Report on

Hydrologic Studies for Sabarmati Riverfront Development Project

For
Ahmedabad Municipal Corporation

National Institute of Hydrology
Roorkee – 247 667 (Uttaranchal)
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1.0 Introduction

The Sabarmati River is one of the four main rivers which traverse the alluvial plains of Gujarat. It rises in the Aravalli hills at a north latitude of 24° 40' and an east longitude of 73° 20' in the Rajasthan state at an elevation of 762 meters near the popular shrine of Amba Bhavani. After traversing a course of about 48 km in Rajasthan, the river enters the Gujarat State. Wakal River joins it from the left, near village Ghonpankhari. It receives the Sei River from the right near Mhauri and then the Harnav River from the left at about 103^{rd} km from the source. Thereafter, it enters the Dharoi reservoir. Downstream of the Dharoi reservoir, Sabarmati is joined the Hathmati River. The river passes through Ahmedabad at about 165 km downstream of Dharoi dam. Further 65 km downstream, another tributary, the Watrak River joins it from the left. Flowing for a further distance of 68 km, the river outfalls into the Gulf of Cambay in the Arabian Sea.

Passing through the centre of Ahmedabad city, the Sabarmati River is a major source of water for the city. The river has been subjected to severe pressure and abuse owing to the fast pace of urban and industrial growth of the city. At present, the Sabarmati riverfront lies neglected and is characterized by unplanned urban development. The river has a wide channel with encroachment by slum dwellers and others at several places. A barrage, known as Wasna Barrage, was constructed on the river downstream of Ahmedabad to utilize the water of the river.

Appropriate development of the Sabarmati river front was planned way back in sixties. It has been proposed to remodel and reshape the channel to a uniform smaller width and the additional land including back filling behind the river banks is to be developed as a riverfront with roads, parks etc. The development of this riverfront can improve the quality of environment and life in Ahmedabad city. To achieve this objective, the Ahmedabad Municipal Corporation established the Sabarmati River Front Development Corporation Ltd. (SRFDCL). Environmental Planning Collaborative (EPC) was assigned the task for preparing a conceptual proposal for the development of the river bank areas within the municipal limits of Ahmedabad. The EPC conceptual proposal contemplates the construction of embankments with retaining walls all along the riverfront (on both banks) in 9 km reach from Subhash Bridge to Wasna Barrage.

1.1 Sabarmati River Front Development Project (SRFDP)

The SRFDCL has taken up task of the execution of comprehensive development of both banks of Sabarmati River from Narmada main canal to Wasna Barrage. The distance from Narmada main canal (NMC) to *Wasna* Barrage is approximately 20 km. In addition to the many socio-economic features, the salient points that are envisaged to from the hydraulics point of view are as follows:

- (i) Construction of embankments on both sides of the river along the entire stretch from Narmada main canal to Wasna Barrage.
- (ii) Retention of water in the river for the whole year by construction of barrage at Kotarpur and Dudheshwar
- (iii) The laying of water supply lines, trunk sewers and pumping stations and along both the banks, and the extension of storm water drains flowing into the river.

M/s C C Patel & Associates Pvt. Ltd. have completed a study of the river floods and evaluated the hydraulic profile of likely floods. In the above background, the Municipal Commissioner, Ahmedabad Municipal Corporation (AMC) has approached the National Institute of Hydrology (NIH) for carrying out the hydrological studies for the Sabarmati Riverfront Development Project. It is proposed to carry out the work as described here.

1.2 Objectives

The study will be completed in two phases and the objectives will be as follows:

1. To review the study conducted earlier on flood behavior due to constricting of the river width for different discharges as worked out from the data collected and as suggested by different experts.
2. To compute and confirm the HFL for a design flood of 4.75 lakh cusec (13450 cumecs) along the river after the execution of the project.

1.3 Deliverables

On completion of the study, the Institute would submit:

1. Review of earlier studies on flood behavior due to constricting of the river width for different discharges;
2. Computation of HFLs for the design flood of 4.75 lakh cusec along the river after the execution of the riverfront development project i.e. after the constriction of river reach.

2.0 STUDY AREA

The study area for this project is the Sabarmati River reach from Narmada main canal crossing to *Vasana* Barrage; the approximately length of the reach is 20 km. The physical features and the hydraulics of this reach are described briefly in the following.

2.1 Physical Features of the River

This section describe the physical features of the study are, as per the findings of the detailed survey of the river and adjacent areas undertaken for developing the SRFD Project (SRFDCL, 2004).

Subhash Bridge to *Vasana* Barrage (Inside the city area):

The river runs a meandering course of about 9 km from Subhash Bridge up to the Wasna Barrage through the city with an average width varying from 325 to 500 m, with two meandering loops at *Gaikwad Haveli* and *Wadaj.* The average reduced levels (RL) of the riverbed at *Subhash* Bridge and *Wasna* Barrage are 39.2 m and 37.4 m respectively, and the average slope is mild. The height of the banks ranges from 4 to 9 m. A negative slope is observed from *Sardar* Bridge to *Wasna* Barrage. The edge is not clearly defined by embankments or retaining walls at most places. The river edge gently slopes down to the riverbed at several places, which have vegetation and have been encroached by slum settlements. The RL of the top gate of the *Wasna* Barrage is 41.756 m. Filling Wasna Barrage up to these level results in flooding of the nearby areas in monsoons.

Narmada Main Canal (NMC) to *Subhash* Bridge (Outside City area):

The river runs a meandering course of total 11.65 km from Narmada Main Canal to *Subhash* Bridge with an average width varying from 296 to 732 m. There are three meandering loops at old village sites of *Kotarpur*, *Ashram Bapu's Ashram* (near Koteshwar) and near AEC at *Subhash* Bridge. The average reduced levels of the riverbed at Narmada Main Canal and Subhash Bridge are 44.73 m and 39.25 m respectively, and the slope of the river is generally mild. The height of the banks ranges from 4.75 to 12.50 m. The edge is not clearly defined by embankments or retaining walls at most places, and the river edge gently slopes down to the riverbed at several places, which have vegetations.

2.2 Hydraulics features

The linear waterway required for the bridges on the Sabarmati River as per CWC (1987) report is 176 m. The Subhash Bridge, Gandhi Bridge (and its widening), Nehru Bridge and Sardar Bridge (and its widening) are all designed for an estimated flood of 4 lakh cusec. Ellis Bridge (and its widening) is designed for an estimated flood of 5 lakh cusec. This data is based on the report of R & B (1989). The flood observed in 1973, prior to the construction of the Dharoi Dam was estimated at being 5 lakh cusec. However, no verification of this estimate is available. The Wasna barrage was non-existent in 1973. The flood magnitude for the event was approximately 4.75 lakh cusec. The reduced level for the bridges in this reach along with their corresponding Soffit levels and corresponding HFLs for this flood are reported in Table 1 (SRFDCL, 2004). As per the present status, the 'afflux bunds' or embankments constructed on both the banks are designed to provide protection against a 5 lakh cusec flood.

Table 1 HFL in relation to bridge levels in the case of a 4.75 lakh cusec flood.

