CHAPTER 4

MODELLING OF WATER SURFACE PROFILE USING ARIIMA MODEL AND CRITICAL ANALYSIS OF SHORT TERM AND LONG TERM MEASURES TO MEETIGATE TAPI RIVER FLOODS

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MODELLING OF WATER SURFACE PROFILE USING ARIIMA MODEL AND CRITICAL ANALYSIS OF SHORT TERM AND LONG TERM MEASURES TO MITIGATE TAPI RIVER FLOODS

In reservoir operation problems, to achieve the best possible performance of the system, decisions need to be taken on releases and storage over a period of time considering the variations in inflows and demands. In the past, various researchers applied different kinds of mathematical programming techniques like linear programming, dynamic programming, nonlinear programming (NLP), etc., to solve such reservoir operation problems. An extensive review of these techniques can be found. But as far as reservoir operation is concerned, no standard algorithm is available, as each problem has its own individual physical and operational characteristics.

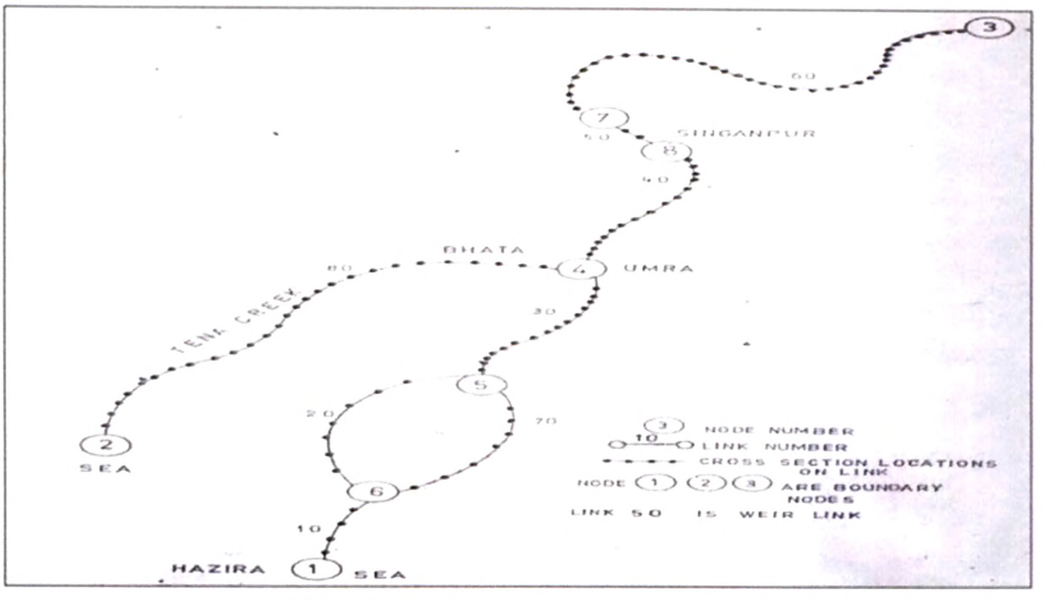
In the case of multipurpose reservoir operation, the goals are more complex than the single purpose reservoir operation and often involve various problems such as insufficient inflows and larger demands. In order to achieve the best possible performance of such a reservoir system, a model should be formulated as close to reality as possible. In this process, the model is expected to solve problems having nonlinearities in their domain. For example, a typical hydropower production function is complex, with nonlinear relationships in objectives and constraints. So the linear programming methods cannot be used. The dynamic programming approach faces the additional problem of dimensionality, whereas the nonlinear programming methods have the limitation of slow rate of convergence, requiring large amount of computational storage and time compared with other methods. Also, often NLP results in local optimal solutions.

In spite of development of many conventional techniques for optimization, each of these techniques has its own limitations. To overcome those limitations, recently met heuristic techniques are being used for optimization. By using these techniques, the given problem can be represented more realistically. These also provide case in handling the nonlinear and no convex relationships of the formulated model. Genetic Algorithms (GAs) and Particle Swarm Optimization (PSO) are some of the techniques in this category. These evolutionary algorithms search from a population of points, so there is a greater possibility to cover the whole space and reaching the global optimum.

Genetic Algorithm is one of the population-based search techniques, which works on the concept of "survival of the fittest". In the field of water resources, in earlier studies, few applications of the GA technique to derive reservoir operating policies have been reported and they illustrated the utility of evolutionary techniques for reservoir operation problems. Though GA has many advantages over conventional methods, it also has some drawbacks, such as slow rate of convergence and requiring a large number of simulations to arrive at an optimum solution.

4.1 ARIMA - 1D Mathematical Model

One dimensional mathematical model Auto Regression Integrated Mathematical Analysis (ARIMA) prepared by using MATLAB release version 2012 to determine Tapi river flood water surface profile under different flood frequency i.e. outflow ranging from 5668.93 to 22675.73 Cumecs (2 lacs Cusecs to 8 lacs Cusecs). This model is 1-D mathematical model for numerical simulation of unsteady water and sediment movement in multiply connected network of mobile bed channels. This model is capable of handling unsteady water and sediment flows in multiply connected channels highly non uniform sediment and grain sorting and armoring process. The model can simulate processes such as; sediment sorting, bed armoring, flow dependent friction factor and alternate drying and flooding of perched channels. The flow over the weir can also be handled. Continuity and Momentum equations are the Governing Equations for water flow. Model uses widely applied Pressiman 4 point weighted implicit finite difference scheme. For solution of governing equations terms in the equation are discritised in x-t plane and system of linearised simultaneous difference between equations is obtained i.e. Coefficient matrix is a banded matrix. ARIMA model uses Double sweep algorithm. The entire network of channel is schematized into links (Channel) and Nodes (junctions or any bifurcation points or end or beginning of channels) so that each link has one node at each end and each node has at least one link (Channel) starting from it or ending at it. Each link there are grid points where the cross sectional data given. The nodes could of internal and boundary nodes shown in below schematic diagram.



Schematic Diagram of Tapi Creek Channel Network

4.1.1 Input Data Requirements

Model needs following type of input data:

1. Topographic data i.e. Channel Cross Sections, layout & connectivity, Configuration of weirs.
2. Hydrologic Data i.e. Inflow hydrographs for upstream & downstream boundary condition, bed roughness.
3. Sediment Data i.e. Size, Properties, Distributions, Sediment inflow hydrographs by class.
4. Calibration & Verification Data i.e. Discharge hydrographs, sediment transport rates by size class, observed changes in bed levels and composition.

4.1.2 Solution of Water Flow Equations

- Formulation of set of linearised difference equations for each link.
- Carrying out forward and backward sweep in each channel and storing the coefficients
- Formulation of node matrix
- Solution of node matrix to obtained the water levels at each node
- Computing the water level and discharged at each grid point in each channel using the coefficients stored and water levels at nodes

During these computations the numerical parameter such as distance step (Δx) time step (Δt), time weighting coefficient (θ), and space weighting coefficient (ψ) are involved. In the present studies Δx was variable depending upon grid point spacing. Δt time step was adopted as 10 minutes. Value of θ and ψ were 0.55 or 0.50 respectively. The θ value of 0.55 was adequate to avoid the damping of flood or tidal wave.

4.1.3 Model Assumptions

St. Venant Hypothesis for water flow is assumed (i.e. uniform velocity and horizontal. Distribution, applicability of steady state resistance law for unsteady flow and small bed slopes).

- Channel network pattern assumed (i.e. total no. of channels, and their inter-connections) must remain same during a particular simulation.
- Cross sections are assumed to rise or fall without changing its shape.
- Continuous lateral flows not considered However, in additions due to rainfall could be represented by channel joining at regular interval.
- Other restrictions associated with sediment routing processes (i.e. those required for sorting, armoring sediment discharge, friction prediction etc).

4.1.4 Model Equations

Model uses St. Venant equations for water flow, equations for sediment continuity and provides alternatives sediment discharge and friction factor predictions. Generally for governing equations for channel geometry, hydraulic sorting and armoring of bed surface are given below separately. Governing equations are water continuity equation, momentum equation, sediment discharge predictor, friction factor prediction sediment continuity equation, channel geometry equation, hydraulic sorting of bed material, armoring of bed surface listed below from Equation No. 4.1 to 4.32.

Water Continuity Equation

$$\frac{\partial h}{\partial t} + \frac{\partial q}{\partial x} = 0 \qquad \text{.......... (4.1)}$$

De Saint Venant Equations

The one dimensional modeling of unsteady flow in open channels is most often performed by supplementing De Saint Venant equations that describe the propagation of a wave, flow depth, velocity flow depth and the rate of flow.

$$\frac{\partial M}{\partial t} = \frac{d}{dt} \int_{x_1}^{x_2} (\rho A + \rho A_2 V_2 - \rho A_1 V_1 - \rho q_1 (x_2 - x_1)) dx = 0 \qquad \text{......... (4.2)}$$

Momentum Equation for Water

$$\frac{\partial q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{q^2}{h}+\frac{1}{2}gh^2\right)+gh\frac{\partial z}{\partial x}+ghS_f=0 \qquad \text{.......... (4.3)}$$

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}(QV+gA\hbar)=gA((S_o-S_f)+V_x q_1 \qquad \text{.......... (4.4)}$$

$$\frac{\partial V}{\partial t}+\frac{\partial V}{\partial x}+g\frac{\partial V}{\partial x}-g\left(S_o-S_f\right)=0 \qquad \text{.......... (4.5)}$$

$$S_f=S_o \quad -\frac{\partial h}{\partial x} \quad -\frac{\partial}{\partial x}\left(\frac{V^2}{2g}\right) \quad -\frac{1}{g}\frac{\partial V}{\partial t} \qquad \text{.......... (4.6)}$$

Steady, uniform

Kinematic Wave Aprox.

Steady, nonuniform

Diffusion Wave Aprox.

Steady, nonuniform

Quasi-Steady Dynamic Wave Aprox.

Unsteady, nonuniform

Full Dynamic Wave Equation

Cross-wave pattern in a curved channel

$$\frac{B}{\tan\beta}=\left(r_e+\frac{B}{2}\right)\tan\theta_o \qquad \text{.......... (4.7)}$$

Easement Curves

$$\tan^{-1}\frac{B}{\left(2r_c+\frac{B}{2}\right)\tan\beta} \qquad \text{.......... (4.8)}$$

Banking of the Channel Bottom

$$\phi=\tan^{-1}\frac{V^2}{g r_c} \qquad \text{.......... (4.9)}$$

Sediment Discharge Predictor

F1 (Qs, D50, Q, A, d, Sf, ACF) = 0 .... (4.10)

Friction Factor Predictor:

F2 (A, A, d50, S f, D, ACF) = 0 .... (4.11)

Sediment Continuity Equation

$$\frac{\partial}{\partial t}\left[(1-p)z+\frac{q_s h}{q}\right]+\frac{\partial q_s}{\partial x}=0 \quad \text{.......... (4.12)}$$

$$(1-P)\frac{Bdz}{dt}+\frac{dQs}{dx}=0 \quad \text{.......... (4.13)}$$

Channel Geometry Equation

A = A (d, x) .... (4.14)

B = B (d, x) .... (4.15)

Hydraulic Sorting of Bed Material

$$D50^{n} \Longrightarrow D50^{n+1} \quad \text{.......... (4.16)}$$

Armoring or Bed Surface

$ACF^{N} = ACF^{N+1}$ .... (4.17)

Aggradations due to sediment overloading

$$\left[(1-p)z+\frac{q_s h}{g}\right]{}^{k+1}_{1}=\left[(1-p)z+\frac{q_s h}{q}\right]{}^{k}_{1}+\frac{\Delta t}{\Delta x}\left[(q_{so}+\Delta q_s)-(q_s)^{k}_{1}\right] \quad \text{.......... (4.18)}$$

Simple Standing Oscillation

$$T=\frac{21}{gh} \quad \text{.......... (4.19)}$$

Height at any time of any harmonic constituent

H.Cos (NT – g) .... (4.20)

Elevation for sea level resulting from a moving atmospheric pressure

$$\frac{13(29.8-P)}{1-\frac{k^2}{68}D} \quad \text{.......... (4.21)}$$

Predictor

In the following equations, a superscript *indicates value of the variable computed at the end of the predictor part.

