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To Increase Carrying Capacity of Tapi River Using HEC-RAS Software

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Abstract: ***In the present study,One-Dimensional steady flow analysis using flood events and unsteady flow analysis using daily discharge flow and daily tidal level. Tapi River from weir cum causeway to Magadalla Bridge and also evaluation for flood situation in 1883,1884,1942,2006, and 2013 is carried out using steady flow and unsteady flow analysis. HEC-RAS software is used for analysis.***

Keywords: HEC-RAS, One-Dimensional steady flow analysis, flood control, Tapi River, discharge

1. Introduction

HEC-RAS is a one-dimensional steady flow hydraulic model designed to aid hydraulic engineers in channel flow analysis and floodplain determination. The results of the model can be applied in floodplain management and flood insurance studies. If you recall from hydraulics, steady flow describes conditions in which depth and velocity at a given channel location do not change with time. Gradually varied flow is characterized by minor changes in water depth and velocity from cross-section to cross-section. The primary procedure used by HEC-RAS to compute water surface profiles assumes a steady, gradually varied flow scenario, and is called the direct step method. The basic computational procedure is based on an iterative solution of the energy equation:

$$H = Z + Y + \frac{\alpha V^2}{2g}$$

Which states that the total energy (H) at any given location along the stream is the sum of potential energy (Z + Y) and kinetic energy ($aV^2/2g$). The change in energy between two cross-sections is called head loss (h_L). The energy equation parameters are illustrated in the following graphic:

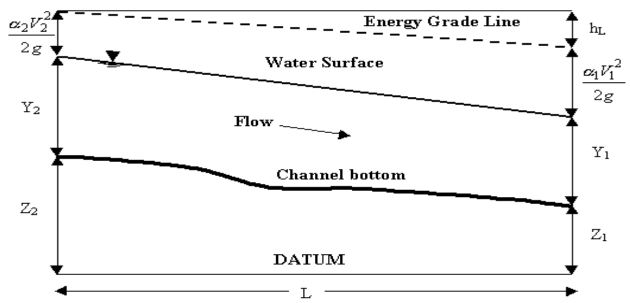

Figure 1

Given the flow and water surface elevation at one cross-section, the goal of the direct step method is to compute the water surface elevation at the adjacent cross-section. Whether the computations proceed from upstream to downstream or vice versa, depends on the flow regime. The dimensionless Froude number (Fr) is used to characterize flow regime, where:

•Fr< 1 denotes subcritical flow
•Fr> 1 denotes supercritical flow
•Fr = 1 denotes critical flow

For a subcritical flow scenario, which is very common in natural and man-made channels, direct step computations would begin at the downstream end of the reach, and progress upstream between adjacent cross-sections. For supercritical flow, the computations would begin at the upstream end of the reach and proceed downstream.

The development of the program (HEC-RAS) was done at the Hydrologic Engineering Centre (HEC), which is a part of the Institute for Water Resources (IWR), U.S. Army Corps of Engineers; Fig 2 shows the main window of HEC-RAS.

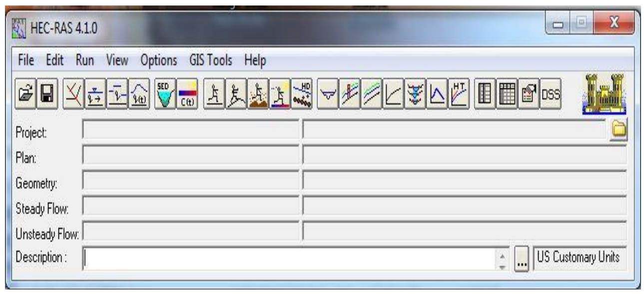

Figure 2: HEC-RAS main window

HEC-RAS has the ability to make the calculations of water surface profiles for steady and gradually varied flow as well as for subcritical, super critical, and mixed flow regime. In addition to this, HEC-RAS is capable to do modeling for sediment transport, which is notoriously difficult. Therefore, modeling sediment transport is based on assumptions and empirical theory that is sensitive to several physical variables (Brunner, 2010).

For making such calculations, HEC-RAS requires boundary conditions for each type of data. These boundary conditions are important to determine the mathematical solutions to the problems. Boundary conditions are required to obtain the solution to the set of differential equations describing the problem over the domain of interest. In HEC-RAS, there are several boundary conditions available for steady flow and sediments analysis computations. Boundary conditions can be either externally specified at the ends of the network system (upstream or downstream) or internally used for connections to junctions.

2. HEC-RAS Parameters

Steady and unsteady flow analysis when carried out using HEC-RAS software which parameters are required as inputs are discussed in the subsequent paragraphs.

3. Definition Sketch

Fig. 3 represents a definition sketch for the stream. In this the river width is divided into three segments as main channel, left bank flood way and right bank flood way. The location of left bank station and right bank station are also marked in the definition sketch. The normal water surface and flood water surface are shown in figure.

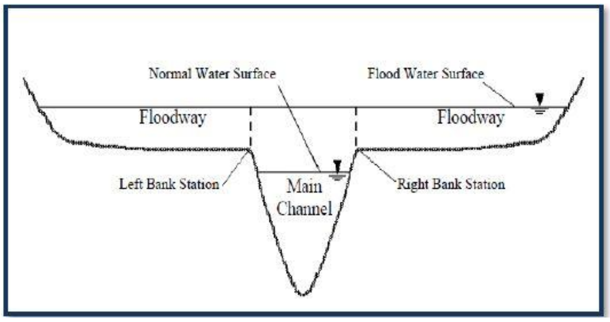

Figure 3: Stream Schematic Diagrams

At each cross-section, HEC-RAS uses several input parameters to describe shape, elevation, and relative location along the stream:

- River station (cross-section) number
- Lateral and elevation coordinates for each (dry, unflooded) terrain point
- Left and right bank station locations
- Reach lengths between the left floodway, stream Centerline, and right floodway of adjacent cross-sections (The three reach lengths represent the average flow path through each segment of the cross-section pair. As such, the three reach lengths between adjacent cross-sections may differ in magnitude due to bends in the stream.)
- Manning's roughness coefficients (may vary horizontally or vertically)
- Channel contraction and expansion coefficients
- Geometric description of any hydraulic structure, such as bridges, culverts, and weirs.