Bridge	Bridge levels in m.		HFLs	
	Top of Bridge	Soffit of Bridge	Natural conditions	275m waterway
60 m Ring Road	58.585	55.523	54.37	55.67
Indira Bridge	58.809	55.828	52.38	53.69
Railway Bridge	55.470	53.094	50.45	50.29
Subhash Bridge	55.410	52.298	50.05	49.95

2.3 Scope of the Project

The SRFD project is envisaged to take up three additional new bridges in phased manner. The proposed bridges are: (i) two nos. at Ch. 1195 and 2058 m in the reach from *Subhsah* Bridge to *Gandhi* bridge, (ii) third at Ch. 8030 m in the reach from *Sardar* ridge to *Vasana* Barrage (refer Figure 1). On basis of the earlier studies by SRFDCL, the estimation of HFLs for a 5 lakhs cusec flood with the existing riverbank levels data showed a uniform bridge width of 275 m was optimal to achieve the objectives of the design. This means that the natural existing width has to be constricted to 275 m. Hence, the hydrological analysis has five focal parts: (i) computing the design flood at the upstream i.e. Narmada Main Canal; (ii) safe routing of the flood wave through constricted widths; (iii) describing the surface profile of this flood at different bridge sections and salient points of the reach including the backwater effect; (iv) optimal estimation of roughness coefficient (n) for the increased velocity of flow for the constricted sections; (v) computing the scour using the optimal parameters obtained thereof. On basis of the detailed analysis, recommendations have to be made for bridge sections, embankments along with the downstream protection measures.

3.0 Comments on the Report on Design Flood and Flood Profile

[Sabarmati River Front Development Scheme under Environmental Planning Collaborative (EPC) by M/s C.C. Patel & Associates Pvt. Ltd. A/34, GIDC, Electronics Estate Gandhinagar, Gujarat, 1999.]

3.1 General

The report presents the water surface profiles of Sabarmati River for the reach falling between Subhash Bridge and Wasna Barrage in Ahmedabad (Gujarat). The water surface profiles for the floods of 4 lakh cusec and 4.75 lakh cusec are computed using the popular HEC-2 computer package of Hydrologic Engineering Centre, USA, considering both natural and constricted river reach conditions. These profiles are derived for suggesting a river front development plan in such a way that the constriction of the river reach does not augment the hazards more than those under natural river condition. From the view-point of the practice usually followed in the country, the report correctly employs all the steps for deriving the water surface profiles. In the present work, the water surface profile have been computed using HEC-2 Program and these will be verified by application of the DAMBRK program.

3.2 Approach Adopted

3.2.1 Estimation of Design Flood

The design flood is calculated using two approaches: (i) unit hydrograph approach and (ii) flood frequency analysis (FFA). For the former the Sabarmati catchment up to NMC is divided into three sub-catchments as (a) Dharoi (5475 Sq. km), (b) Hathmati tributary (524 Sq. km), and (c) the balance up to NMC (4420 Sq. km). The former uses the 100 year routed flood from Dharoi and Hathmati reservoir along the catchment at (c) that would reach NMC. The following observations are noted in regard to design flood calculations:

(i) If the flood at Dharoi is impinged at 621 ft. (i.e. just at FRL which is 622 ft.), the corresponding routed flow is calculated as 4.55 lac cusec, however the detail computations for the same are not mentioned in the report.

(ii) For the Hathmati confluence with Sabarmati and the Narmada Main canal crossing, the Snyder's method is used for computing synthetic UH. The calculations at Annexure-3.3.2 shows; L = 106 km., and tp = 5.32 h. This means the average velocity of flow to be approximately L/tp = 5.5 m/s, which seems slight higher though the exact values can only be checked through experiment and modeling. In deriving the UH, it is advisable to follow the recommendations of CWC (1987) report to calculate the salient UH parameters.

(iii) For the statistical approach, it is mentioned that the annual peak series data is not available. A regional flood frequency analysis on the basis of the geomorphological features could have given a validation of the UH approach results.

3.2.2 Use of HEC-2 Package

It is of common knowledge that the HEC-2 program utilizes two fundamental equations for computation of water surface profiles utilizing standard step method: (1) Law of mass conservation, i.e. continuity equation and (2) Law of energy conservation, i.e. Bernoulli equation. In addition, the HEC-2 package considers one-dimensional flow. The two governing equations are as such an approximation of the Saint Venant's equations:

Continuity equation

$$\frac{\partial h}{\partial t}+u\frac{\partial h}{\partial x}+h\frac{\partial u}{\partial x}=0 \qquad (1)$$

Momentum equation

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+g\frac{\partial h}{\partial x}+g(S_f-S_0)=0 \qquad (2)$$

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where h is the depth of flow, u is the velocity of flow, g is the gravitational acceleration, S_f is the friction slope, S_o is the bed slope, and x and t are the space and time co-ordinates, respectively. If all the components of the momentum equation are considered, the resulting formulation is known as the dynamic wave model. It becomes a diffusion wave model if the first two terms of Eq. (2) are ignored, and it is kinematic wave model if the first three terms are ignored.

Since the reach-length for the study is about 22 km, and the peak discharge is assumed to stay for time greater than the time of travel between two considered sections of the channel, the variation of velocity with time can safely be ignored. In such cases the velocity (u) may be approximated to be in a steady flow regime. For this condition, the first term of Eq. (2) viz., $\partial u / \partial t$ can be neglected. However, the second term in Eq. (2) which relates to the inertial (or acceleration) component of the momentum equation, can only be ignored if the velocities are (reasonably) uniform. Here it is worth mentioning that the inclusion/exclusion of a component in/from the momentum equation depends on the relative importance of each term and the slope of the considered channel forms a major governing factor in such a decision. In case the change of *u* along the reach, i.e. $\partial u / \partial x$ and change of depth of flow along x, i.e. $\partial h / \partial x$ are considered to be small, the results (reported in the report) remain consistent with the formulation of the HEC-2 model for a unit width rectangular channel.

The average slope of the reach (study area) is very mild. Since the analysis of flow profiles in such channels require the use of full St. Venant's equation, i.e. all the components of Eq. (2), there might be some errors in profile estimations. However, if the stage-discharge relation can be adequately described by a steady state (no loop curve) stage-discharge relation, one gets a kinematic wave model and, therefore, the results using HEC-2 shall correctly depict the flow profiles. Since the report does not give any stage-discharge variations in the reach, the computations of water surface profile using HEC-2 program have to be verified with a dynamic wave routing model e.g. the NWS DAMBRK model.

For unsteady flow conditions, due to the presence of hysteresis in the rating curves, the peak stage occurs later than the peak discharge and the latter does not correspond to the discharge corresponding to the peak stage. Therefore, it is proposed to check the available stage-discharge curves for atleast two sections along the reach and to compute the water surface profile by using the NWS DAMBRK model. In addition to the above, the following are some additional suggestions that might improve the water surface elevation results.

(i) Since the longitudinal profile shows a great variation in slopes in different sub-reaches, a closer spacing of cross sections would also improve the results of surface profile computations using HEC-2.

(ii) To eliminate the effect of downstream boundary condition, i.e. the initial high flood level, the last sub-reach could have been hypothetically extended beyond the Wasna Barrage, or the actual downstream cross-sections considered in HEC-2 computations.

3.2.3 Estimation of Manning's coefficient (n)

The flow is mainly governed by the Manning's roughness coefficient (n), which depends on the nature of the channel surface and the slope of hydraulic gradient besides the river geometry. This hydraulic principle is followed in estimating water surface profiles for the given flood peaks. The values of n adopted for the analysis in the report are as follows.

Table 2 Roughness coefficient (Manning's n) used in Review report (1999)

n =	0.035 for channel section downstream of Wasna Barrage
	0.06 for over bank portions downstream of Wasna Barrage

Since the river having considerable depth of alluvium stratum experiences sub-surface scour flow during high flood occurrence, sub-surface scour area that augments the channel-flow area needs to be considered reasonably based on the scour-depth analysis. Looking at the sensitivity of n-parameter on flood computation and profile description, a justification for n-values would be checked by sensitivity analysis. The n-values given by Chow (1959) in Table 5-6 of the book are as follows.