$$h_i^* = h_i^k - \frac{\Delta t}{\Delta x}(q_{1+1}^k - q_i^k) \qquad \text{.......... (4.22)}$$

$$q_i^k - \frac{\Delta t}{\Delta x}\left[\frac{(q_{i+1}^k)^2}{h_{i+1}^k} - \frac{(q_i^k)^2}{h_i^k} + \frac{g}{2}\{(h_{i+1}^k)^2 - (h_i^k)^2\}\right] - gh_i^k \frac{\Delta t}{\Delta x}(z_{i+1}^k - z_i^k) - gh_i^k \Delta t\ (q_i^k n)^2/(h_i^k)^{3.33} \quad \text{... (4.23)}$$

$$z_i^* = z_i^k + \frac{1}{1-p}\left[(\frac{q_s h}{q})_i^k - (\frac{q_s h}{q})_i^*\right] - \frac{\Delta t}{(1-p)\Delta x}\left[(q_s)_{i+1}^k - (q_s)_i^k\right] \qquad \text{.......... (4.24)}$$

$$(q_s)_i^* = a(\frac{q_i^*}{h_i^*})^b \qquad \text{.......... (4.25)}$$

Corrector

$$h_i^{**} = h_i^* - \frac{\Delta t}{\Delta x}[q_i^* - q_{i-1}^*] \qquad \text{.......... (4.26)}$$

$$q_i^{**} = q_i^* - \frac{\Delta t}{\Delta x}\left[\frac{(q_i^*)^2}{h_i^*} - \frac{(q_{i-1}^*)^2}{h_{i-1}^*} + \frac{g}{2}\{(h_i^*)^2 - (h_{i-1}^*)^2\}\right] - gh_i^* \frac{\Delta t}{\Delta x}(z_i^* - z_{i-1}^*) - gh_i^* \Delta t\ \frac{(q_{in}^*)^2}{(h_i^*)^2} \quad \text{.... (4.27)}$$

$$z_i^{**} = z_i^* + \frac{1}{1-p}\left[(\frac{q_s h}{q})_i^* - (\frac{q_s h}{q})_i^{**}\right] - \frac{\Delta t}{(1-p)\Delta x}[(q_s)_i^* - (q_s)_{i-1}^*] \qquad \text{.......... (4.28)}$$

$$(q_s)_i^{**} = a\left(\frac{q_i^{**}}{h_i^{**}}\right) \qquad \text{.......... (4.29)}$$

$$h_i^{k+1} = \frac{1}{2}(h_i^k + h_i^{**}) \qquad \text{.......... (4.30)}$$

$$q_i^{k+1} = \frac{1}{2}(q_i^k + q_i^{**}) \qquad \text{.......... (4.31)}$$

$$z_i^{k+1} = \frac{1}{2}(z_i^k + z_i^{**}) \qquad \text{.......... (4.32)}$$

Where,

q = Water Discharge

A = Cross Sectional Area

Y = Water Surface Elevation

z = Bed Elevation

S_f = Energy Slope

B = Water Surface Width

d = Flow Depth

k = Conveyance

D_{50} = Medium Size of Bed Material

K = Speed in knots

α = Momentum Correction Factor

p = Porosity of Bed Material

h= the flow depth (L)

g= acceleration due to gravity (L/T^2)

r_c = radius of the circular curve

M = mass

β= Sin^{-1}

T= Time

u= the flow velocity (L/T)

S_o= bed slope

T= independent variable of time (T)

v= velocity

Δt = Change in Time

q_1 = volumetric rate of lateral inflow or outflow per unit length of the channel between sect 1 and 2.

x = independent variable representing the coordinate in the longitudinal direction (flow direction) (L)

Model Solution Procedure/Scheme

In general model follows Preissmann Implicit Scheme for discrediting the water and sediment flow and continuity equations. The solution procedure include water flood routing with forward and backward sweeps in each branch, formulation of node matrix solution of sediment continuity equation and then grain sorting and armoring. Analytical solutions of all these simultaneous equations are not possible due to following reasons: (a) Inherent non – linear equations (b) Tabular nature of equations for channel geometry (i.e. equations 4.14 & 4.15). (c) Adhoc procedure (as against mathematical relationships) for equations for bed material sorting and armoring of bed surface (equation 4.16 & 4.17). (d) Necessity to solve equation of sediment continuity for each size fraction followed by reconstitution of total change in bed elevation. Therefore, decoupled solution approach is adopted for solution of these equations as described below: The solution proceeds in three stages.

Stage 1 Equations for sediments discharge (equation 4.10), friction factor (equation 4.11), channel geometry (equation 4.14 & 4.15), and discredited equations of water flow are solved in a hydraulic sweep. During this sweep, bed elevation (z), medium diameter of sediment and armoring factor (AFC) are kept constant as if bed is frozen temporarily. Thus, during the sweep at grid point (I) water flow (QI), water level(y) and sediment transport capacity Qs (i, j) for each size fraction j of bed material are computed.

Stage 2 During this stage discritised equation of sediment continuity is solved in downstream sweep to yield new bed levels at grid point i.e. the sediment discharge Qs^{n+1} computed in stage1 is treated constant assuming that it is unaffected by evolution process (bed level change), armoring & grain sorting.

Stage 3 In this stage accounting procedure is executed using aggradations or degradation computed in stage 2 (i.e. sorting) bed material to compute new D50 and computation of $ACF^{n+1} - n$ (armoring factors).

The above procedure is called uncoupled as it is assumed that these processes (in above stages) occur sequentially (not concurrently) in a given time step. This simultaneous violation of all mechanisms involved becomes necessary due to practical difficulties associated with the lack of closed form representation of armoring and sorting processes. Such decoupled approach models are based on assumption that the change in any one variable during a time step is small enough to affect other variables during the time step. Required sequence of operations is as follows:

1 Load boundary conditions (water and sediment inflow and down steam water levels)

2 Compute water depth, friction slope, water surface width and water and sediment discharge at all grid points (through simultaneous solutions of equations 4.1, 4.3, 4.10, 4.11, 4.13, 4.14, 4.15) using latest values of z, D_{50}, ACF.

3 Using estimated sediment discharge (Qs) and water surface width (B) computed in 2*, a new estimate of bed surface elevation is obtained by solution.

4 Using changes in bed elevation in 3, new estimates of armoring factor and medium dam are computed using equations (4.10) and (4.17). Steps *2 to *4 are repeated till successive estimates of bed elevations (Z^{n+1}) no longer changes.

Supplementary Relations Used in ARIMA

The model uses following supplementary relations for simulating different processes:

(i) Total Sediment Load Prediction

Following sediment transport formula have been coded in CHARIMA:

(a) TLTM formula by Karim & Kennedy

(b) England & Hanson formula.

(c) Modified Ackers & White formula

(d) Power law predictor

(ii) Dune Height Predication

(a) Yalin's relation

(b) Allen's relation

(iii) Hydraulic Sorting of Bed Material

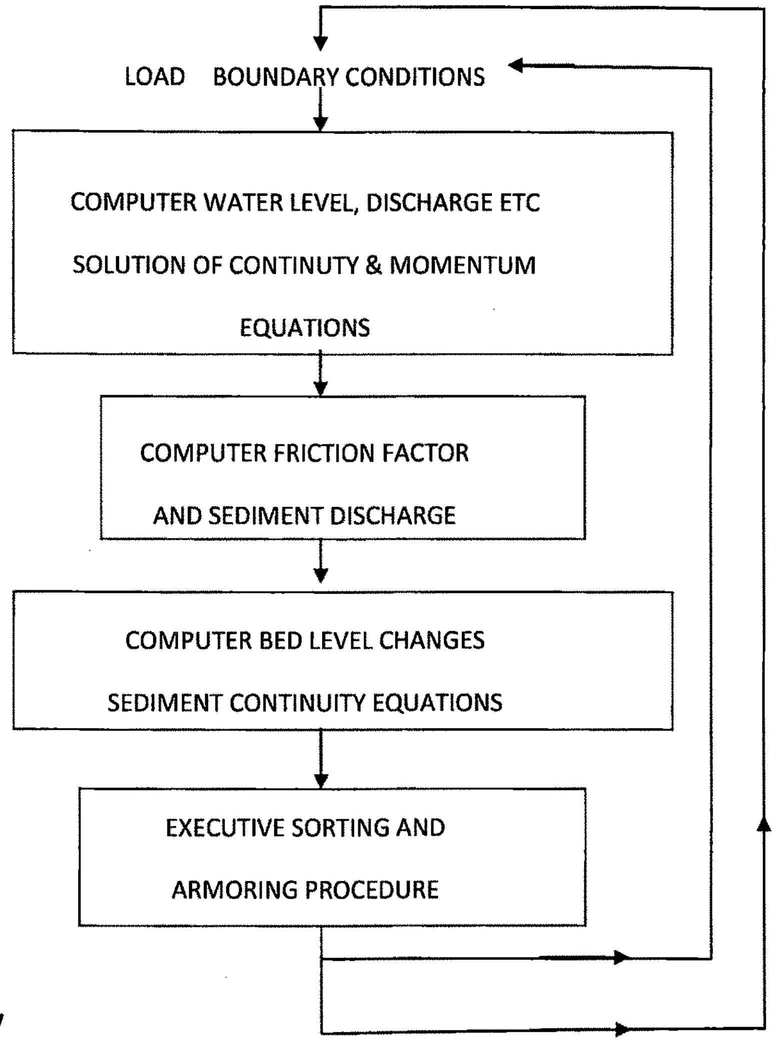
(iv) Changes in Bed Material Composition

(v) Armoring of Bed Surface / Armoring Factor

(vi) Effect of Bed Forms on Armoring

(vii) Effect of Armoring on Sediment Discharge and Mixed Layer Thickness

4.2 Flow Chart for Model Programming



4.2.1 Methodology

These studies were taken up with the help of mathematical model capable of handling unsteady flow in river channel network comprising junctions, bifurcations and loops. The total rivers reach of about 66 km from mouth at Hazira to about 15 km upstream of Kathor Bridge was simulated along with existing flood embankments from Surat to Kathor. The Tena creek from mouth its junction with Tapi River near Bhata village was also reproduced in the node. The tidal water level at Outer Hazira was used as downstream boundary condition. The model equations numbered from 1 to 32 and flow chart program shown below. This model covers conditions such as effect of river flow at meandering shape, armoring of sediments, Tapi river carrying capacity and existing flood protection embankments etc. First the model the results of analysis shown in combine graph which shows submergence level of different locations at Surat city under varying flood scenario.

4.3 Tapi River Water Surface Profile at Different location of Surat City

The predicted flood water surface profile compared with actual flood observed flood Aug 2006 for validation of model, same shown in Table.No.4.1 and 4.2. By using flow chart for model programming, Tapi river flood water surface profile calculated for important locations (Magdalla Bridge, Umra/ Bhata, Nehru Bridge, Singanpur Weir, Varivay , Amroli Bridge, Kathor Bridge) in Surat city for different flood frequency and scenario used to forecast flood submergence shown in Graph.No.4.1 to 4.6.

Table.4.1 Predicted Water Levels along Tapi River for different discharges at Spring Tide under Existing Condition

Sl No	C/S No.	Ch km	Thwg m	Bride	Left Bank	Right Bank	Discharge in m^3/s (Cusecs)										Remark
							5669 m^3/s (2 Lac)		8503 m^3/s (3 lac)		14172 m^3/s (5 lac)		19076 m^3/s (6.73 lac)		22676 m^3/s (8 lac)		
							WL m	Vel m/s	WL m	Vel m/s	WL m	Vel m/s	WL m	Vel m/s	WL m	Vel m/s	
1	1	0.00	-4.75				5.26	1.60	5.26	-0.51	5.26	-0.39	5.26	-0.26	5.26	-0.09	
2	2	1.00	-6.50				5.24	1.60	5.24	-0.63	5.25	-0.46	BRAS	-0.28	5.25	-0.02	
3	3	2.00	-5.00				5.23	1.59	5.24	-0.48	5.24	-0.33	5.26	-0.16	5.23	0.07	
4	4	2.99	-5.00				5.22	1.59	5.23	-0.42	5.24	-0.24	5.25	-0.07	5.23	0.18	
5	5	3.99	-6.50				5.22	1.59	5.23	-0.42	5.24	-0.18	5.25	0.02	5.23	0.29	
6	6	5.01	-3.96				5.18	1.58	5.20	-0.50	5.22	-0.13	5.24	0.17	5.18	0.57	
7	7	6.01	-5.43				5.14	1.57	5.19	-0.44	5.22	-0.03	5.24	0.31	5.21	0.73	
8	8	7.00	-4.82				5.11	1.56	5.17	-0.39	5.21	0.10	5.25	0.50	5.18	1.00	
9	9	8.00	-5.43				5.09	1.55	5.15	-0.09	5.21	0.15	5.26	0.35	5.27	0.54	
10	10	9.00	-5.00				5.08	1.55	5.14	0.07	5.19	0.50	5.25	0.86	5.27	1.16	
11	11	9.85	-5.00				5.05	1.54	5.12	0.31	5.18	1.10	5.22	1.74	5.26	2.31	
12	12	11.00	-6.00				5.05	1.54	5.12	0.34	5.18	1.11	5.22	1.76	5.26	2.43	
13	13	11.99	-6.50				5.05	1.54	5.15	0.55	5.35	1.39	5.53	2.00	5.82	2.44	
14	14	13.01	-8.60				5.02	1.53	5.11	1.05	5.59	2.09	5.90	2.85	6.34	3.43	
15	15	13.51	-5.50	-5.50			5.02	1.53	5.12	1.00	5.80	1.86	6.26	2.45	6.85	2.90	Magdalla Port
16	16	14.01	-3.60				5.00	1.52	5.13	1.03	5.89	1.86	6.47	2.33	7.13	2.75	
17	17	14.51	-3.00				4.99	1.52	5.17	0.97	6.03	1.62	6.73	1.91	7.47	2.19	
18	Magdalla	15.01	-2.90	-2.90	4.91	6.24	4.97	1.51	5.16	1.29	6.03	2.24	6.70	2.72	7.35	3.32	Magdalla Bridge
19	45	15.39	-2.51		6.89	3.93	4.97	1.51	5.21	1.49	6.11	2.44	6.83	2.86	7.54	3.42	
20	44	15.73	-2.75		6.39	3.86	4.99	1.52	5.29	1.24	6.32	1.98	7.11	2.32	7.91	2.76	
21	43	16.51	-1.36		6.98	6.59	5.00	1.52	5.30	1.54	6.44	2.15	7.27	2.30	8.13	2.58	
22	42	16.49	-3.00		6.98	6.62	5.03	1.53	5.39	1.41	6.66	1.83	7.50	1.91	8.38	2.10	
23	41	16.76	-2.81		7.80	6.52	5.06	1.54	5.46	1.48	6.78	1.80	7.61	1.84	8.49	2.00	
24	40	17.12	-3.36		7.24	6.61	5.09	1.55	5.54	1.42	6.89	1.89	7.73	1.92	8.59	2.10	
25	39	17.37	-4.97		7.29	5.79	5.10	1.56	5.59	1.42	7.02	1.63	7.87	1.66	8.72	1.85	
26	38	17.61	-5.14		7.82	5.30	5.12	1.56	5.65	1.43	7.12	1.49	7.95	1.53	8.80	1.71	
27	37	18.05	-4.74		7.61	3.93	5.18	1.58	5.78	1.12	7.25	1.24	8.06	1.31	8.90	1.48	
28	36	18.34	-4.50		4.92	4.02	5.21	1.59	5.84	1.04	7.30	1.16	8.11	1.23	8.95	1.40	
29	35	18.57	-4.98		5.11	4.30	5.23	1.59	5.86	1.06	7.30	1.39	8.09	1.57	8.89	1.86	
30	34	18.82	-5.10		8.33	4.17	5.24	1.60	5.89	1.18	7.35	1.34	8.15	1.44	8.97	1.63	
31	33	19.26	-3.64		6.26	4.74	5.25	1.60	5.92	1.47	7.32	1.97	8.08	2.25	8.81	2.71	Crematorium
32	32	19.50	-2.96		8.76	4.70	5.22	1.59	5.87	2.03	7.18	2.81	7.87	3.28	8.45	4.04	