4. Evaluation of Flood Events Using Steady Flow Analysis

Data Analysis is the key tool in understanding the behaviour of the river cross-sections under the effect of various flood events. The flood events of 1883, 1884, 1942, 2006, and 2013 were considered for steady flow analysis of Tapi river for a reach of 15 km. The study reach is influenced by high tide and low tides. The study reach is a part of confluence of Tapi river and Arabian sea. Therefore, high tides and low tides have effect on the dispersion of flood water in to sea. The study of behavior of river cross-sections under various flood discharges along with tidal condition are carried out. The carrying capacity of study reach section are accessed for five flood events.

5. Results

5.1 Hydraulic Geometry Assessment Based on Flood Data of 1883

In this uniform flow computation is carried out using HEC-RAS and flood data of 1983 having peak discharge 28480cumecs.From cross section 49 to cross section 8 overtopping are observed and no overtopping in cross section 7 to 1. The result of simulated water levels for section numbers 11 to 49 are as shown in Table 1 and figure 4 to 8.

Table 1: Flood Event of 1883

Flood event 1883= 28480					
Reach	River Sta	W.S. Elev	VelChnl	Flow Area	Froude# Chl
		(m)	(m/s)	(m2)	
A	49	17.1	1.86	15303.56	0.14
A	48	17.13	1.52	18745.79	0.11
A	47	17.13	1.46	19570.81	0.11
A	46	17.13	1.47	19324.46	0.11
A	45	17.1	1.6	17805.78	0.12
A	44	17.07	1.71	16667.31	0.13
A	43	17.06	1.7	16715.09	0.13
A	42	17.06	1.66	17112.56	0.13
A	41	16.92	2.23	12789.28	0.18
A	40	16.86	2.4	11871.07	0.2
A	39	16.8	2.55	11178.25	0.21
A	38	16.74	2.68	10619.8	0.22
A	37	16.64	2.9	9816.99	0.23
A	36	16.71	2.41	11797.06	0.19
A	35	16.72	2.28	12497.45	0.19
A	34	16.7	2.27	12541.86	0.18
A	33	16.71	2.13	13350.63	0.18
A	32	16.59	2.52	11289.98	0.21
A	31	16.54	2.62	10869.52	0.22
A	30	16.52	2.61	10919.03	0.22
A	29	16.5	2.58	11043.45	0.21
A	28	16.35	2.97	9577	0.25
A	27	16.33	2.92	9756.76	0.25
A	26	16.21	3.18	8948.83	0.27
A	25	16.14	3.25	8761.79	0.28
A	24	16.07	3.34	8520.64	0.29
A	23	15.81	3.86	7380.03	0.33
A	22	15.83	3.61	7891.21	0.31
A	21	15.68	3.83	7439.04	0.34
A	20	15.49	4.1	6939.65	0.37
A	19	15.28	4.39	6494.17	0.4
A	18	14.98	4.8	5937.83	0.44
A	17	14.19	5.91	4821.13	0.52
A	16	14.39	5.07	5614.79	0.46
A	15	14.42	4.68	6082.39	0.42
A	14	14.02	5.21	5466.02	0.48
A	13	13.94	5.08	5608.91	0.46
A	12	14.09	4.34	6559.72	0.4
A	11	14.12	3.99	7140.15	0.37
A	10	14.37	2.69	10568.03	0.27
A	9	14.38	2.47	11507.61	0.24
A	8	14.27	2.72	10453.74	0.26
A	7	14.25	2.64	10784.5	0.26

A	6	14.13	2.9	9812.5	0.29
A	5	14.07	2.94	9682.23	0.3
A	4	13.93	3.19	8924.76	0.32
A	3	13.69	3.64	7825.47	0.39
A	2	13.56	3.72	7650.32	0.38
A	1	13.57	3.4	8370.18	0.33