Table 3 Roughness coefficient (Manning's n) given by Chow (1959) for major streams having top width > 100 ft

Type of channel	Roughness coefficient	
	Minimum	Maximum
Regular sections with no boulders	0.025	0.06
Irregular and rough sections	0.035	0.10

The normal 'n' recommended by Chow (1959) is 0.03, and for flood banks with heavy stand of timbers with flood reaching below branches (as the features prevail in the study area), n may vary between 0.08-0.2. To validate the adopted values of Manning's 'n', a sensitivity analysis of Manning's 'n' may have to be carried out.

3.2.4 Consideration of scour in HEC-2 computations

In alluvial Sabarmati reach, scour depths are considered to be of 1 and 2 m under natural and constricted river reach conditions, respectively, and their variation assumed to be parabolic across the cross-section, the maximum depth of scour at the center of the reach and zero at the banks. Firstly, the shape of the cross-section assumed by the section in a regime condition is semi-elliptical rather than parabolic. Secondly, the regular variation of scour depth across the cross-section forces the flow to occupy larger space in the inactive zone than required. The implication is to reduce the computed flow depths at various cross-sections for a given discharge. The permissible velocities for erodible channels which scours but don't silt are given by Chow (1959) in Table 7.3 as 3.75 f/s. This figure or recommendations of the Indian standard codes need a reference while analyzing this aspect.

3.2.5 Comments on the Conclusions of the Report

From the recommendations of the review report, it is felt that the following points should be incorporated:

1. In regard to 100 year return period of flood, which is calculated at 5.25 lakh cusecs, the detailed comment has been given in the design flood section at 3.2.1. In addition a 10% reduction factor for computing the design flood has been computed based on simultaneous maximum rainfall which may not occur over the entire catchment and a reference of clause 7.3 of IS 7784 (Part 1): 1993 for giving weightage to the observed data is made.
2. 2. In regard to point (5), the structures/channel cross-sections should be designed for 4 lakh cusecs flood with a check flood of 4.75 lakh cusecs. However, the ongoing works are designed for 4.75 lakh cusecs.

3. In regard to point (8), the detail calculations for scour is not described in the report. It should have been considered in the HEC-2 while calculating the surface profiles. Similarly, siltation aspect is also not discussed in the report. It should have been described in detail while computing scouring. However, scour during high flood will give lower water surface elevation and it is a conservative approach.

4. It might be a good idea to estimate higher floods that might occur in the future, and flood plane zoning (inundation maps for Ahmedabad city) may be carried out based on this value.

3.2.6 Comments on the recommendations

The structure should be designed for a design flood and additional free boards should be adequately provided as per the existing standard recommendations. The scour and siltation should be computed for certain salient points in the channels for different velocities. As mentioned above, this could have been taken care of while running the HEC model.

However, the scour during high floods will yield lower water surface elevation and it is a conservative approach. A gauge and discharge site may be planned on Sabarmati river at Narmada main canal crossing upstream of the city. This shall ensure that the river flow can be measured without any backwater effect due to storage at Wasna barrage and data collected would be of immense use in future.

3.2.7 Summary

The Sabarmati River Front Development Scheme is not a flood control scheme. However, the hydraulic analysis is required for analyzing the post-project hydraulics conditions like surface profiles at different cross-sections, the possibility of flood inundation, ,safe carrying capacity of the channel and deriving a flood plane zoning for the area. This is needed so as to ensure that the project does not significantly aggravate the hydraulic and environmental conditions that are existing in the pre-project case. The Sabarmati River Front Development Corporation Ltd. (SRFDCL) under Environmental Planning Collaborative (EPC) was assigned the task for preparing a conceptual proposal for the development of the river bank areas within the municipal limits of Ahmedabad. The SRFDCL assigned the detail hydraulics study to M/s C.C. Patel & Associates who completed submitted the report in 1999. The report presents the water surface profiles of Sabarmati River for the reach falling between *Subhash* Bridge and *Vasna* Barrage in Ahmedabad (Gujarat). The report computes the following hydrological variables for this reach: (i) design flood on basis of a 100 year recurrence return period 1 day PMP, considering the maximum flow from two reservoirs upstream of the reach and routing the flood along the balance channel reach; (ii) the water surface profiles for the floods of 4 lac cusecs and 4.75 lac cusecs (taking the 1973 flood event) are computed using the HEC-2 computer package of Hydrologic Engineering Centre, USA, considering both natural and constricted river reach (275 m) conditions. These profiles are derived for suggesting a river front development plan in such a way that the constriction of the river reach does not augment the hazards more than those under natural river condition; (iii) the maximum depth that could possibly be encountered under natural and constricted river reach conditions, respectively, and their variation assuming a parabolic cross-section.

Ahmedabad Municipal Corporation (AMC) has approached the National Institute of Hydrology (NIH) to review the SRFDP report results in regard to the flood behavior due to constricting of the river width for different discharges as worked out from the data collected and as suggested by different experts. This review report forms the first phase of this consultancy.

The findings of the review report can be summarized as follows:

1. The calculation of 100 year return period flood and the subsequent 10% reduction factor for computing the design flood is based on the argument that the simultaneous maximum rainfall may not occur over the entire catchment. The study by M/s C.C. Patel Associates was completed in the year 1999. Subsequently, there have many developments in the catchments up stream of Ahmedabad and due to these; the utilization of water has increased. A reference of clause 7.3 of IS 7784 (Part 1): 1993 for giving weightage to the observed data is made.

2. As explained above, the design flood considered is 4.75 lakh cusecs instead of estimated design flood of 5.25 lakh cusec.

3. In the later part of the report it is mentioned that the structures/channel cross-sections should be designed for 4 lakh cusecs of flood with a check flood of 4.75 lakh cusecs. However the ongoing works are designed for 4.75 lakh cusecs flood.

4. The detail calculations for finding the maximum possible scour depth is not described in the report, the same reason holds for the siltation aspect. However the scour during high flood will give lower water surface elevation which implies higher safety of the structure.

5. The Manning's roughness parameter - n used for computation of water surface profile has important influence on computed water surface elevation. Hence it is required to check it's sensitivity by sensitivity analysis.

The following are the recommendations of this report:

(i) A gauge and discharge site may be planned on Sabarmati River at Narmada main canal crossing upstream of the city. This shall ensure that the flow can be measured without the backwater effect due to storage at *Vasana* Barrage.

(ii) As the HEC 2 model has certain limitations when applied for channels with a mild slope, a suitable dynamic wave routing model with routing option such as the NWS DAMBRK model might yield more representative water surface profiles.

(iii) The adaptation of n values for profile calculations needs to be checked by sensitivity analysis.

(iv) Flood plane zoning may be done in future as a part of disaster management.

4.0 Flood Profile

[Sabarmati River Front Development Scheme under Environmental Planning Collaborative (EPC) by M/s C.C. Patel & Associates Pvt. Ltd. A/34, GIDC, Electronics Estate Gandhinagar, Gujarat, 1999].

4.1 General

The report presents the water surface profiles of Sabarmati River for the reach falling between Subhash Bridge and Vasana Barrage in Ahmedabad (Gujarat). The water surface profiles for the floods of 4 lakh cusec and 4.75 lakh cusec are computed using the popular HEC-2 computer package of Hydrologic Engineering Centre, USA, considering both natural and constricted river reach conditions. These profiles are derived for suggesting a river front development plan in such a way that the constriction of the river reach does not augment the hazards more than those under natural river condition. From the view-point of the practice usually followed in the country, the report correctly employs all the steps for deriving the water surface profiles. However, since a model can only mimic the complex natural conditions, the computation of water surface profiles using HEC-2 program is also subject to certain limitations. Since the derivation of the design flood is beyond the scope of the present work, the report is restricted to the derivation of water surface profiles for the data made available by the agency.

