Journal of Civil Engineering and Technology Research

Volume No. 12

Issue No. 1

January - April 2024

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Journal of Civil Engineering and Technology Research

Aims and Scope

Journal of Civil Engineering and Technology Research intended to bring together the information in different areas of civil engineering around the world. The aim of this journal is to combine theory and practice in civil engineering and thus advancement of civil engineering sciences. It will provide a platform for academicians, researches and engineers to share their experience and solution to problems in different areas of civil engineering.

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(Volume No. 12, Issue No. 1, January - April 2024)

Contents

Sediment Classification Using Acoustic Signal Processing

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ABSTRACT

Sedimentation in reservoir is one of the prime concern in today's reducing water resources potential. However there are very limited technologies available to measure the underwater sedimentation. Most Popular among them is under water Acoustic technology. However these systems suffer the drawback of limitation in operating frequency and the non-linearity of underwater acoustics. Most of the systems are designed for different conditions, which does not suite Indian waters. However as the hardware is proprietary, we are forced to use them with a compromise to its accuracy. This Paper describes the design of a new acoustic system with advanced signal processing technology to overcome the uncertainties of the underwater objects. By this method the sedimentation volume as well as the sediment type could be classified to an acceptable accuracy. This paper is based on the case study conducted with different under water objects in attest tank at CWPRS. Classification is done based on the Knowledge based training given to the signal analysis algorithm. Few noise suppressing algorithms are also explained briefly

Keywords—Bathymetry, Underwater Acoustics, Sediment Classification

1. INTRODUCTION

Acoustic technology is the proven technology applied in under water imaging and different kind of decision making applications. Despite that measurement accuracy is totally depend on the medium which is used for application and environmental conditions around that water bodies. Today all reservoirs in India are getting silted-up, which is of a great concern for the dam operators, the users, it also becomes hurdles in the nation development. However there is very limited research going on in the area of underwater acoustic technology as it is inter connected to the measurement technology. Underwater acoustic noise, its path and other acoustic parameter has been addressed for sediment classification study. The noise due to multipath, sediments, currents, animals corrupts the echo being processed. The problem which is to be solved is noise characterization and that means characterize source of noise, signal enhancement of the underwater echoes which means measurements of weak signals even in the presence of large interference specifically for a reservoir environment. The issue in present literature is that such studies are scattered in terms of domains and application, no available literature has been able to address the issue of noise and its correlation or its dependence on two or more random variable for reservoir platforms. The sedimentation and chemical contamination profile of have been dealt with numerous model studies for various lakes in India including Khadakwasla reservoir, however the sound behavior depends on the characteristics of the water quality, under water materials, atmosphere around water bodies and hence a site specific study is required. But before that experiment has to be performed in the lab to see what kind of result we will gate. The transmission paths through different mediums contribute differently and for any noise control effort the precise contributions needs to be done. This data in combination with soil testing, temperature and other hydrological data including sediment data is a new area of research. [1,2,3]

Comparison of single beam echoes with multi frequency will be carried out for a sample site by which a bench mark / standard measurement procedure for experiment will be prepared. The conventional way of estimating the sedimentation and classify the sediment using different criteria has two methods, one is empirical method and second one is the model base approach. Imperial method has been used for this particular experiment. Floating sediment and its correlation to the settled sediments have been considered for study and the generalized model based analysis will be attempted to differentiate different types of sediment and study effect of acoustic signal on this sediment. The in-depth study will be undertaken for a specific reservoir in India, in this case it is Khadakwasla reservoir used for the onsite validation of the study. MATLAB based simulation is used for result.

Single beam technology involves a number of disciplines including underwater acoustics, detection theory statistics, and digital signal processing. Many excellent texts are available that provide in-depth mathematical treatment of each of these fields. The purpose of this research paper is not to cover all related topics in rigorous mathematical detail, but instead to present you with a simple, clear understanding of the fundamental concepts required to develop the full potential of a Sediment Classification Using Acoustic Signal Processing or acoustic technology. Ideas are presented in a graphical and descriptive way, with minimal use so it would become so much easier to pursue topics in greater detail. Most of the concepts explained in this research paper are common to all acoustic signal processing.[6,7,9]

2. MODEL SETUP

In order to classify different sediment on the bases of their performance when they come into the contact with acoustic pulses, detail study of different parameters are required. So a test tank has set up in room temperature 210 c to study those parameters. Size of this test tank is 2m x 2m. Layers of different types of sediment which has thickness of 20 cm is applied at the bottom of this test tank.

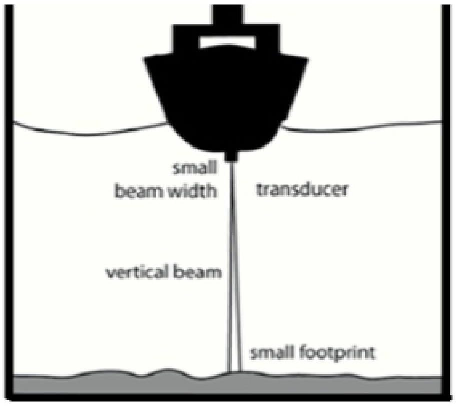

Figure 1 sketch of test tank experiment

Different material is used as sediment, such as sand with maximum diameter of particles is up to ½ mm and sedimentary rocks with maximum diameter of particle is up to 2 mm. then sound pulses are transmitted which will penetrate in the sediment. Beam width of this pulse is 1.50 and ping rate is 3-5 Hz for sand and 3 Hz for the sedimentary rocks. Direction of this beam will be near to normal axis. By studding the penetration of sound signal in different sediment, which has different particle size and varying different parameters regarding echo sounder like velocity of the pulse, transmission power of the pulse as well as frequency of the pulse will has various effect.

Figure 2 Single beam Echo sounder

There are total eight types of sediment which are enlisted in this study, each sediment has different particle size. And each sediment react with acoustic signal differently due to its nature. Following are the sediment with its particle size.

List of different sediment used in experiment –

Name of the sediment	Maximum diameter of the particle (mm)
Very coarse sand	2
Coarse sand	1
Medium sand	1/2
Fine sand	1/4
Very fine sand	1/8
Silt	1/16
Clay	1/256
Colloid	1/1000

3. EXPERIMENTAL RESULT

There are several experiment has been performed and different parameter are use in this study. By varying this parameter we can study the effect on sound signal of this parameter. And we can easily classify different sediment.

Experimental data on medium type sand with maximum diameter of particle is up to 1/2 mm, Beam width is 1.50 and Ping rate 3-5 Hz. Direction of beam is near to normal axis.

Velocity = 1480 M/s	Measured distance for 1W Power		Measured distance for 2W Power		Measured distance for 5W Power	
Actual Distance In meters	Channel 1= 200 KHz	Channel 2 = 33 KHz	Channel 1	Channel 2	Channel 1	Channel 2
1.7	1.14	1.34	1.12	0.6	1.1	0.62
1.6	1.05	1.58	1	0.59	1.02	0.61
1.5	0.95	1.47	0.91	0.63	0.88	0.61
1.4	0.84	1.18	0.79	0.64	0.8	0.63
1.3	0.75	1.15	0.7	0.52	0.71	0.59
1.2	0.65	1.02	0.61	0.52	0.61	0.61

The two experimental data table that show above are two different case studies that are performed in Experimental data on sedimentary rocks with maximum diameter of particle is up to 2 mm, Beam width is 1.50 and Ping rate 3 Hz. Direction of beam is near to normal axis.

Both case study took place in the same environment. Only difference in this case study is that parameter like transmission power are very in the both case from 1W, 2W, 5W while velocity is kept constant at 1480 m/s. Signal penetration is measured for both channel. Channel 1 operating frequency is 200 KHz and Channel 2 operating frequency is 33 KHz. Maximum distance which have measured is 1.7 meter in 2 meter test tank. Following are the graph of the signal penetration in the different types of sediment.

Experimental data on sedimentary rocks with maximum diameter of particle is up to 2 mm, Beam width is 1.50 and Ping rate 3 Hz. Direction of beam is near to normal axis.

Velocity = 1480 M/s	Measured distance for 1W Power		Measured distance for 2W Power		Measured distance for 5W Power	
Actual Distance In meters	Channel 1= 200 KHz	Channel 2 = 33 KHz	Channel 1	Channel 2	Channel 1	Channel 2
1.7	1.54	1.62	1.53	1.61	1.52	0.52
1.6	1.44	1.52	1.42	1.52	1.41	0.52
1.5	1.34	2.45	1.32	1.39	1.32	0.52
1.4	1.28	2.39	1.23	1.3	1.23	0.52
1.3	1.13	2.3	1.14	1.2	1.13	0.52
1.2	1.05	1.11	1.03	1.09	1.03	0.52
1.1	0.95	1.01	0.92	0.52	0.93	0.52
1	0.83	0.91	0.83	0.52	0.82	0.52

Both case study clearly indicate that change of transmission power has severe effect on the acoustic signal. 200 KHz transmission frequency and 5w transmission power is the ideal operational parameter in this condition because at this operating range signal penetration at its peak.

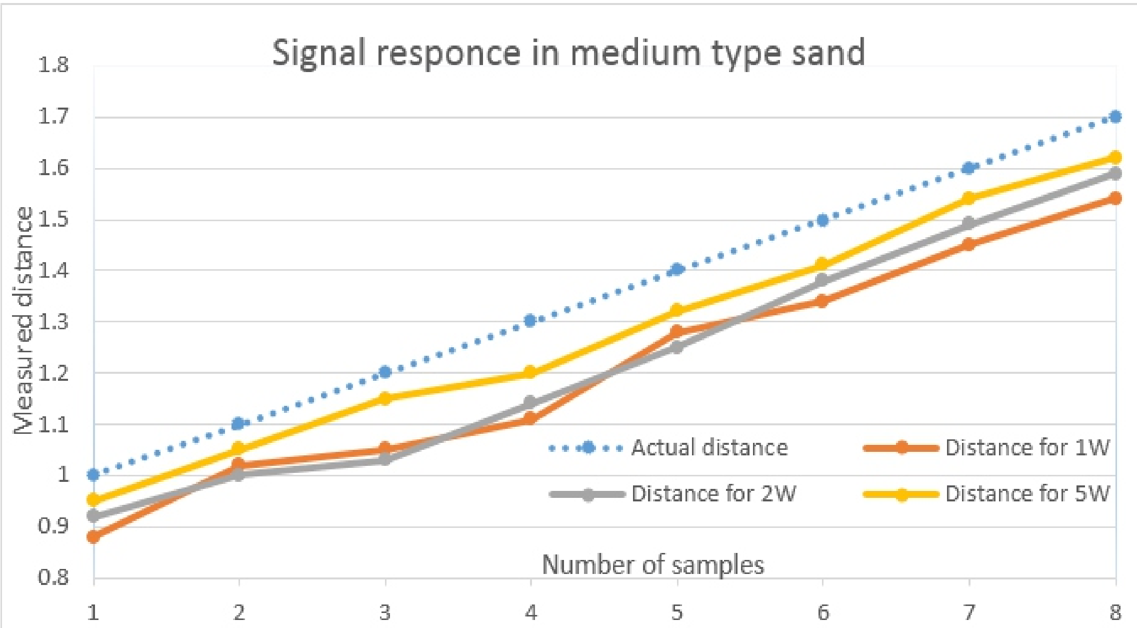

Figure 2 Signal response in medium type sand

Case study clearly specify that signal penitration in the medium sand with maximum dimeter of the partical is ½ mm is showing 80% to 85% accuracy. At transmittion power 5W signal penitration is maximum.

Experimental data with change in velocity while keeping power constant at 1W and frequency at 200 KHz. Thickness of the silt is 0.2m (20 cm).

Changing velocity in m/s	Actual height in meters	Measured height	Actual height	Measured height
1350	1.8	1.45	1.5	1.2
1375	1.8	1.45	1.5	1.17
1400	1.8	1.5	1.5	1.23
1425	1.8	1.54	1.5	1.27
1450	1.8	1.61	1.5	1.31
1475	1.8	1.63	1.5	1.35
1500	1.8	1.64	1.5	1.36
1525	1.8	1.67	1.5	1.37
1550	1.8	1.72	1.5	1.39
1575	1.8	1.75	1.5	1.4
1600	1.8	1.76	1.5	1.42

This is another case study in which transmission power kept constant at 1W and transmission frequency kept constant at 200 KHz and velocity of the acoustic signal is kept changing at some interval. Study clearly show that at 1600 m/s velocity signal penetration is maximum that means as velocity of the signal increase, signal penetration is increase as well.

Experimental setup for change in time gate on the pulse while keeping velocity constant at 1480 m/s and thickness of the silt is 0.2 m (20 cm).

Changing time gate	Actual height for channel 1(meter)	Measured height for channel 1(meter)	Actual height for channel 2 (meter)	Measured height for channel 2 (meter)
2%	1.7	1.6	1.7	1.68
	1.4	1.3	1.4	1.36
4%	1.7	1.57	1.7	1.65
	1.4	1.28	1.4	1.36
8%	1.7	1.58	1.7	1.66
	1.4	1.29	1.4	1.36
20% (Fail reading)	1.7	0.89	1.7	0.46
	1.4	1.29	1.4	0.46

In this case study time gate of the acoustic signal is changed and other parameter like velocity is kept constant at 1480 m/sand frequency of channel 1 is 200 KHz and channel 2 is 33 KHz. The study shows that change in time gate of the pulse of acoustic signal has huge impact of the penetration capacity of the signal in the sediment.

Experimental data with change in velocity while keeping power constant at 1W and frequency at 200 KHz. Thickness of the silt is 0.2m (20 cm).

CONCLUSION

Characteristic of acoustic signal has been studied. Different parameter like transmission power, velocity of acoustic Signal, changing transmission frequency, time gate of the acoustic signal pulse and other parameter has severe effect on the signal behavior as well as its penetration capacity completely depend on this parameter. Change in any of this parameter could change entire scenario of experiment. Above case study is the proof of this concept.

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Equations

Primary equation used for data calculation in acoustic theory is given below.

$$V = SL + G - 40 \log R - 2aR + TS + 2B(\Theta\phi) \qquad (1)$$

V = the received voltage of the echo in decibels (dB),

SL = the transmitted source level in dB @ 1m/ µpa,

G = the receiving gain of the system in dB per µpa at 1m.

40 log R = the two-way spreading loss in dB where R is the range in meters,

a = the sound attenuation coefficient in dB/m,

Ts = the acoustic target strength,

B (Θϕ) = the transducer directivity pattern function.

This is the equation which is used for acoustic signal processing. If the value of V in the given equation is greater than background noise signal penetration take place.

Both transmitted sound and the return echo are greatest at the acoustic axis. And that will be perpendicular to the face of the transducer.

Numerical Modelling of Wave Propagation in Gulf of Khambat

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ABSTRACT

Shallow-water wave transformation strongly depends upon coastal geomorphology, bottom sediment characteristics and sea bed slope. As waves propagate into shallower water they reach a point where the water depth cannot support the local wave height and eventually breaks due to wave transformation. Accurate prediction of wave parameters is vital for the coastal infrastructure developments . This phenomenon can be simulated using mathematical modeling technique. This paper aims in simulating the shallow water transformation in the Gulf of Khambhat having higher tidal range using a numerical model. Variations in wave characteristics and their physical significance is discussed . A Oil company has a proposal for development of anchorages of ships and development of a coal jetty at Hazira in the Gulf of Khambhat. The tidal range at Hazira is high. Due to presence of shoals, dissipation of wave energy takes place and at low water, significant reduction in wave height is observed at the anchorages points in the absence of effect of local wind. However, local wind prevailing in the domain contributes to increase in wave heights at the sites of port development. From the numerical simulation of wave transformation the maximum significant wave height at the anchorage was estimated to compute the yearly downtime for berthing operations.

Keywords—Tide, Wave Transformation, Spectral Wave, Gulf

1. INTRODUCTION

Prediction of design wave parameters viz wave height, wave period and their directional distribution is vital for any costal infrastructure development. Most of the time near shore wave data are not available and the available offshore wave data needs to be transformed to near shore using mathematical model technique. Transformation waves is site specific and depends upon coastal geomorphology, sea bed slope and bottom sediment characteristics it also depends upon the shape of the area. Gulf of Khambhat is located on the western coast of India in the Arabian Sea between the Saurashtra peninsula and mainland Gujarat. Oil industries such as Reliance Industries, M/s Essar M/s VOTL have developed private ports for import and export of oil products in the gulf near Hazira and Dahej. The Gulf is barely a few Kilometers wide and it opens out southward like a funnel. Because of the funnel shape and semi-enclosed nature of coast, it experiences very high tides, the tidal height increases tremendously in the upstream. The bathymetry in the Gulf of Khambhat near Hazira and Dahej is complex. The depth contours show that there is wide stretch of tidal flats and also shoals in the vicinity of site of developme-

-nts. As the tidal range is more than 5m, large area is subjected to flooding and drying. This paper aims in simulating the shallow water transformation in a region having higher tidal range, using numerical model, to determine feasibility of all weather operations at the oil handling jetties and anchorages.

2. METHODOLOGY

The offshore wave data reported by India Meteorological Department (IMD) as observed from ships plying in deep waters off Hazira were transformed to nearshore location using Spectral Wave(SW) model MIKE -21 to get the near-shore wave climate at Hazira. MIKE – 21 SW model is a spectral wind wave model based on unstructured mesh developed by Danish Hydraulic Institute, Denmark. The model simulates the growth, decay and transformation of wind generated waves and swell in offshore and coastal areas.

2.1 Site conditions

Hazira is situated at 21°05'4"N latitude and 72°36'33"E longitude. The bathymetry in the Gulf of Khambhat near Hazira is complex (Fig. 1). The depth contours show that there is wide stretch of tidal flats and also shoals in the vicinity of proposed project site. As the tidal range is more than 5m, large area is subjected to flooding and drying.

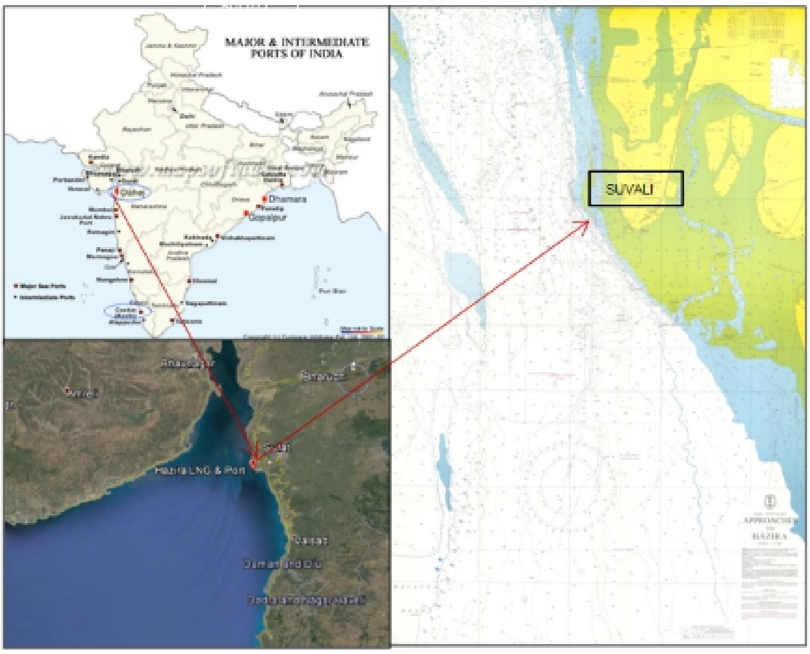


Figure 1 Location map of Hazira

2.2 Tide and wave data

The observed tide data at Hazira show the tidal range of the order of 5.7 m. The highest high water level is at + 7.4 m and lowest low water level is at +1.7 m shown in figure 2.