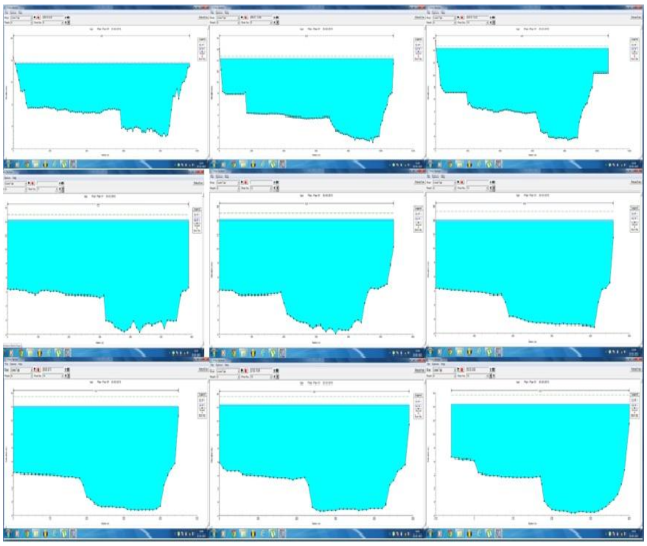

Figure 4: Water Surface Profile at cross section 11 to 20

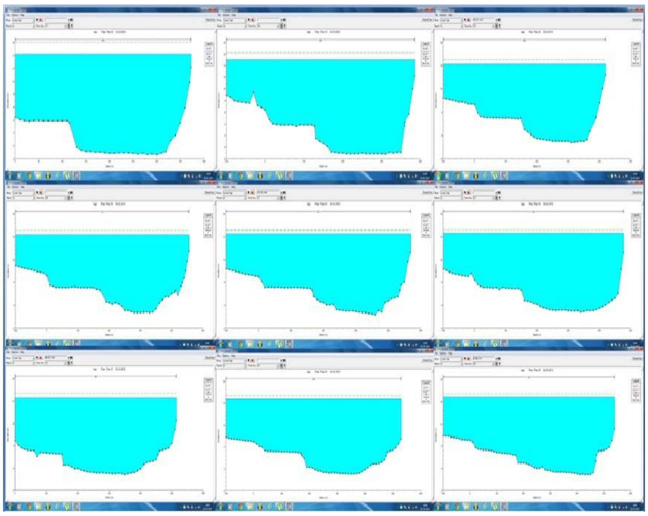

Figure 5: Water Surface Profile at cross section 21 to 29

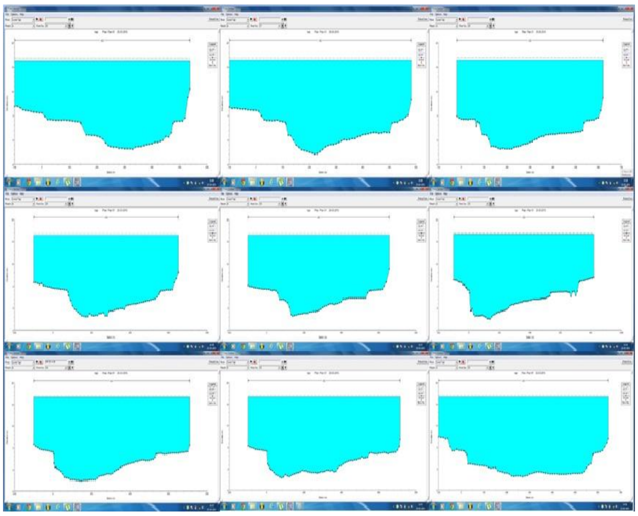

Figure 6: Water Surface Profile at cross section 30 to 38

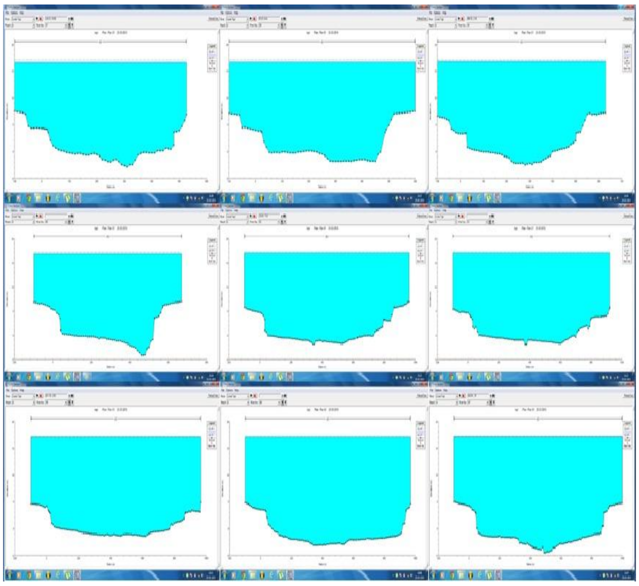

Figure 7: Water Surface Profile at cross section 39 to 47

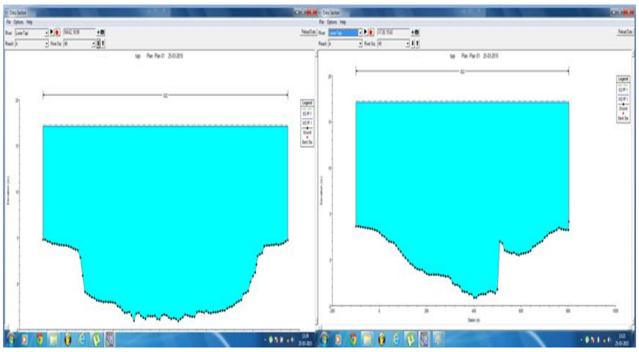

Figure 8: Water Surface Profile at cross section 48, 49

5.2 Hydraulic Geometry Assessment Based On Flood Data Of 1884

In this uniform flow computation is carried out using HEC-RAS and flood data of 1884 having peak discharge 23974.5cumecs. From cross section 49 to cross section 10 overtopping are observed and no overtopping in cross section 9 to 1. The result of simulated water levels for section numbers 10 to 49 are as shown in Table 2 and figure 9 to 13.

Table 2: Flood event of 1884

Flood event 1884= 23974.5					
Reach	River Sta	W.S. Elev	VelChnl	Flow Area	Froude # Chl
		(m)	(m/s)	(m2)	
A	49	15.74	1.7	14071.73	0.14
A	48	15.76	1.39	17306.63	0.11
A	47	15.76	1.33	18062.32	0.1
A	46	15.76	1.34	17861.62	0.1
A	45	15.73	1.47	16349.96	0.12
A	44	15.71	1.57	15300.47	0.13
A	43	15.7	1.56	15322.14	0.13
A	42	15.69	1.53	15666.5	0.13
A	41	15.58	2.05	11685.88	0.17
A	40	15.52	2.21	10840.53	0.19
A	39	15.47	2.35	10206.35	0.2
A	38	15.41	2.47	9692.77	0.21
A	37	15.33	2.66	8996.17	0.22
A	36	15.39	2.22	10811.08	0.19
A	35	15.39	2.1	11418.32	0.18

A	34	15.38	2.09	11468.43	0.18
A	33	15.38	1.98	12127.15	0.17
A	32	15.28	2.32	10321.2	0.2
A	31	15.23	2.42	9916.43	0.21
A	30	15.21	2.41	9929.79	0.21
A	29	15.19	2.38	10062.08	0.21
A	28	15.06	2.74	8754.76	0.24
A	27	15.04	2.7	8877.84	0.24
A	26	14.93	2.95	8136.45	0.26
A	25	14.87	3.02	7946.9	0.27
A	24	14.8	3.1	7721.38	0.28
A	23	14.59	3.55	6749.95	0.31
A	22	14.59	3.34	7177.18	0.3
A	21	14.45	3.56	6740.82	0.33
A	20	14.28	3.83	6263.7	0.36
A	19	14.09	4.08	5873.15	0.39
A	18	13.81	4.46	5373.47	0.43
A	17	13.19	5.39	4447.33	0.5
A	16	13.32	4.68	5124.05	0.45
A	15	13.33	4.32	5552.06	0.41
A	14	12.99	4.79	5006.08	0.46
A	13	12.92	4.67	5138.79	0.45
A	12	13.03	4	5986.15	0.38
A	11	13.04	3.69	6505.41	0.35
A	10	13.24	2.56	9359.39	0.28
A	9	13.24	2.33	10287.51	0.24
A	8	13.15	2.55	9391.1	0.26
A	7	13.12	2.48	9665.3	0.25
A	6	13.01	2.74	8765.15	0.28
A	5	12.95	2.78	8611.83	0.29
A	4	12.81	3.03	7917.41	0.33
A	3	12.56	3.51	6827.1	0.4
A	2	12.45	3.54	6771.02	0.38
A	1	12.46	3.2	7499.32	0.33