4.2. Basis of the Surface profile calculations by M/S C. C. Patel & Associates Pvt. Ltd. in (March-1999)

The finding in the report of M/S C. C. Patel & Associates Pvt. Ltd. (1999) in regard to computation of surface profiles was based on the following observations:

1. In 1973, the Vasana barrage was not in existence at Ahmedabad city, and therefore, there was no downstream control where the discharge passing through the river could have been accurately measured. Similarly, Dharoi dam was also not constructed, rather the work of construction was in progress. Therefore, there was no upstream control for measurement of actual discharge passing through river Sabarmati. The report of M/S C. C. Patel & Associates Pvt. Ltd. was primarily based on the flood level observed at Subhash Bridge in the year 1973.

2. Based on the observed flood levels at Subhash Bridge in the year 1973, the flood discharge predicted by M/S C. C. Patel & Associates Pvt. Ltd. was 4.0 lakh cusecs.

4.3 Recent floods observed in the month of August-2006 at Ahmedabad

In the recent flood of August-2006, simultaneous gauging and flood measurements were taken at Subhash Bridge and Vasana Barrage lying within a reach of 9 km. The real time data of Vasana Barrage and flood gauging at Subhash Bridge was recorded by AMC for four subsequent floods in August-2006, and these are reported in Table 4. To summarize, the observed maximum flood at Vasana Barrage during this event occurred on 17th August 2006. The magnitude of this flood was 3.10 lakh cusecs and the corresponding gauge was 42.160m. The concurrent gauge level at 9.0 km upstream at Subhash Bridge was 47.48m. The present study makes use of these observations for working out the hydraulic gradient and also for estimating approximately the value of Manning's roughness "n" of this reach.

5.0 Methodology for the Flood Profile Computation

The observed flood of 3.1 lakh cusec was simulated using the HEC-2 model considering the constricted conditions, i.e. taking into account the ongoing construction work and already constructed diaphragm wall and lower promenade and filling on the over banks which has been carried out up to RL 42.40m from the railway bridge to the Sardar Bridge. This accounts for a reach of approximately 7.5 km out of total reach of 9 km. Simulation of the flood shows that substantial scouring may take place in the jacketed reach (constricted width between diaphragms). The Manning's roughness "n" values for both banks and channel were computed by matching the (available) water levels observed at the *Vasana* Barrage during the Aug. 2006 flood event. The n-values computed for this data were taken as a guide for water profile computations for 4.00 and 4.75 lakh cusec floods.

Table 4 Observed flood data of Sabarmati River at Ahmedabad in August-2006.

Time in hrs. starting on 19-8-2006	Dharoi				Subhash Bridge		Vasana Barrage		
	Inflow (cusec)	Outflow (cusec)	Current Level (ft)	High Flood (ft)	Depth of flow (ft)	Reduced Level (m)	Outflow (cusec)	Reduced Level (ft)	Reduced Level (m)
00.00 am	45260	59982	619.00	622.006	0.00	41.08	25216	127.00	38.71
01.00 am	57204	59982	618.99	622.006	0.00	41.08	25216	127.00	38.71
02.00 am	74704	69982	619.05	622.006	0.00	41.08	25216	127.00	38.71
03.00 am	88871	74755	619.15	622.006	0.00	41.08	25216	127.00	38.71
04.00 am	103922	74755	619.25	622.006	0.00	41.08	25216	127.00	38.71
05.00 am	100866	74755	619.34	622.006	0.00	41.08	25216	127.00	38.71
06.00 am	95033	74755	619.41	622.006	0.00	41.08	25216	127.00	38.71
07.00 am	103644	74755	619.51	622.006	0.00	41.08	25216	127.00	38.71
08.00 am	112533	75350	619.64	622.006	0.00	41.08	25216	127.00	38.71
09.00 am	124722	100000	619.76	622.006	0.00	41.08	25216	127.00	38.71
10.00 am	156111	125000	619.91	622.006	0.00	41.08	25216	127.00	38.71
11.00 am	172222	125000	620.07	622.006	2.62	41.88	25216	127.00	38.71
12.00 pm	189166	150000	620.20	622.006	5.90	42.88	44016	128.00	39.01
01.00 pm	207777	200000	620.27	622.006	10.23	44.20	76304	129.75	39.55
02.00 pm	215000	225000	620.35	622.006	12.00	44.74	121402	131.75	40.16
03.00 pm	253611	250000	620.37	622.006	13.02	45.04	140896	132.50	40.39
04.00 pm	228890	200000	620.37	622.006	13.84	45.30	167126	133.50	40.69
05.00 pm	209166	200000	620.40	622.006	14.40	45.47	167126	133.50	40.69
06.00 pm	269000	225000	620.63	622.006	14.40	45.48	173182	133.75	40.77
07.00 pm	261944	250000	620.80	622.006	14.76	45.58	180182	134.00	40.84
08.00 pm	261944	250000	620.80	622.006	15.00	45.65	180844	134.00	40.84
09.00 pm	228889	250000	620.73	622.006	15.30	45.75	187073	134.25	40.92
10.00 pm	120555	250000	620.30	622.006	16.47	46.10	205598	134.75	41.07
11.00 pm	109722	200000	620.00	622.006	17.50	46.40	236120	135.75	41.38
00.00 am	115833	200000	619.71	622.006	18.43	46.70	254648	136.50	41.61
01.00 am	122222	150000	619.58	622.006	19.25	46.95	268042	137.00	41.76
02.00 am	87777	50000	619.71	622.006	19.58	47.05	277666	137.25	41.83
03.00 am	108055	100000	619.91	622.006	19.91	47.15	286478	137.50	41.91
04.00 am	135277	100000	620.03	622.006	20.25	47.25	293900	137.75	41.99
05.00 am	172222	150000	620.27	622.006	20.73	47.40	293900	137.75	41.99
06.00 am	137777	150000	620.23	622.006	20.73	47.40	310432	138.25	42.14
07.00 am	219444	150000	620.46	622.006	20.73	47.40	**310438**	**138.25**	**42.14**
08.00 am	231950	200000	620.65	622.006	20.73	47.40	310432	138.25	42.14
09.00 am	219444	225000	620.46	622.006	20.07	47.20	302860	138.00	42.06
10.00 am	185800	225000	620.33	622.006	19.25	46.95	293900	137.75	41.99
11.00 am	93000	225000	619.89	622.006	18.04	46.58	261752	136.75	41.68
12.00 pm	101000	200000	619.55	622.006	16.47	46.10	246136	136.25	41.53
01.00 pm	121670	200000	619.28	622.006	15.81	45.90	214164	135.00	41.15
02.00 pm	87800	200000	618.89	622.006	14.83	45.60	199748	134.50	41.00
03.00 pm	87800	175000	618.65	622.006	14.76	45.58	180844	134.00	40.84
04.00 pm	87800	150000	618.30	622.006	15.25	45.73	180844	134.00	40.84