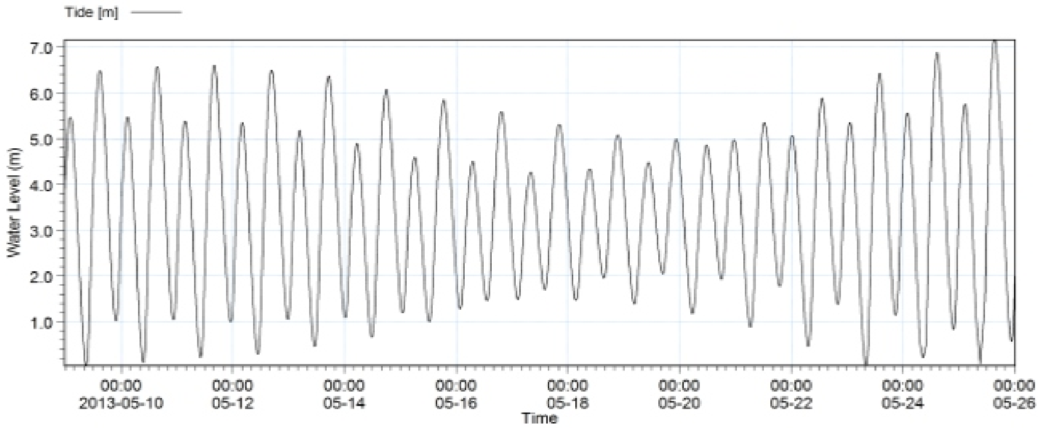
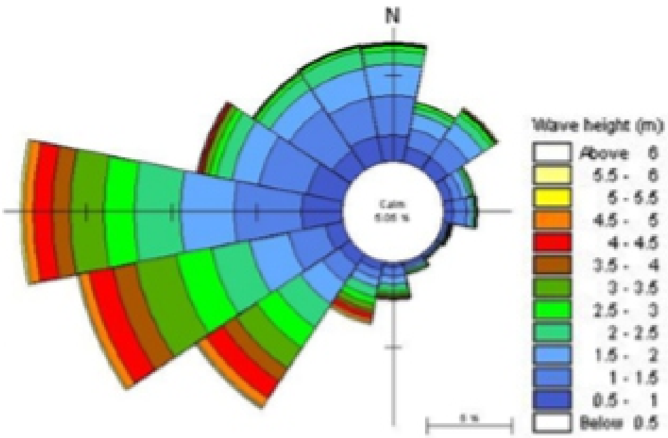


Figure 2 Observed tide at Hazira

The offshore wave data reported by India Meteorological Department (IMD) as observed from ships plying in deep waters off Hazira were analysed. The frequency distribution of wave heights from different directions for entire year of the above offshore data is given in the form of wave rose diagrams and is shown in Fig.3. It is seen from the deepwater data that the predominant wave directions in the deep sea off Hazira are from SW to NW. It may be noted that the wave height based on ship observed data corresponds to significant wave height, which represents average energy of the random wave train.



3.0 WAVE TRANSFORMATION

Coastal area of 250 km by 200 km (Fig. 4) with an unstructured mesh was considered for studies with MIKE –21 SW model, which extends up to 50 m depth contour in deep sea and high water line near the shore. Depth adaptive mesh with coarse grid in the offshore and fine mesh near the project site was used for the simulation studies. The input wave conditions were derived from the offshore wave climate. The model was validated with measured wave data available at Pipavav at latitude 20.88^0 N and longitude 71.49^0 E during monsoon season from July 1999 to December1999. Validation of the model was done by considering effect of local wind on the wave transformation.

Initially, to observe effect of tidal range on wave transformation at Hazira, the model was run for the tidal cycle of two weeks shown in Fig.2 . Time history of significant wave height during the tidal cycle for waves incident from SW direction with incident wave height of 5.0m is shown in Fig.5. The model was run for waves incident from SW, WSW and West directions for incident wave height of 5m. The model was run for the highest high water level at 7.4m and lowest low water at 1.7m, first without considering effect of local wind and then with wind speed of 15m/s.

Plots of wave height contours and wave vectors in the model area and near the proposed jetties are shown in Figs. 6 to 8 for all the incident wave directions at high water and low water respectively, considering effect of local wind on wave transformation.

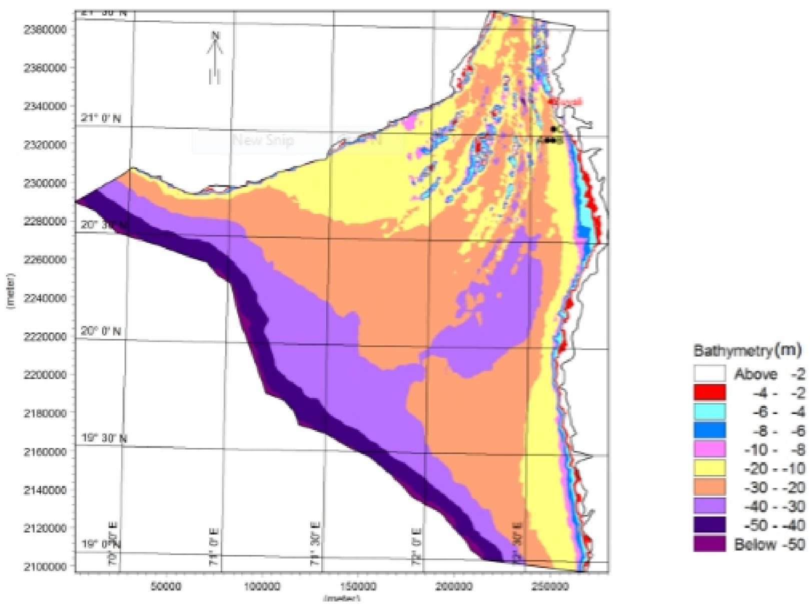


Figure.4. Model area for mike-21

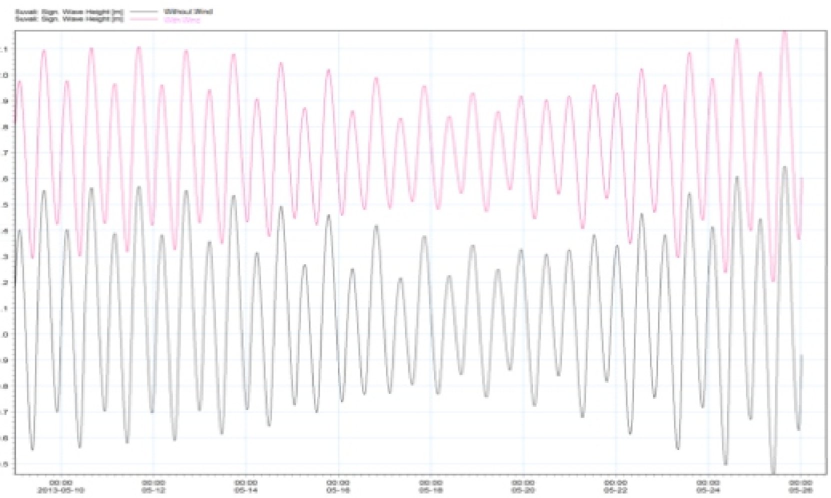


Figure.5. Time history of significant wave height at Hazira

4.0 DISCUSSIONS

An anchorage is a location at sea where ships can lower anchors. These locations usually have adequate depth for ships with larger draft and provide protection from weather conditions, and other hazards. Oil industries such as Reliance Industries, M/s Essar and M/s VOTL are engaged in import and export of oil products near Hazira and Dahej. For carrying these products Panamax vessels are used and three anchorage locations (A, B and C) were proposed near Hazira.

Wave height and directions were extracted at the three anchorages as shown in Fig. 4. These wave heights are shown in figure 6 to 8 at high water and low water respectively for the three predominant incident wave directions.

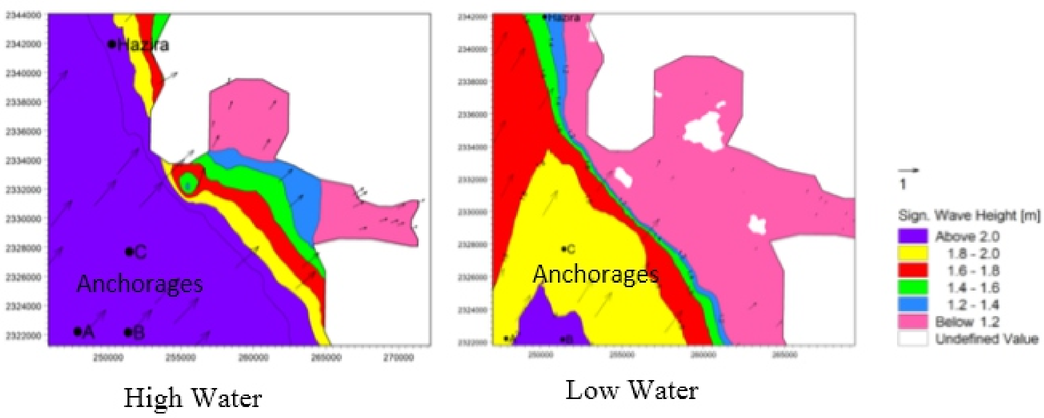


Figure.6. Wave height distribution at the proposed development at Hazira for waves from South West direction

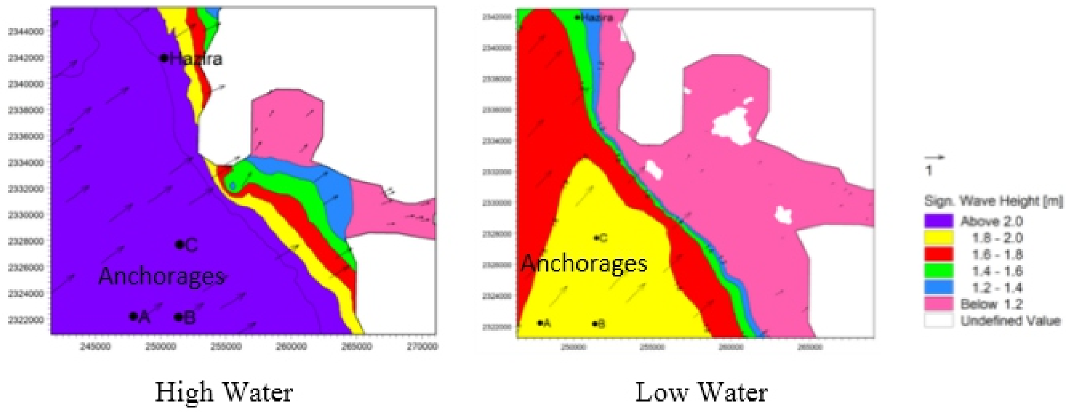


Figure.7. Wave height distribution at the proposed development at Hazira for waves from WSW direction

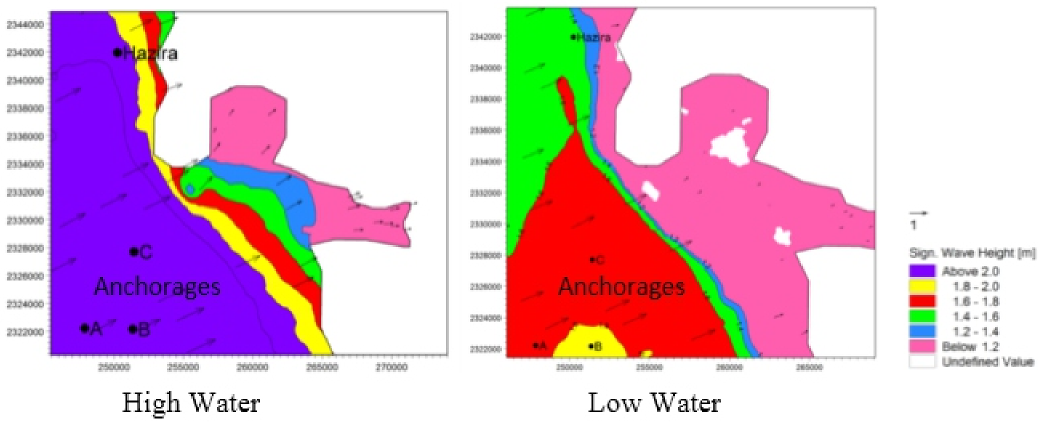


Figure.8. Wave height distribution at the proposed development at Hazira for waves from West direction

5.0 CONCLUSION

Numerical model MIKE 21 SW was used for simulation of wave transformation from deep water to near shore for deciding locations of anchorages for oil tankers in Gulf of Khambat As the tidal range is more than 5m, large area is subjected to flooding and drying. Due to presence of shoals dissipation of wave energy takes place at low water and significant reduction in wave height is observed at anchorage location in the absence of effect of local wind. However, local wind prevailing in the domain contributes to increase in wave heights at the proposed site of port development. It was observed that the anchorages location proposed by Reliance Industries at offshore Hazira, Gujarat provide protection from weather conditions, and other hazards for Panamax and similar vessels. At these anchorage location the maximum significant wave height at high water and low water is of the order of 2.7m and 2.0 m respectively. Thus numerical modeling technique was found to be a useful tool for estimation of wave climate in the coastal region having complex bathymetry with high tidal range.

Acknowledgements

Authors are grateful to Mr. S. Govindan, Director, Central Water and Power Research Station for his continuous encouragement and motivation during the course of studies and for granting permission to publish this paper.

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Hydraulic Design of a Major Bridge across River Narmada

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ABSTRACT

Assessment of flow conditions in the vicinity of a bridge across a major river is very difficult without model studies. Model studies are therefore invariably conducted to finalize hydraulic design parameters for the major bridges. Similarly a hydraulic model study for a proposed road bridge across river Narmada as a part of the Vadodara-Mumbai Expressway of National Highway Authority of India (NHAI) has been carried out.

The proposed bridge in this case comes under the tidal reach of river Narmada. The Hydraulic model studies were conducted on a physical model with a horizontal scale of 1:500 and vertical scale of 1:100 in conjunction with one-dimensional mathematical model study. Different aspects such as flow conditions at the proposed bridge site, adequacy of the bridge alignment and waterway, alignment of guide bunds, afflux due to bridge and scouring at the bridge piers etc. were studied on the model. Foundation level for bridge piers were decided considering maximum discharge intensity observed on the model.

Measures suggested to mitigate the problem identified during the course of experiments are discussed in this paper.

Keywords—Afflux, Bridge, Flow Conditions, Guide Bund, Model studies, Scour, Waterway.

1. INTRODUCTION

National Highway Authority of India (NHAI) has proposed to construct 8-lane expressway from Vadodara to Mumbai. The proposed bridge across river Narmada is located about 8 km downstream of Bharuch and 9 km upstream of Bhadbhut, Fig.1., where the river width is about 1800 m. The bridge is designed for a discharge of 74,500 m3/s. The HFL of the river at the site is assessed as 11.486 m empirically. The total waterway of the bridge is proposed to be 1788 m. The bridge consists of 26 bays of varying spans and foundation at different levels depending on the sub-soil strata. The hydraulic model studies for the proposed road bridge across Narmada river was conducted to predict changes in flow conditions, due to construction of road bridge, near the proposed bridge site and in the upstream reach and to ensure proper siting of the bridge, to assess adequacy of the proposed waterway, assessment of expected scour at the bridge piers and examine the need for bank protection works, if any. Narmada river reach near proposed bridge is subjected to tides from Gulf of khambat and floods from upstream. The large area of the region near proposed bridge is inundated during high tide and floods. A physical model was therefore constructed with the help of topographic data for the reach from Bhadbhut to about 17 km upstream upto golden bridge at Bharuch. In addition 1-D mathematical model study was also carried out to establish gauge-discharge relationship at the downstream boundary of physical model.

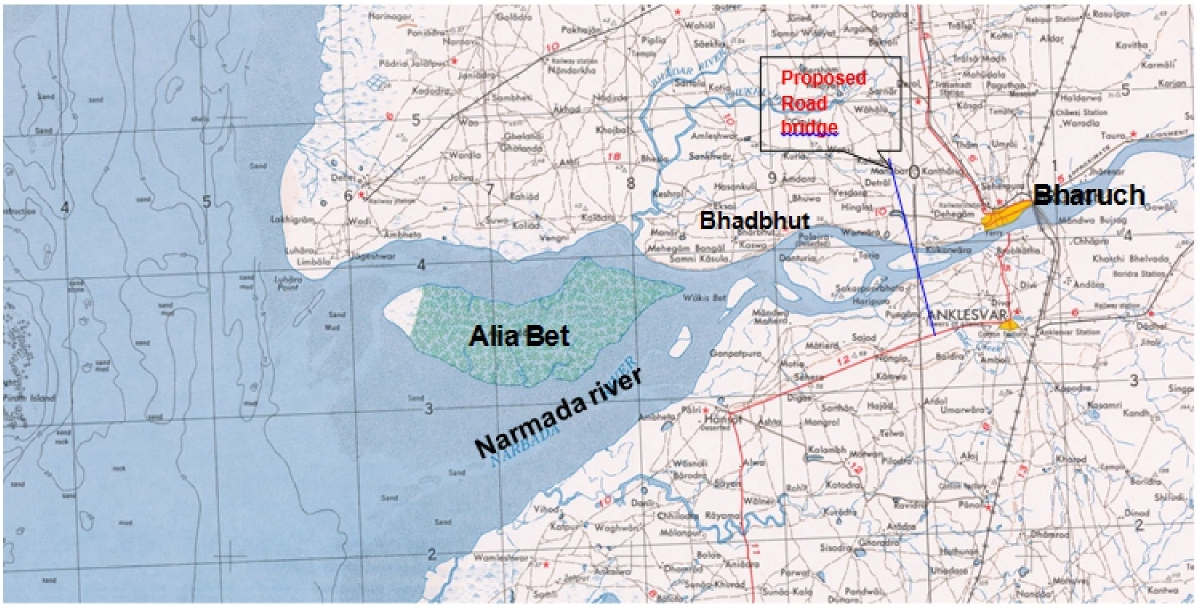


Figure 1 : Location of proposed road bridge 8 km downstream of Bharuch

2. Data required for the study

For any hydraulic model study generally topographical, hydrological, hydraulic and sediment data is required.