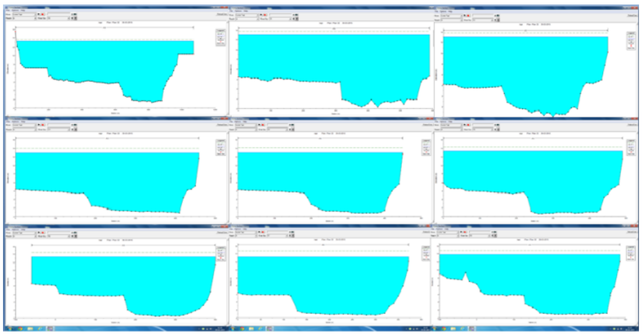

Figure 9: Water Surface Profile at cross section 10 to 18

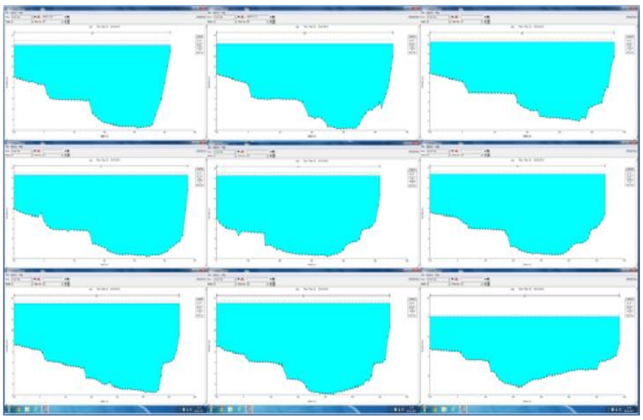

Figure 10: Water Surface Profile at cross section 19 to 27

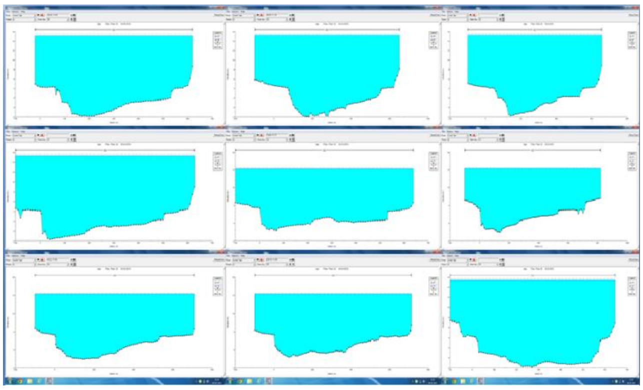

Figure 11: Water Surface Profile at cross section 28 to 36

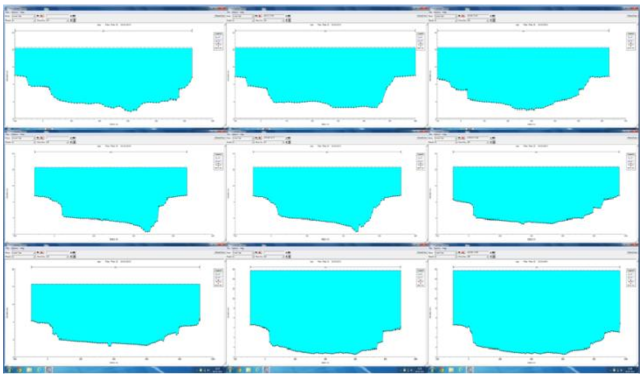

Figure 12: Water Surface Profile at cross section 37 to 45

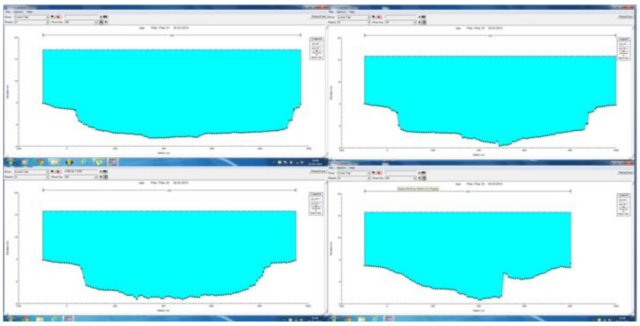

Figure 13: Water Surface Profile at cross section 46 to 49

5.3 Hydraulic Geometry Assessment Based on Flood Data of 1942

In this uniform flow computation is carried out using HEC-RAS and flood data of 1942 having peak discharge 24371.2cumecs.From cross section 49 to cross section 10 overtopping are observed and no overtopping in cross section 9 to 1. The result of simulated water levels for section numbers 10 to 49 are as shown in Table 3 and figure 14 to 18.

Table 3 Flood Event of 1942

Flood event 1942= 24371.2					
Reach	River Sta	W.S. Elev	VelChnl	Flow Area	Froude # Chl
		(m)	(m/s)	(m2)	
A	49	15.86	1.72	14183.64	0.14
A	48	15.89	1.4	17437.37	0.11
A	47	15.89	1.34	18199.35	0.11
A	46	15.88	1.35	17994.5	0.11
A	45	15.85	1.48	16482.19	0.12
A	44	15.83	1.58	15424.63	0.13
A	43	15.82	1.58	15448.66	0.13
A	42	15.82	1.54	15797.84	0.13