05.00 pm	93900	150000	618.17	622.006	16.07	45.98	199748	134.50	41.00
06.00 pm	93900	150000	618.00	622.006	17.05	46.28	205598	134.75	41.07
07.00 pm	74440	150000	617.74	622.006	17.72	46.48	214164	135.00	41.15
08.00 pm	44722	100000	617.55	622.006	18.53	46.73	236120	135.75	41.38
09.00 pm	38890	100000	617.34	622.006	18.90	46.83	246136	136.25	41.53
10.00 pm	40277	75000	617.22	622.006	19.03	46.88	254648	136.50	41.61
11.00 pm	34445	75000	617.08	622.006	19.03	46.88	254648	136.50	41.61
00.00 am	32500	50000	617.02	622.006	18.70	46.78	246136	136.25	41.53
01.00 am	78888	50000	617.12	622.006	18.37	46.68	246136	136.25	41.53
02.00 am	113888	75000	617.34	622.006	18.04	46.58	239204	136.00	41.45
03.00 am	104166	75000	617.44	622.006	17.38	46.38	239204	136.00	41.45
04.00 am	95278	75000	617.51	622.006	17.06	46.28	229024	135.50	41.30
05.00 am	83888	75000	617.54	622.006	16.73	46.18	214164	135.00	41.15
06.00 am	83000	75000	617.54	622.006	16.67	46.00	199748	134.50	41.00
07.00 am	75000	75000	617.54	622.006	15.74	45.80	180844	134.00	40.84
08.00 am	75000	75000	617.54	622.006	15.25	45.73	180000	134.00	40.84
09.00 am	57500	75000	617.48	622.006	14.43	45.48	173000	133.75	40.77
10.00 am	54700	75000	617.42	622.006	13.45	45.18	167126	133.50	40.69
11.00 am	54700	75000	617.33	622.006	12.63	44.93	153786	133.00	40.54
12.00 pm	28700	50000	617.17	622.006	11.81	44.68	140896	132.50	40.39
01.00 pm	34900	30000	617.15	622.006	11.22	44.50	128366	132.00	40.23
02.00 pm	41600	40000	617.00	622.006	10.82	44.30	128366	132.00	40.23
03.00 pm	60300	50000	617.25	622.006	10.82	44.37	121402	131.75	40.16
04.00 pm	58900	50000	617.28	622.006	11.31	44.53	121402	131.75	40.16

5.1 Use of HEC-2 Package

As explained in the earlier part of the report, the HEC-2 package considers one-dimensional flow and utilizes two fundamental equations for computation of water surface profiles using the Standard Step method: (1) Law of mass conservation, i.e. continuity equation and (2) Law of energy conservation, i.e. Bernoulli equation. The two governing equations are as such an approximation of the Saint Venant's equations. Since the reach-length for the study is about 9 km, and the peak discharge is assumed to stay for time greater than the time of travel between two considered sections of the channel, the variation of velocity with time can be safely ignored. In such cases the velocity (u) may be approximated to be in a steady flow regime. For this condition, the first term of Eq. (2) viz., $\partial u / \partial t$ can be neglected. However, the second term in Eq. (2) which relates to the inertial (or acceleration) component of the momentum equation can be ignored only if the velocities are (reasonably) uniform, and it can be checked analyzing the flood profile computations with the same data using DAMBRK or any other model containing hydro-dynamic module and it is discussed later in this report.

5.2 Estimation of Manning's coefficient (n) using HEC-2

The flow in a natural open channel is largely governed by the Manning's roughness coefficient (n), which depends on the nature of the channel surface and the slope of hydraulic gradient besides the river geometry. This hydraulic principle is followed in estimating water surface profiles for the given flood peaks. The initial values of n adopted for this analysis is taken from the recommendations of Chow (1959), which are given in Table 2 and 3.

Since the river bed having considerable depth of alluvium stratum experiences scour during high floods, the augmented scour area to the channel flow area needs to be considered reasonably based on the scour-depth analysis. This aspect is however difficult to incorporate in HEC-2 hydraulically, specifically the variation of Manning's n with increasing scour with the quantity of flow. In a simplified way, the effect is analyzed by carrying out a sensitivity analysis of parameter-n on flood profile computations considering 1 m scour depth throughout the cross-section and all along the considered reach. To this end, the flood of 3.1 lakhs cusec was simulated with n varying in the range of 0.022 to 0.05 for channel bed and a constant value for banks, and the flood profile computed. Table 5(a) and Figure 3 show the results of sensitivity of n (channel), designated as n_c. It is seen that the variation of n_c has a significant bearing on the computed depth of flow. The larger the n_c-value, the larger the depth of flow, and vice versa.

Table 5(a) Results of sensitivity analysis of n (channel).

Sl.	n_c	Depth (m)	Sl.	n_c	Depth (m)	Sl.	n_c	Depth (m)	Sl.	n_c	Depth (m)
1	0.0236	50.87	51	0.032	51.41	101	0.0421	52.26	151	0.047	52.86
2	0.0237	50.88	52	0.0321	51.42	102	0.0422	52.27	152	0.047	52.88
3	0.0239	50.89	53	0.0323	51.43	103	0.0423	52.28	153	0.047	52.89
4	0.0239	50.89	54	0.0324	51.44	104	0.0424	52.29	154	0.047	52.91
5	0.0244	50.9	55	0.0325	51.45	105	0.0425	52.31	155	0.048	52.92
6	0.0247	50.91	56	0.0326	51.46	106	0.0426	52.32	156	0.048	52.93
7	0.0248	50.94	57	0.0327	51.47	107	0.0427	52.33	157	0.048	52.95
8	0.025	50.95	58	0.0329	51.48	108	0.0428	52.34	158	0.048	52.96
9	0.0253	50.96	59	0.033	51.49	109	0.0429	52.35	159	0.048	52.98
10	0.0255	50.97	60	0.0331	51.5	110	0.043	52.36	160	0.048	52.97
11	0.0262	50.98	61	0.0332	51.51	111	0.0431	52.37	161	0.048	52.98
12	0.0264	50.99	62	0.0333	51.52	112	0.0432	52.38	162	0.048	52.99
13	0.0266	51.0	63	0.0335	51.53	113	0.0433	52.39	163	0.048	53.01
14	0.0268	51.02	64	0.0336	51.54	114	0.0434	52.4	164	0.048	53.02
15	0.0269	51.04	65	0.0337	51.55	115	0.0435	52.41	165	0.049	53.03
16	0.0271	51.05	66	0.0338	51.56	116	0.0436	52.42	166	0.049	53.04
17	0.0272	51.06	67	0.0339	51.57	117	0.0437	52.44	167	0.049	53.06
18	0.0273	51.07	68	0.034	51.58	118	0.0438	52.45	168	0.049	53.07
19	0.0274	51.08	69	0.0342	51.59	119	0.0439	52.46	169	0.049	53.08
20	0.0276	51.09	70	0.0343	51.6	120	0.044	52.47	170	0.049	53.09
21	0.0277	51.1	71	0.0344	51.61	121	0.0441	52.48	171	0.049	53.11
22	0.0278	51.11	72	0.0345	51.62	122	0.0442	52.49	172	0.049	53.12

23	0.028	51.12	73	0.0346	51.63	123	0.0443	52.5	173	0.049	53.13
24	0.0281	51.13	74	0.0347	51.64	124	0.0444	52.51	174	0.049	53.14
25	0.0282	51.14	75	0.0349	51.65	125	0.0445	52.53	175	0.05	53.16
26	0.0286	51.15	76	0.0396	51.99	126	0.0446	52.54	176	0.05	53.17
27	0.0287	51.16	77	0.0397	52.0	127	0.0447	52.55	177	0.05	53.18
28	0.0289	51.17	78	0.0398	52.01	128	0.0448	52.56	178	0.05	53.19
29	0.029	51.18	79	0.0399	52.02	129	0.0449	52.57	179	0.05	53.21
30	0.0292	51.19	80	0.04	52.03	130	0.045	52.58			
31	0.0293	51.2	81	0.0401	52.04	131	0.0451	52.59			
32	0.0294	51.21	82	0.0402	52.05	132	0.0452	52.6			
33	0.0296	51.22	83	0.0403	52.06	133	0.0453	52.61			
34	0.0297	51.23	84	0.0404	52.07	134	0.0454	52.63			
35	0.0298	51.24	85	0.0405	52.08	135	0.0455	52.64			
36	0.03	51.25	86	0.0406	52.1	136	0.0456	52.65			
37	0.0301	51.26	87	0.0407	52.11	137	0.0457	52.67			
38	0.0303	51.27	88	0.0408	52.12	138	0.0458	52.68			
39	0.0305	51.29	89	0.0409	52.13	139	0.0459	52.69			
40	0.0306	51.3	90	0.041	52.14	140	0.046	52.7			
41	0.0308	51.31	91	0.0411	52.15	141	0.0461	52.72			
42	0.0309	51.32	92	0.0412	52.16	142	0.0462	52.73			
43	0.031	51.33	93	0.0413	52.17	143	0.0463	52.74			
44	0.0311	51.34	94	0.0414	52.18	144	0.0464	52.76			
45	0.0313	51.35	95	0.0415	52.19	145	0.0465	52.77			
46	0.0314	51.36	96	0.0416	52.21	146	0.0466	52.78			
47	0.0315	51.37	97	0.0417	52.22	147	0.0467	52.79			
48	0.0316	51.38	98	0.0418	52.23	148	0.0468	52.81			
49	0.0318	51.39	99	0.0419	52.24	149	0.0469	52.82			
50	0.0319	51.4	100	0.042	52.25	150	0.047	52.83			

Note: n_c is the Manning's coefficient for the river channel.