2.1 Topographical data

The river cross section data was available for the reach from Bhadbhut to Zadeshwar (about 5.2 km upstream of Golden bridge at Bharuch). These data included river cross sections at an interval of about 1000 m in the reach from Bhadbhut to Zadeshwar. The levels along the cross sections were at an interval of 5 to 10 m in deep channel and at 100 m at spill portion. The cross sections were extended on both sides upto high flood level (HFL) / high tide level or high bank level. Figure 2 shows the location cross sections and the extent of river reach surveyed for the study purpose. These were utilized for development of mathematical model and reproduction of bed profile in physical model.

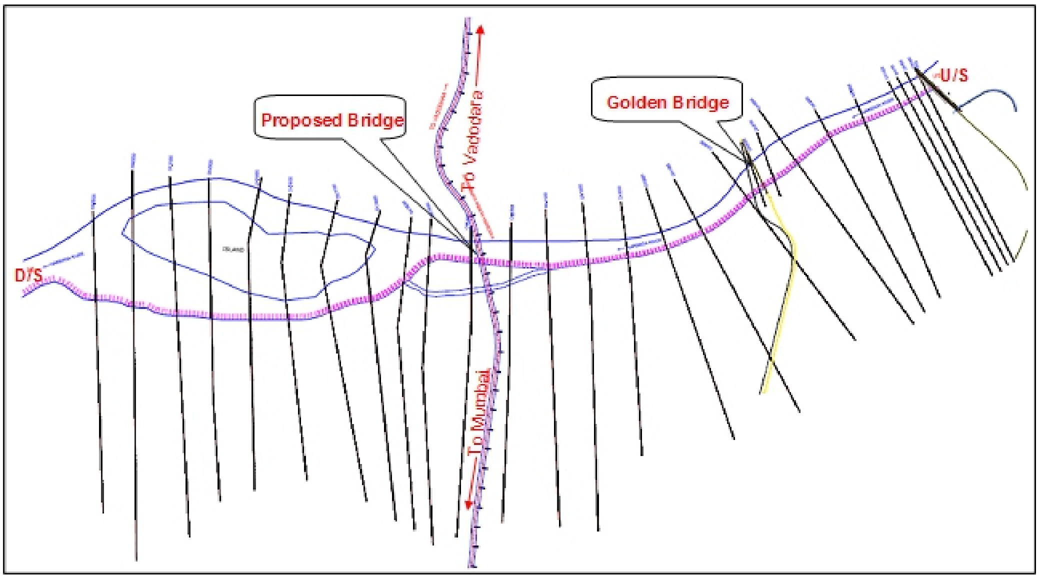


Figure2 Location and extent of cross-sections

2.2 Hydraulic data

The hydraulic data such as gauge-discharge data was not available at any location in the vicinity of proposed bridge site. Daily water level data was available at Golden bridge site. Daily discharge data was available at Garudeshwar (about 110 km upstream on the mainstream Narmada) and Chandwada (located on a major tributary). With the help of above data, a gauge discharge relation was established at Golden bridge site (Fig. 3).

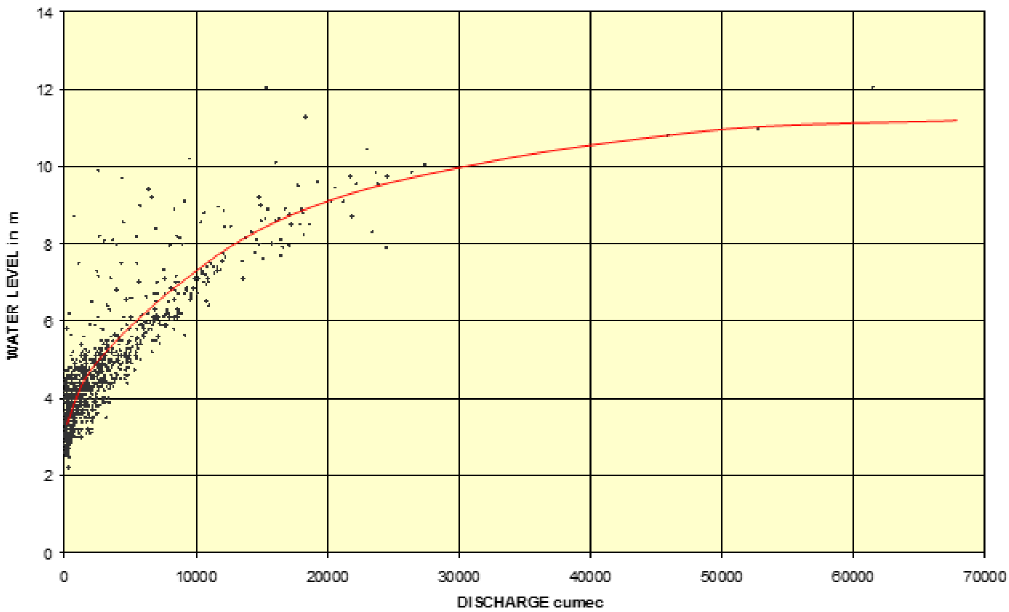


Figure 3 Gauge-Discharge curve for river Narmada at Golden Bridge

2.3 Sediment data

The grain-size distributions for the river bed material at few locations in the reach near proposed bridge were also available (Fig.4), which were utilized for computation of scour.

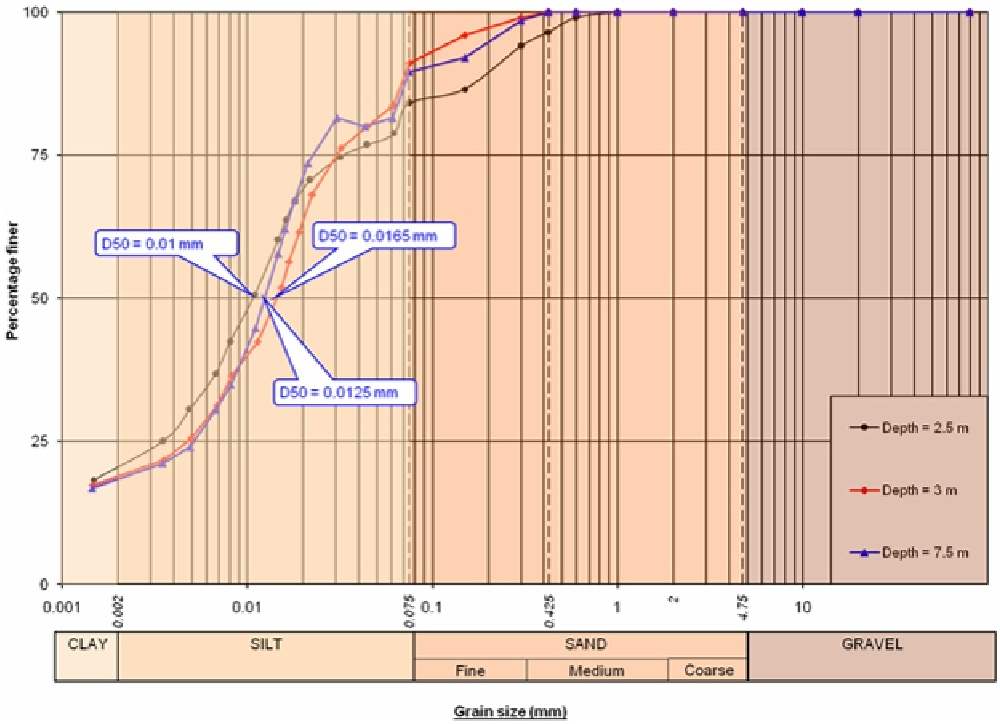


Figure 4 Grain size distribution curve for bed material in the vicinity of proposed bridge

3.0 RIVER DETAILS

The proposed bridge site is located about 8 km downstream of Bharuch. At the proposed site river Narmada is about 7500 m wide, whereas deep channel is towards right bank and about 1100 to 1400 m wide. Right bank is very firm and almost vertical having RL of about 15 m. Left bank is at a comparatively low level of RL 13.8 to 13.9 m and the flood plain level in the vicinity of proposed location is nearly at RL 8 m. The proposed bridge across the river Narmada is in the tidal zone. The tidal range / variation in the water level at the site is of the order of 5 m. An island of size 7 km x 2 km (approximately) is present in the river khadir starting at about 3 km downstream of proposed site. This island bifurcates the river in two channels, the right channel carries more discharge than the left one. During ebbing condition channels and islands are clearly visible. There exists a 60 to 70 m wide channel at higher elevation within the river khadir itself, which become active during the tidal flooding condition. There is some habitation on the island and people are doing farming on the dry land. Most of the area on island remains above the river water level except at the time of very high floods.

3.1 Influence of Tidal Water Flow in Narmada River

The tidal rise in river Narmada is felt up to 32 km (19.9 mile) above Bharuch. The tidal flow in Narmada estuary becomes complicated due to following reasons :

- Presence of shallow outer bar area between deep sea and the mouth
- Steep bed slope of the river at entrance
- Presence of Alia bet and other small islands between Mehgham and Bhadbhut.
- During period of low flows from upstream, the flow is dominated by tide. During high floods from upstream the flow is dominated by flood and remains unidirectional i.e. towards sea. In the lower reaches downstream of Bhadbhut river bed remains flooded during period of High tide.

4.0 MODEL STUDIES

4.1 Methodology

The choice of any prediction or simulation technique generally depends on desired accuracy and extent and quality of data available for representing the prototype topography and boundary conditions. In the present study, calibration and validation data available were inadequate; therefore, a 1-D mathematical model was developed to make up for inadequacies in the Gauge – Discharge data. Calibration of physical model was done with the help of water level data at Golden bridge along with the results of one-dimensional mathematical model. The HECRAS was used for the 1-D mathematical model studies.

4.2 Schematisation of Narmada River

The topographical reach of about 47 kms of Narmada river from sea mouth to Bharuch including Golden Bridge was reproduced in the mathematical model. The Narmada river cross sections near sea mouth were not available, therefore, the C-Map Software data for this reach was utilised for the present studies. The tidal water level data of spring tide was used to formulate tidal data at downstream boundary. Figures 5 show the schematization of river Narmada in HECRAS along with river reach simulated in the model and the locations of cross sections used for model reproduction.

4.2.1 Upstream boundary condition

The constant discharges of 5000, 10,000 and 15,000 along with the hydrographs corresponding to

20,000, 30,000, 40,000, 50,000, 60,000 and 74500 m3/s, Figure 6, for river Narmada at Bharuch were used as upstream boundary conditions. The timing of peak discharge of flood hydrograph was matched with the time of high water of spring tide to get the worst condition.

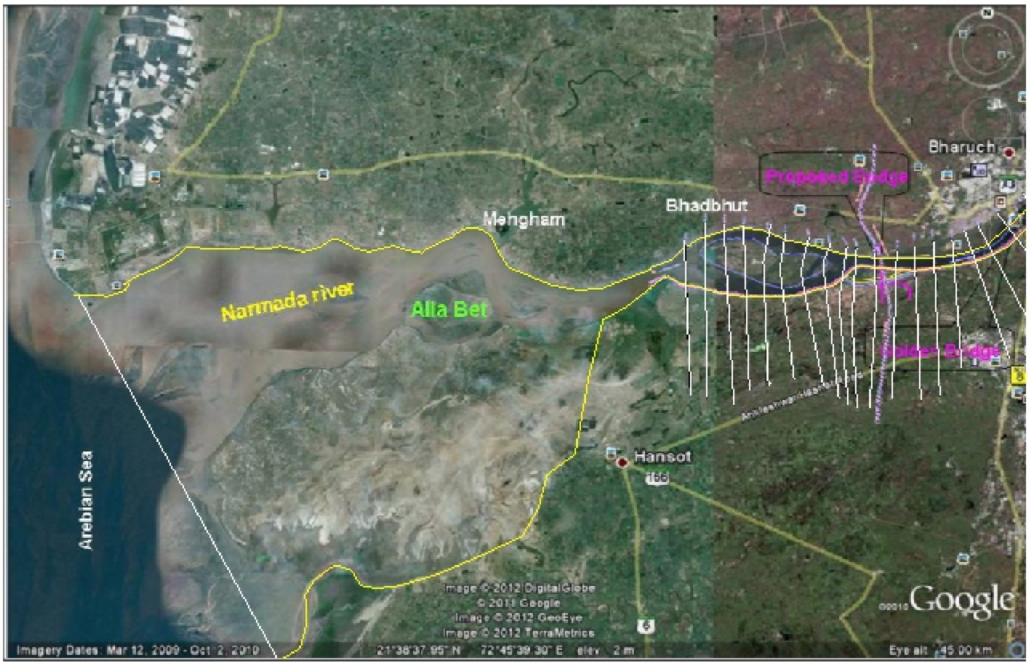


Figure 5 Schematisation of river Narmada for HECRAS

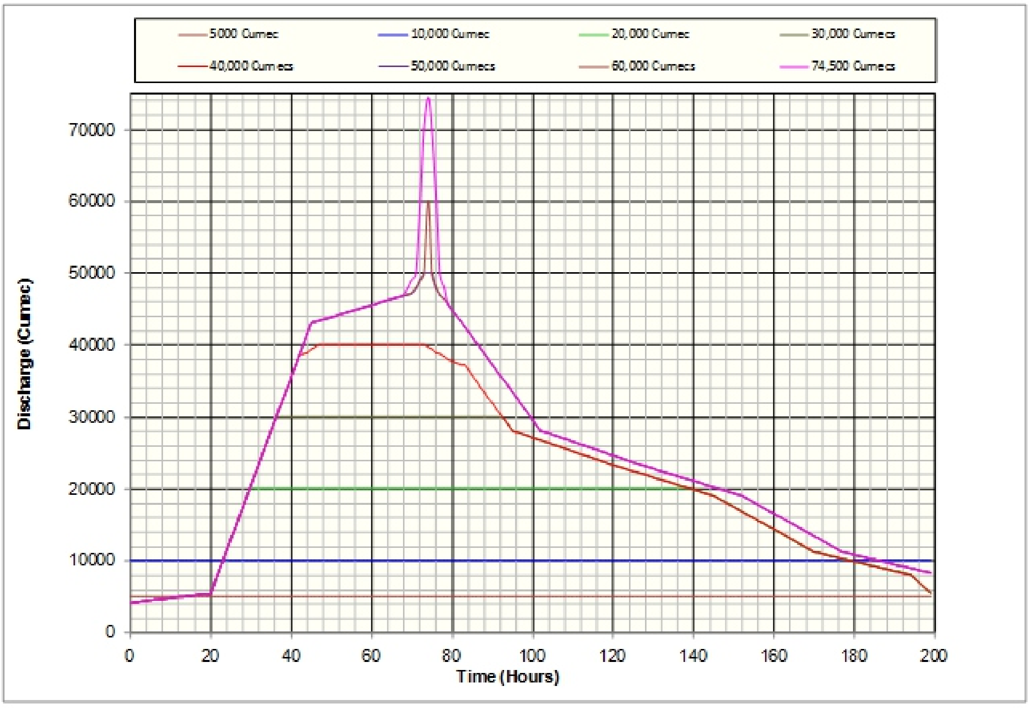


Figure 6 Hydrographs of river Narmada at Bharuch used as upstream boundary for 1-D Model

4.2.2 Downstream Boundary Conditions

The spring tide levels shown in Fig.7 were used as downstream boundary conditions which was available at CWPRS from previous studies. This spring tide has high water level of 2.33 m and low water level of -6.33 m with respect to GTS.

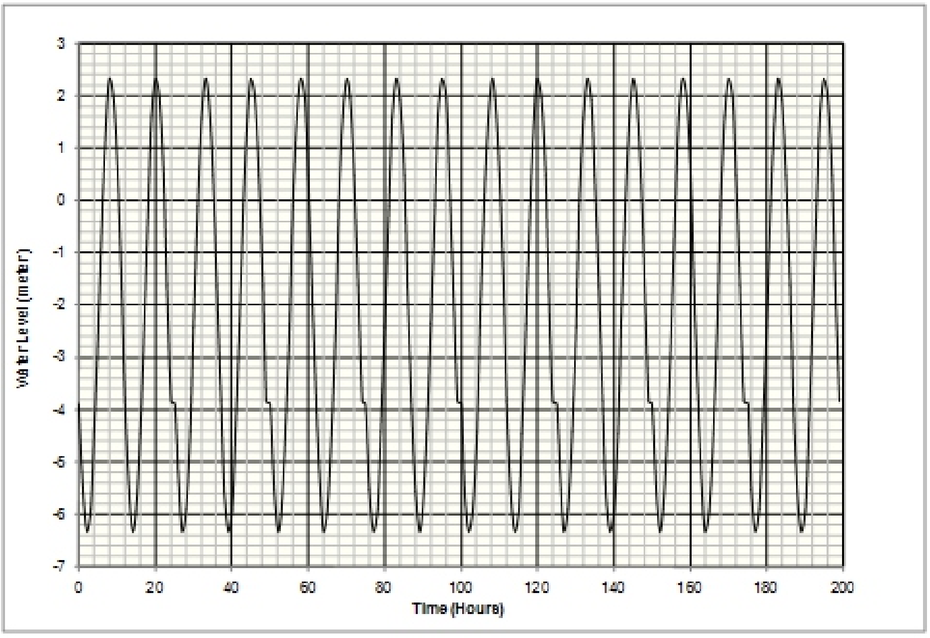


Figure 7 Spring Tide at sea mouth used as Downstream Boundary for 1-D Model

4.2.3 Model calibration and validation

The most important parameters in calibration of 1–D models are channel bed roughness and channel widths at water surface. From the information about the bed conditions and from calibration runs using gauge discharge data at Golden bridge, Bharuch as shown in figure 4, the Manning's roughness values adopted in different reaches of the Narmada river were 0.022 to 0.025. For the validation runs, the spring tide of high water level of 2.33 m and low water level of -6.32 m (Figure 7) was used as downstream boundary condition. The respective flood hydrographs (Figure 6) with peak of the flood hydrograph approaching at the time of high water were used as upstream boundary conditions. These model runs were carried out for different combinations of the boundary conditions as spring tide at downstream boundary with peak of the flood hydrograph for different floods arriving at the time of high water level. The comparison of water levels shows that the mathematical model prediction is in close agreement with the G-Q relationship at Golden bridge, Bharuch, (Figure 8).

