0.100	1000.000	2.550	0.070	0.144
52	1990	4.900	2.550	52.000	2.550	0.049	2039.216	2.550	0.098	1019.608	2.550	0.068	0.146
53	1991	3.680	2.470	53.000	2.470	0.047	2145.749	2.470	0.093	1072.874	2.470	0.065	0.153
54	1992	1.840	2.430	54.000	2.430	0.045	2222.222	2.430	0.090	1111.111	2.430	0.064	0.157
55	1993	3.350	3.380	55.000	3.380	0.061	1627.219	3.380	0.123	813.609	3.380	0.080	0.126
56	1994	8.870	2.360	56.000	2.360	0.042	2372.881	2.360	0.084	1186.441	2.360	0.060	0.167
57	1995	4.010	2.350	57.000	2.350	0.041	2425.532	2.350	0.082	1212.766	2.350	0.059	0.170
58	1996	2.120	2.320	58.000	2.320	0.040	2500.000	2.320	0.080	1250.000	2.320	0.057	0.175
59	1997	4.940	2.310	59.000	2.310	0.039	2554.113	2.310	0.078	1277.056	2.310	0.056	0.178
60	1998	10.530	2.150	60.000	2.150	0.036	2790.698	2.150	0.072	1395.349	2.150	0.053	0.190
61	1999	3.300	2.150	61.000	2.150	0.035	2837.209	2.150	0.070	1418.605	2.150	0.052	0.194
62	2000	2.380	2.120	62.000	2.120	0.034	2924.528	2.120	0.068	1462.264	2.120	0.050	0.199
63	2001	3.090	2.080	63.000	2.080	0.033	3028.846	2.080	0.066	1514.423	2.080	0.049	0.205
64	2002	4.320	1.840	64.000	1.840	0.029	3478.261	1.840	0.058	1739.130	1.840	0.044	0.225
65	2003	3.320	1.790	65.000	1.790	0.028	3631.285	1.790	0.055	1815.642	1.790	0.043	0.233
66	2004	3.890	1.620	66.000	1.620	0.025	4074.074	1.620	0.049	2037.037	1.620	0.040	0.252
67	2005	4.680	1.580	67.000	1.580	0.024	4240.506	1.580	0.047	2120.253	1.580	0.039	0.260
68	2006	12.050	1.550	68.000	1.550	0.023	4387.097	1.550	0.046	2193.548	1.550	0.038	0.267
69	2007	6.370	1.330	69.000	1.330	0.019	5187.970	1.330	0.039	2593.985	1.330	0.034	0.296
70	2008	2.080	1.120	70.000	1.120	0.016	6250.000	1.120	0.032	3125.000	1.120	0.030	0.330
71	2009	2.150	0.660	71.000	0.660	0.009	10757.576	0.660	0.019	5378.788	0.660	0.023	0.428
72	2010	2.320	0.500	72.000	0.500	0.007	14400.000	0.500	0.014	7200.000	0.500	0.021	0.480
73	2011	2.310	0.500	73.000	0.500	0.007	14600.000	0.500	0.014	7300.000	0.500	0.021	0.487
74	2012	3.350	0.500	74.000	0.500	0.007	14800.000	0.500	0.014	7400.000	0.500	0.020	0.493

Below Graph.No.5.10 Shows flood probability analysis at Ghala station no lower Tapi basin: issues experienced floods during 1959, 1968, 1978, 1979, 1994, 1998, 2006 i.e.1 in 8 years.



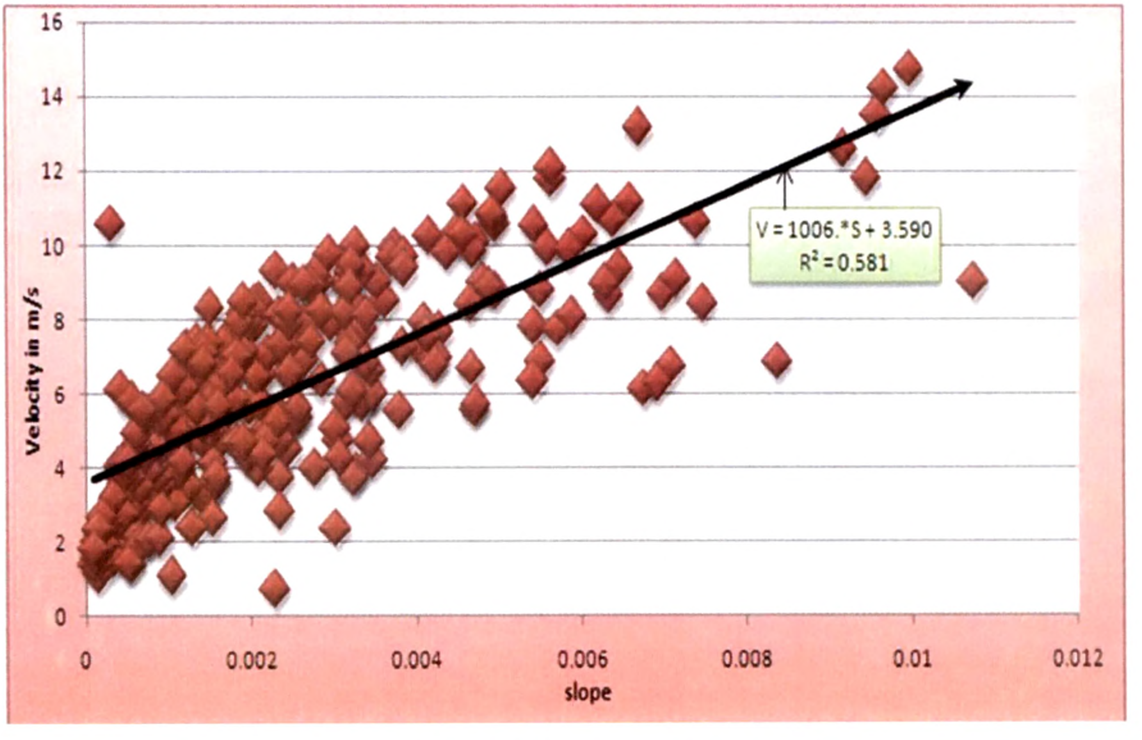
Graph.5.10 Flood Probability Analysis at Ghala - floods during 1959, 1968, 1978, 1979, 1994, 1998, 2006

Below Graph.No.5.11 shows Flood frequency analysis at Ghala gauge station: 27-years measured data shows good trend fit, R2 = 0.93 but, extreme values are not yet well captured.



Graph.5.11 Flood frequency analysis at Ghala gauge station

Below Graph.No.5.12 shows Cross-section velocities are calculated using Manning's equation.



Graph.5.12 Cross-section Velocities