A	41	15.7	2.07	11786.12	0.17
A	40	15.65	2.23	10934.15	0.19
A	39	15.59	2.37	10294.66	0.2
A	38	15.53	2.49	9777.03	0.21
A	37	15.45	2.69	9070.8	0.23
A	36	15.51	2.24	10900.7	0.19
A	35	15.51	2.12	11516.39	0.18
A	34	15.5	2.11	11565.99	0.18
A	33	15.5	1.99	12238.31	0.17
A	32	15.4	2.34	10409.25	0.2
A	31	15.35	2.44	10003.06	0.21
A	30	15.33	2.43	10019.7	0.21
A	29	15.31	2.4	10151.26	0.21
A	28	15.18	2.76	8829.51	0.24
A	27	15.16	2.72	8957.73	0.24
A	26	15.05	2.97	8210.3	0.26
A	25	14.99	3.04	8020.97	0.27
A	24	14.92	3.13	7794.02	0.28
A	23	14.7	3.58	6807.28	0.31
A	22	14.7	3.37	7242.11	0.3
A	21	14.56	3.58	6804.3	0.33
A	20	14.39	3.85	6325.16	0.36
A	19	14.2	4.11	5929.64	0.39
A	18	13.92	4.49	5424.84	0.43
A	17	13.28	5.44	4481.64	0.5
A	16	13.42	4.71	5168.91	0.45
A	15	13.43	4.35	5600.5	0.41
A	14	13.09	4.83	5048.19	0.46
A	13	13.01	4.7	5181.82	0.45
A	12	13.13	4.04	6038.55	0.39
A	11	13.14	3.71	6563.37	0.36
A	10	13.34	2.57	9469.43	0.28
A	9	13.34	2.34	10398.62	0.24
A	8	13.25	2.57	9486.82	0.26
A	7	13.23	2.5	9766.85	0.25
A	6	13.11	2.75	8860.14	0.28
A	5	13.05	2.8	8708.82	0.29
A	4	12.91	3.04	8009.04	0.32
A	3	12.66	3.52	6917.9	0.4
A	2	12.55	3.56	6850.77	0.38
A	1	12.56	3.22	7578.46	0.33

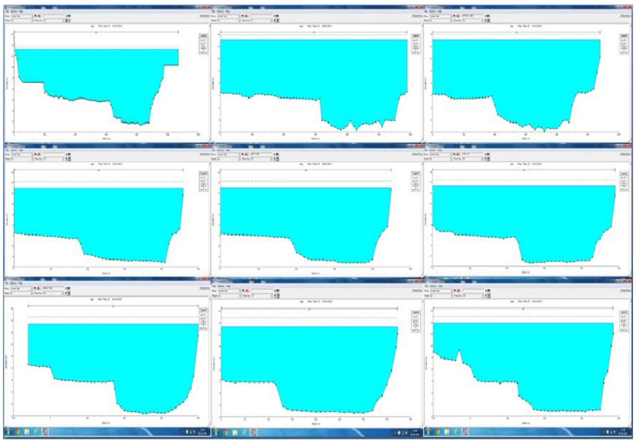

Figure 14: Water Surface Profile at cross section 10 to 18

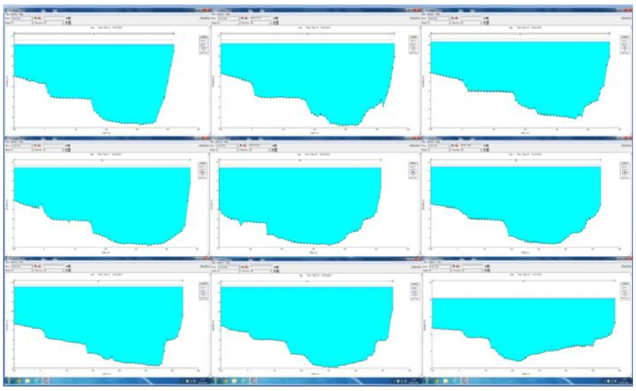

Figure 15: Water Surface Profile at cross section 19 to 27

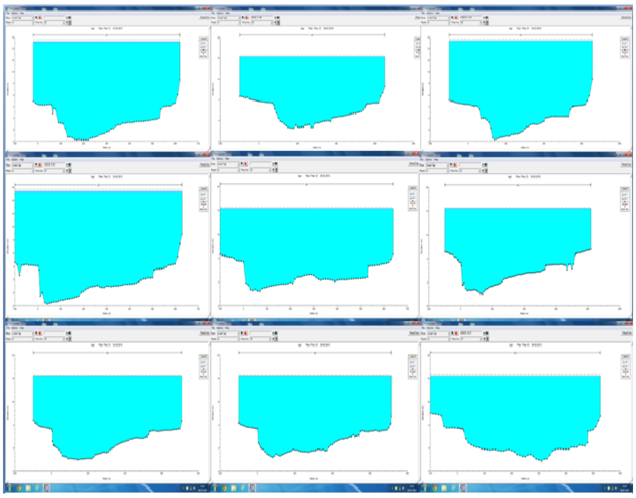

Figure 16: Water Surface Profile at cross section 28 to 36

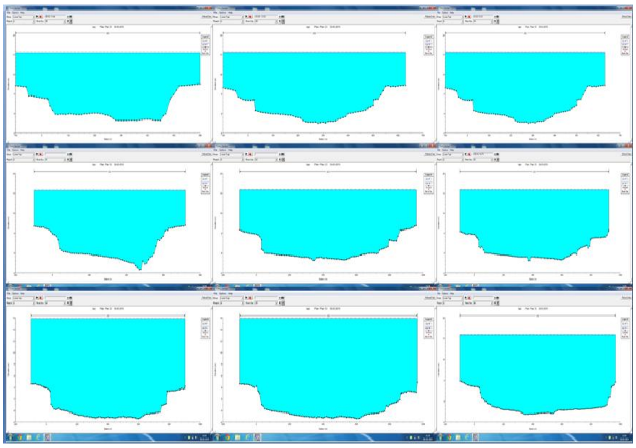

Figure 17: Water Surface Profile at cross section 37 to 46

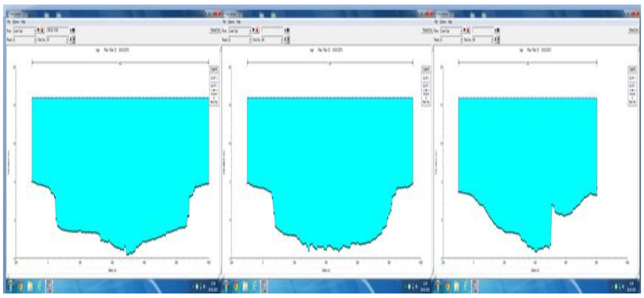

Figure 18: Water Surface Profile at cross section 47 to 49

5.4 Hydraulic Geometry Assessment Based on Flood Data of 2006

In this uniform flow computation is carried out using HEC-RAS and flood data of 2006 having peak discharge 25788cumecs.From cross section 49 to cross section 9 overtopping are observed and no overtopping in cross section 8 to 1. The result of simulated water levels for section numbers 1 to 49 are as shown in Table 4 and figure 19 to 24.