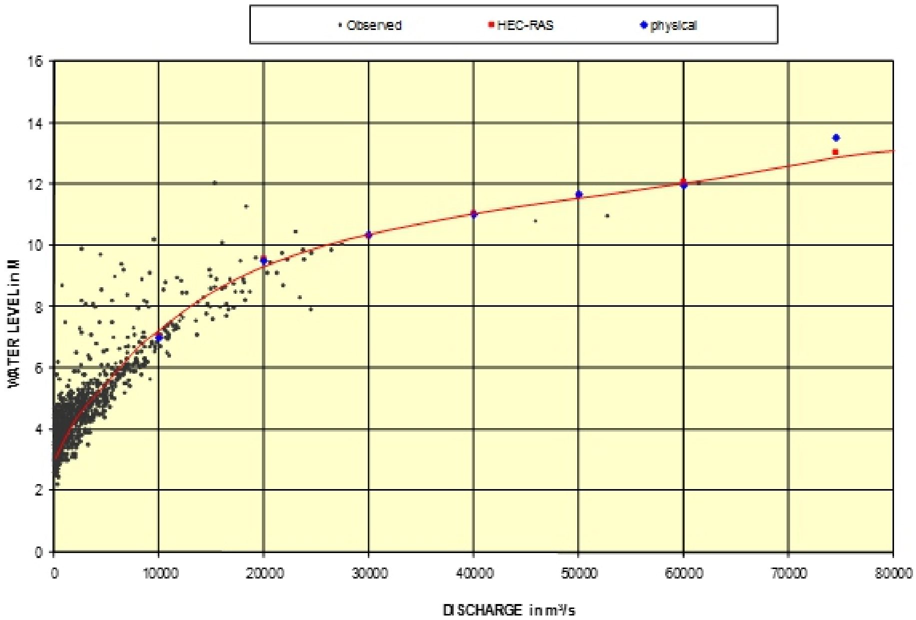


Figure8 The comparision of G-Q relation at Golden bridge with observed levels on physical and mathematical models

5.0 PHYSICAL MODEL STUDIES

A physical model was constructed from Bhadbhut as a downstream boundary to Golden bridge at Bharuch as upstream boundary, Photo 1, covering river reach of about 18 km. As length of the river reach to be reproduced was much more compared to water depth, distorted scales were used for the design of model to facilitate measurement of required model parameters. The design of model scales were arrived at giving due consideration for movement of bed material in the model on the basis of tractive force theory for incipient condition and also roughness co-efficient for reproducing proper water surface slopes. Horizontal scale of 1:500 and vertical scale of 1:100 were adopted for the model. The channel of the river was made mobile by laying sand of D50 = 2.1 mm computed for the tractive force in model for bank full stage discharge.

Photo 1 Model of Narmada river

The purpose of physical model was to obsetve flow conditions in the reach around the proposed bridge as well as on some reach upstream and downstream. The components of bridge structure i.e. piers; foundations, deck etc. were crafted in teakwood with a smooth finish of oil paint. Initially guide bund was reproduced in cement mortar with smooth finish of oil paint and as per the design suggested by project authority. There were total 25 piers out of which 5 were in deep channel, 5 on the high bank on right side and remaining on left side on the flood plain. The river cross sections at an interval of 1000 m were used to reproduce main river channel and the spill areas / flood plains. The Golden bridge at Bharuch was also reproduced on the model. Figures 9 and 10 shows the cross-sections of river Narmada upstream and downstream of proposed bridge location respectively. The different flood discharges were used as upstream boundary and validated water levels of 1-D mathematical model were used as downstream boundary. It was observed from the google images of the area under study that there was habitation on the left side flood plain of river Narmada and very thick vegetation / bushes existed on the left side of main river channel. Considering the ground reality, additional roughness on the left side of river channel / flood plain was provided on the model with the aid of stone aggregate / tiny strips of G.I.sheet protruding above river flood plain, to simulate the increased resistance to flow at these locations.

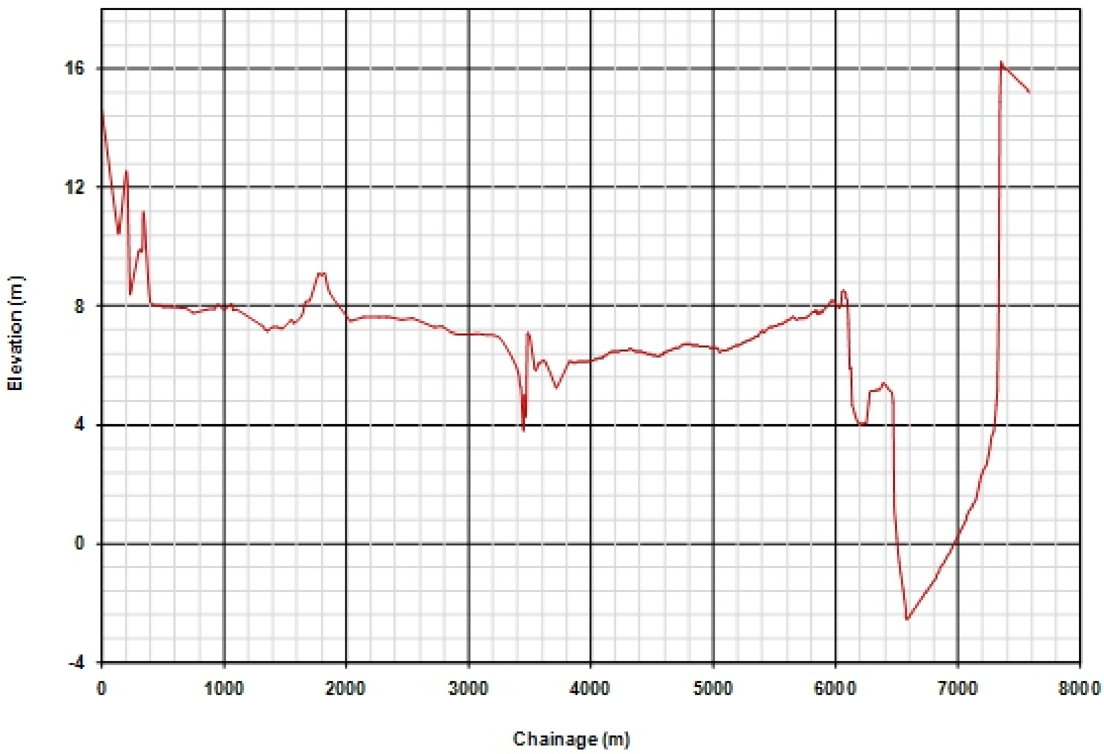


Figure9 River cross-section upstream of proposed bridge site

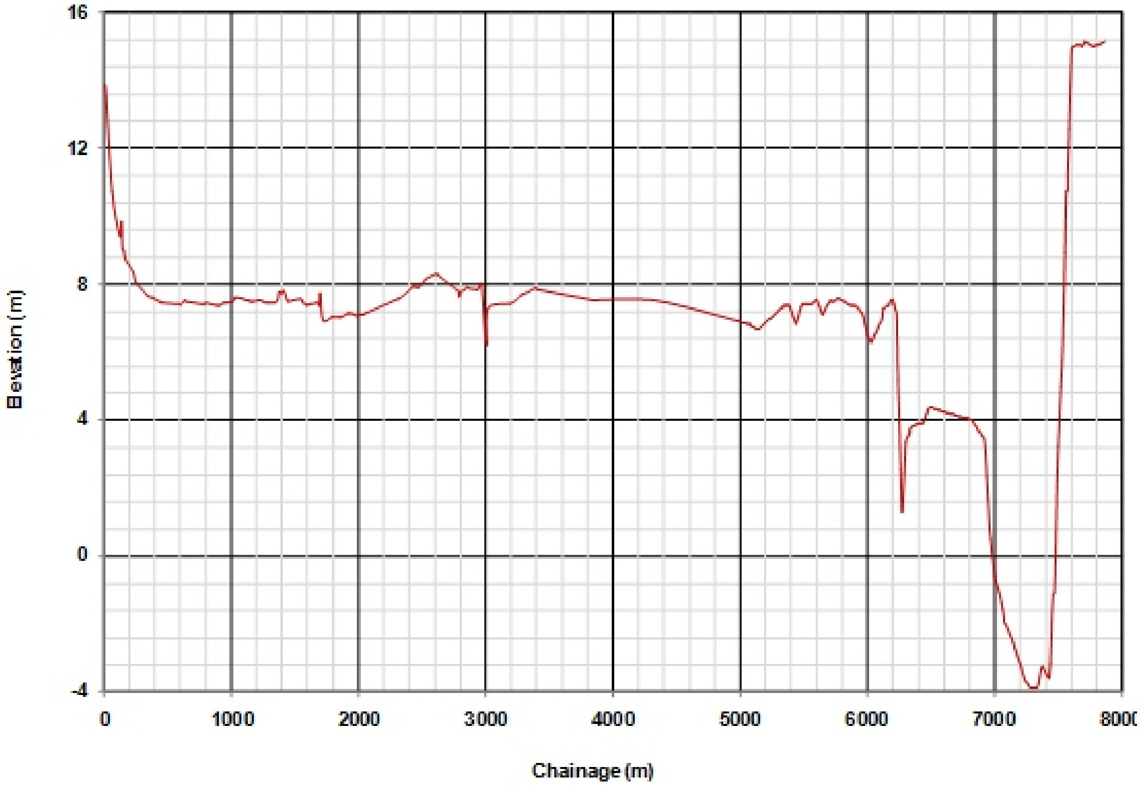


Figure10 River cross-section downstream of proposed bridge site

5.1 Proving Studies

After the construction, physical model was calibrated to ensure correct simulation of hydraulic parameters like discharges and their corresponding water levels at different locations. Two gauges were fixed one at 8 km upstream and the other at 10 km downstream from the axis of proposed bridge. Experiments were conducted for different discharge stages by maintaining downstream gauge for a known value of particular discharge and corresponding value of upstream gauge was observed. The results of both physical and mathematical model are shown in the following Table – 1. Table shows that physical model results are in close agreement with the predicted HECRAS results. Thus the physical model was accepted as proved.

Table 1 Comparison of water levels

Sl. No.	Discharge Q in m^3/s	Water levels obtained from HEC-RAS model at		Water levels observed on Physical model at	
		Upstream Gauge in m	Downstream Gauge in m	Upstream Gauge in m	Downstream Gauge in m
1	10000	7	4.86	7	4.86
2	20000	9.56	7.015	9.5	7.015
3	30000	10.33	7.93	10.35	7.93
4	40000	11.05	8.7	11	8.7
5	50000	11.66	9.21	11.6	9.21
6	60000	12.1	9.75	11.95	9.75
7	74500	13.05	10.47	13.15	10.47

5.2 Model Observation with Existing Conditions (pre-bridge)

After calibration studies, experiments were conducted on the model for the existing condition i.e. pre-bridge scenario. Water levels, velocities and depths were measured at different locations i.e. both upstream and downstream of the proposed bridge axis for different discharge stages ranging from 20000 m3/s to 74500 m3/s. Selection of the discharge stages were made to include discharge at bank full stage, 20 year return period flood and design discharge. Flow patterns were observed in the model for various discharge stages. Photos2 show the flow conditions for the design discharge of 74500 m3/s. Water levels and Velocities were measured under the existing pre-bridge condition in the model. Velocities varied from 1.67 m/s to 6.69 m/s (Figure11) and Water levels varied from 12.15 m to 12.35 m(Figure12) for design discharge of 74500 m3/s, near the bridge axis. It was observed that flow mainly concentrates along the left side of deep channel of river for discharge 74500 m3/s, at the proposed bridge axis, its upstream as well as downstream (Photo2).

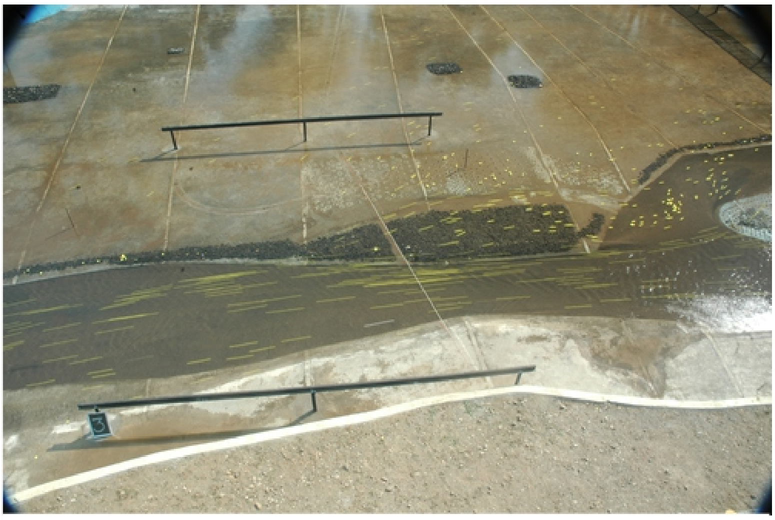


Photo2 Pre bridge Flow conditions for discharge of 74500 m3/s

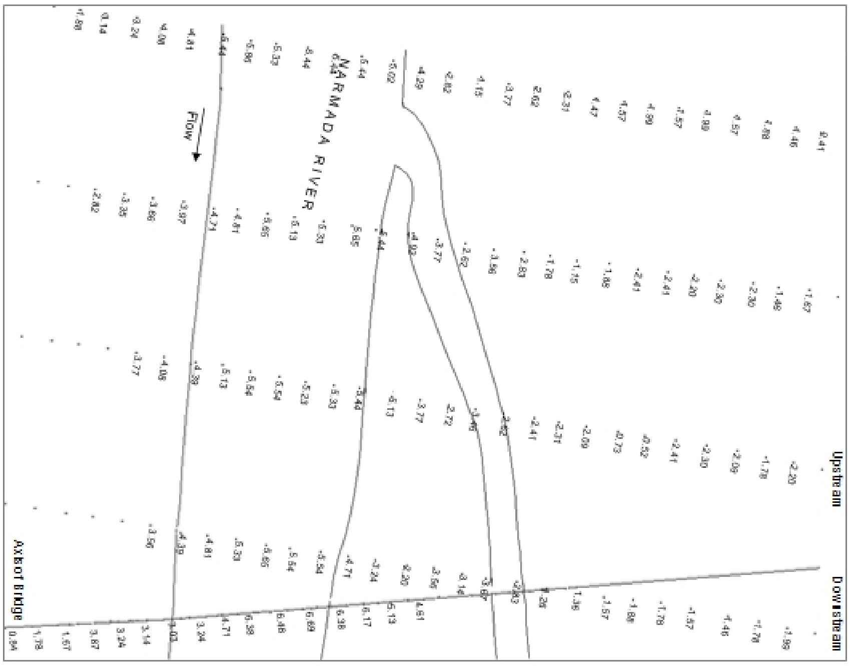


Figure11 Velocities for discharge of 74500 m^3/s

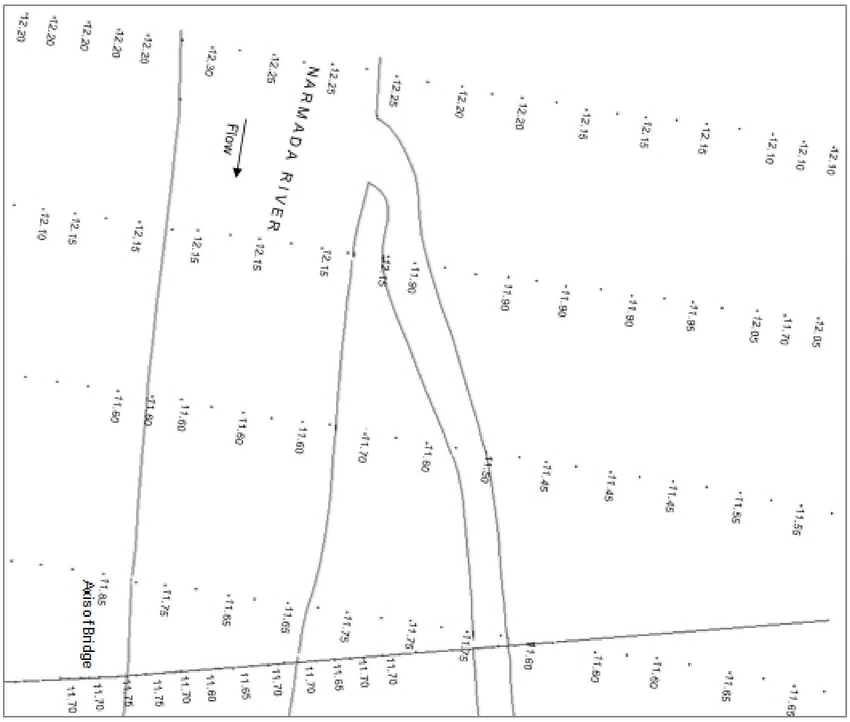


Figure12 Water levels for discharge of 74500 m^3/s

5.3 Model Observations with Bridge in Position

The bridge was reproduced in the model as per original design. Experiments were conducted for different discharges after reproduction of bridge and other related structures like guide bund, approach embankments etc. Controlling the discharge at upstream and water level at downstream, water levels, velocities and depths were measured for different discharge stages at different cross-sections including the bridge location. The Guide bund was initially reproduced as per the original design and it was noticed that flow was not properly negotiating the curve of guide bund and getting deflected towards the deep channel thereby causing concentration of flow near the left side of deep channel, (Photo 3). Therefore, several alignments for the guide bund were tried by receding upstream part of the guide bund towards left side and an alignment by receding straight part of the guide bund by 50 towards left. Thus increasing the angle of straight portion with bridge axis from 900 to 950 as shown in figure 13 was found suitable. The flow conditions with this alignment were satisfactory. The flow was seen approaching favorably and following the curve of guide bund for most of the length, (Photo 4) and distribution of flow was comparatively uniform.

Photo3 Flow conditions with original alignment of Guide bund

Photo4 Flow conditions with modified alignment of Guide bund

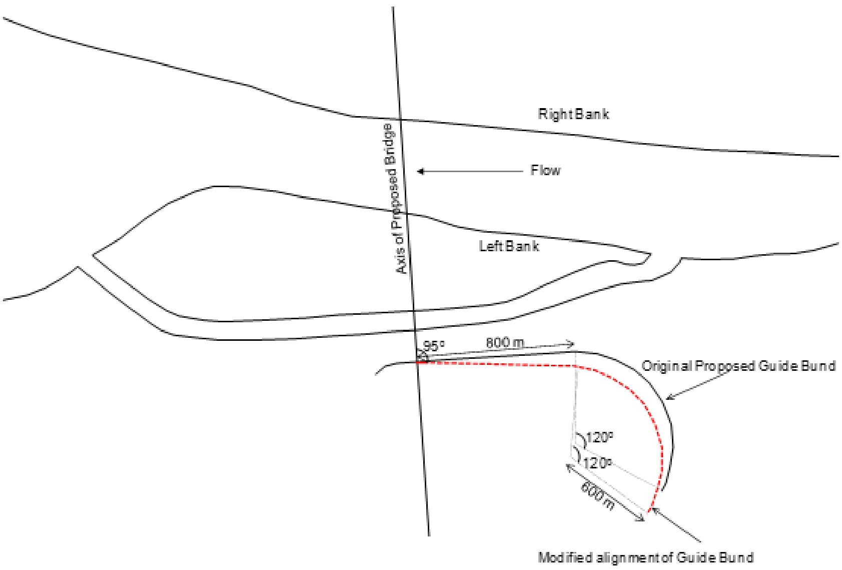


Figure13 Original and modified alignment of Guide Bund

For the discharges of 50,000m3/s and 74,500m3/s, it was observed that flow velocities were fairly uniform just upstream of the bridge, some scouring of bed in the deep channel near left bank was noticed, which was obvious for such higher stage discharges. It was observed that major quantum of flow passes through the deep channel portion of the river towards right side. About 68% of flow passes through the deep channel and rest 32% passes over the spill portion on left side for design discharge of 74,500 m3/s, whereas 76% of total flow passes through deep channel for a river discharge of 50,000m3/s. There was no significant turbulence / undulations observed in the approach flow i.e. immediate upstream of the bridge even for the design discharge of 74,500m3/s. Right bank is steep and at higher level as compared to left side. For the discharges more than 20,000 m3/s river discharge starts spilling over the flood plain on left side. The water levels, velocities and discharge intensities are presented in Figures 14 to 16 respectively for the discharges of 50,000m3/s and 74,500m3/s. It was observed that water level varied from RL 10.55 m to 10.85 m near the bridge axis, velocity varied from 2.09 m/s to 5.4 m/s and discharge intensity varied from 2.98m3/s/m to 71.78m3/s/m for discharge of 50,000m3/s/m. And water level varied from RL 11.95 m to 12.25 m near the bridge axis, velocity varied from 3.64 m/s to 6.59 m/s and discharge intensity varied from 4.70m3/s/m to 94.59m3/s/m for discharge of 74,500m3/s/m. Therefore, the value of design HFL was suggested as RL 12.25 m and maximum discharge intensity as 94.59m3/s/m for working out the scour and other related parameters. Water levels upstream of bridge were also observed for the 1 in 20 year (Navigational) flood of 40,370m3/s and shown in figure 17. Water level for this condition varied from 9.65 m at the left side to 10.15 m near the deep channel. Therefore, HFL for navigational flood was suggested as 10.15 m.