Table 5(b) Results of sensitivity analysis of n_{b1} (left bank) and n_{b2} (right bank).

Sl	n_{b1}	n_{b1}	n_c	**Depth (m)**	**Sl.**	n_{b1}	n_{b1}	n_c	**Depth (m)**
1	0.001	0.001	0.0225	51.31	6	0.031	0.031	0.0225	51.31
	0.001	0.001	0.03	51.25		0.031	0.031	0.03	51.25
	0.001	0.001	0.02	51.29		0.031	0.031	0.02	51.29
	0.001	0.001	0.022	51.3		0.031	0.031	0.022	51.3
	0.001	0.001	0.025	50.95		0.031	0.031	0.025	50.95
2	0.03	0.03	0.0225	51.31	7	0.003	0.003	0.0225	51.31
	0.03	0.03	0.03	51.25		0.003	0.003	0.03	51.25
	0.03	0.03	0.02	51.29		0.003	0.003	0.02	51.29
	0.03	0.03	0.022	51.3		0.003	0.003	0.022	51.3
	0.03	0.03	0.025	50.95		0.003	0.003	0.025	50.95
3	0.0225	0.0225	0.0225	51.31	8	0.11	0.11	0.0225	51.31
	0.0225	0.0225	0.03	51.25		0.11	0.11	0.03	51.25
	0.0225	0.0225	0.02	51.29		0.11	0.11	0.02	51.29
	0.0225	0.0225	0.022	51.3		0.11	0.11	0.022	51.3
	0.0225	0.0225	0.025	50.95		0.11	0.11	0.025	50.95
4	0.035	0.035	0.0225	51.31	9	0.55	0.55	0.0225	51.31

	0.035	0.035	0.03	51.25		0.55	0.55	0.03	51.25
	0.035	0.035	0.02	51.29		0.55	0.55	0.02	51.29
	0.035	0.035	0.022	51.3		0.55	0.55	0.022	51.3
	0.035	0.035	0.025	50.95		0.55	0.55	0.025	50.95
5	0.033	0.033	0.0225	51.31	10	0.99	0.99	0.0225	51.31
	0.033	0.033	0.03	51.25		0.99	0.99	0.03	51.25
	0.033	0.033	0.02	51.29		0.99	0.99	0.02	51.29
	0.033	0.033	0.022	51.3		0.99	0.99	0.022	51.3
	0.033	0.033	0.025	50.95		0.99	0.99	0.025	50.95

Note: n_{bi} is the Manning's coefficientfor the river banks, i= 1 and 2 are for left and right banks, respectively.

A similar test was carried out to study the sensitivity of n for left (n_{b1}) and right (n_{b2}) banks on the computed depth of flow. The normal n-value recommended by Chow (1959) for flood banks with heavy stand of timbers with flood reaching below branches (as these features dominates the study reach) is in the range of 0.03-0.6. The range of simulated n for this test was kept in between 0.01 and 1, and the results are shown in Table 5(b) and Figure 4. The results show that fluctuations in the depth are largely unaffected by varying n (bank). It is largely due to the fact that the lower velocities of flow in the vicinity of river banks have lower contribution to the total discharge passing through the whole cross-section. In other words, the flow contribution from the cross-sectional area near banks is comparatively much smaller than that from the middle, i.e. channel, portion of the cross-section. With these results in background, the n-values are calibrated for the year 2006 event using HEC-2 model as discussed in the following section.

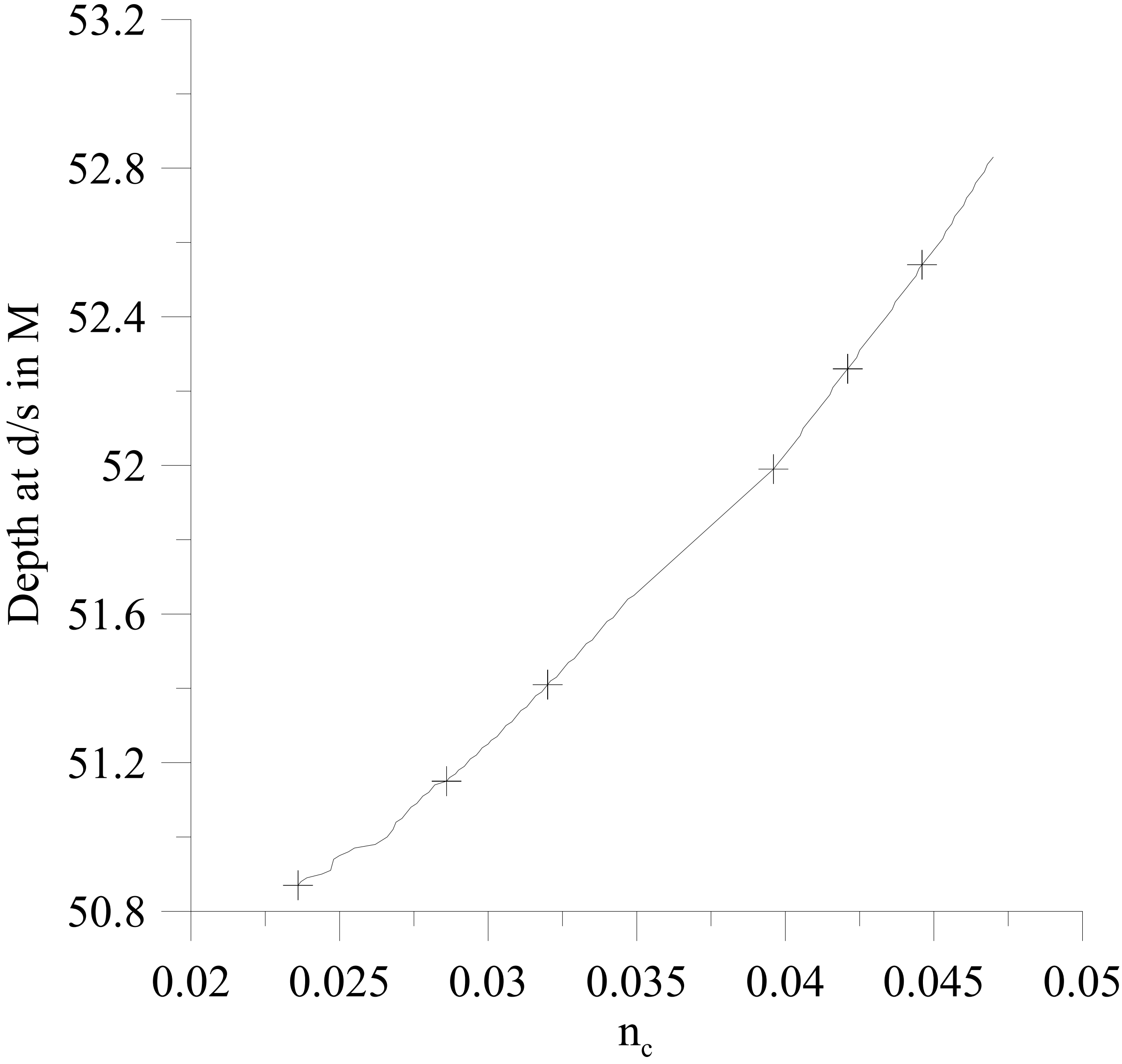

Figure 3 Sensitivity of n (channel).