Table 4: Flood Event of 2006

Flood event 2006= 25788					
Reach	River Sta	W.S. Elev	VelChnl	Flow Area	Froude # Chl
		(m)	(m/s)	(m2)	
A	49	16.3	1.77	14578.16	0.14
A	48	16.33	1.44	17898.26	0.11
A	47	16.33	1.38	18682.43	0.11
A	46	16.32	1.4	18462.98	0.11
A	45	16.29	1.52	16948.4	0.12
A	44	16.27	1.63	15862.36	0.13
A	43	16.26	1.62	15894.74	0.13
A	42	16.25	1.59	16260.93	0.13
A	41	16.13	2.12	12139.53	0.18
A	40	16.07	2.29	11264.24	0.19
A	39	16.02	2.43	10605.97	0.2
A	38	15.96	2.56	10073.78	0.22
A	37	15.87	2.76	9333.59	0.23
A	36	15.94	2.3	11216.31	0.19
A	35	15.94	2.17	11861.79	0.18
A	34	15.92	2.17	11909.56	0.18
A	33	15.93	2.04	12629.88	0.18
A	32	15.82	2.41	10719.37	0.2
A	31	15.77	2.5	10308.14	0.21
A	30	15.75	2.49	10336.33	0.22
A	29	15.73	2.46	10465.38	0.21
A	28	15.59	2.84	9092.75	0.24
A	27	15.57	2.79	9239.08	0.24
A	26	15.46	3.04	8470.38	0.27
A	25	15.39	3.11	8281.83	0.28
A	24	15.32	3.2	8049.89	0.29
A	23	15.09	3.68	7009.11	0.32
A	22	15.1	3.45	7470.77	0.31
A	21	14.96	3.67	7027.89	0.33
A	20	14.78	3.94	6541.62	0.37
A	19	14.58	4.21	6128.57	0.39
A	18	14.29	4.6	5605.69	0.43
A	17	13.6	5.6	4602.03	0.51
A	16	13.76	4.84	5326.59	0.45
A	15	13.78	4.47	5770.83	0.42
A	14	13.42	4.96	5196.13	0.47
A	13	13.34	4.84	5333.02	0.45
A	12	13.47	4.14	6222.82	0.39
A	11	13.49	3.81	6767.21	0.36
A	10	13.71	2.62	9856.94	0.27
A	9	13.71	2.39	10789.86	0.24
A	8	13.61	2.62	9826.27	0.26
A	7	13.59	2.55	10124.56	0.25
A	6	13.47	2.8	9194.95	0.29
A	5	13.41	2.85	9050.57	0.29
A	4	13.27	3.1	8331.65	0.32
A	3	13.02	3.56	7237.62	0.4
A	2	12.91	3.62	7131.94	0.38
A	1	12.92	3.28	7857.25	0.33