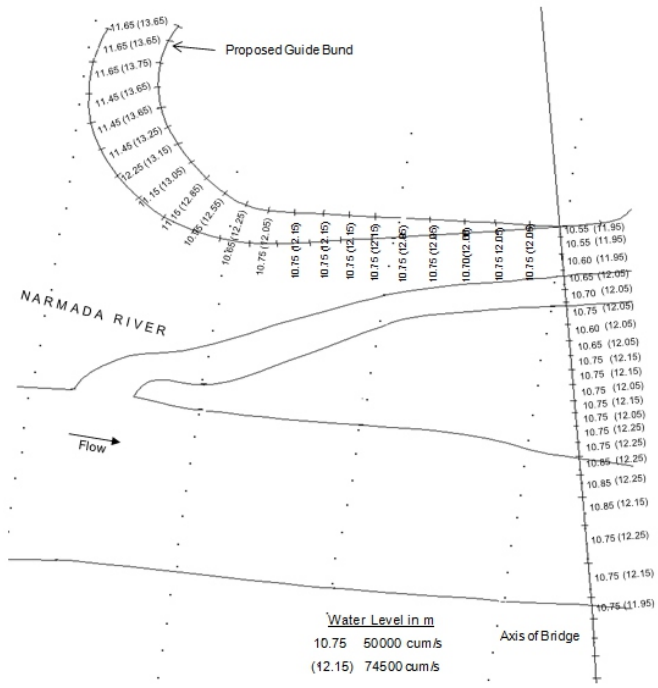
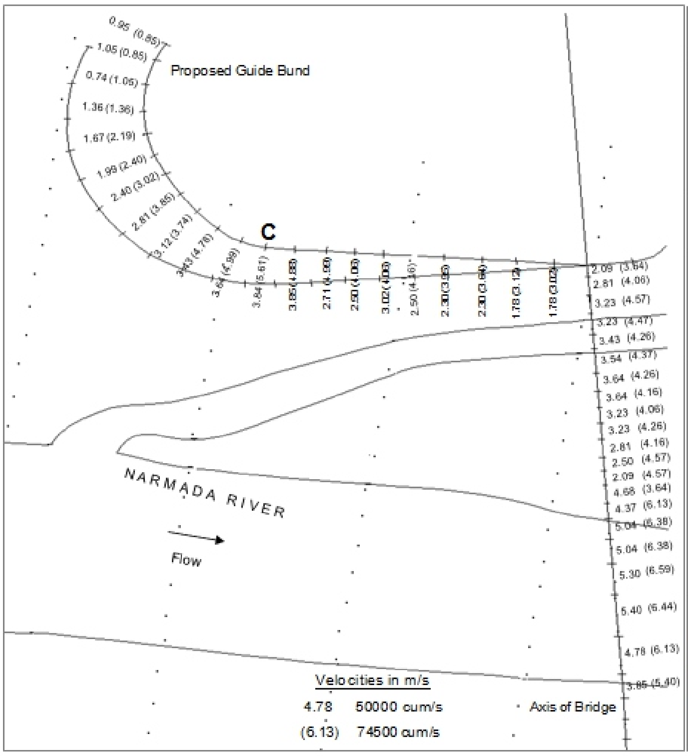


Figure14 Water levels for discharges of 50000 and 74500 m3/s

Water levels and velocities were also observed along the guide bund for discharges of 50,000m3/s and 74,500m3/s(Figure 14 and 15). At the upstream curved portion of guide bund the water level observed for design discharge was 13.75 m. The velocities observed at the shank portion were more than 3.0 m/s. At the beginning of curvature of guide bund (point C) velocity observed was of the order of 5.61m/s which require adequate protection works. The approach flow conditions at the head of guide bund were found favorable and velocity was also less than 1.0 m/s. The flow conditions behind the guide bund were very calm and no return flow could be noticed at this location.



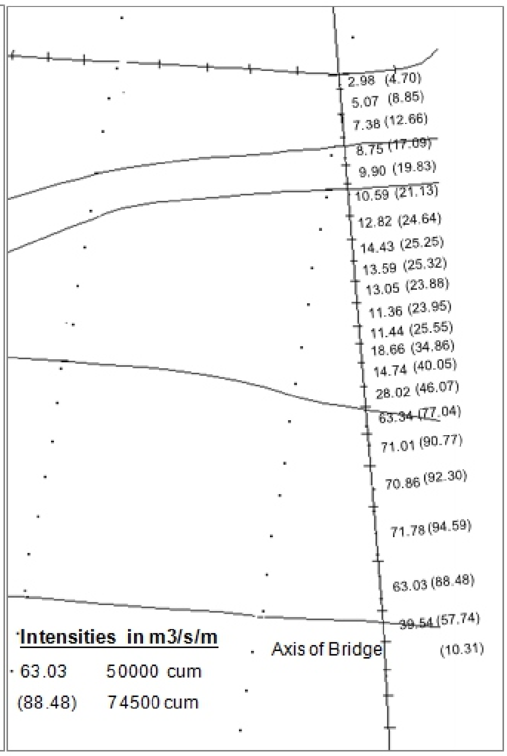


Figure15 Velocity for discharges of 50000 and 74500 m^3/s **Figure16** Discharge Intensity for discharges of 50000 and 74500 m^3/s

Approach road embankment on left side beyond the guide bund in a length of about 6.0 km was also reproduced on the model. It was observed on the model that an opening (culvert type) may be provided in the embankment portion at a distance of 1170 m from guide bund to drain out water from the upstream side of embankment. Another opening at a distance of about 2275 m from guide bund at the location of existing natural drain was also provided on the model. Any opening beyond this point (i.e. towards left) will not affect the flow conditions near bridge, however any opening closer than the first opening may adversely affect flow conditions near the downstream portion of guide bund or the embankment hence was not considered. The flow beyond downstream side of guide bund was also observed traveling in the downstream and not going back towards the embankment hence flow conditions near the approach embankment were also satisfactory.

6.0 ANALYSIS OF RESULTS

6.1 Adequacy of Waterway

It was observed on the model that with the total waterway of 1788 m and clear waterway of 1730 m, the 1 in 100 year flood of 74,500 m3/s could pass easily without any excessive scour and with an afflux of 0.4 m to 0.5 m. Hence waterway provided was considered adequate.

6.2 Scour around Bridge Piers

Due to the construction of the bridge, the natural water way gets constricted and generally there is relative rise in the flow velocities at the bridge. This leads to erosion of bed and banks of river. The flow obstructed by the piers, gets separated and causes horse shoe type vortex and results in removal of bed material thus forming scour holes around bridge piers. During the experiments in the physical model, scouring pattern observed around bridge piers did not show sign of excessive scouring in any specific area. Scour around the bridge piers calculated in original design and calculations on the basis of model observations are summarised below:

6.2.1 Original Computation by the Designers

For design of foundation level, the maximum scour depth at bridge piers was originally calculated as 28.74m considering average discharge intensity of 49.67m3/s/m, an assumed silt factor of 2 and HFL at RL 11.30m.

6.2.2 Physical Model Studies and Lacey-Inglis / Shen's Formula

The clear waterway provided (1730 m) is more than the lacey's waterway of 1318 m and the intensity distribution varies a lot due to local topography and approach flow conditions. Therefore, maximum observed intensity of 94.59 m3/s/m is considered for working out scour. For the actual silt factor of 0.23 (D_{50}=0.0165mm) and observed HFL of 12.25m general scour was worked out as 33.88m. The total scour is arrived at by calculating local scour (14 m) for the obstruction created by 10m diameter well foundations. The foundation level calculated by both Lacey's (-51.60m) as well as Shen's method (-50.0m) are almost same.

6.3 Afflux due to Bridge

For different discharge stages, water levels were measured upstream of bridge over a reach of 8.0 km both for the pre-bridge and post-bridge conditions and the longitudinal water surface profile for design discharge of 74,500 m3/s and for 50,000 m3/s is drawn in Figure17. It was observed that the afflux due to bridge for the design discharge wass only of the order of 0.14 m at 7.5 km upstream.

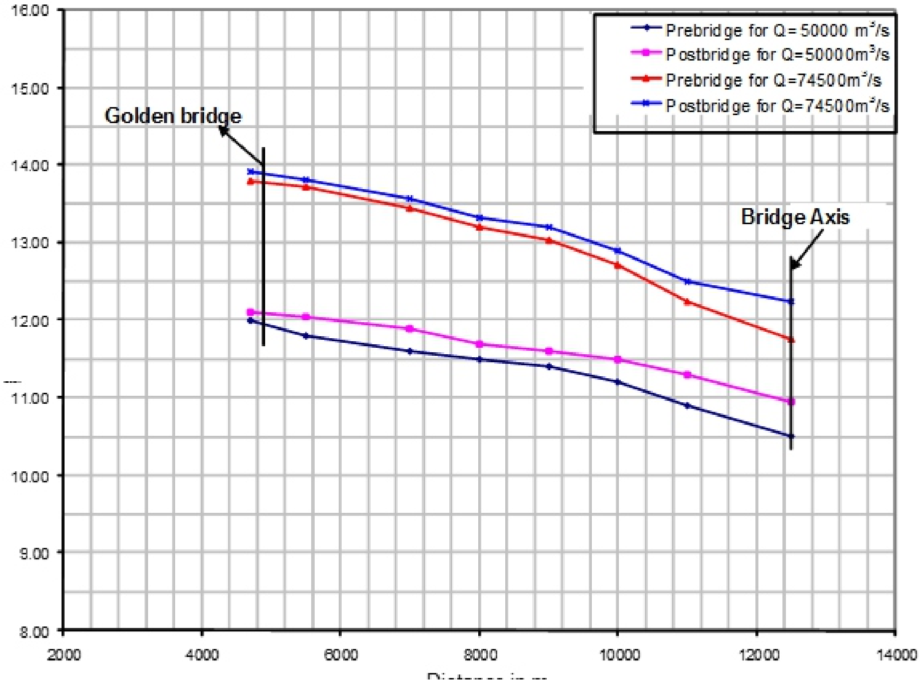


Figure 17 Longitudinal Water Surface profiles for discharges of 50000 and 74500 m3/s

6.4 Effect of bridge and Guide Bund

Initially studies were carried out with bridge and proposed alignment of guide bund as per original design. These studies indicated that the concentration of the flow was mainly on right side and spans on left side of the bridge were found relatively inactive for lower stages of discharges upto 30,000 cum/s. For higher discharges beyond 30,000 cum/s, discharge distribution was relatively better with modified layout of guide bund. For more uniformity in discharge distribution, in addition to velocity, uniformity in water depth is also required, which was not possible in the present case due to constraint of very less width of deep channel as compared to flood plain and guide bund was located on flood plain. Higher flow velocities were observed at the end of shank portion of guide bund from where the curved part starts.

7.0 CONCLUSIONS

Based on the analysis of data, one-dimensional mathematical model studies and studies conducted on hydraulic model, a total water way of 1788 m provided in the design was found to be sufficient to pass the design discharge of 74,500 m3/s. Flow conditions were more or less even and there was not much disturbance observed for the discharges up to 40,370 m3/s. For higher flows, minor undulations were seen. The flow was observed on the flood plain on left side for discharges more than 20,000 m3/s, however 60 to 70% of flow was through deep channel on the right side. The proposed bottom level of bridge deck was well above observed HFL of RL 12.25 m. From maximum scour depth analysis and adequate provision of grip length foundation level of piers recommended to be provided at R.L. – 50.0 m. Higher velocities in the range of 3.0 m/s to 5.6 m/s were observed along the guide bund for design discharge, therefore to avoid possibility of erosion on sides of guide bund and river bed, protection work in the form of stones in crates laid over suitable synthetic filter along with launching apron were recommended. It was also recommended that no opening in the approach embankment should be provided before 1170 m from the guide bund, for draining of water from upstream of embankment.

Acknowledgement

Authors express deep sense of gratitude to Shri S. Govindan, Director, CWPRS for constant encouragement and valuable suggestions during preparation of papers and consent to publish this paper. The co-operation extended by all the CWPRS staff members in conducting studies is great- fully acknowledged.

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Trend Analysis of Temperature Time Series in Geba River Basin, Northern Ethiopia

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ABSTRACT

Understanding the effects of global climate changes on the environment is crucial for sustainable water resources development, and management. Identification and analysis of trends in historical climate records at regional scale can provide additional information on the likely scenarios of future climate conditions. The present study was taken up to analyze trends in historical air temperature records in the semi-arid Geba River Basin located in Northern Ethiopia. Air temperature records for seven climate stations located in the basin for the 43 year period (1971-2013) were used. Annual series of average daily maximum and average daily minimum temperature were subject to trend analysis using non-parametric Mann-Kendall test at 95% confidence level. Results indicated that although daily maximum temperature revealed a rising trend at three stations, it was statistically significant at only two stations (increase of 0.053 oC per decade). A decreasing trend in maximum temperature was noted in the remaining four stations, but the trend was statistically significant at only two stations (decrease of 0.084 oC per decade). On the other hand, annual average daily minimum temperature indicated a statistically significant positive trend at all stations except two where statistically significant decreasing trends were evident. Results of this study provide useful inputs to hydrologic modelers who are concerned with understanding the effects of climate changes on water resources and to decision makers involved in the development of suitable climate change mitigation strategies in this semi-arid river basin..

Keywords—Climate Change, Geba River Basin, Mann-Kendall Test, Semi-arid Environment, Temperature Change, Trend Analysis

1. INTRODUCTION

Global warming resulting in climate changes across various spatial and temporal scales of the Earth varies is a cause for major concern in recent times. Air temperature is rising on the African land mass and the surrounding oceans (Bryan et al., 2013). During the period 1901-2005 the earth's averaged surface temperature has increased by 0.74°C and among regions the rates of climate change is significantly different as reported by the latest estimates by IPCC (2007a). Furthermore, the globally averaged surface temperature show a warming of 0.85 °C, over the period 1880 to 2012, when multiple independently produced datasets exist whereas, the total increase between the average of the 1850–1900 period and the 2003–2012 period is 0.78 °C, based on the single longest dataset available (IPCC, 2013). Collier et al. (2008) investigated that many semi-arid parts of the developing world that are likely to become hotter and dryer with time as changes in temperature patterns are widely experienced. Compared to the global average, the average temperature rise in Africa is faster and is likely to continue in the future (Hulme, 2001). Collier et al. (2008) suggested that this warming occurred at the rate of about 0.5ºC per decade with a slightly larger warming where crops are grown close to the thermal acceptance limits.

Opiyo et al. (2014) investigated the temporal trends such as monthly, seasonal and annual temperature variability for the period 1979 to 2012 in Turkana, Kenya. Results showed that except for the months August and November which showed statistically significant increasing trends for maximum temperature in the entire study, there is a mix of positive and negative trends, though they are not statistically significant at 90 and 95% significance level. Schreck and Semazzi (2004) and Omendi et al. (2013) highlighted that there is a general increase in warm extremes particularly at night while cloud extremes are decreasing in the horn of Africa. Kinh Uyu et al. (2000), Schrech and Semazzi (2004) studied the climate variability across the Eastern Africa region; results revealed that there has been temperature variability in the region, especially within the arid and semi-arid environments. In addition, by the end of 21st century, roughly 1.5 times the global mean response as a region of climate models suggest that median temperature increases between 3 °C and 4 °C in Africa (IPCC, 2007; Bryan et al., 2013).

Ethiopia is likely to be highly susceptible to future climate change like other African countries (Conway and Schipper, 2011). Some studies have indicated that warming has occurred across much of Ethiopia, particularly since the 1970s, at a variable rate (Conway, 2000; Conway et al., 2004; IPCC, 2007b; Gebrehiwot and van der Veen, 2013). A study made by Ethiopian National Meteorological Services Agency (ENMSA) also reveals that there has been a warming trend in temperature over the past 50 years in the country. The average annual minimum temperature over the country has been increasing by about 0.25°C every ten years while average annual maximum temperature has been increasing by about 0.10°C every decade (ENMSA, 2001).

The aim of this study is to evaluate temperature trends in the semi-arid Geba River Basin located in Northern Ethiopia. The main purpose of this study is the detection and analysis of significant trends or fluctuations in historical air temperature records within the basin. Accordingly, a thorough analysis of climatic data from 1971 to 2013 was carried out in order to identify seasonality, variability and trends of temperature at different time scales.

2. MATERIALS AND METHODOLOGY

2.1 Study Area

The Geba River basin, located in Northern Ethiopia between 38o38'E and 39o48'E and 13o18'N and 14o15'N, is a major tributary of the Tekeze (the Sudanese called Atbara) river, which is the last tributary of the Nile. The basin is surrounded by the Danakil basin in the east, by the Tekeze River basin in the south and the Werie River basin in the west. The Geba River originates from the Mugulat mountains near Adigrat in the north and the Atsbi horst in the northeast and flows south and then westwards to join the River Tekeze on its way to Sudan (Zenebe, 2009). Figure 1 shows the location and the main tributaries of the Geba basin. The basin has a total area of 5137 km2. The Geba joins the Tekeze River at the confluence known as Chemoy.