Table.4.1 Predicted Water Levels along Tapi River for different discharges at Spring Tide under Existing Condition (Continue)

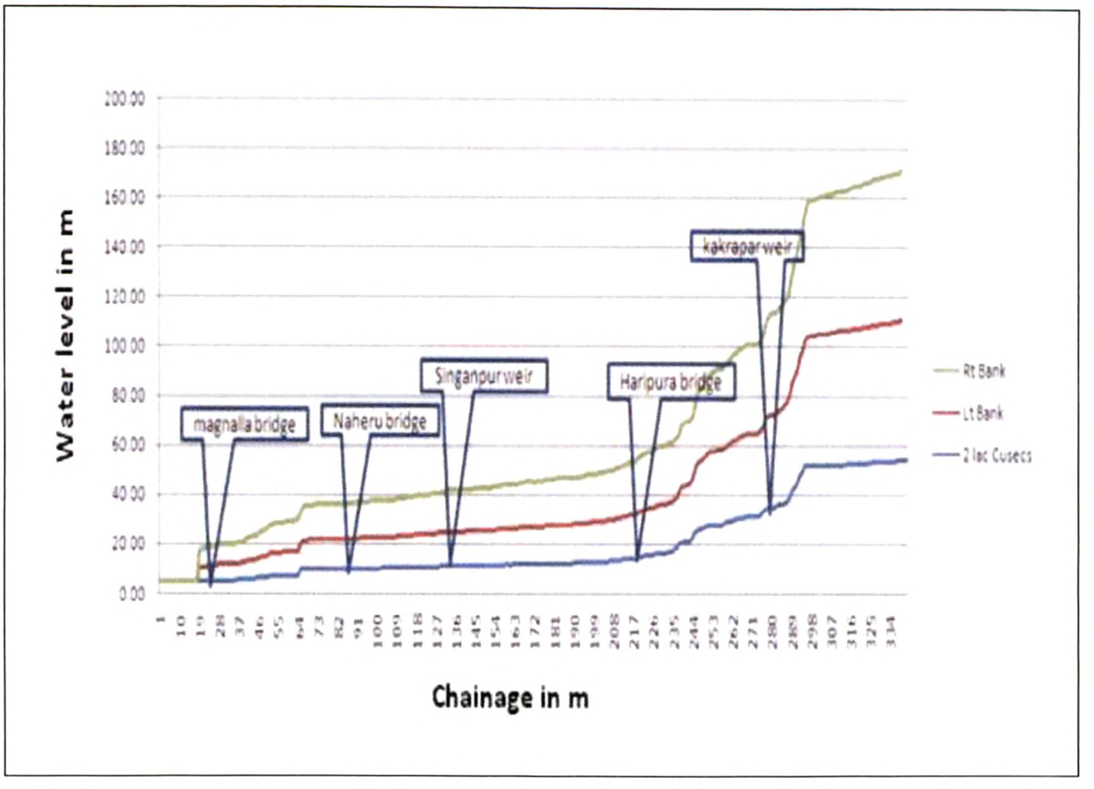
33	31	19.77	-3.04		8.81	4.19	5.22	1.59	5.87	2.08	7.18	2.81	7.87	3.31	8.45	3.77	
34	30	20.04	-2.78		8.50	4.50	5.27	1.61	5.96	2.17	7.31	2.95	8.02	3.47	8.61	3.96	
35	29	20.26	-2.84	-2.84	8.77	4.73	5.32	1.62	6.06	2.13	7.46	2.90	8.20	3.40	8.83	3.89	Satkeval Temple
36	28	20.53	-2.80		8.08	4.80	5.33	1.62	6.08	2.62	7.46	3.49	8.17	4.09	8.76	4.66	
37	27	20.84	-2.80		8.51	5.64	5.45	1.66	6.27	2.68	7.69	3.62	8.45	4.24	9.08	4.82	
38	26	21.06	-2.90		9.18	5.70	5.53	1.69	6.43	2.56	7.96	3.38	8.80	3.92	9.52	4.43	
39	25	21.31	-3.96	-3.96	9.26	5.61	5.63	1.72	6.60	2.45	8.19	3.23	9.08	3.75	9.86	4.22	Ambaji Temple
40	24	21.53	-2.65		9.84	5.04	5.71	1.74	6.73	2.39	8.36	3.15	9.29	3.64	10.10	4.09	
41	23	21.83	-3.06		9.82	7.18	5.78	1.76	6.82	2.63	8.48	3.38	9.43	3.87	10.26	4.32	
42	22	22.00	-4.12		7.06	6.61	5.86	1.79	6.94	2.58	8.62	3.33	9.58	3.82	10.42	4.27	
43	21	22.14	-4.30		9.77	7.63	5.90	1.80	7.01	2.65	8.70	3.41	9.67	3.91	10.52	4.36	
44	20	22.46	-5.16		9.35	6.11	5.99	1.82	7.12	2.88	8.76	3.91	9.66	4.60	10.42	5.24	
45	19	22.80	-5.20		9.52	6.74	6.15	1.87	7.39	2.50	9.23	3.28	10.29	3.79	11.22	4.25	
46	18	23.10	-6.30		10.06	7.14	6.19	1.89	7.46	2.79	9.30	3.63	10.36	4.18	11.28	4.68	Sardar Bridge
47	17	23.37	-6.48		5.70	7.26	6.32	1.93	7.67	2.46	9.60	3.26	10.70	3.80	11.67	4.28	
48	16	23.66	-5.36	-5.36	4.89	8.83	6.39	1.95	7.77	2.57	9.74	3.30	10.89	3.79	11.90	4.22	Nanpura Jetty
49	15	23.90	-1.89		5.98	5.82	6.45	1.97	7.84	2.80	9.82	3.49	10.98	3.95	12.00	4.36	
50	14	24.20	-1.89		8.22	3.97	6.72	2.05	8.13	2.32	10.19	2.91	11.41	3.30	12.50	3.64	Swami Vivekanand Bridge
51	13	24.44	-1.99	-1.99	8.10	4.57	6.84	2.08	8.26	2.12	10.35	2.69	11.60	3.06	12.71	3.39	Hope Bridge
52	12	24.50	-1.80	-1.80	6.61	8.36	6.89	2.10	8.34	1.83	1.48	2.28	11.76	2.58	12.91	2.83	Nehru Bridge
53	11	24.67	-1.46		7.06	5.22	6.95	2.12	8.40	1.75	10.54	2.21	11.83	2.51	12.98	2.77	
54	10	24.88	-1.48		7.18	8.80	7.01	2.14	8.47	1.54	10.64	1.93	11.96	2.17	13.13	2.39	
55	9	25.09	-1.55		13.73	8.88	7.05	2.15	8.51	1.55	10.69	1.90	12.01	2.13	13.19	2.33	
56	8	25.25	-1.19		13.71	8.25	7.07	2.16	8.54	1.57	10.72	1.93	12.04	2.16	13.22	2.35	
57	7	25.66	-1.34		13.71	8.43	7.16	2.18	8.62	1.39	10.80	1.76	12.13	1.99	13.32	2.19	
58	6	25.93	-1.52		13.29	13.60	7.20	2.19	8.67	1.37	10.86	1.74	12.19	1.98	13.38	2.19	
59	5	26.11	-1.74		13.57	13.60	7.20	2.19	8.66	1.60	10.84	2.00	12.17	2.25	13.36	2.45	
60	4	26.29	-1.61		13.44	13.62	7.21	2.20	8.68	1.77	10.84	2.21	12.16	2.48	13.34	2.70	
61	3	26.59	-1.16		13.76	13.62	7.27	2.22	8.72	1.88	10.89	2.35	12.21	2.63	13.40	2.84	
62	2	26.88	-1.42		13.77	14.34	7.37	2.25	8.84	1.51	11.05	1.93	12.40	2.19	13.59	2.40	
63	1	27.12	-1.26		14.04	13.64	7.38	2.25	8.85	1.72	11.05	2.17	12.39	2.43	13.58	2.65	
64	S Weir	27.38	6.00	6.00	6.00	6.00	7.38	2.25	8.85	0.00	11.05	0.00	12.39	0.00	13.58	0.00	S Weir
65		27.40					9.84	3.00	11.04	0.00	12.30	0.00	13.72	0.00	15.00	2.88	
66	6	27.41	-0.34	-0.34	14.10	13.76	9.84	3.00	11.04	1.63	12.30	2.34	13.72	2.64	15.00	2.88	SMC Office
67	7	27.59	-1.84		13.79	10.07	9.88	3.01	11.10	1.37	12.41	2.01	13.84	2.32	15.13	2.57	
68	8	27.87	-2.24		14.12	14.99	9.89	3.01	11.11	1.47	12.43	2.12	13.87	2.42	15.17	2.65	HT Tower
69	9	28.16	-3.29		12.07	11.02	9.92	3.02	11.15	1.42	12.50	2.05	13.95	2.33	15.25	2.56	HT Tower
70	10	28.37	-3.94		13.07	12.94	9.94	3.03	11.18	1.34	12.56	1.94	14.02	2.21	15.33	2.43	
71	11	28.53	-4.04		14.32	10.77	9.96	3.04	11.21	1.17	12.62	1.69	14.09	1.95	15.42	2.15	
72	12	28.70	-2.34		14.40	13.94	9.97	3.04	11.23	1.12	12.66	1.59	14.15	1.79	15.48	1.94	
73	13	28.88	-5.04		16.04	11.22	9.99	3.05	11.26	0.89	12.72	1.29	14.22	1.48	15.56	1.63	
74	14	29.06	-4.10		14.78	13.90	10.00	3.05	11.27	0.83	12.74	1.20	14.24	1.38	15.59	1.52	HT Tower
75	15	29.24	-6.40	-6.40	14.33	14.21	10.00	3.05	11.27	0.93	12.74	1.31	14.24	1.48	15.59	1.62	Jahangirpura Intake well
76	16	29.40	-5.70		14.61	14.06	10.00	3.05	11.28	0.94	12.75	1.33	14.25	1.50	15.60	1.62	
77	17	29.55	-5.40		14.86	12.11	10.01	3.05	11.29	0.84	12.78	1.19	14.28	1.36	15.63	1.49	Dabholi Gamtal
78	18	29.75	-4.40		15.53	14.24	10.02	3.05	11.30	0.87	12.79	1.23	14.29	1.40	15.64	1.53	
79	19	29.94	-4.40		14.22	9.58	10.03	3.06	11.31	0.83	12.80	1.19	14.31	1.37	15.66	1.50	
80	20	30.13	-2.40		14.73	12.48	10.02	3.05	11.30	1.01	12.79	1.41	14.29	1.58	15.64	1.71	

Table.4.2 Predicted Water Levels along Tapi River for different discharges at Spring Tide under Existing Condition