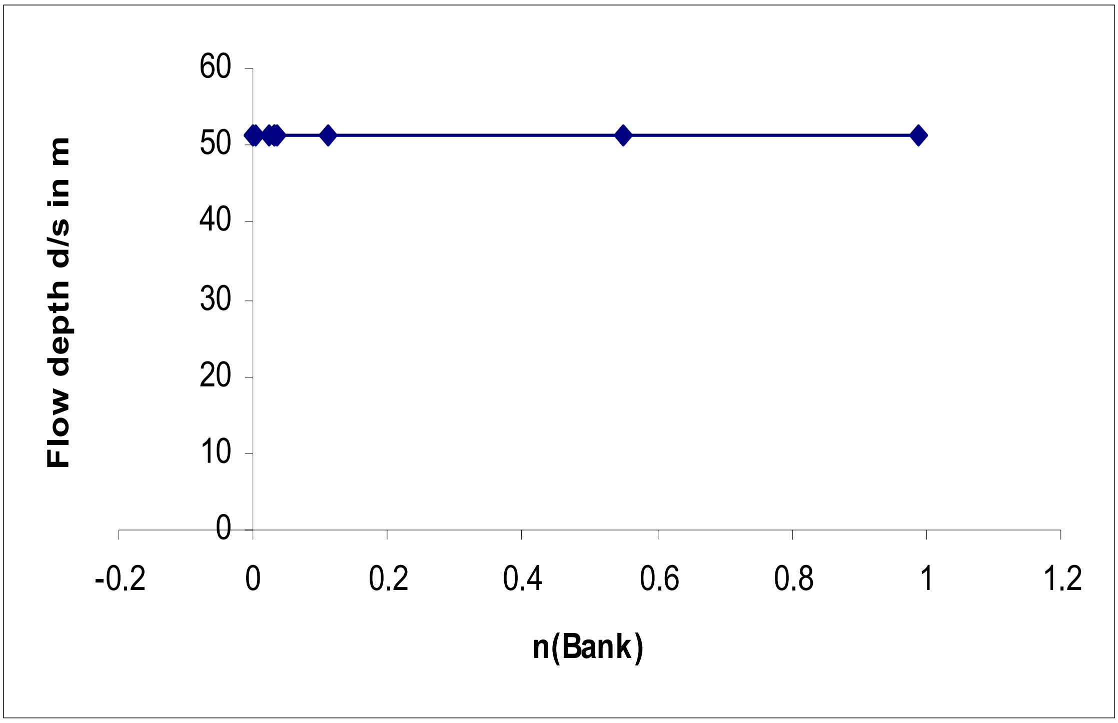

Figure 4 Sensitivity of n (Bank).

5.3 HEC-2 application to 2006 observed flood data

The methods briefly followed for this analysis are as follows:

1. Using the data of Aug-2006 event (peak of 3.1 lakh cusec), HEC-2 model was used to compute the flow profile for the reach. For calibration of 'n', the flow depth at Vasana Barrage is cross-checked. The conditions are considered to be valid for a constricted channel since the embankment construction for a 7 km reach was complete by Aug. 2006.
2. The calibrated 'n' is used for the 4.00 and 4.75 lakh cusec flood and the flow depth is computed at specific points along the reach.

Based on the observed flood event data of August-2006 (Table 4), the optimum values of Manning's coefficient for which the flow profile matched reasonably are given as follows: Peak flow = 8787 cumec (3.1 lakh cusec), river water level = 47.4 m at Vasana Barrage. Notably, the level computed closely matches the observed level. Here, the n-values used were as follows: n_b (left) = 0.026, n_b (right) = 0.026 and n_c = 0.021. Taking these values as a guide, the flow profiles were simulated for 4.00 and 4.75 lakh cusec floods considering the jacketed (or constricted) reach of Sabarmati River. The results of the present analysis and that of M/S C. C. Patel & Associates Pvt. Ltd are reported in Tables 6 (a) and (b). It is observed from the Tables that the results of M/S C. C. Patel which relied on 4.75 lakh cusec event (approximate without any record) and that must have occurred in natural reach conditions shows flood levels higher than those derived from the present analysis. Since the present analysis relies on observed data of a recent (2006) flood event with constricted conditions, this method has the efficacy to predict the future flow profiles of constricted channel better than the inputs used earlier.

Table 6(a) Results of flow depths for 4.00 lakh cusec (constricted condition)

Bridge location	M/S C. C. Patel & Associates report Constricted Condition	M/S C. C. Patel & Associates report Natural Condition	Simulated results based on 2006 flood event
Vasana barrage	42.92 m	42.92 m	43.17 m
Sardar bridge	47.04 m	46.87 m	46.20 m
Ellis bridge	47.77 m	47.10 m	46.89 m
Nehru bridge	48.12 m	47.38 m	47.20 m
Gandhi bridge	49.11 m	48.31 m	48.25 m
Subhash bridge	50.05 m	49.61 m	49.15 m

Table 6(b) Results of flow depths for 4.75 lakh cusecs flood (constricted condition)

Bridge location	M/S C. C Patel & Associates report (Constricted Condition)	M/S C. C. Patel & Associates report Natural Condition	Simulated results based on calibration of 2006 flood event
Vasana barrage	43.44 m	43.44 m	43.81 m
Sardar bridge	48.56 m	48.06 m	47.21 m
Ellis bridge	49.21 m	48.24 m	47.93 m
Nehru bridge	49.56 m	48.51 m	48.29 m
Gandhi bridge	50.44 m	49.56 m	49.41 m
Subhash bridge	51.34 m	50.79 m	50.31 m

Here it is worth mentioning that the HEC-2 model has its own limitations in flow profile computations. For example, inclusion/exclusion of a component in/from the momentum equation depends on the relative importance of each term and the slope of the considered channel forms a major governing factor in such a decision. The model provides consistent results for cases where the change of velocity along the reach and change of depth of flow are small. For the study area, the average slope of the reach is in the order of 1/2300 which can be described as a very mild slope. Since the analysis of flow profiles in such channels require the use of full St. Venant's equation, i.e. all the components of Eq. (2), there might be some errors in profile estimations. However, if the stage-discharge relation can be adequately described by a steady state (no loop curve) stage-discharge relation, one gets a kinematic wave model and for this situation, HEC-2 shall correctly depict the flow profiles. The observed data of the year 2006 flood event (Table 4) was used to plot the stage-discharge curve as shown in Figure 5. The rating curve as such does not show any hysteresis or loop, and therefore, the results of HEC-2 model are equally reliable. However, to further check the reliability of the HEC-2 results, NWS DAMBRK model was used with its routing option to derive the maximum water surface elevations attained by different floods at various locations of the considered study reach.