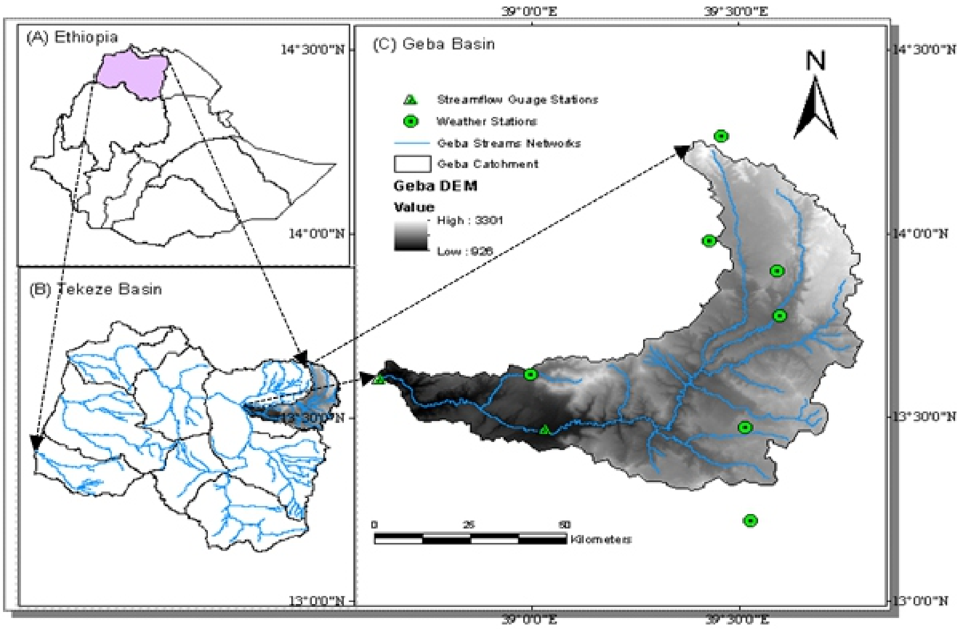


Figure 1 The Geba River basin location

The elevation ranges from 920 m above mean sea level at Chemoy valley at the last outlet point where it joins the Tekeze River to 3301 m above mean sea level at the Mugulat Mountains near Adigrat. Climatic conditions in the study area are quite diverse due to considerable differences in the altitude and relief. About 80% of the annual precipitation in the Geba basin occurs in the Kiremt (rainy) season from June to September and 63% of the annual precipitation is the peak which is recorded in July and August. The mean temperature varies from a minimum average of 6.5oC in the Atsbi plateaus to a maximum average of 32oC at Agbe in the Avergele lowlands.

2.2. Methodology and Data Analysis

A brief overview of the methodology used in data analyses is presented here. In this study, the trend analysis of time series temperature for the Geba river basin is examined, for the period 1971-2013. The daily minimum and maximum temperature is the available meteorological data used in this study. The daily minimum and maximum temperature data were acquired from the Ethiopia National Meteorological Agency for seven meteorological stations in the Geba River basin. The temporal variability of the monthly and annual temperature trends along with their significance are analyzed using Mann-Kendall test and Sen's method which were applied for a given confidence level in order to detect trends.

Mann-Kendall analysis: The non-parametric Mann-Kendall test is a statistical procedure that is appropriate for analyzing trends in data over time (Mann, 1945; Kendall, 1975). Any assumptions as to the statistical distribution of the data are not required by the Mann-Kendall test (e.g., normal, lognormal, etc.,) and irregular sampling intervals and missing data can be used with data sets in this test (Gilbert, 1987). The trend in the data can be measured by the Mann-Kendall Statistic (S). Positive values designate an increase in the climatic variables over time, whereas negative values indicate a decrease in climatic variables over time. The magnitude of the Mann-Kendall Statistic is proportional to the strength of the trend (i.e., large magnitudes indicate a strong trend). Time sequential order of the data used for the Mann-Kendall Analysis is very important. To determine the sign of the difference between consecutive sample results is the first step.

Sign (Xj-Xk) is an indicator function that results in the values -1, 0, or 1 according to the sign of Xj-Xk where j>k ((Motiee and McBean, 2009), the function is calculated as follows:

$$\mathrm{Sign}\ (\mathrm{Xj}-\mathrm{Xk}) = \begin{cases} +1 & if\ (Xj-Xk) > 0 \\ 0 & if\ (Xj-Xk) = 0 \\ -1 & if\ (Xj-Xk) < 0 \end{cases} \qquad (1)$$

The Mann-Kendall statistic (S) is defined as follows (as the sum of the number of positive differences minus the number of negative differences):

$$\mathrm{S} = \sum_{k=1}^{n-1} \sum_{j=k+1}^{n} \mathrm{sign}\ (\mathrm{Xj}-\mathrm{Xk}) \qquad (2)$$

When n value is bigger than 10, the standard normal statistical variables, Z can be described in the following equation:

$$Z = \begin{cases} \frac{S-1}{\sqrt{\text{var}(S)}} & \text{if } S > 0 \\ 0 & \text{if } S = 0 \\ \frac{S+1}{\sqrt{\text{var}(S)}} & \text{if } S < 0 \end{cases} \tag{3}$$

Where,

$$Var(S) = \frac{n(n-1)(2n+5) - \sum_{p-1}^{q} t_p(t_p-1)(2t_p+5)}{18} \tag{4}$$

Where tp is the number of ties for the pth value and q is the number of tied values this means the summation term in the numerator is used only if the data series contains tied values. The standardized test statistic Zs is calculated as follows (Douglas, 2000).

Sen's estimator analysis: In a time series if a linear trend is present, by using a simple non-parametric procedure developed by Sen (1968); the true slope (change per unit time) can be estimated. The slope estimates for the pairs of data are first computed by:

$$\beta = Median\left[\frac{X_j - X_i}{j-k}\right] \text{ for all } k<j \tag{5}$$

Where, Xj and Xk are data values at times j and k (j > k), respectively.

3. RESULTS AND DISCUSSION

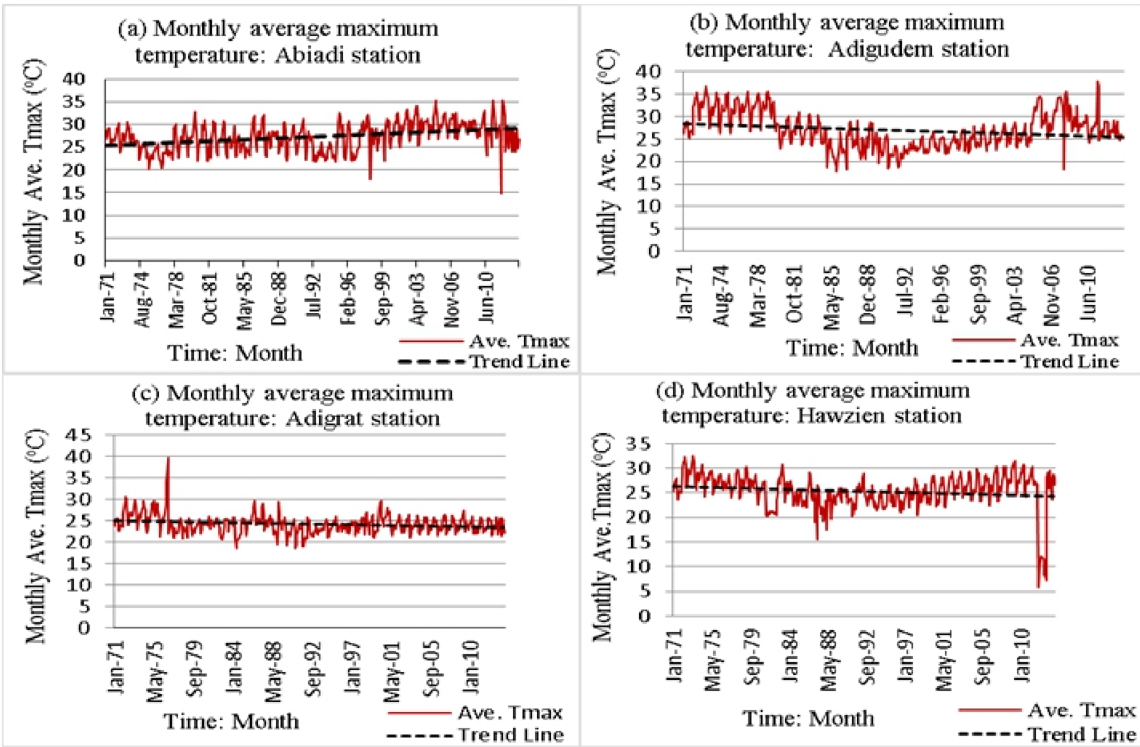
3.1. Analysis of Temperature

Results obtained by application of the Mann-Kendall test on monthly average daily minimum and monthly average daily maximum and annual average daily minimum and maximum temperature data for all the seven stations in the Geba river basin are shown in Tables 1 and 2. If the p value is less than the significance level α (alpha) = 0.05, H0 is rejected. Rejecting H0 indicates that there is a trend in the time series, while accepting H0 indicates no trend was detected in the time series. On rejecting the null hypothesis, the result is said to be statistically significant.

Table 1 indicates that the Null Hypothesis was rejected for all the stations except Hawzien. The monthly maximum temperature shows a greater number of decreasing trends than increasing trends. Decreasing trends were observed at five stations with four stations showing statistically significant trends (Figure 2). Hawzien showed a decreasing trend in monthly maximum temperature but it is not statistically significant at 95% significance level (p=0.264). A statistically significant increasing trend in the monthly maximum temperature was, however, observed at Abiadi and Wukro stations (p=< 0.0001) though the magnitude of the Sen's slope is slightly small 0.008 and 0.009 degree Celsius per year respectively.

Table 1 Results of the Mann-Kendall test for the monthly average maximum daily temperature data

Station name	Mean	Std. deviation	p for α=0.05	Sen's slope, β ($^{O}C/y$)	Trend nature	Trend significance
Abiadi	27.321	3.083	< 0.0001	0.008	Positive	Yes
Adigudem	26.935	4.024	0.002	-0.004	Negative	Yes
Adigrat	24.221	2.226	< 0.0001	-0.002	Negative	Yes
Hawzien	25.346	3.578	0.264	-0.001	Negative	No
Mekelle	25.287	2.553	< 0.0001	-0.007	Negative	Yes
Senkata	24.273	2.917	< 0.0001	-0.008	Negative	Yes
Wukro	26.974	3.365	< 0.0001	0.009	Positive	Yes



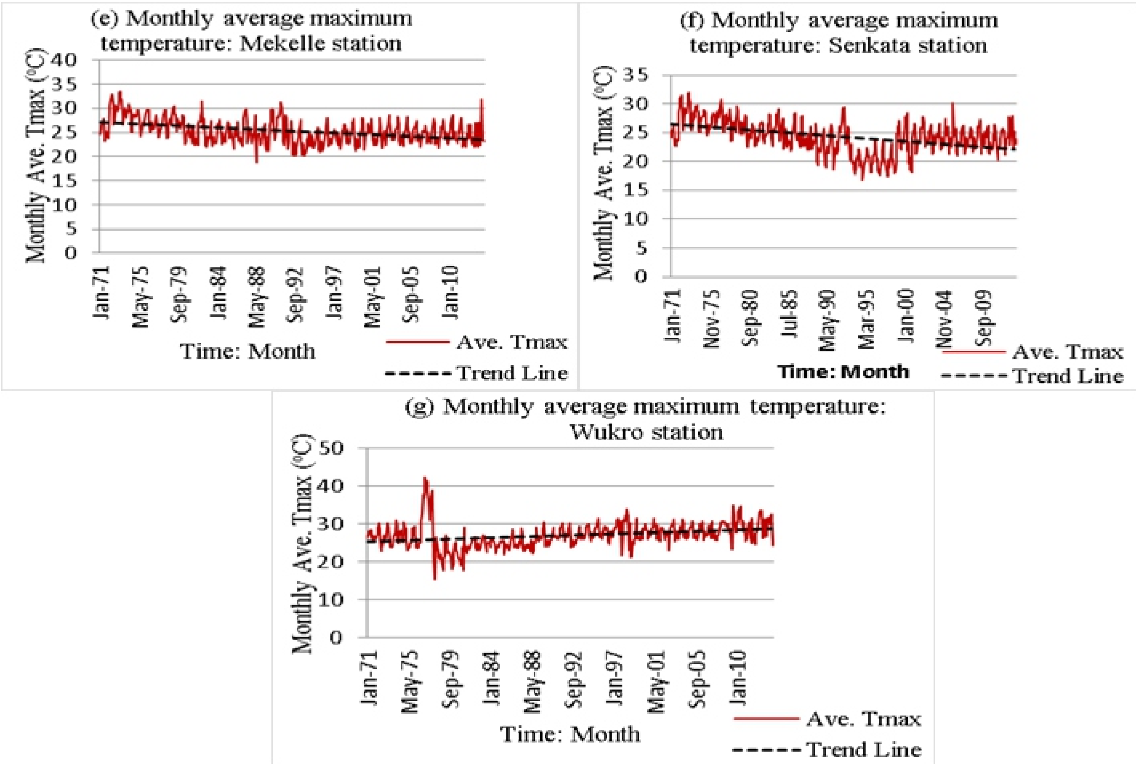


Figure 2 Linear trend line corresponding to monthly average maximum temperature at the seven stations (a) Abiadi (b) Adigudem (c) Adigrat (d) Hawzien (e) Mekelle (f) Senkata and (g) Wukro

The trends in the time series of monthly average minimum temperature were also investigated. Except Adigudem, all the six stations exhibited a decreasing trend out of which four (Abiadi, Adigrat, Hawzien and Wukro) were statistically significant (Table 2). The variable monthly average minimum temperature exhibited a decreasing trend at two stations, but none of these were statistically significant (p=0.069) and (p=0.803) respectively. The increasing trends in monthly average minimum temperature (Figure 3) were observed at one station in which it was not statistically significant (Adigudem, p=0.448).

Table 2 Results of the Mann-Kendall test for the monthly average minimum temperature data

Station name	Mean	Std. deviation	p for α=0.05	Sen's slope (^{O}C/y)	Trend nature	Trend significanc e
Abiadi	13.111	2.56	< 0.0001	-0.004	Negative	Yes
Adigudem	10.658	3.086	0.448	7.07E-04	Positive	No
Adigrat	8.861	4.109	0	-0.005	Negative	Yes
Hawzien	11.069	2.383	0.002	-0.002	Negative	Yes
Mekelle	11.364	2.14	0.069	0.001	Negative	No
Senkata	11.005	2.608	0.803	-1.96E-04	Negative	No
Wukro	12.088	3.404	< 0.0001	-0.009	Negative	Yes

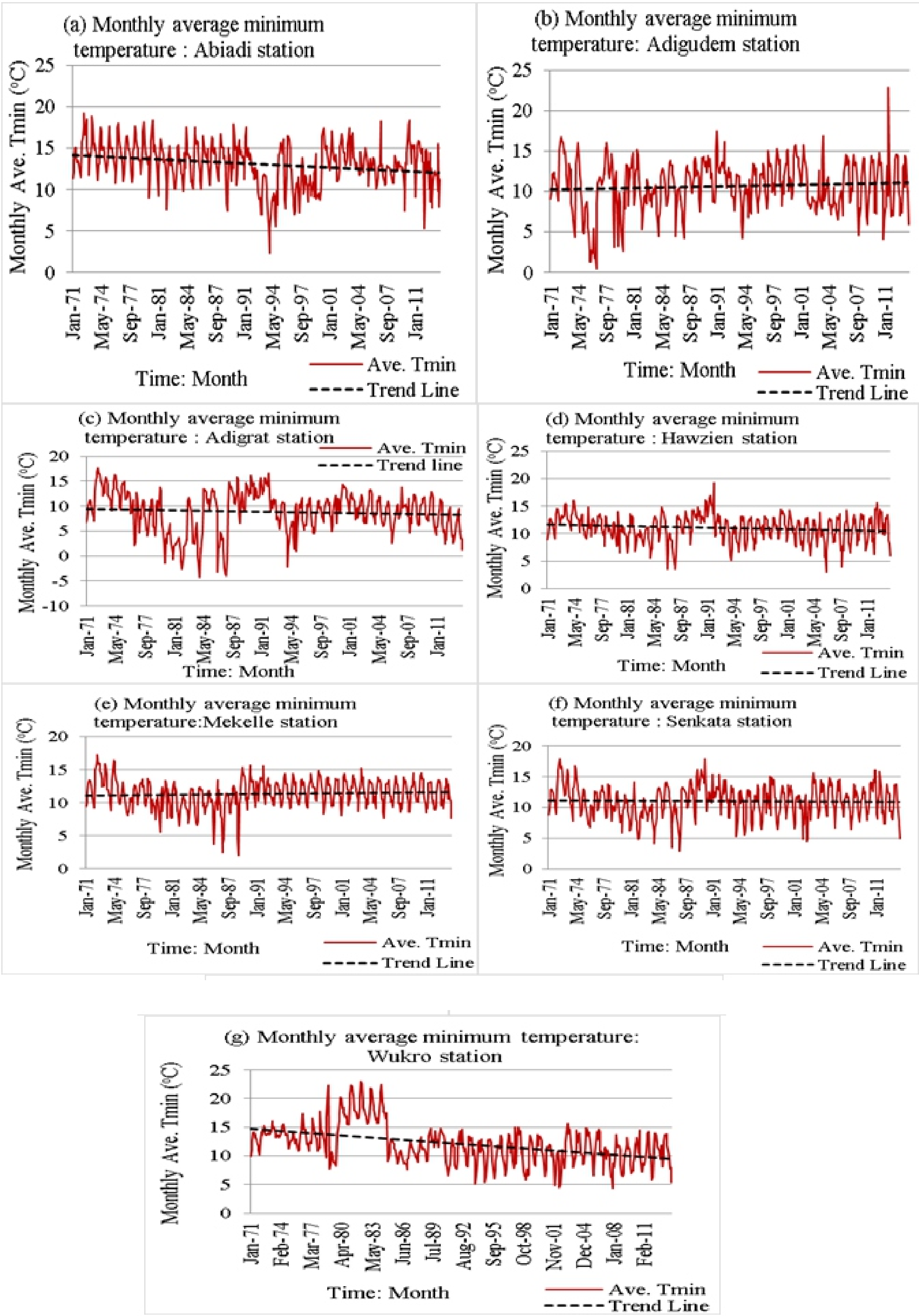
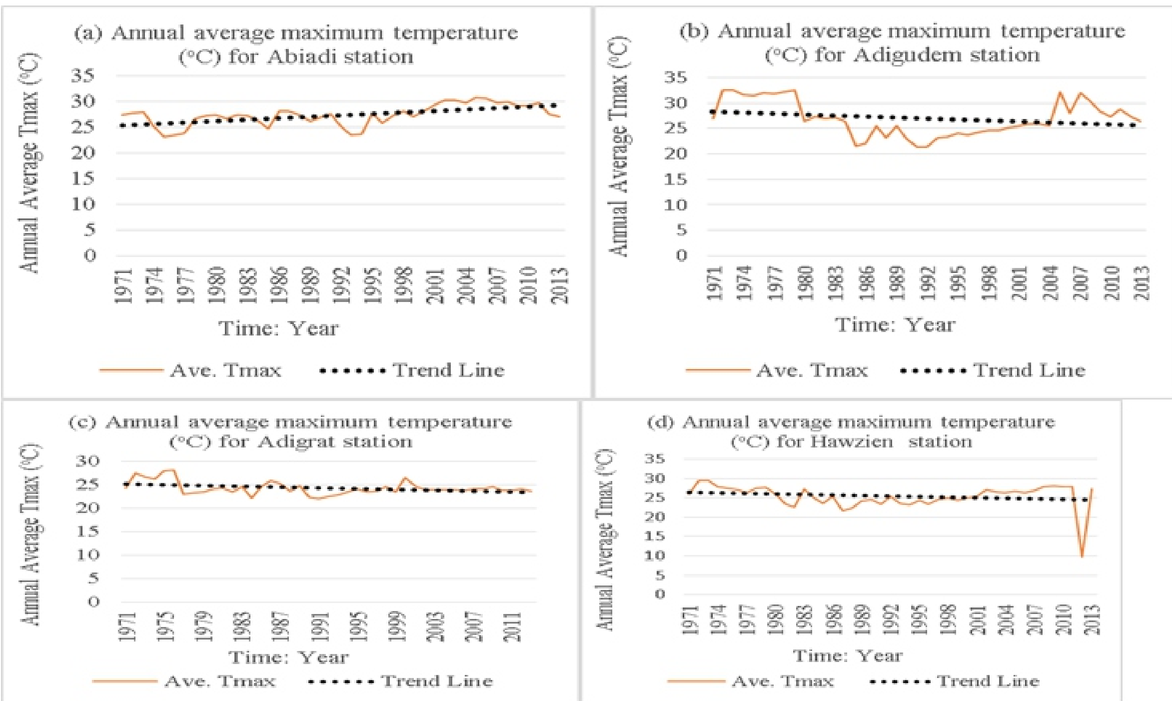