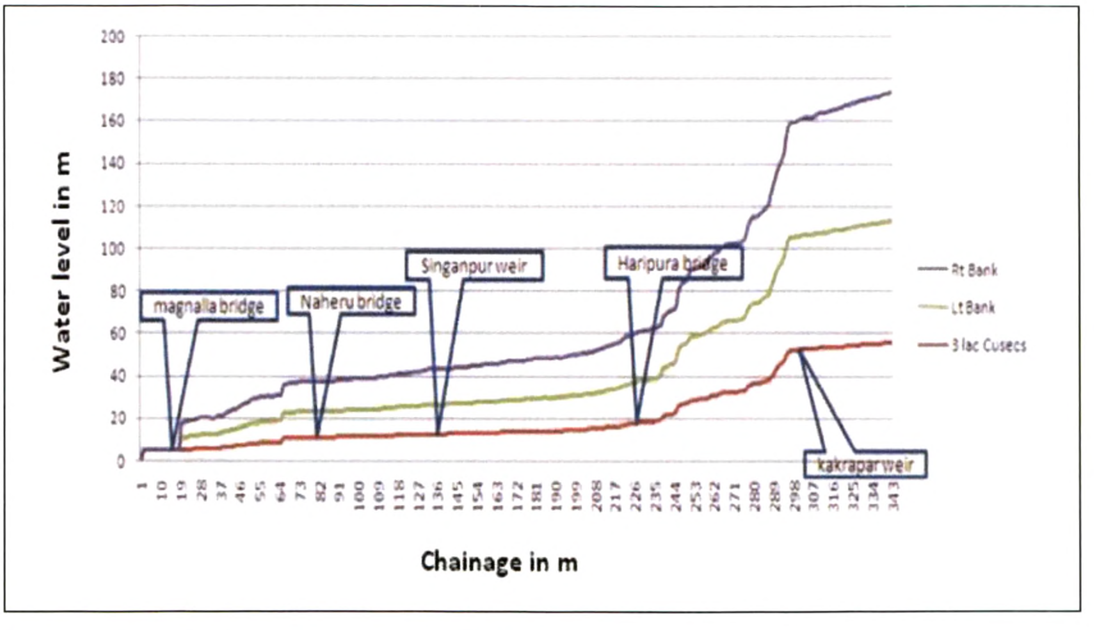
Sl No	C/S No.	Ch km	Thwg m	Bride	Left Bank	Right Bank	Discharge in m^3/s (Cusecs)										Remark
							25510 m^3/s (9 Lac) SW (0.9& 0.7)		28345 m^3/s (10 lac) SW (0.9& 0.7)		34014 m^3/s (12 lac) SW (0.9& 0.7)		39683 m^3/s (14 lac) SW (0.9& 0.7)		45351 m^3/s (16 lac) SW (0.9& 0.7)		
							WL m	Vel m/s	WL m	Vel m/s	WL m	Vel m/s	WL m	Vel m/s	WL m	Vel m/s	
1	1	0.00	-4.75				5.26	0.01	5.26	0.14	5.26	0.28	5.26	0.50	5.26	0.60	
2	2	1.00	-6.50				5.24	0.10	5.25	0.30	5.24	0.50	5.25	0.79	5.23	0.98	
3	3	2.00	-5.00				5.24	0.18	5.23	0.35	5.23	0.52	5.25	0.81	5.26	0.92	
4	4	2.99	-5.00				5.22	0.30	5.23	0.48	5.22	0.69	5.25	0.94	5.26	1.15	
5	5	3.99	-6.50				5.22	0.42	5.23	0.62	5.22	0.84	5.25	1.12	5.26	1.36	
6	6	5.01	-3.96				5.21	0.75	5.19	1.08	5.21	1.35	5.29	1.82	5.26	2.09	
7	7	6.01	-5.43				5.22	0.96	5.28	1.25	5.35	1.58	5.43	2.04	5.65	2.21	
8	8	7.00	-4.82				5.26	1.20	5.35	1.55	5.43	1.89	5.75	2.16	5.82	2.58	
9	9	8.00	-5.43				5.37	0.63	5.55	0.76	5.79	0.84	6.11	0.99	6,46	1.06	
10	10	9.00	-5.00				5.37	1.30	5.58	1.48	5.78	1.66	6.16	1.73	6.44	2.02	
11	11	9.85	-5.00				5.38	2.52	5.54	2.93	5.76	3.21	6.05	3.46	6.31	4.08	
12	12	11.00	-6.00				5.38	2.68	5.54	3.07	5.76	3.38	6.05	3.71	6.31	4.26	
13	13	11.99	-6.50				6.00	2.58	6.33	2.75	6.65	2.90	7.03	3.03	7.57	3.25	
14	14	13.01	-8.60				6.52	3.62	6.81	3.98	7.09	4.33	7.40	4.65	7.82	5.27	
15	15	13.51	-5.50	-5.50			7.06	3.05	7.45	3.33	7.82	3.60	8.22	3.85	8.83	4.31	Magdallaport
16	16	14.01	-3.60				7.36	2.87	7.79	3.10	8.21	3.31	8.65	3.51	9.35	3.88	
17	17	14.51	-3.00				7.71	2.25	8.19	2.40	8.66	2.55	9.13	2.67	9.92	2.94	
18	Magdalla	15.01	-2.90	-2.90	4.91	6.24	7.57	3.43	7.98	3.77	8.37	4.11	8.75	4.40	9.35	5.07	Magdalla Bridge
19	45	15.39	-2.51		6.89	3.93	7.76	3.50	8.21	3.80	8.63	4.09	9.05	4.34	9.75	4.92	
20	44	15.73	-2.75		6.39	3.86	8.14	2.83	8.64	3.07	9.11	3.31	9.58	3.49	10.41	3.95	
21	43	16.15	-1.36		6.98	6.59	8.37	2.59	8.91	2.73	9.43	2.87	9.93	2.95	10.86	3.24	
22	42	16.49	-3.00		6.98	6.615	8.62	2.10	9.16	2.19	9.70	2.29	10.21	2.33	11.19	2.56	
23	41	16.76	-2.81		7.795	6.52	8.73	1.98	9.27	2.05	9.81	2.12	10.32	2.14	11.31	2.35	
24	40	17.12	-3.36		7.24	6.61	8.82	2.07	9.3609	2.1458	9.89	2.23	10.39	2.23	11.37	2.47	
25	39	17.37	-4.97		7.29	5.79	8.95	1.83	9.4807	1.905	10.01	1.98	10.50	2.00	11.49	2.23	
26	38	17.61	-5.14		7.82	5.295	9.02	1.69	9.56	1.76	10.08	1.84	10.58	1.86	11.57	2.08	
27	37	18.05	-4.74		7.61	3.93	9.13	1.48	9.66	1.55	10.19	1.63	10.68	1.66	11.68	1.88	
28	36	18.34	-4.50		4.92	4.02	9.18	1.40	9.71	1.47	10.23	1.55	10.74	1.58	11.73	1.80	
29	35	18.57	-4.98		5.105	4.295	9.13	1.87	9.64	2.02	10.15	2.17	10.65	2.26	11.58	2.64	
30	34	18.82	-5.10		8.325	4.17	9.21	1.63	9.74	1.72	10.26	1.81	10.79	1.87	11.75	2.12	
31	33	19.26	-3.64		6.26	4.74	9.06	2.73	9.54	2.96	10.01	3.20	10.55	3.40	11.30	3.99	Crematorium
32	32	19.50	-2.96		8.76	4.7	8.70	4.07	9.09	4.47	9.44	4.89	9.99	5.23	10.20	6.44	
33	31	19.77	-3.04		8.805	4.19	8.70	4.11	9.09	4.57	9.44	5.01	9.99	5.49	10.20	6.14	
34	30	20.04	-2.78		8.495	4.5	8.89	4.30	9.31	4.77	9.69	5.23	10.19	5.80	10.52	6.38	
35	29	20.26	-2.84	-2.84	8.77	4.73	9.15	4.21	9.61	4.65	10.04	5.09	10.61	5.61	10.97	6.27	Satkeval Temple
36	28	20.53	-2.80		8.075	4.08	9.06	5.05	9.47	5.59	9.84	6.13	10.30	6.81	10.65	7.48	

Table.4.2 Predicted Water Levels along Tapi River for different discharges at Spring Tide under Existing Condition (Continue)

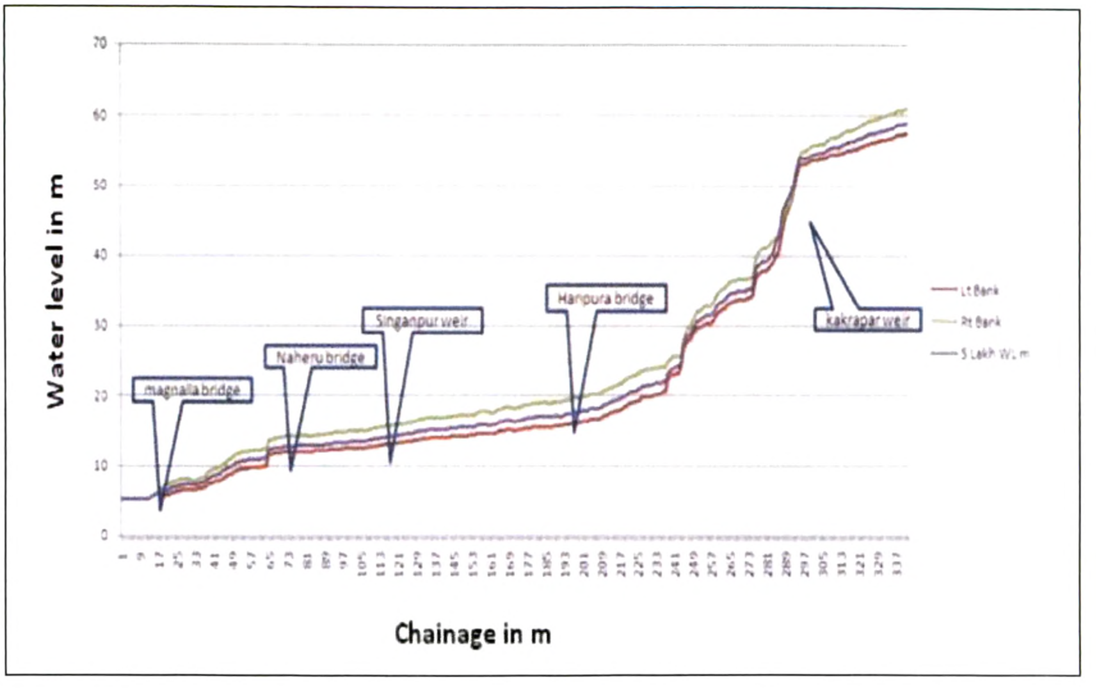
37	27	20.84	-2.80		8.51	5.64	9.41	5.21	9.88	5.74	10.31	6.27	10.87	6.92	11.35	7.53	
38	26	21.06	-2.90		9.18	5.7	9.93	4.76	10.50	5.20	11.06	5.63	11.80	6.12	12.45	6.59	
39	25	21.31	-3.96	-3.96	9.26	5.61	10.30	4.52	10.93	4.93	11.54	5.33	12.35	5.77	13.06	6.21	Ambaji Temple
40	24	21.53	-2.65		9.94	5.04	10.57	4.38	11.23	4.77	11.88	5.14	12.73	5.56	13.48	5.98	
41	23	21.83	-3.06		9.82	7.18	10.73	4.60	11.41	4.99	12.07	5.36	12.94	5.77	13.70	6.19	
42	22	22.00	-4.12		7.06	6.61	10.91	4.56	11.60	4.95	12.27	5.32	13.14	5.74	13.90	6.17	
43	21	22.14	-4.30		9.765	7.63	11.01	4.65	11.71	5.04	12.38	5.41	13.26	5.83	14.03	6.26	
44	20	22.46	-5.16		9.345	6.11	10.84	5.65	11.42	6.24	11.95	6.82	12.60	7.53	13.07	8.29	
45	19	22.80	-5.20		9.515	6.74	11.77	4.53	12.55	4.93	13.32	5.30	14.28	5,73	15.18	6.14	
46	18	23.10	-6.30		`10.06	7.135	11.82	4.99	12.59	5.42	13.33	5.82	14.26	6.30	15.13	6.76	
47	17	23.37	-6.48		5.7	7.26	12.25	4.58	13.06	4.99	13.85	5.39	14.83	5.85	15.75	6.30	
48	16	23.66	-5.36	-5.36	4.89	8.825	12.50	4.49	13.36	4.86	14.19	5.21	15.22	5.62	16.20	6.01	Nanpura Jetty
49	15	23.90	-1.89		5.975	5.82	12.62	4.62	13.49	4.98	14.33	5.31	15.39	5.70	16.38	6.07	
50	14	24.20	-1.89		8.22	3.97	13.17	3.86	14.10	4.15	15.01	4.42	16.14	4.74	17.22	5.04	
51	13	24.44	-1.99	-1.99	8.095	4.565	13.39	3.59	14.34	3.87	15.26	4.13	16.42	4.44	17.51	4.73	Hope Bridge
52	12	24.50	-1.80	-1.80	6.61	8.36	13.61	3.00	14.60	3.21	15.56	3.42	16.77	3.66	17.91	3.88	Nehru Bridge
53	11	24.67	-1.46		7.06	5.22	13.69	2.93	14.68	3.15	15.64	3.36	16.85	3.60	17.99	3.83	
54	10	24.88	-1.48		7.18	8.8	13.85	2.53	14.86	2.71	15.85	2.88	17.08	3.08	18.25	3.27	
55	9	25.09	-1.55		13.725	8.875	13.91	2.46	14.93	2.62	15.92	2.78	17.16	2.97	18.35	3.15	
56	8	25.25	-1.19		13.71	8.25	13.94	2.47	14.96	2.64	15.96	2.79	17.20	2.97	18.38	3.15	
57	7	25.56	-1.34		13.705	8.43	14.04	2.32	15.06	2.48	16.06	2.64	17.31	2.83	18.49	3.00	
58	6	25.93	-1.52		13.285	13.6	14.11	2.32	15.13	2.49	16.13	2.65	17.37	2.85	18.55	3.03	
59	5	26.11	-1.74		13.565	13.6	14.08	2.57	15.10	2.74	16.10	2.90	17.34	3.09	18.52	3.26	
60	4	26.29	-1.61		13.435	13.62	14.05	2.84	15.07	3.03	16.05	3.21	17.28	3.42	18.45	3.61	
61	3	26.59	-1.16		13.76	13.62	14.12	2.97	15.13	3.15	16.12	3.32	17.35	3.52	18.52	3.70	
62	2	26.88	-1.42		13.765	14.34	14.32	2.55	15.34	2.73	16.33	2.91	17.57	3.12	18.74	3.31	
63	1	27.12	-1.26		14.04	13.64	14.31	2.79	15.32	2.98	16.31	3.16	17.54	3.38	18.71	3.58	
64	S Weir	27.38	6.00	6.00	6	6	14.31	0.00	15.32	0.00	16.31	0.00	17.54	0.00	18.71	0.00	S Weir
65		27.40					15.80	0.00	16.93	0.00	18.03	0.00	19.41	0.00	20.73	0.00	
66	6	27.41	-0.34	-0.34	14.095	13.76	15.80	3.03	16.93	3.23	18.03	3.42	19.41	3.65	20.73	3.85	SMC Office
67	7	27.59	-1.84		13.79	10.07	15.93	2.73	17.07	2.95	18.17	3.15	19.55	3.40	20.87	3.61	
68	8	27.87	-2.24		14.115	14.99	15.98	2.79	17.12	2.99	18.23	3.17	19.62	3.39	20.95	3.58	HT Tower
69	9	28.16	-3.29		12.065	11.02	16.07	2.70	17.21	2.89	18.33	3.07	19.72	3.29	21.05	3.48	HT Tower
70	10	28.37	-3.29		13.065	12.94	16.14	2.57	17.29	2.76	18.41	2.94	19.82	3.15	21.15	3.34	
71	11	28.53	-4.04		14.32	10.765	16.24	2.28	17.40	2.46	18.53	2.62	19.94	2.82	21.29	3.00	
72	12	28.70	-2.34		14.4	13.935	16.32	2.03	17.50	2.16	18.64	2.28	20.08	2.42	21.44	2.55	
73	13	28.88	-5.04		16.04	11.22	16.40	1.73	17.58	1.86	18.73	1.98	20.17	2.12	21.54	2.24	
74	14	29.06	-4.10		14.775	13.9	16.43	1.61	17.61	1.73	18.77	1.85	20.21	1.98	21.58	2.11	HT Tower
75	15	29.24	-6.40	-6.40	14.325	14.21	16.43	1.71	17.61	1.82	18.76	1.93	20.21	2.06	21.58	2.18	Jahangirpura Intake Well
76	16	29.40	-5.70		14.61	14.055	16.44	1.70	17.63	1.80	18.79	1.90	20.23	2.01	21.61	2.12	