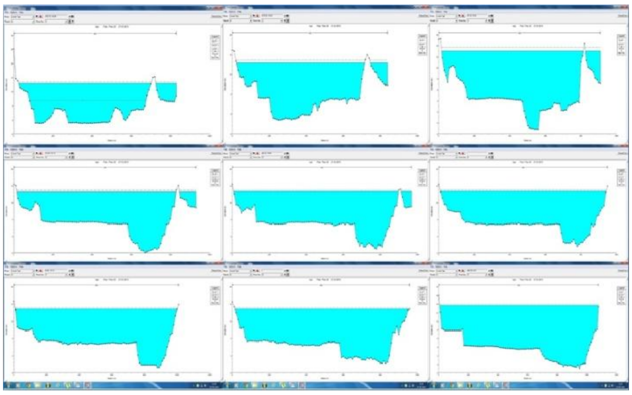

Figure 19: Water Surface Profile at cross section 1 to 9

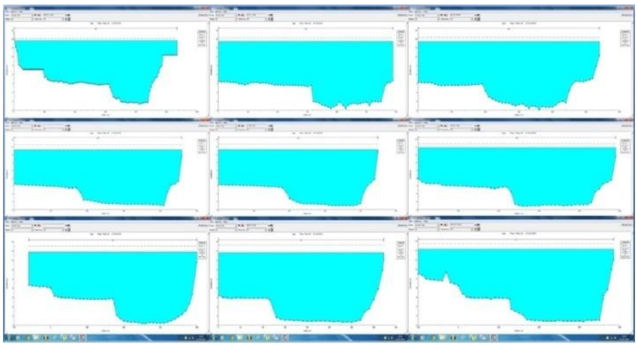

Figure 20: Water Surface Profile at cross section 10 to 18

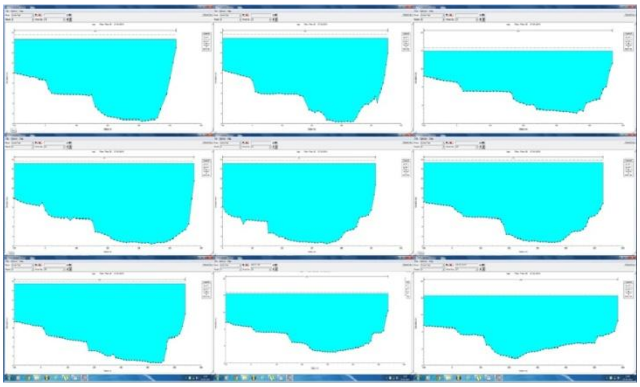

Figure 21: Water Surface Profile at cross section 19 to 28

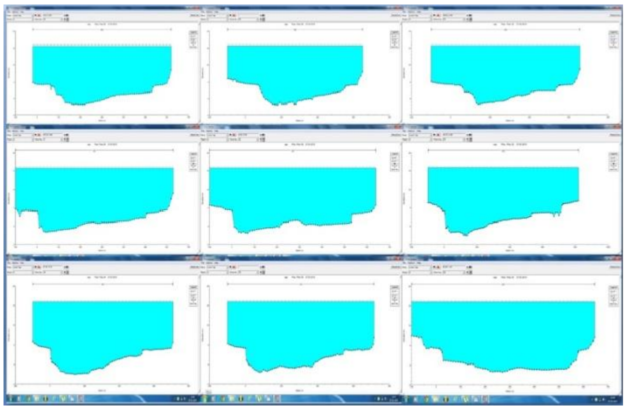

Figure 22: Water Surface Profile at cross section 29 to 36

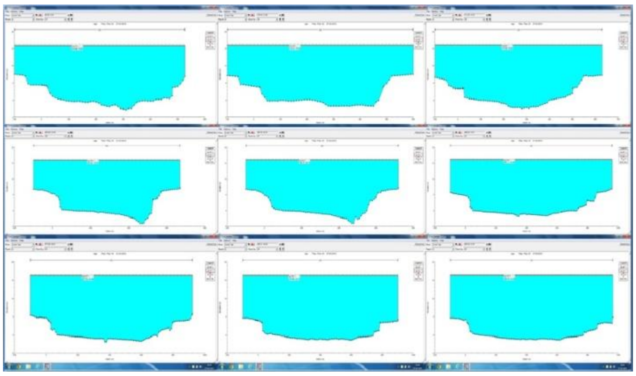

Figure 23: Water Surface Profile at cross section 37 to 45

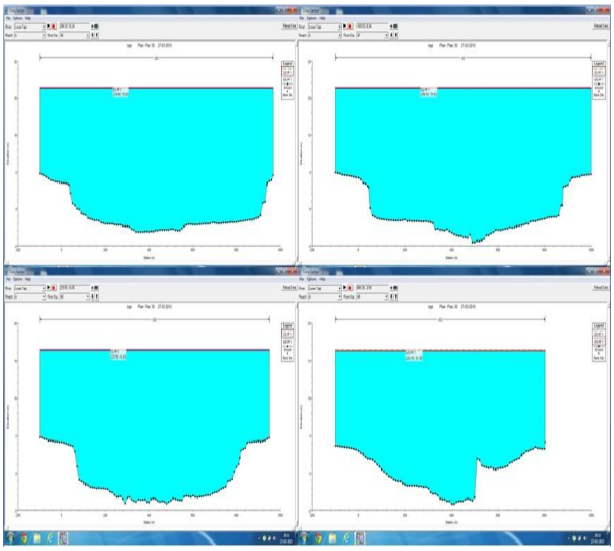

Figure 24: Water Surface Profile at cross section 46 to 49

5.5 Hydraulic Geometry Assessment Based on Flood Data of 2013

In this uniform flow computation is carried out using HEC-RAS and flood data of 2013 having peak discharge 15050cumecs.From cross section 49 to cross section 8 overtopping are observed and no overtopping in cross section 7 to 1. The result of simulated water levels for section numbers 10 to 49 are as shown in Table 5 and figure 25 to 28.

Table 5: Flood Event of 2013

Flood event 2013= 15050					
Reach	River Sta	W.S. Elev	VelChnl	Flow Area	Froude # Chl
		(m)	(m/s)	(m2)	
A	49	12.63	1.34	11267.78	0.12
A	48	12.64	1.07	14032.34	0.09
A	47	12.64	1.03	14630.85	0.09
A	46	12.64	1.04	14533.62	0.09
A	45	12.62	1.15	13038.22	0.11
A	44	12.6	1.23	12190.55	0.11
A	43	12.6	1.24	12152.98	0.11
A	42	12.59	1.22	12377	0.11
A	41	12.51	1.64	9172.4	0.16
A	40	12.47	1.77	8492.07	0.17
A	39	12.43	1.88	7990.72	0.18
A	38	12.39	1.99	7577.79	0.19
A	37	12.34	2.11	7120.6	0.2
A	36	12.37	1.76	8561.42	0.17
A	35	12.37	1.68	8957.63	0.16
A	34	12.36	1.67	9020.85	0.16
A	33	12.35	1.61	9339.16	0.16
A	32	12.29	1.86	8109.48	0.18
A	31	12.25	1.94	7740.51	0.19
A	30	12.22	1.96	7670.9	0.2
A	29	12.21	1.92	7821.07	0.19
A	28	12.12	2.19	6873.13	0.21
A	27	12.1	2.19	6867.85	0.22
A	26	12.01	2.4	6276.6	0.24
A	25	11.96	2.47	6081.34	0.26
A	24	11.9	2.55	5890.85	0.27
A	23	11.77	2.84	5299.05	0.28
A	22	11.75	2.72	5536.32	0.28
A	21	11.63	2.93	5135.08	0.31
A	20	11.47	3.2	4706.82	0.35
A	19	11.33	3.39	4443.24	0.37
A	18	11.12	3.7	4064.66	0.41
A	17	10.77	4.24	3549.12	0.44
A	16	10.79	3.8	3962.12	0.41
A	15	10.78	3.5	4299.65	0.38
A	14	10.54	3.85	3909.13	0.42
A	13	10.47	3.75	4015.34	0.4
A	12	10.51	3.25	4624.88	0.35
A	11	10.49	3.01	5002.38	0.33
A	10	10.57	2.3	6540.08	0.29
A	9	10.57	2.02	7435.46	0.24
A	8	10.5	2.16	6955.34	0.25
A	7	10.46	2.13	7062.74	0.25
A	6	10.36	2.38	6329.94	0.29
A	5	10.28	2.46	6126.01	0.3
A	4	10.14	2.69	5584.85	0.33
A	3	9.83	3.34	4501.73	0.45
A	2	9.75	3.18	4734.7	0.4
A	1	9.76	2.75	5471.14	0.32