Figure 3 Linear trend line corresponding to monthly average minimum temperature at the seven stations (a) Abiadi (b) Adigudem (c) Adigrat (d) Hawzien (e) Mekelle (f) Senkata and (g) Wukro

As shown in Table 3 the Mann-Kendall test results of the Geba River basin average annual maximum temperature for many years. As can be seen, the annual average maximum temperature showed a rising trend at Abiadi, Hawzien and Wukro stations and except the Hawzien station both Abiadi and Wukro stations have shown statistically significant for the annual maximum temperature with a changing rate of 0.053 oC per decade. Besides there is a decreasing trend (Figure 4) in the stations Adigudem, Adigrat, Mekelle and Senkata of which only Mekelle and Senkata are showing statistically significant decreasing trend at 5% level of significance with a changing rate of 0.084 oC per decade. Therefore, change trend of annual maximum temperature is likely obvious, and the changing rate is deceased by -0.031 oC per decade this indicated that there is slowly decreasing in the annual average maximum temperature in the study area, although an overall increase of 0.20oC per decade has been detected for the entire country over the last five decades (NMA, 2007).

Table 3 Results of the Mann-Kendall test for the annual average maximum temperature data

Station name	Mean	Std. deviation	p for α=0.05	Sen's slope (^{O}C/y)	Trend nature	Trend significance
Abiadi	27.32	2.042	0	0.092	Positive	Yes
Adigudem	26.936	3.491	0.603	-0.033	Negative	No
Adigrat	24.221	1.409	0.295	-0.018	Negative	No
Hawzien	25.347	3.095	0.787	0.007	Positive	No
Mekelle	25.288	1.795	0.009	-0.065	Negative	Yes
Senkata	24.273	2.354	0	-0.098	Negative	Yes
Wukro	26.975	2.637	< 0.0001	0.113	Positive	Yes



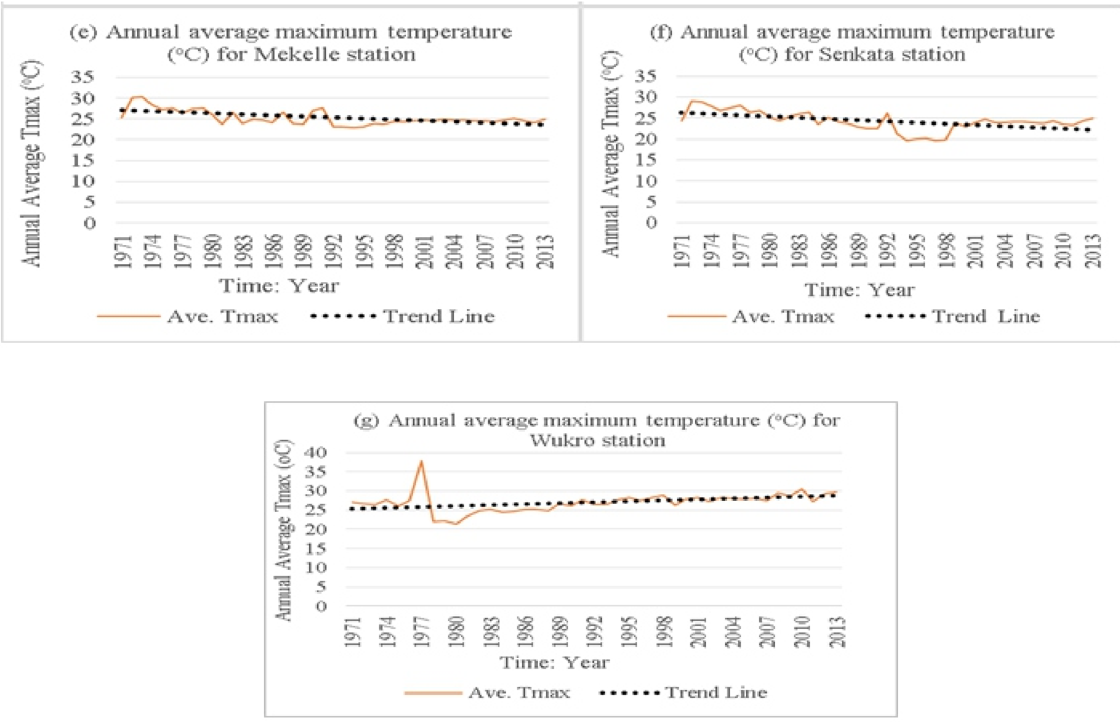


Figure 4 Linear trend line corresponding to annual average maximum temperature at the seven stations (a) Abiadi (b) Adigudem (c) Adigrat (d) Hawzien (e) Mekelle (f) Senkata and (g) Wukro

For the annual minimum temperature data, the Mann-Kendall test shows that there is an increasing trend for Adigudem and Mekelle though they are not statistically significant. The Mann-Kendall test is statistically significant for Abiadi and Wukro stations for their decreasing trend. For both of these stations, therefore, null hypothesis H0 is rejected and thereby implying that there is trend that can be seen in the data (Figure 5). As seen in Table 4 below outputs of the statistical techniques for annual average minimum temperature revealed that there is a statistically significant decreasing trends occurred in the stations Abiadi and Wukro (p=0.003) and (p=0.000) respectively. Although there are a mix of positive (Adigudem and Mekelle) and negative (Adigrat, Hawzien and Senkata) trends in the stations they felt to show the statistical significant at 95% confidence level for the variable annual average minimum temperature.

Table 4 Results of the Mann-Kendall test for the annual average minimum temperature data

Station name	Mean	Std. deviation	p for α=0.05	Sen's slope (^{O}C/y)	Trend nature	Trend significanc e
Abiadi	13.112	1.719	0.003	-0.058	Negative	Yes
Adigudem	10.659	2.015	0.755	0.009	Positive	No
Adigrat	8.862	3.211	0.243	-0.047	Negative	No
Hawzien	11.069	1.491	0.079	-0.026	Negative	No
Mekelle	11.364	1.181	0.675	0.008	Positive	No
Senkata	11.005	1.519	0.884	-0.001	Negative	No
Wukro	12.089	2.739	0	-0.091	Negative	Yes

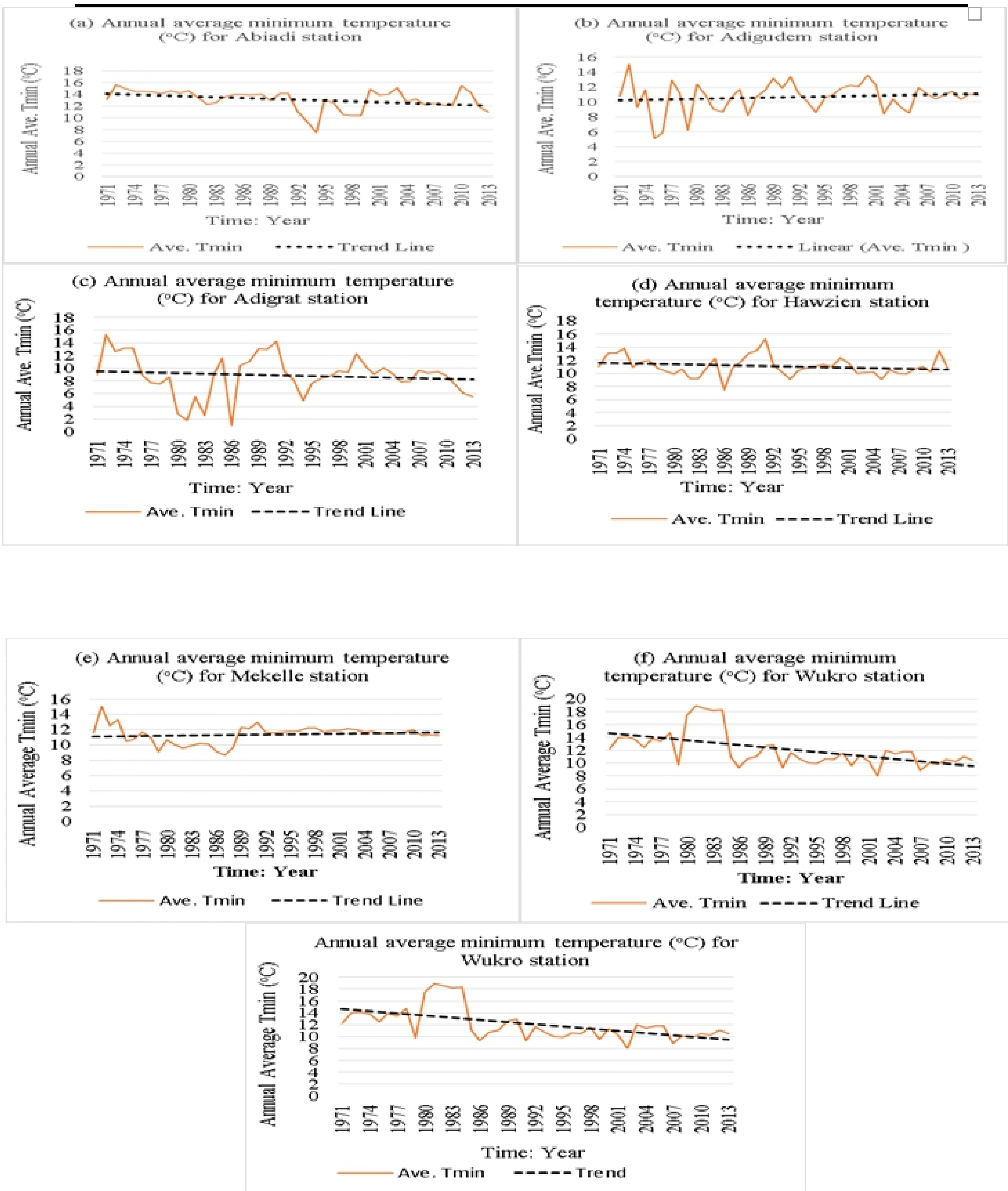


Figure 5 Linear trend line corresponding to annual average minimum temperature data for each of the stations in the Geba basin

4. CONCLUSIONS

The analysis of monthly and annual daily minimum and daily maximum temperature records for different stations in Geba River basin have shown a mix of decreasing and increasing trend for the study period. The monthly and annual maximum temperature exhibits a statistically significant increasing trend in the Abiadi and Wukro stations and at Hawzien station the maximum annual temperature and at Adigudem station monthly minimum temperature exhibit positive trend though they are statistically insignificant at 95 % confidence level. Monthly average minimum temperature also showed a decreasing trend at six stations of which four are significant and two are not and one station showed an increasing trends and statistically insignificant trend at the 95% confidence level and for the annual average minimum temperature five stations showing a decreasing trend only two stations are statistically significant. In general, the findings showed that a slightly decrease in the monthly and annual temperature occurred in the 1971 to 2013 span of time. Finally results of this study contribute to climate change research in the region and provide inputs for better planning in the adapting to changing climate.

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Groundwater System Modelling With Parametric Uncertainty Characterization For Sai-Gomti Inter-Fluvial Plain

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ABSTRACT

The groundwater resource is vital for water supply, irrigation and urbanization in Indo-Gangatic alluvial plains. The aquifers underlying the Indo-Gangetic alluvial river systems are recharged by rains and also by seepage from the irrigation canal commands. However, the resource is threatened due to more water demand for advancement in agricultural sector together with rapid industrialization. Withdrawal of groundwater matching with annual groundwater recharge is generally recommended as sustainable utilization. Excessive withdrawal of groundwater, mainly for irrigation, has resulted in a significant depletion of the groundwater table. The problems of excessive groundwater extraction in the tail reaches, and waterlogging and salinization in the head reaches are often found in the same canal command. In such areas, there is considerable potential for better water management through groundwater system modelling adopting conjunctive utilization of surface water with groundwater. Present work simulates the groundwater system in Sai-Gomti interfluvial region which is a part of Indo-Gangatic alluvial plain in Uttar Pradesh, India. Visual MODFLOW was calibrated and validated for water level data available for 8 years (2005 to 2013). The steady state value of hydraulic conductivity was used as the initial value for the transient state model. PEST (Parameter Estimation) analysis was performed to optimize the hydraulic conductivity and storivity. After a number of trial runs, where the input/output stresses were varied, computed water level matched reasonably to observed values. An uncertainty analysis was also performed due to possible uncertainty in hydraulic conductivity. Further, effect of change in recharge rate and withdrawal rate is also investigated to predict the corresponding changes in water levels.

Keywords—Groundwater simulation, VISUAL MODFLOW, Sai-Gomti interfluve region, uncertainty analysis

1. INTRODUCTION

Water table is declining in various parts of the world due to overexploitation of groundwater resources. The population growth, urbanization and effects of global climate change, such as, longer spells of summer and erratic or failing monsoon; have resulted in increasing urban an agricultural water demands. This, in turn, has stressed aquifer systems where groundwater is the dominant source of water supply (Taniguchi et al. 2009). During last few decades groundwater level in several parts of country has been declining due to increase in withdrawal of groundwater for different activities. Rapid increase in population and changing lifestyles are major factors causing rise to groundwater crisis. Intense competition among agriculture, industry and domestic sectors are responsible for declining groundwater table (Srivastava, 2003). In this scenario of declining groundwater level, it is imperative to investigate the groundwater system with respect to recharge, withdrawal and storage capacity and groundwater system modelling for groundwater management.

Chakravorty et al. (2014) investigated the effect of conjunctive use on waterlogging in lower Gandak basin of Bihar. Groundwater flow modelling of Hindon-Yamuna interfluve region, western Uttar Pradesh was conducted by Alam F and Umar R (2013). Gosh and Kashyap (2012) utilized optimization technique in pre-calibrated simulation model of groundwater flow. Optimized sustainable groundwater extraction management of Lucknow city was carried out by Singh et al. (2012). Ahmed et al. (2008) investigated water balance studies in parts of Krishni-Yamuna interstream area in western Uttar Pradesh. Local scale groundwater flow model was developed by Ebraheem et al. (2004); Palma and Bentley (2007) for groundwater resources management. Groundwater system modelling of Azraq basin, Jordan was performed by Abdulla et al. (2000).

The present work simulates the groundwater system to predict the groundwater level in Sai-Gomti interfluvial region which is a part of Indo-Gangatic alluvial plain in Uttar Pradesh, India. Also, characterization of uncertainty in aquifer parameters has been performed using Fuzzy-alpha cut (FAC) technique.

2. GROUNDWATER SYSTEM SIMULATION

The present work utilized Visual MODFLOW groundwater model to simulate the groundwater flow processes. It solves numerically the 3-D groundwater flow equation (Equation 1) for porous media by finite-difference method.

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial x}\left(K_{zz}\frac{\partial h}{\partial z}\right)-W=S_s\frac{\partial h}{\partial t} \tag{1}$$

Where, x, y, z are Cartesian coordinate axes, h = potentiometric head [L], Kxx, Kyy, Kzz = hydraulic conductivities along x, y and z axes [LT^{-1}], W = volumetric flux/unit volume and represents sources and/or sinks of water [T^{-1}], S_S = specific storage of the porous material [L^{-1}], and t = time.

3. STUDY AREA

The study area, Sai-Gomti inter-fluvial plain, is a part of Indo-Gangatic alluvial plains. The location map of study area is shown in Figure 1. The study area falls in Barabanki, Raebareli, Sultanpur, Pratapgarh and Jaunpur districts of Uttar Pradesh, India. The study area lies between latitude $26^{O}45'36”$ and $25^{O}41'60”$ N, and longitude $81^{O}6'36”$ and $82^{O}49'12”$ E and is estimated to be 8287.50 km^2.The area is representative of the whole of the Sarda Sahayak Canal command. It is expected that the methodology adopted and conclusions arrived, would be applicable elsewhere in the canal command area. The model area is bounded in the North by the river Gomti and in the South by Sai River. The confluence of Sai-Gomti River in Jaunpur district forms an eastern extremity of the area and the feeder channel section between Gomti and Sai aqueducts lying between 153.4 km and 184.16 km from the western boundary. The average rainfall is 1,110 mm/year, out of which 1000 mm/year occurs during the 40-45 monsoon days between June and August (Livingston 2009; Foster and Choudhary 2009). The temperature is high in months of April-June. The maximum temperature rises up to 42^{0}C in month of June and minimum temperature drops to 10^{0}C during month of January.