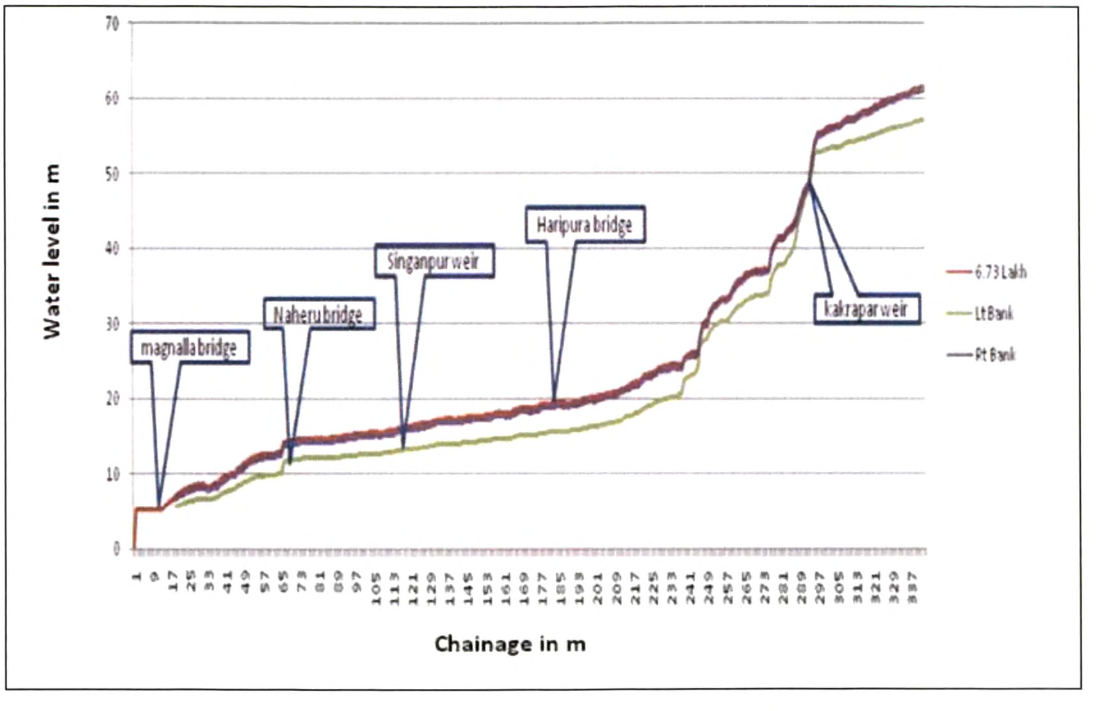
Graph.4.1 Water Surface profile of Tapi River at Surat 5669 m^3/s (2 lac Cusecs)



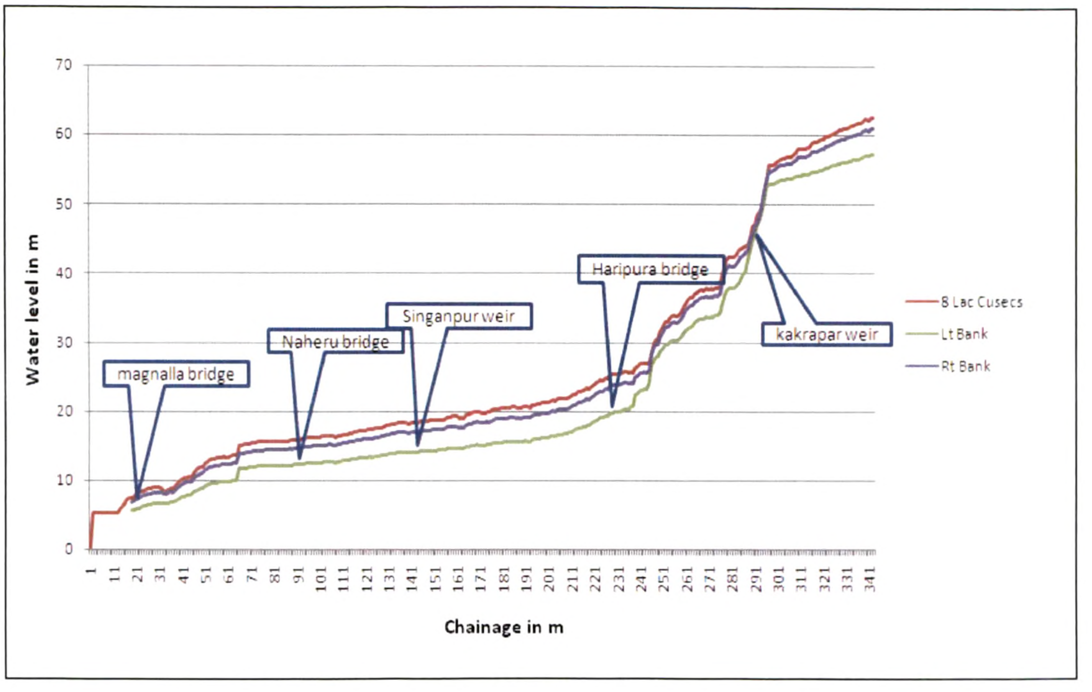
Graph.4.2 Water Surface profile of Tapi River at Surat 8503 m^3/s (3 lac Cusecs)



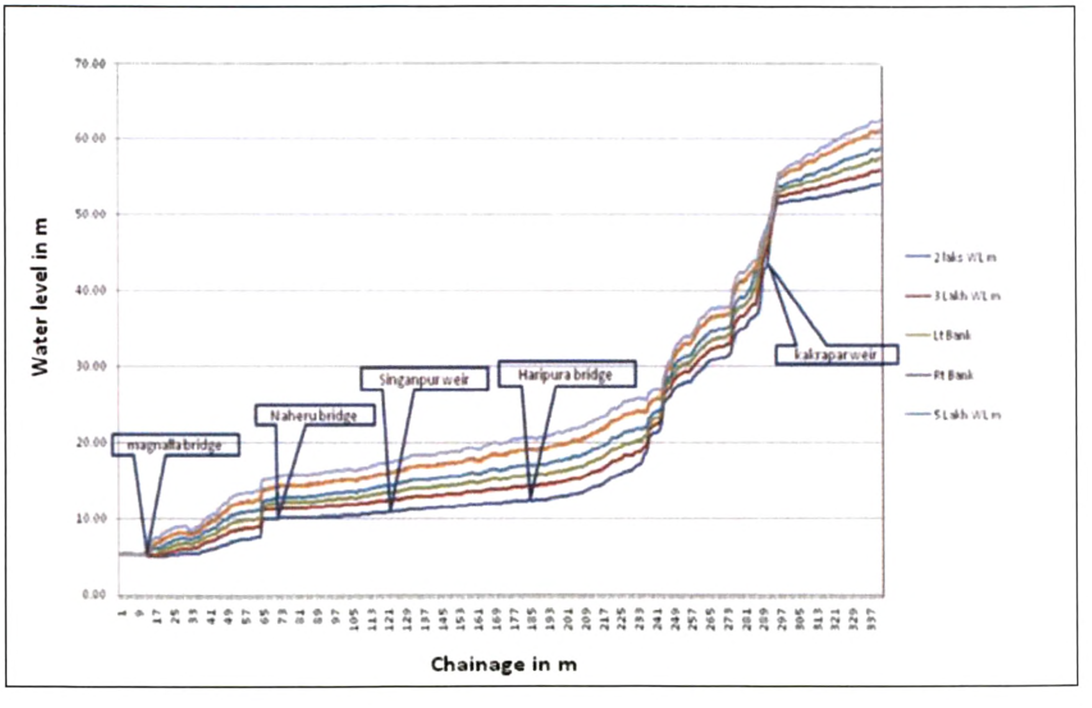
Graph.4.3 Water Surface profile of Tapi River at Surat 14172 m^3/s (5 lac Cusecs)



Graph.4.4 Water Surface profile of Tapi River at Surat 19076 m^3/s (6.73 lac Cusecs)



Graph.4.5 Water Surface profile of Tapi River at Surat 22676 m^3/s (8 lac Cusecs)



Graph.4.6 Tapi River Water Surface Profile for Different Flood Scenario – Surat

4.4 Validation of Mathematical Model by Comparing Observed Flood Water Levels

4.4.1 Mathematical Model Studies

The mathematical model entitle Autoregressive Integrated Moving Average (ARIMA) developed for Tapi river mouth at Hazira to 66 km upstream (about 15 km upstream of Kathor Bridge) included the river channel loop around Kadia Island and the Tena creek from its mouth to its junction with Tapi river near Bhata. The roughness values varying from 0.02 to 0.035 were adopted for different reaches. The model was capable of handling tide as downstream boundary and flood hydrographs as upstream boundary. Validation shown in Table 4.5.