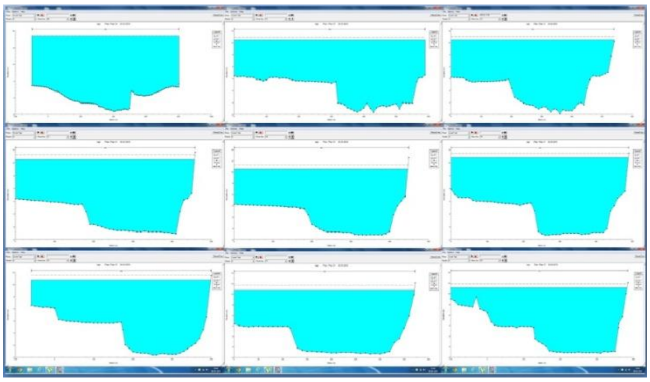

Figure 25: Water Surface Profile at cross section 9 to 17

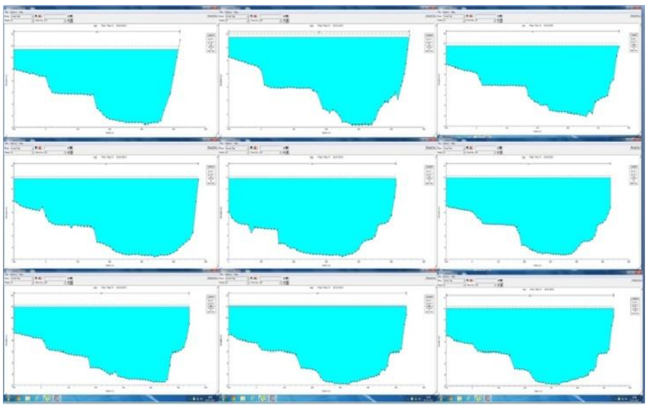

Figure 26: Water Surface Profile at cross section 18 to 26

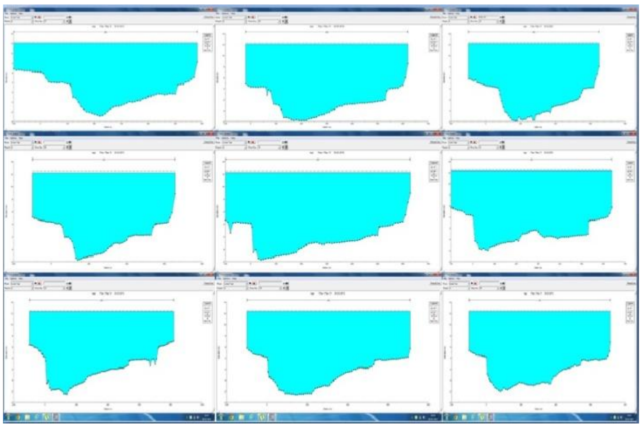

Figure 27: Water Surface Profile at cross section 27 to 35

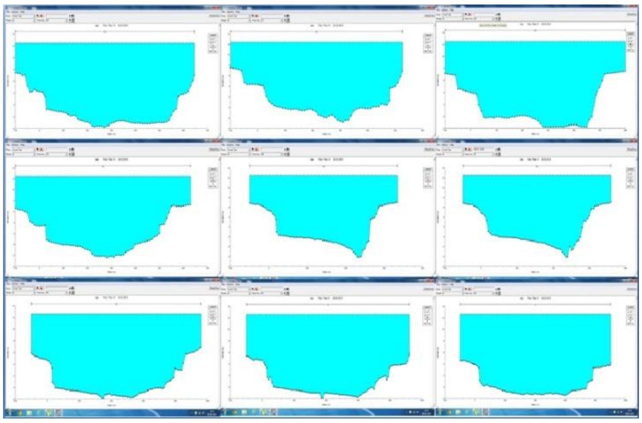

Figure 28: Water Surface Profile at cross section 36 to 44

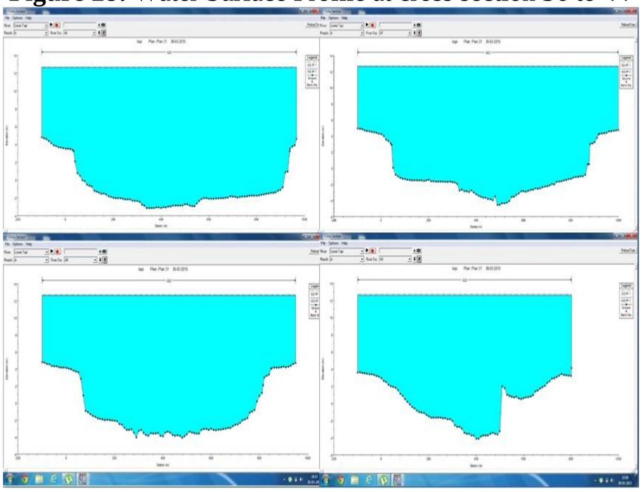

Figure 29: Water Surface Profile at cross section 46 to 49

6. Conclusions

From the present study following points can be summarized as outcome of the study.
In present study , Water Surface Level analysis has been done by using steady flow analysis in HEC-RAS; Twelve flood events has been considered in the present study.

5.1 For the flood event of 1883, the peak discharge was 28480cumecs and overflow occurred on one or both the banks of 41 cross-sections out of 49 cross sections.

5.2 The peak discharge of 1884 flood was 23974.5cumecs. For this flood overflow occurred on one or both the banks of 40 cross section out of 49 for the study reach under consideration.

5.3 The peak discharge of 1942 flood was 24371.2cumecs. For this flood overflow occurred on one or both the banks of 40 cross section out of 49 for the study reach under consideration.

5.4 The peak discharge of 2006 flood was 25788cumecs. For this flood overflow occurred on one or both the banks of 41 cross section out of 49 for the study reach under consideration.

5.6 The peak discharge of 2013 flood was 15050cumecs. For this flood overflow occurred on one or both the banks of 42 cross section out of 49 for the study reach under consideration.

7. Overall Conclusion

From the flood data analysis it has been concluded that various cross sections are critical and overflows during all flood events. As the discharge carrying capacity of Tapi river at Surat is approximately 8450 cumecs (3,00,000 cusecs), the flood of higher magnitude causes flooding in the various low lying areas of the city. As a measure of protection levees has to be constructed at these cross-sections so as to increase the capacity of river to carry this discharge safety. These proposed levees may prevent critical flood situations at Surat city.

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