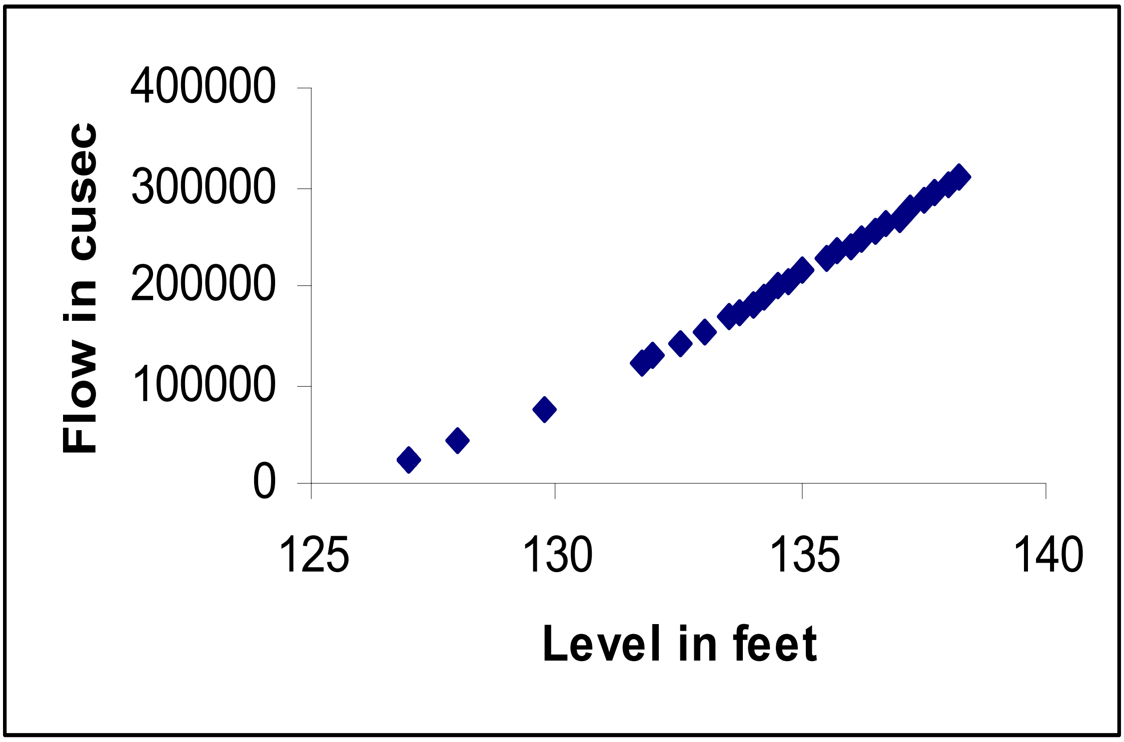

Figure 5 Stage-discharge curve at Vasana Barrage for the 2006 flood event.

5.4 Computation of water surface profile using DAMBRK Model

The National Weather Service (1981) dam break flood forecasting model (NWS-DAMBRK) uses the weighted 4-point finite difference implicit (or Preissmann) scheme for the solution of the St. Venant's equations. In order to cope with stability, convergence, and for other reasons, the stage-discharge relationship, a frequently used downstream boundary condition, is expressed in terms of Manning's equation to reproduce the above-described hysteresis effect. Using the NWS DAMBRK model flood profiles for the considered reach are computed for various flood discharges considering the scour depth of 1 m in the river bed, and n-values varying with depth of flow, as follows.

1. The profile for 3.1 lakh cusec flood was computed for 3 cross-sectional average n-values, viz., 0.020, 0.022, and 0.025 and 1 m scour throughout the cross-section and reach. The results showed a variation of 0.95 m depth of flow at Vasna barrage when computed with maximum and minimum n-values. It indicates that the consideration of a cross-sectional average n-value in the analysis can significantly affect the flow depths and, therefore, would involve (subjective) judgment.

2. Therefore, the above results were matched closely with those derived considering more reasonable n-values that vary with the depth of flow and these were as follows: 0.03, 0.025, 0.022, 0.018, 0.018, 0.018, starting from river bed level to the top of river banks.

3. The above pattern of n-values varying with depth of flow were considered for deriving water surface profiles for 4.0 and 4.75 lakh cusec floods. These values were further refined as: 0.032, 0.03, 0.027, 0.022, 0.021, 0.021, starting from river bed to top of banks. These values took into account the recommendations of Chow (1959) (Table 3) and the above sensitivity analysis. In addition, the higher values provide an additional factor of safety to the hydraulic structures. The results obtained with these n-values, with 1 m scour throughout the cross-section and all along the reach, and with constricted (275 m river width) of the river, are shown in Tables 7 (a) and (b) for 4.0 and 4.75 lakh cusec floods, respectively. Apparently, the maximum depths of flow attained for both the floods, viz., 4.0 and 4.75 lakh cusec, are little higher than those presented in the recent report, but generally significantly lower than those presented by M/s C.C. Patel & Associates. Thus, the DAMBRK results not only verify the results of the present report but also enhance the level of confidence with respect to the safety of hydraulic structures.

6.0 Summary of Flood Profile

The Sabarmati River Front Development Scheme is not a flood control scheme. However, the hydraulic analysis is required for analyzing the post-project hydraulic conditions like surface profiles at different cross-sections, the possibility of flood inundation, safe carrying capacity of the channel, and deriving a flood plain zoning for the area. This is needed to ensure that the project does not significantly aggravate the hydraulic and environmental conditions that exist in the pre-project conditions.

Table 7(a) Results of flow depths for 4.0 lakh cusec floods considering constricted conditions along with 1 m river bed scour in DAMBRK application

Bridge location	M/S C. C. Patel & Associates report (4.00 lakh cusec (constricted condition)	HEC-2 analysis based on observed flood (4.00 lakh cusecs (constricted condition)	DAMBRK analysis with varying n-value and considering 1 m river bed scour
Vasana barrage	42.92 m	43.17 m	43.21 m
Sardar bridge	47.04 m	46.20 m	45.55 m
Ellis bridge	47.77 m	46.89 m	46.61 m
Nehru bridge	48.12 m	47.20 m	46.92 m
Gandhi bridge	49.11 m	48.25 m	47.58 m
Subhash bridge	50.05 m	49.15 m	49.20 m

Table 7(b) Results of flow depths for 4.75 lakh cusec floods considering constricted conditions along with 1 m river bed scour in DAMBRK application

Bridge location	M/S C. C. Patel & Associates report (4.75 lakh cusec (constricted condition)	HEC-2 analysis based on observed flood (4.75 lakh cusecs (constricted condition)	DAMBRK analysis with varying n-value and considering 1 m river bed scour
Vasana barrage	43.44 m	43.81 m	43.86 m
Sardar bridge	48.56 m	47.21 m	46.21 m
Ellis bridge	49.21 m	47.93 m	47.37 m
Nehru bridge	49.56 m	48.29 m	47.70m
Gandhi bridge	50.44 m	49.41 m	48.35 m
Subhash bridge	51.34 m	50.31 m	50.07 m

This report also provides the water surface profiles calculations of Sabarmati River for the reach falling between *Subhash* Bridge and *Vasna* Barrage in Ahmedabad (Gujarat). The report computes the following hydrological variables for this reach: (i) the water surface profiles for the floods of 4.00 lakh cusec and 4.75 lakh cusec (1973 flood event); (ii) the maximum depth that could possibly be encountered under constricted river reach conditions, and their variation considering scour and appropriate values of Manning's roughness coefficient. The analysis was carried out using both HEC-2 and DAMBRK models, and the results are presented in Tables 7 (a) and (b). The selection of Manning's coefficient of the channel and banks is based on the recommendation of Chow (1959) and a detailed sensitivity analysis and simulation of the year 2006 flood.

Finally, it was found that the water levels computed at the various locations in the Sabarmati River in constricted condition of channel width 275m by using the HEC-2 model and the DAMBRK model by adopting the parameters calculated using the data of recent flood of 2006, are lower than those computed by M/s C.C. Patel & Associates for natural as well as for constricted condition. Further, due to improvement in channel carrying capacities in the constricted reach, the maximum water levels corresponding to the design flood are likely to be lower than those expected under the natural conditions.

References:

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