4.4.2 Simulation of September 1998 Flood

The mathematical model run for this simulation was taken under condition i.e. with September 1998 flood hydrograph as upstream boundary and predicted tidal levels at Outer Hazira as downstream boundary. The observed high flood levels on 17.09.1998 at various locations along Tapi reach between Magdalla Bridge Kathor Bridge are superimposed on the water surface profile predicated from mathematical model for comparison the predicted water surface profile show good agreement with the observed high flood levels especially at the location Magdalla Bridge, Nehru Bridge and Kathor Bridge where gauges are installed. In general there is good agreement between predicated and observed flood levels over the entire reach. Comparison of predicted and observed flood levels predicated on East of Magdalla – Icchapur are about 8.50 m which is close to the observed levels of 7.90 m at GAIL and 8.50 m at Hazira branch canal velocities predicted in different reaches of Tapi rivers are 1.5 m/s to 2.5 m/s in the reach downstream of Magdalla Bridge. 1.5 m/s to 3.0 m/s in the reach form Umra to Kathor. These velocities are quite realistic and reasonable. Mathematical model was thus very well validated for flooding situation of 1998. The predicted water levels in the reach upstream of Singanpur weir were higher than the observed levels by 0.5 m to 0.9 m. The model run with reduced peak discharge also indicated that the observed water levels in this reach correspond to discharge of 1840 m^3/s (6.5 lac cfs). It may be mentioned here that the discharges at Kakrapar are given as upstream boundary 15 km upstream of Kathor Bridge. In reality there could be some reduction in peak discharge due to routing from Kakrapar to Kathor. This explains the difference in observed and predicted flood levels. The difference could be attributed to the lack of exact widths of cross sections at higher elevations and the difference in actual and simulated discharge

4.4.3 Flood level Predictions for 28315 m^3/s (10 lac Cusecs) Discharge (without flood Embankment Downstream of Nehru Bridge)

With tidal conditions at Outer Hazira during1998 flood (neap tide) and extension of flood hydrograph up to 28315 m^3/s (10 lac Cusecs) i.e., the mathematical model was run as explained. Predicted water surface profile for 22650 m^3/s (8 lac Cusecs) and 28315 m^3/s (10 lac Cusecs) are presented. The predicted water levels along Tapi River were 7.86 m at Magdalla Bridge 10.63 m at Umra, 12.93 m at Nehru Bridge, 15.23 m at Singanpur weir and 21.55 m at Kathor Bridge.

Another model run with 28315 m^3/s peak flood discharge from upstream boundary and highest spring tide (with 5.3 m HWL) at downstream boundary was taken. The water surface profile during peak flood discharge. The predicted flood levels at different locations. It could be seen that the flood levels upstream of Magdalla Bridge nearly remain same under both conditions indicating no effect of tide in this reach. On the downstream of Magdalla Bridge the water levels rise by about 0.9 m at KRIBHCO jetty and by about 1.5 m at L & T. In comparison to flood levels of 1998 the rise in flood levels will be 1.12 m at Magdalla Bridge, 2.0 m at Umra, 1.5 m at Nehru Bridge, 1.7 m at Singanpur weir and about 3.0 m at Kathor Bridge. Rise in flood levels around ONGC will be about 2.0 m above 1998 flood levels.

4.4.4 Need of Flood Protections Works Along Right Bank from Bhata to Magdalla Bridge and Further Downstream up to KRIBHCO / L & T Jetty

The analysis of flood levels predicted for the flood discharge of 28315 m^3/s (10 lac Cusecs) with highest spring tide indicate that in general the flood levels in this reach will exceed the natural bank levels by about 4 m (near Bhata) to 2.0 m (near KRIBHCO jetty). In general the bank levels and predicted flood levels along this reach will be as given below Table.No.4.3.

Table.4.3 Bank Levels and Predicted Flood Levels

Locations	Right bank Level (m)	Predicted HFL for 28315 m^3/s	1998 HFL (m)
Bhata	6.4	10.66	8.55
Bhatpur	4.5	09.00	-
Magdalla Bridge	4.5	07.92	6.80
Magdalla port	4.2	07.81	-
KRIBHCO Jetty	4.2	06.05	-
L & T	4.0	05.65	-

Considering the fact that the natural ground levels in most of Hazira Industrial Area are between 4.0 m to 5.0 m and the finished ground levels vary from 5.5 m to about 7.0. Some industries with low FGLs are likely to suffer from the flood water entry from river side in the reach downstream of Magdalla Bridge. The report by Surat Irrigation Circle has already recommended flood protection works from Bhata to Magdalla Bridge but no works downstream of Magdalla Bridge are proposed in comprehensive planning. Considering the predicted flood levels and right bank levels presented in above table it could be seen that flood protection works along right bank in the reach downstream of Magdalla Bridge will be essential, if other effective measures such a as Ukai reservoir operation or diversion of Tapi flood are not found feasible. The flood levels predicted by CDO Gujarat at Magdalla are even higher than those predicted by CWPRS.

4.4.5 Flood Level Predictions for 28315 m^3/s (10 lac Cusecs) with Flood Embankments from Nehru Bridge to Hazira and Nehru Bridge to Magdalla Bridge.

Analysis of results of these studies is presented and the comparison of predicted flood levels with the flood levels predicted for other conditions are shown in Table. No. 4.3 and 4.4. These results show that the construction of flood embankments on both the banks in the reach Nehru Bridge to Hazira will result in rise in flood levels along Tapi River especially in the reach Hazira to Singanpur. The comparison of predicted water levels without and with flood embankments indicated that the fleed levels at Magdalla Bridge will go up from 7.92 m to 9.55 m, at Bhata/Umra from 10.66 m to 11.28 m at Nehru Bridge from 12.93 m to 14.85 m and at Singanpur weir from 15.24 m to 16.43 m. the rise in flood levels in the reach upstream of Singanpur weir between Variav and Kathor will be between 0.7 m to 0.1 m this comparatives marginal. Thus, the Surat city and surrounding urban developments between Singanpur weirs to Magdalla Bridge will be subject to high rise in flood levels of about 1.20 m to 1.6 m. In the reach them Magdalla Bridge to L&T the flood levels with flood embankments will vary from 9.55 m to about 8.0 m under worst condition. Thus, the flood levels will further rise by 1.6 to 2.9 m in this reach after construction of flood embankments. In comparison to 1998 flood levels the rise in flood levels in the reach from Hazira to Bhata will be between 2 to 2.5 m if the flood embankments are provided and flood discharge of 28.315 $m^{3/}s$ (10 lac Cusecs) arrives at the time of highest spring tide. Even with partial flood embankment from Nehru Bridge to Magdalla Bridge flood levels will be 8.26 m at Magdalla. 10.70 m at Umra, 14.60 m at Nehru Bridge, 16.21 m at Singanpur weir, 18.02 m at Variav and 18.69 m at Amroli. Thus, there is substantial rise in flood level even with these flood embankments in the reach Magdalla to Singanpur. However, at downstream of Magdalla the flood levels reduce in comparison to those under condition i.e. embankment up to Hazira. The water level near L & T or Limla/Kawas outfall will be around 7.3 m with the partial embankment. For the discharge of 6.73 lac cfs the rise in flood level with flood level with flood embankment will be about 0.5 to 0.6 m between Nehru Bridge and Singanpur weir. Thus, if at all necessary the flood embankment from Nehru Bridge to Magdalla Bridge could be provided to give protection up to a flood discharge of 6.73 lac cfs since the rise in flood levels for this discharge is moderate as compared to flood discharge of 28315 m^3/s (10 lac Cusecs).

Keeping in view the average ground levels of 4.5 m to 6 m along right bank in the reach between Bhata to Hazira with the flood levels of 11.28 at Bhata and about 8.0 m at L&T Hazira the height of flood embankment will be 5 to 7 m considering 1.5 m of free board. Construction and maintenance of such high flood embankments for the length of about 18 km will be a huge task and costly affair apart from possibility of rise in flood levels all along the river reach up to Singanpur. Also the storm water drainage of the area protected by the flood embankments will pose many problems. Considering all these aspects the provision for such flood embankments fro entire reach between Bhata to Hazira should be the last option. If at all embankment becomes necessary it should be between Nehru Bridge to Magdalla Bridge. But before taking up such embankment proposal the other alternatives such as improving flood moderation at Ukai and possibility of diversion of Tapi floods to adjacent creeks (Kim and Seena) before Singanpur weir need to be studied in details. The proposal of effective flood of Ukai is better than the proposal of flood embankments.

4.4.6 Validation of Model – Channel Roughness and Water Surface

4.4.6.1 Roughness and Water Surface Widths in Model

The most important parameters in calibration of such 1 – D models are channel bed roughness and channel widths at water surface. From the information about the bed conditions, previous experience of CWPRS on Tapi studies and from estimation of roughness from the observed flood data, the following Manning's roughness values were selected in different reaches of the Tapi creek.

From mouth Magdalla port	→	0.02
From Magdalla put to Singapur Weir	→	0.025
Upstream Of Singapur Weir	→	0.03 to 0.035

The water surface widths at different levels are interpolated from the given input data of cross sectional widths at different water levels. Whenever the water level in the model exceeds the highest level in the input data, the width at the highest level given in the data is adopted for the further higher water levels.

Table.No.4.4 shows flood water levels along Tapi River for different flood condition, Table.No.4.5 shows comparison of calculated flood levels with observed flood level, CWPRS and CDO, Table.No.4.6 shows Ukai reservoir operation trials with August 1998 flood inflow hydrograph, Table.No.4.7 shows Ukai reservoir operation trials with August 1968 flood inflow hydrograph, Table.No.4.8 shows Ukai reservoir operation trials with PMF hydrograph at Ukai. Table.No.4.9 shows comparison of maximum water level in Ukai reservoir with different operational conditions, Table.no.4.10 shows water levels in Tapi River for different discharges.

Table.4.4 Flood Water Levels along Tapi River for Different Flood Condition

Maximum Upstream Flood Discharge (m^3/s)	Water Levels at Various Locations (m)						
	Magdalla Bridge	Umra/ Bhata	Nehru Bridge	Singanpur Weir	Varivay	Amroli Bridge	Kathor Bridge
19057 (6.73 Lac cfs)	6.80	8.62	11.36	13.56	14.99	15.58	18.77
22650 (8. 0 Lac cfs)	7.13	9.51	11.95	14.29	16.01	16.79	20.24
(7 Lac cfs)	6.80	8.57	11.41	13.91	14.27	14.80	18.32
25482 (9.0 Lac cfs)	7.39	9.92	12.41	14.78	16.72	17.55	21.00
28315 (10 Lac cfs)	7.86	10.63	12.90	15.24	17.34	18.17	21.55
28315 (10 Lac cfs)	7.92	10.66	12.93	15.24	17.43	18.29	21.76
(A) 28315 (10 Lac cfs)	8.26	10.76	14.60	16.21	18.02	18.69	21.88
(10 Lac cfs)	7.94	10.68	12.95	15.24	17.45	18.32	21.78
(B) 19057 (9.73 Lac cfs)	7.08	8.82	12.00	14.07	15.16	15.75	18.90

*Observed HFLs of 1998

CONDITION I: - 1998 flood hydrographs from upstream boundary and at downstream boundary tidal water levels as per the curve derived using high and low water levels for Outer Hazira from tide tables. (The flood embankments as existing from Kathor to Nehru Bridge)

CONDITION II: - 1998 flood hydrographs from upstream beyond 19057 m^3/s (6.73 lac Cusecs) upto 28315 m^3/s (10 lac Cusecs) and tidal condition downstream same as under Condition I. (Flood embankments as existing)

CONDITION III: - At upstream boundary flood hydrograph of 1998 extended from 19057 m^3/s (6.72 lac Cusecs) up to 28315 m^3/s (10 lac Cusecs). At downstream boundary Spring tide water level curve with HWL of 5.3m given as Boundary condition (Flood embankments as existing)

CONDITION IV: - Condition III with additional flood embankments on either banks from Nehru Bridge to river mouth at Hazira

CONDITION V: - Condition III with additional flood embankments on both banks from Nehru to Magdalla Bridge

Table.4.5 Comparison of Calculated Flood Levels with Observed Flood Level, CWPRS and CDO

Tapi Flood Discharge	Prediction by	Predicted Water at Various Locations (m)						
		Magdalla Bridge	Umra/ Bhata	Nehru Bridge	Singanpur Weir	Varivay	Amroli Bridge	Kathor Bridge
1998 Flood 19057 m^3/s (7 lac Cusecs)	CDO	(Reported - at port) 7.80	8.80	11.50	12.81	14.31	14.87	18.40
	CWPRS	6.80	8.62	11.36	13.56	14.99	15.58	18.77
	Observed level In Sept. 1998	6.80	8.55	11.40	13.90	14.23	14.77	18.29
	Calculated (ARIMA Model)	6.80	8.57	11.41	13.91	14.27	14.80	18.32
28315 m^3/s (10 lac Cusecs)	CDO	9.88	-	13.95	15.57	17.05	17.61	21.29
	With Sept. 1998 tide	7.86	10.63	12.90	15.24	17.34	18.17	21.55
	With highest Spring tide	7.92	10.66	12.93	15.24	17.43	18.29	21.76
	Calculated (ARIMA Model)	7.94	10.68	12.95	15.24	17.45	18.32	21.78

Table.4.6 Ukai Reservoir Operation Trials with August 1998 Flood Inflow Hydrograph