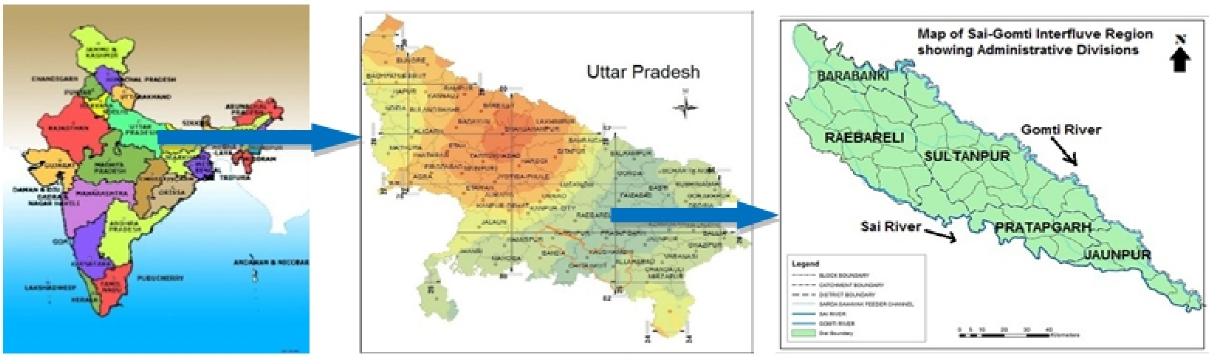

Figure 1 Location map study are: (a) India (b) Uttar Pradesh (c) Study area

3.1 Geology of study area

The Sai-Gomti inter-fluvial plain forms a part of central Ganga plain. It is laid by soft/ unconsolidated sediments. The soft sediments have assumed an enormous thickness which varies from place to place. The observations of deep drilling study conducted by CGWB indicated that it is 487m thick at Janauli in Raebareli district where granite basement was encountered; 399m at Kandhai in Pratapgarh district (Vindhayan sandstone as basement) and 745m at Leduka in Jaunpur district. The alluvium comprises alternation of sand-silt-clay sequence, sometimes the sequence gets admixed with concentrations of calcium carbonate or Kankar as called in local language.

4. MODEL INPUTS AND BOUNDARY CONDITIONS

The area of 8287.50 km^2 has been divided into equal sizes grid network of 30 columns and 86 rows. Thus, the area has been divided into 2580 cells of equal dimensions. Each cell measures 2500 meters in length and with identical width. Hence, each cell represents 6.25 km^2. Aquifer geometry and characteristics of the area indicate presence of phreatic aquifer together with semi-confined aquifer. This zone as a whole has been simulated in the model. Due to inadequacy of the data on aquifer parameters,

,the figures for hydraulic conductivity and specific yield were adopted from the similar contiguous areas and the available reports. Single layer groundwater flow model has been developed based on the available information related to aquifer characteristics, rainfall, and other data sets of study area.

4.1 Hydraulic properties

The hydrological setting of the Sai-Gomti interfluve region is described by Bhatnagar (1966) and CGWB (2009). Since, location specific input parameters were not available, representative values of hydraulic conductivity and specific storage for the aquifer system were taken from standard literature. The input value of hydraulic conductivity (K) was taken as 7.0 m/day and specific storage value of 0.001/m with coefficient of storage (S) as 0.15. These values were modified during the process of calibration of the model. After the parameter estimation (PEST) run these values were modified as hydraulic conductivity (K) = 6.0057 m/day and specific storage value of 0.900/m with coefficient of storage (S) as 0.16.

4.2 Estimation of stresses

In present study, stress period of 8 years (2005-2013) has been considered. Inflow and outflow are the main constraints of stresses in groundwater system. The outflow from groundwater systems can be inform of pumpage, evapo-transpiration or rivers and boundaries. Recharge due to rainfall and return flow from irrigation are main components of inflow in groundwater systems. The outflow from groundwater system is called discharge and inflow is called as recharge. Blockwise groundwater draft figure (CGWB, 2003) has been used for cell-wise distribution of draft.

4.3 Estimation of recharge

Recharge due to inflow from river/canal and head dependent boundaries would be taken care by the model automatically depending on the heads in the boundaries. The areal recharge due to rainfall has been taken as 20% of rainfall. The estimated values were applied to the respective grid in the model using recharge boundaries.

4.4 Boundary conditions

Consideration of boundary conditions of a groundwater model is one of the most important aspects of calibrating and validating the model. The imprecision in assigning boundary conditions can result in imaginary predictions of water levels. In present study, the western and eastern boundaries bounded by Sai and Gomti rivers were considered as River Boundaries. These two boundaries meet on the south-eastern corner at the confluence of the two rivers. Cluster of few grid cells in western part of the area are simulated as General Head Boundaries, as these grid cells are not bounded by either of the rivers. Heads were assigned to General Head Boundaries with the help of water level data.

5. MODEL CALIBRATION AND VALIDATION

The purpose of model calibration is that the model can replicate field measured heads and flows. Calibration can be carried out by trial and error adjustment of parameters or by using an automated parameter estimation (PEST). In present study, automated parameter estimation (PEST) technique has been used.

5.1 Steady state calibration

The steady state condition is a condition that existed in groundwater system before any development had taken place. Matching the initial heads observed for groundwater system with the hydraulic heads simulated by MODFLOW is called steady state calibration. In present study, the aquifer system was taken to be in steady state during November 2005. It was chosen to run and calibrate the model under steady state for this period using 36 observation wells in the study area. The groundwater head in the aquifer model was computed by using Visual MODFLOW. Waterloo Hydrogeologic Software (WHS) solver package of MODFLOW has been used for groundwater flow computation. The computed water level accuracy was judged by comparing the mean error with mean absolute and Root Mean Squared (RMS) error (Anderson and Woessner 1992). Mean error is 0.005 m and RMS error in the present simulation is 0.08 m. The absolute residual mean is 0.056 m. Figure 2 shows the computed vs. observed head for steady state calibration.

5.2 Transient state calibration

The model was calibrated in transient state from 2005 to 2012 (7 years). Visual MODFLOW uses boundary conditions imposed by the user to determine the length of each stress period. After a number of trial runs, compute water levels were matched fairly reasonably to observed values. The RMS error for the transient state model is 0.442 m. A comparison of observed and computed heads at different observation wells are shown in Figure 3.

The calibrated model provided hydraulic conductive (K) value as 6.50 m/d and coefficient of storage (S) as 0.23. Also, the calibrated model was validated with the available data of year 2013 and acceptable difference between observed and calculated values was observed.

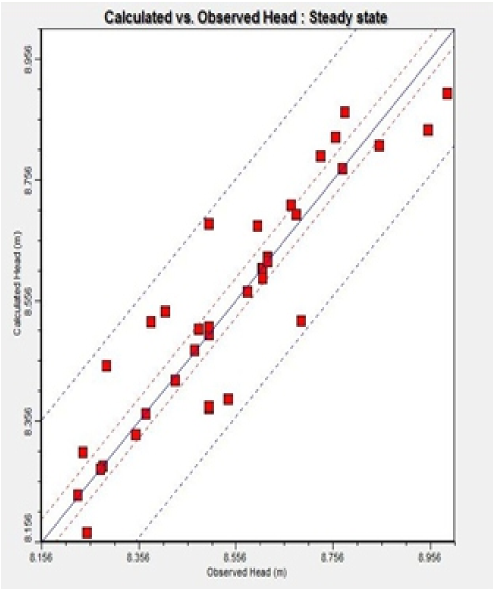

Figure 2 Steady state calibration

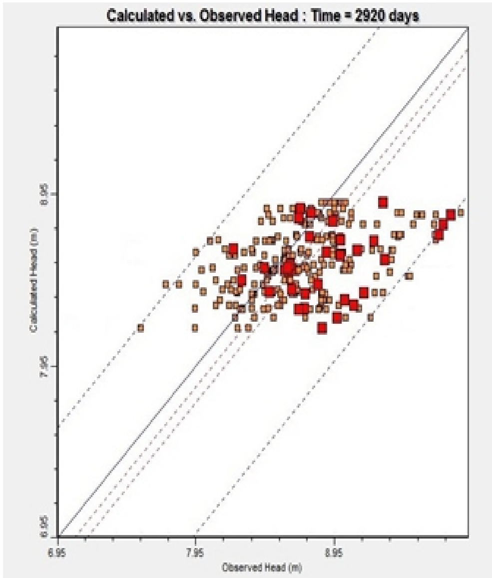

Figure 3 Transient state calibration

6. PREDICTION AND UNCERTAINTY ASSESSMENT

Three different scenarios were considered to predict the behavior of the groundwater regime in Sai-Gomti interfluve region during the period 2014-2017. These scenarios are explained below.

6.1 Scenario 1: Increase in current withdrawal rate

During this prediction scenario the current withdrawal rate was increased by 20% from 2014 to 2017, over a period of 3 years. The initial recharge was kept constant throughout the prediction period. It was observed during this prediction run that the blocks: Goasainganj, Trivediganj, Haidergarh , Jagdishpur, Sukul bazar have higher drawdown ranging between 8.77 to 8.85 m. These results show that the areas nearby River Gomti are significantly affected. The maximum drawdown of 8.85 m was observed in Haidergarh block of Barabanki district (Figure 4).

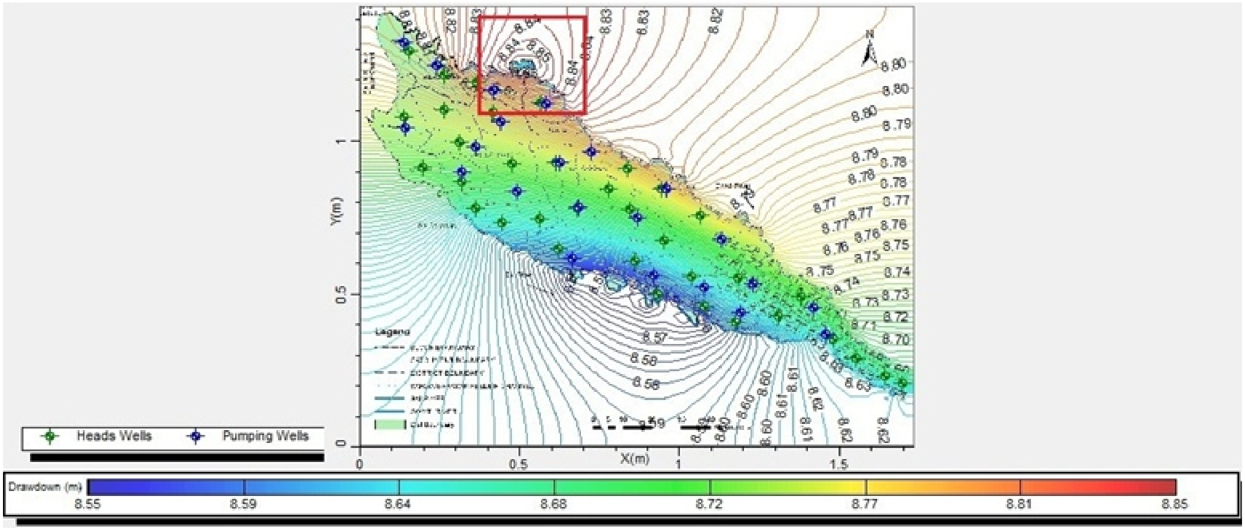

Figure 4 Drawdown prediction in scenario 1

6.2 Scenario 2: Decrease in recharge

In this prediction scenario, the combined effect of increased withdrawal rate by 20% and reduced rainfall by 20% was examined. The combined effect of both these factors revealed that the almost all the blocks in study area have declining water level trend with maximum drawdown of 8.90 m in Haidergarh block of Barabanki followed by Trivediganj , Musafirkhana, Kurwar blocks of Sultanpur and Shivgarh, Singhpur blocks of Raebareli district (Figure 5). The drawdown in these areas is higher as compared to scenario 1.

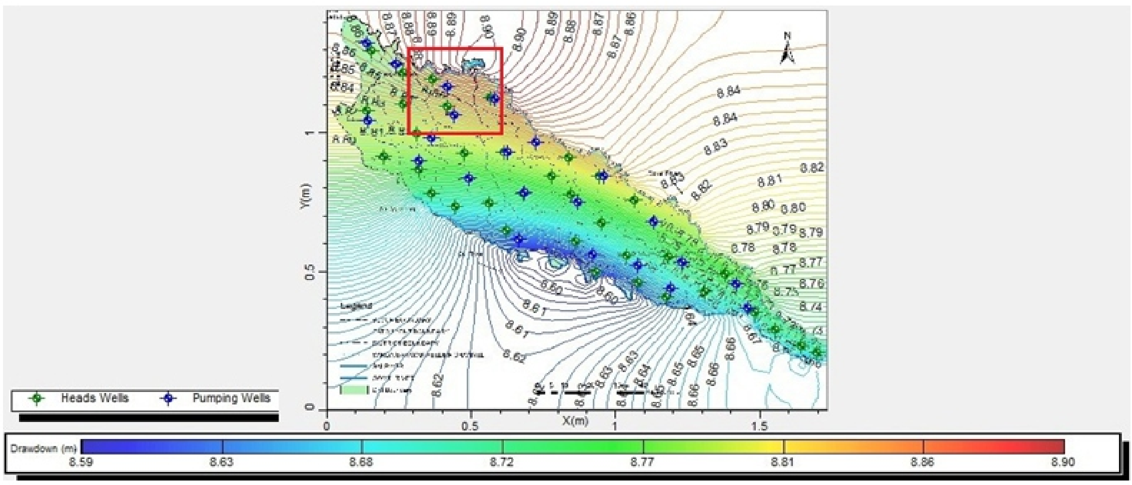

Figure 5 Drawdown prediction in scenario 2

6.3 Scenario 3: Introducing recharge through canal

In order to mitigate the groundwater depletion at some locations in study area, additional recharge of 300 mm/year was applied over the study area. This is a practically possible scenario. The recharge from surface water structures and through water harvesting can positively help affected areas (Figure 6).

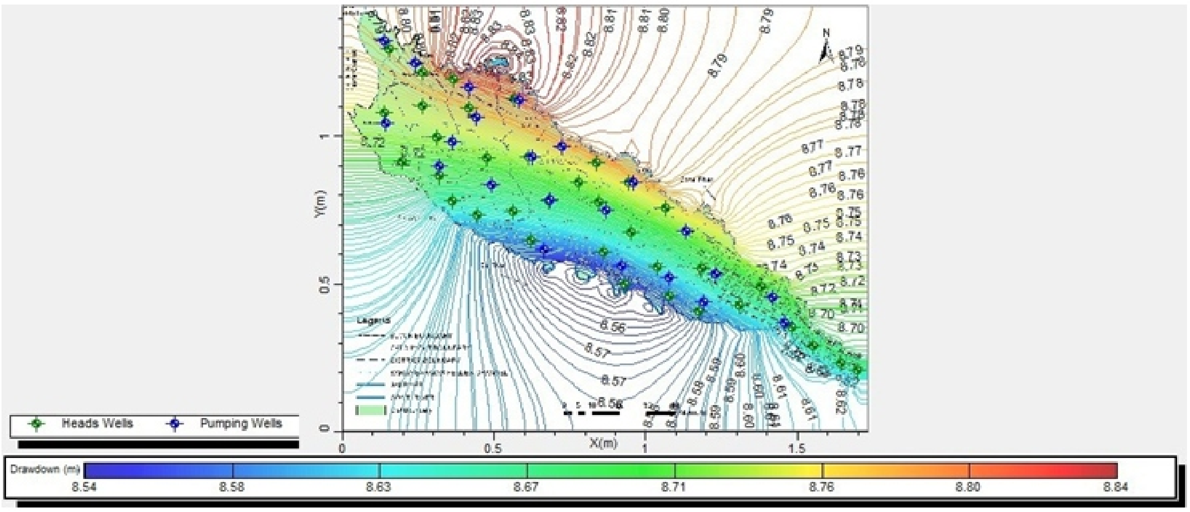

Figure 6 Drawdown prediction in scenario 3

6.4 Uncertainty Analysis

Groundwater system modeling is composed of many non-deterministic components with non-linear coupling. It is expected to face the high degree of uncertainty in groundwater system modelling which results imprecision in predicted groundwater level. Parametric variation in model inputs can be handled with statistical techniques. Fuzzy numbers are used as alternative tool to address the parametric uncertainty when the model input parameters are limited or imprecise (Singh, 2011). In present study, fuzzy alpha-cut (FAC) technique has been used to handle the uncertainty in hydraulic conductivity of groundwater flow model Visual MODFLOW. In this technique, the uncertainty in parameter is represented using fuzzy set theory.

The uncertain parameters are assumed to be fuzzy numbers with some membership functions. Figure 7 shows a parameter P represented as a triangular fuzzy number with support of A_0. The wider the support of the membership function, the higher the uncertainty. The fuzzy set that contains all elements with a membership of α ε [0,1] and above is called the α-*cut* of the membership function. At a resolution level of α, it will have support of A_α. The higher the value of α, the higher the confidence in the parameter (Li & Vincent, 1995).

The membership function is cut horizontally at a finite number of α -levels between 0 and 1. For each α -level of the parameter, the minimum and maximum values of the output are determined. This data is then directly used to construct the corresponding fuzziness (membership function) of the output which is

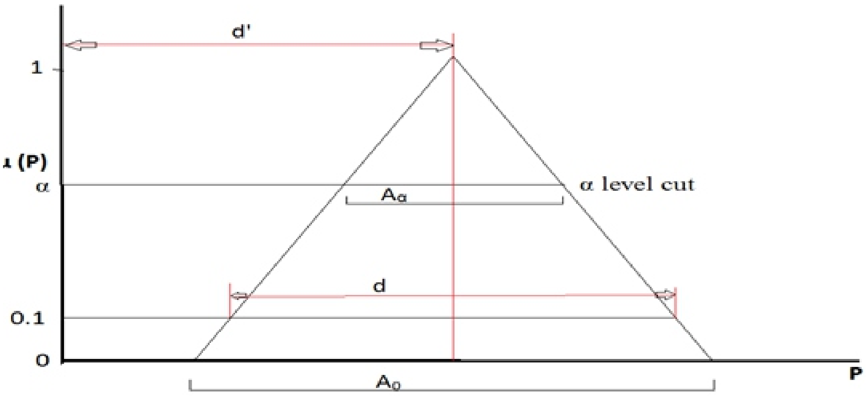

Figure 7 Fuzzy number, its support and α-cut

used as a measure of uncertainty. The measure of uncertainty is given by the ratio d/d' (Abebe et al., 2000).

In present study, uncertainty analysis was performed to investigate uncertainty in hydraulic conductivity. Uncertainty of ± 15% was assumed in hydraulic conductivity from the mean value of 6 m/d. The resulting uncertainty was below 1% which suggests that head value is least sensitive to change in value of hydraulic conductivity up to ± 15% uncertainty.

7. CONCLUSIONS

The present study provides a framework for understanding of aquifer system of Sai-Gomti inter-fluvial plain, Uttar Pradesh, India, under different stresses with parametric uncertainties. The groundwater system has been simulated for both steady and transient state conditions using groundwater simulation model Visual MODFLOW. The developed model has been employed to predict the groundwater level under increased withdrawal rate and varying recharge rate scenarios. Also, uncertainty analysis performed using Fuzzy alpha-cut technique. Results of uncertainty analysis revels that the uncertainty associated with groundwater level prediction is less than 1% against ± 15% uncertainty in hydraulic conductivity.

Acknowledgements

We duly acknowledge the financial assistance from Science and Engineering Research Board (SERB), Department of Science and Technology, Government of India, New Delhi. The financial assistance received for this study was used for Junior Research Fellowship of second author.

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