Time (Hours)	Ukai Inflow (m^3/s)	Ukai Outflow (m^3/s)	Ukai Reservoir Level (m) with				
			1998 Outflows		Outflows restricted to		
			Observed in m	Computed in m	3 lac Cusecs	3.5 lac Cusecs	4 lac Cusecs
1	7890	826	104.17	104.17	104.17	104.17	104.17
2	6901	3313	104.21	104.20	104.20	104.20	104.20
3	7574	3313	104.24	104.22	104.22	104.22	104.22
4	11297	6199	104.30	104.24	104.24	104.24	104.24
5	12331	6199	104.35	104.27	104.27	104.27	104.27
6	12414	10760	104.38	104.31	104.31	104.31	104.31
7	13278	10760	104.39	104.31	104.33	104.32	104.31
8	12278	10760	104.41	104.33	104.35	104.34	104.33
9	12948	10760	104.42	104.34	104.38	104.35	104.34
10	12885	10760	104.43	104.35	104.40	104.37	104.35
11	12957	10760	104.45	104.36	104.42	104.38	104.36
12	12623	10760	104.45	104.38	104.45	104.40	104.38
13	1339	10760	104.47	140.38	104.47	104.42	104.38
14	14423	10760	104.40	104.40	104.49	104.43	104.40
15	18914	10760	104.56	140.42	104.53	104.46	104.42
16	22576	10760	104.65	104.46	104.58	104.51	104.46
17	22727	10760	104.74	104.53	104.66	104.57	104.53
18	1597	10760	104.78	104.59	104.73	104.64	104.59
19	1564	10760	104.81	104.62	104.77	104.67	104.62
20	16052	12430	104.84	104.64	104.81	104.70	104.64
21	19391	12430	104.89	104.66	104.85	104.74	104.67
22	15662	12430	104.91	104.70	104.91	104.79	104.71
23	14944	12430	104.93	104.72	104.95	104.82	104.73
24	17803	13450	104.96	104.73	104.98	104.85	104.75
25	15975	13450	104.97	104.75	105.03	104.89	104.79
26	18560	13450	105.01	104.77	105.07	104.92	104.81
27	19720	13450	105.06	104.79	105.13	104.97	104.85
28	17302	14328	105.08	104.83	105.18	105.02	104.89
29	18719	14328	105.11	104.84	105.23	105.06	104.93
30	25520	14328	105.18	104.87	105.28	105.10	104.97

Table.4.7 Ukai Reservoir Operation Trials with August 1968 Flood Inflow Hydrograph

Time (Hour)	Inflow (m^3/s)	Ukai Reservoir Level (m) with maximum Outflow Restricted to						
		3 lac Cusecs with Initial level (m)		4 lac Cusecs with Initial level (m)	5 la Cusecs with Initial level (m)		6 lac Cusecs with Initial level (m)	
		103.63	**100.58**	**103.63**	**103.63**	**102.11**	**103.63**	**100.58**
0	0	103.63	100.58	103.63	103.63	102.11	103.63	100.58
1	283	103.63	100.58	103.63	103.63	102.11	103.63	100.58
2	566	103.63	100.58	103.63	103.63	102.11	103.63	100.58
3	849	103.63	100.58	103.63	103.63	102.11	103.63	100.58
4	1133	103.63	100.58	103.63	103.63	102.11	103.63	100.58
5	1416	103.63	100.58	103.63	103.63	102.11	103.63	100.58
6	1699	103.63	100.58	103.63	103.63	102.11	103.63	100.58
7	1982	103.63	100.58	103.63	103.63	102.11	103.63	100.58
8	2855	103.63	100.58	103.63	103.63	102.11	103.63	100.58
9	3728	103.63	100.58	103.63	103.63	102.11	103.63	100.58
10	4601	103.63	100.58	103.63	103.63	102.11	103.63	100.58
11	5475	103.63	100.58	103.63	103.63	102.11	103.63	100.58
12	6348	103.63	100.58	103.63	103.63	102.11	103.63	100.58
13	7221	103.63	100.58	103.63	103.63	102.11	103.63	100.58
14	7716	103.63	100.58	103.63	103.63	102.11	103.63	100.58
15	8212	103.63	100.58	103.63	103.63	102.11	103.63	100.58
16	8707	103.63	100.58	103.63	103.63	102.11	103.63	100.58
17	9203	103.63	100.59	103.63	103.63	102.11	103.63	100.58
18	9698	103.64	100.59	103.63	103.63	102.11	103.63	100.58
19	10194	103.64	100.60	103.63	103.63	102.11	103.63	100.58
20	13401	103.65	100.61	103.63	103.63	102.11	103.63	100.58
21	16608	103.68	100.64	103.64	103.63	102.11	103.63	100.58
22	19815	103.72	100.70	103.67	103.64	102.13	103.63	100.58
23	23021	103.78	100.78	103.72	103.67	102.16	103.65	100.61
24	30525	103.86	100.88	103.78	103.72	102.22	103.68	100.65
25	38029	103.98	101.03	103.88	103.81	102.34	103.75	100.74
26	38053	104.13	101.24	104.02	103.94	102.50	103.86	100.89

Table.4.8 Ukai Reservoir Operation Trials with PMF Hydrograph at Ukai

Time (Hour)	Inflow	Outflow			Reservoir Level (feet)		Reservoir Level (m)	
					With Initial Level (feet)		With Initial Level (m)	
	(Cusecs)	(m^3/s)	(Cusecs)	(m^3/s)	**330.00**	**340.00**	**100.58**	**103.63**
0	141259	4000	141259	4000	330.00	340.00	100.58	103.63
1	141259	4000	141259	4000	330.00	340.00	100.58	103.63
2	141259	4000	141259	4000	330.00	340.00	100.58	103.63
3	141259	4000	141259	4000	330.00	340.00	100.58	103.63
4	141259	4000	141259	4000	330.00	340.00	100.58	103.63
5	141259	4000	141259	4000	330.00	340.00	100.58	103.63
6	141259	4000	141259	4000	330.00	340.00	100.58	103.63
7	147145	4167	147145	4167	330.00	340.00	100.58	103.63
8	153031	4333	153031	4333	330.00	340.00	100.58	103.63
9	158917	4500	158917	4500	330.00	340.00	100.58	103.63
10	164802	4667	164802	4667	330.00	340.00	100.58	103.63
11	170688	4833	170688	4833	330.00	340.00	100.58	103.63
12	176574	5000	176574	5000	330.00	340.00	100.58	103.63
13	194231	5500	194231	5500	330.00	340.00	100.58	103.63
14	211889	6000	211889	6000	330.00	340.00	100.58	103.63
15	229546	65000	229546	6500	330.00	340.00	100.58	103.63
16	247204	7000	247204	7000	330.00	340.00	100.58	103.63
17	264861	7500	264861	7500	330.00	340.00	100.58	103.63
18	282518	8000	282518	8000	330.00	340.00	100.58	103.63
19	306062	8667	306062	8667	330.00	340.00	100.58	103.63
20	329605	9333	329605	9333	330.00	340.00	100.58	103.63
21	353148	10000	353148	10000	330.00	340.00	100.58	103.63
22	412006	11667	412006	11667	330.00	340.00	100.58	103.63
23	470864	13333	470864	13333	330.00	340.00	100.58	103.63
24	529722	15000	529722	15000	330.00	340.00	100.58	103.63

Table.4.9 Comparison of MWL in Ukai Reservoir with Different Operational Conditions

No	Inflow Hydrograph	Peak Inflow m^3/s (Cusecs)	Maximum Outflow Restricted to m^3/s (Cusecs)	Initial Reservoir m (feet)	Maximum Water Level in Ukai Reservoir m (feet)
1	September 1998 flood Ukai inflow hydrograph	29232 (1053500)	19085 (674000)	104.17 (341.75)	105.46 (345.91)
2	September 1998 flood Ukai inflow hydrograph	29832 (1053500)	8495 (300000)	104.17 (341.75)	106.48 (349.34)
				102.11 (335.00)	105.05 (344.84)
3	September 1998 flood Ukai inflow hydrograph	29817 (1053000)	11327 (400000)	104.17 (341.75)	106.19 (348.34)
				102.11 (335.00)	104.67 (343.41)
4	September 1998 flood Ukai inflow hydrograph	29817 (1053000)	11327 (400000)	104.17 (341.75)	105.93 (347.55)
				102.11 (335.00)	103.34 (342.31)
5	August 1968 flood Hygrograph at Ukai	42418 (1498000)	8495 (300000)	103.63 (340.00)	110.49 (362.51)
6	August 1968 flood Hygrograph at Ukai	42418 (1498000)	8495 (300000)	100.58 (335.00)	109.33 (358.70)
7	August 1968 flood Hygrograph at Ukai	42418 (1498000)	11327 (400000)	103.63 (340.00)	109.87 (360.45)
				102.11 (335.00)	109.30 (358.59)
				100.58 (330.00)	108.52 (3258.70)
8	August 1968 flood Hygrograph at Ukai		14158 (500000)	103.63 (340.00)	109.33 (358.70)
				102.11 (335.00)	108.58 (358.70)
9	August 1968 flood Hygrograph at Ukai		16990 (600000)	103.63 (340.00)	108.73 (356.74)
10	August 1968 flood Hygrograph at Ukai		16990 (600000)	100.58 (330.00)	106.83 (350.49)
11	PMF at Ukai	72490 (2560000)	24069 (850000)	103.63 (340.00)	110.35 (362.04)
12	PMF at Ukai	72490 (2560000)	24069 (850000)	100.58 (330.00)	109.19 (358.25)

Table.4.10 Water Levels in Tapi River for Different Discharges

Locations	Water Levels (m) for Different Discharges						Bank Levels (m)	
	7080 Cumecs	8494 Cumecs	11326 Cumecs	14158 Cumecs	19822 Cumecs	28315 Cumecs	Left	Right
	2.5 lac Cusecs	3 lac Cusecs	4 lac Cusecs	5 lac Cusecs	7 lac Cusecs	10 lac Cusecs		
Kawas Outfall	5.30	5.40	5.60	5.70	5.90	6.05	4.50	4.50
Limla Outfall	5.50	6.00	6.20	6.30	6.40	6.70	4.50	4.50
Magdalla Bridge	6.08	6.40	6.70	6.80	7.19	7.92	5.43	4.22
Tena Confluence	6.30	6.74	7.39	8.10	9.14	10.38	8.70	6.40
Umra	5.94	7.05	7.76	8.40	9.22	10.66	10.41	6.50
Nehru Bridge	7.18	8.40	9.68	11.07	11.61	12.90	11.70	11.70
Singanpur Weir	8.47	9.52	10.93	12.39	13.66	15.24	14.41	12.80
Variav	9.50	10.46	11.99	13.50	15.15	17.43	14.60	14.60
Amroli Bridge	10.15	11.06	12.60	14.08	15.85	18.29	11.90	14.30
Surthana	12.26	13.00	14.42	15.85	18.08	20.55	15.00	15.00
Kathore Bridge	13.24	14.01	15.44	16.81	19.02	21.76	23.10	23.10

Note: Above water levels are with Spring tide of 5.03 m at the mouth of river and with existing flood embankments.

4.5 Simulations of September 1998 Flood with Tidal Wave Effect

With the upstream and downstream boundary condition as 1998 flood hydrograph and predicted tidal water levels at Outer Hazira during 15 – 19 September 1998 respectively, the model run was carried out to predict the flood levels are various locations. It could be seen that the predicted water surface profile showing high flood levels along the Tapi River closely follow the observed water levels at different locations. In general, at the locations such a Magdalla Bridge, Nehru Bridge, Singapur Weir, and Kathore Bridge where the water level gauges were installed, the predicted flood levels closely match with observed levels. The predicted levels upstream of Singapur Weir are slightly higher than the observed water levels. In general, there is good agreement of observed and predicted flood levels over the entire model reach as seen the predicted water surface profiles for 8495 m^3/s (3 lac cfs), 14160 m^3/s (5 lac cfs) and 19057 m^3/s (6.73 lac cfs) discharges during 1998 flood. The comparison of predicted and observed water level at Nehru Bridge as a function of time also indicates good agreement. Thus, the model is adequately validated over the entire reach for the 1998 flood. The predicted velocities in different channels reaches are shown in Table.No.4.11.

Table.4.11 Predicted Velocities in Different Channels Reaches

No.	Channel Reach	Velocity (m/s)
1	River mouth to Magdalla Port	1.5 to 2.5
2	Magdalla to Umra	1.7 to 3.0
3	Umra to Singapur Weir	2.0 to 3.0
4	Singapur weir to Kathor	2.0 to 4.0

Above velocities are also realistic and reasonable. The water levels predicted at various locations are shown in Table.No.4.1 and 4.2. Same also shown in Graph.No.4.1 to 4.6.

4.5.1 Studies with 28315 m^3/s (10 lac Cusecs) Discharge with September 1998 Tidal Levels

In order to predict the possible water levels with 28315 m^3/s (10 lac Cusecs) discharge for the 1998 flood situation, the discharge at upstream boundary was increased from 19057 m^3/s (6.71 lac Cusecs) to 28315 m^3/s (10 lac Cusecs) gradually keeping the downstream tidal boundary (i.e. neap tide) unaltered. The predicted water surface profiles for 22650 m^3/s (8 lac Cusecs) and 28315 m^3/s (10 lac Cusecs) are presented. From these predictions, it could be seen that with flood discharge of 28315

m^3/s (10 lac Cusecs) the water levels at Magdalla Bridge, Umra, Nehru Bridge, Singapur Weir and Kathor Bridge will be 7.86 m, 10.63 m, 12.93 m, 15.23 m and 21.55 m respectively. The flood levels at Bhata, Bhatpur and around ONGC could also be about 10.60 m. Thus, there could have been substantial rise in the flood levels of September 1998 if the design discharge of 28315 m^3/s (10 lac Cusecs) had occurred.

4.6 Discussions

Based on the result of analysis from ARIMA mathematical model studies, the prediction of water surface profile plotted in graph no.4.1 to 4.6. Water surface profile levels at different fixed locations along the river reach with special reference to surat city compared with observed flood levels. Tidal wave generation data and result of reservoir operation studies, the discussions already made on various aspects of the research studies. A summary of discussions is presented below.

4.6.1 Tapi Floods of 1994 and 1998 at Surat

Peak floods discharge of 14866 m^3/s (5.25 lac Cusecs) and 19057 m^3/s (6.73 lac Cusecs) were experienced during Tapi floods at Surat September 1994 and 1998 respectively. The 1998 floods levels in ONGC and surrounding industries were about 0.6 m higher than 1994 flood levels. The maximum floods levels in September 1998 were 7.5 m at ONGC, 7.0m at KRIBHCI plant, 7.2 m at KRIBHCO township, 7.9 m at GAIL complex, Floods level along Tapi were 6.80 m at Magdalla Bridge , 8.55 m at Umra. 11.40 At Nehru Bridge. 13.90 m at Singanpur weir, 14.23 m at Variav, 14.77 m at Amroli and 18.29 m at Kathor. These floods resulted in heavy damages of urban properties and infrastructure, buildings, machineries in Hazira Industries Area especially ONGC and KRIBHCO.

Recommendation for detailed, logical short and long term measures to mitigate losses due to Tapi river flood discussed as below.

4.6.2 Immediate Measures

4.6.2.1 Effective operation Ukai Reservoir

Keeping in view the reality that during 1994 and 1998 floods of the order of 14866 m^3/s (5.25 lac Cusecs) and 19057 m^3/s (6.73 lac Cusecs) respectively released from Ukai dam had resulted in Heavy flooding in and around Surat city especially along low level right downstream of Nehru Bridge (where there are no protection works) including Hazira Industrial area, the maximum releases from Ukai may be restricted to 8495 m^3/s (3 lac Cusecs) to 9910 m^3/s (3.5 lac Cusecs) by efficient reservoir operation. As may be restricted by the results of reservoir operation studies with the initial reservoir level at RL 102.11 m (335 ft) the floods with peak discharges of about 29832 m^3/s (10.53 lac i.e. 1998 flood) could be effectively handled without exceeding FRL by restricting outflows to above limits. Even with the initial level of about 103.63 m (340.0 ft) of the same order could be dealt with same outflows but by exceeding FRL by about 1.0m.

4.6.2.2 Review of Ukai Reservoir Operation Policy

It is fact in 1968 high flood of about 42470 m^3/s (15 lac Cusecs) had occurred in Tapi river and the value of spillway design flood is 49500 m^3/s (17.5 lac Cusecs). The revised PMF is about 72490 m^3/s (25.6 lac Cusecs). The Tapi flood protection works are designed for discharge of 28315 m^3/s (10 lac Cusecs) considering the moderation of flood in Ukai reservoir. The flood levels predicted from mathematical model studies for flood discharge of 28315 m^3/s are 7.92 m at Magdalla Bridge 10.66 m at Umra, 12.93 m At Nehru Bridge and 15.24 m at Singanpur weir (without flood embankment downstream of Nehru Bridge). These flood level are 1.1 m to 2 m higher than those of 1998 flood levels and therefore flooding situation will be further worsen for the discharge of 28315 m^3/s (10 lac cfs). Construction of flood embankment Downstream of Nehru Bridge till Hazira will result in further rise in water levels along river reach from Magdalla to Singanpur and upstream. The 1994 and 1998 floods have shown that flood discharge beyond 9910 m^3/s (3.5lac Cusecs) results in flooding along low lying river banks. Considering flood damages those can occur in urban abd industrial areas on downstream reaches by exceeding Ukai outflows beyond 9910 m^3/s (3.5 lac Cusecs). These is need to utilize the strong volume available between FRL (345 ft) and MWL (351 ft) to moderate the high floods similar to that of 1968, Even there is need to floods by restricting the outflows. Considering the results of reservoir operation studies, the Ukai reservoir operation policy may be reviewed to evolve appropriate measures and procedures to effectively deal with floods similar to that of 1968 or higher with special consideration to the aspects of utilization of volume between FRL and MWL and rising of present MWL by about 1.5 m. In view of the fact that measures such as construction of flood embankments and diversion of flow will need period and heavy investment for implementation (apart from many demerits of these) more importance should be given to this recommendation of Ukai reservoir operation procedure.

4.6.2.3 Detailed studies on Ukai Reservoir Operation

Detailed studies on Ukai reservoir Operation may be taken up with different in respect of initial reservoir level, rainfall patterns in catchments and inflow / outflow hydrographs. These studies will be helpful for review of reservoir operation policy.

4.6.2.4 Development of a Mathematical Model of entire Tapi River channel Network u/s of Ukai

A detailed mathematical model of entire Tapi river channel network (comprising major tributaries) of Tapi basin up to Ukai may be developed for predicting time dependent discharge and water levels all along the entire reach of Tapi river. This model may be used to predict inflow hydrographs necessary for detailed Ukai reservoir operation studies. The appropriate authority may take initiative in this respect. This model will also be useful for giving flood warning well in advance.

- **Studies for Possible Rise Flood Levels in the Reach Upstream of Ukai**

The rise in the natural flood levels in the Tapi river reach upstream of Ukai dam due to restricting outflows from Ukai dam under different inflow hydrograph and by rising of MWL. Could be studies with the help of Tapi basin mathematical model mentioned in recommendation (4) above. The comparison of flood levels with and without Ukai reservoir will give an idea about the possible rise in flood levels due to flood moderation in Ukai reservoir. If the rise in natural flood levels due to reservoir operation is considerable then the possible additional flood damages due to this rise may be estimated and compared with possible damages of urban, industrial and agricultural sector which can occur in lower reaches around Surat if flood is not moderated effectively at Ukai. Economic analysis of relative flood damages on upstream and downstream under these situations may be carried out. These aspects may be taken into consideration during review of reservoir operation procedure during emergency situation.

- **Hydrological Data network up gradation / Expansion**

For the effective reservoir operation the timely availability of reliable and adequate quantitative information on hydrological parameters is most important. For this purpose the existing hydrological data acquisition network may be upgraded and expanded (if necessary) to get timely reliable hourly information on hydrological parameters which will be valuable input data for mathematical models for predicting flows along Tapi river at Ukai and Upstream. With this upgraded data acquisition network and the mathematical model of Tapi basin the present period of 35 hours for availability of information of Ukai inflows approaching Ukai may be improved to at least 45 to 50 hours so that reservoir operation could be most effective with minimum flooding of upstream and downstream areas.

- **Availability of Tapi Basin Network Model, Reservoir Operation Model and Information on Hydrological Parameters at Control Room**

The mathematical models for Tapi river network and reservoir operation should be available at Control Cabin where the hourly information on hydrological parameters is also received continuously. Using this data quick runs may be taken on Tapi basin network model and reservoir operation model to estimate the inflow hydrograph at Ukai and then decide in advance appropriate safe outflow hydrograph from the Ukai dam.

- **Study of Rainfall and Gauge Discharge Data of Previous Storms**

Till the time the mathematical model for computing flows in Tapi basin network is developed. The rainfall and gauge discharge data of the past major storms may be studied to evolve co-relation between rainfall and the inflows to Ukai. This could be useful for computing inflows to Ukai dam for a given rainfall in the catchment.

- **Completion of the Ongoing Flood Protection Works between Surat to Kathor**

The incomplete works of flood embankments and of sluice gates pertaining to the ongoing flood protection scheme may be completed as early as possible. The work of rising of flood embankment upstream of Singanpur weir may also be completed at the earliest. Considering the fact that, these incomplete works allowed entry of flood water into Surat city at some locations on both the banks. The completion of these works is at most important.

- **Checking of Existing bank Levels**

The flood level predicted at various locations for various conditions are presented in chapter no.4, Table 4.3 and Table 4.4. These must be studied carefully in relation to existing bank levels over the entire reach. Necessary adequate protection for high flood levels may be provided, if the Ukai reservoir operation policy is not reviewed to reduce the Ukai releases to safer limits.

- **Dredging of Kawas, Limla and Tena nalas and increasing Waterways of Bridges on these Nalas**

The dredging works in Kawas, Limla and Tena nalas may be completed. The dredged material lying along side of these nalas may be lifted and disposed at suitable locations away from the banks so that, it will not find its way to the nalas again. Behind ONGC and KRIBHCO colonies more openings in ESSAR railway embankment may be provided to pass flood water to Tena creek.

• Installation of water level gauges in the reach between Magdalla Bridge and ESSAR Plant

In order to get continuous record of water levels during flood and tide automatic self recording water level gauges may be installed along river bank at Hazira, ESSAR, L&T Jetty, KRIBHCO Jetty and Magdalla Bridge.

• Assessment of Siltation in Ukai Reservoir

The hydrographic Survey of Ukai reservoir may be taken up to assess siltation in the reservoir and corresponding loss of storage capacity. Based on this data the studies for predicting progressive loss in storage capacities in future may be taken up. Actual capacity of reservoir must require for revised reservoir operation calculation after the years at regular interval.

4.6.3 Long Term Measures

The reservoir operation studies have indicated that if Ukai reservoir operation policy is modified to permit use of volume between FRL to MWL and also raising of present MWL by about 1.5 m then the floods even up to 42475 m^3/s (15 lac Cusecs) could be handled effectively by releasing safe discharge of about 8495 to 11327 m^3/s (3 to 4 lac Cusecs) from Ukai with appropriate initial reservoir levels. But if reservoir operation policy is not modified and the use of storage above FRL and raising of MWL is not permitted then the outflows from Ukai reservoir could be about 16990 m^3/s (6 lac Cusecs) or more as seen from Table XVIII. This will lead to flooding in and around Surat and Hazira. Following long term measures may be taken to avoid possible flooding under this situation.

4.6.3.1 Diverting Part of Tapi Flood to Adjoining Creeks on North

The feasibility of diverting about 5660 m^3/s to 8494 m^3/s (2 to 3 lac Cusecs) of flood discharge on upstream of Singanpur weir (out of total 16990 m3/s (6 lac Cusecs) released from Ukai) towards Sena / Kim creeks on north may be studied. Considering the possible locations at Variav and upstream of Kathor Bridge. For diverting of the natural nalas discharge upstream of Singapur, the natural topography of the region, the existing capacities of the natural nalas joining Kim and Sena creeks and existing land use along the tentative alignment of diversion channel may be studied to assess feasibility of such diversion of Tapi floods water into Kim / Sena creek. A control structure (weir) at the mouth of the diversion canal should be provided so that the appropriate flow will be drawn in diversion canal after total flow exceeds 8494 m^3/s (3 lac Cusecs). The diversion channel should have appropriate carrying capacity. If the diversion proposal is found feasible then the detail design of diversion structure and the canal may be studied on a physical and mathematical model. Such diversion of Tapi flood discharge can have indirect benefits such as creation of fresh water lake in Sena/Kim creeks, salinity control and supply of fresh water in surrounding areas. These advantages may also be kept in view while considering the proposal.

4.6.3.2 Provision of Flood Embankment form Nehru Bridge to Magdalla Bridge on Both Banks

If reservoir operation policy is not modified to moderate high floods by restricting the outflows as suggested and also the proposal of diverting the part of Tapi flood (on upstream of Singanpur weir) to the Kim. Sena creek on north not found feasible for implementation. Then the Tapi reach downstream of Ukai till Hazira could be subjected to discharges ranging from 11327 m^3/s to 28315 m^3/s (4 lac to 10 lac Cusecs) especially during high floods. This flooding on large scale as experienced in 1998 or even more. The flood embankments on both banks from Nehru Bridge to Magdalla Bridge to Hazira may further aggravate flooding situation. Under this situation the flood embankments from Nehru Bridge to Magdalla Bridge only could provide necessary protection especially along low lying right bank and therefore should be considered as next alternative. While designing this floods embankments the floods levels predicted at different locations for the discharge of 19057 m^3/s (6.73 lac Cusecs) and 28315 m^3/s (10 lac Cusecs) be taken into account to decide the top level of the embankments at different locations. With these partial flood embankments the major flow entering from Tena nala and Kotars along low lying right bank in the reach between Bhata and Magdalla Bridge will be prevented. However it may be kept in mind that with the partial flood embankments from Nehru Bridge to Magdalla Bridge the flood levels between Magdalla Bridge to KRIBHCO / L&T will vary from 8.26 to about 7.5 m for the discharge of 28315 m^3/s (10 lac Cusecs). Therefore, entry of flood water through Limla, Kawas and other small nalas to the Hazira industrial area is very much likely during high floods. Provision of flood embankment in this reach in this reaches only on right bank may give necessary protection but will reflect in rise in flood levels on upstream. Therefore, extension of embankments only on right bank downstream of Magdalla. The provision of local protections to the important installations of industries or individual campus with acceptance of short period flooding of remaining area is another alternative to extension of flood embankments.