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Journal of Flood Engineering and Science Research

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Typical subjects covered by this journal include:

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Contents

Flood Estimation By Different Flood Frequency Analysis Methods – A Comparison

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A B S T R A C T

The Flood Frequency Analysis (FFA) is a central step in a hydrological risk assessment. In general the FFA aims at estimating flood quantiles which are the discharge values whose return period is large. In the present work, an annual maximum flood peak series considering 80 observations for Alaknanda River at Karnaprayag, Uttarakhand has been used for flood frequency analysis. The analysis of flood discharge for Alaknanda River has been carried out by Normal Distribution, Log Normal Distribution, Gumbel Distribution, Pearson Type-III Distribution, Log Pearson Type III Distribution, Powel Method, Ven Te Chow Method, Foaster Type-III Method, Stochastic Modelling Method, Modified Stochastic Modelling Method, and Wakeby Distribution Method. A single computer package has been compiled and developed incorporating probability distributions of above mentioned eleven different flood frequency analysis methods. The models were compared using statistical measures such as root mean square error, bias, and standard error. The goodness of fits of all these models was evaluated using the test of D-Index. The study clearly indicates that the Gumbel Distribution could be regarded as the best for sample flood data for the Alaknanda River. The study also reveals that the methods like Pearson Type-III Distribution, Wakeby Distribution, Foaster Type-III Distribution can also be adopted for flood frequency analysis for the sample data of Alaknanda river. The software developed shall be useful to Field Irrigation Engineers in their endeavour and the application of package shall result in considerable saving in time, cost and manpower without sacrificing accuracy.

1.0 INTRODUCTION

The Flood Frequency Analysis (FFA) is a central step in a hydrological risk assessment. In general the FFA aims at estimating "flood quantiles" which are the discharge values whose return period is large (usually greater than 10 years). It has many operational applications, including design of civil engineering structures or mapping of flood-prone areas

The sample data can be either annual flood series or partial duration series. In annual flood series highest flood for each year are considered while in partial duration series all the floods above particular threshold are considered. The partial duration series is used only for estimating the magnitudes and frequency of events which have low recurrence interval. However, Langbein (1949) has shown that the discharge calculated from the partial duration series differs very little from that calculated from an annual maximum series for recurrence interval greater than 10 years. Various parameter estimation techniques like method of moments, method of maximum likelihood, method of least square, method of probability weighted moment etc. are available in the literature.

For estimating the quantiles at different recurrence intervals, commonly used probability distributions for flood frequency analysis are:

I. Normal Distribution
ii. Log Normal Distribution
iii. Gumbel Distribution
iv. Pearson Type-III Distribution
v. Log Pearson Type-III Distribution
vi. Powel Method
vii. Ven Te Chow Method
viii. Foaster Type-III Method
ix. Stochastic Modelling Method
x. Modified Stochastic Modelling Method
xi. Wakeby Distribution Method, etc.

In frequency analysis correct inference about the distribution which fits the peak flood series of a site is crucial as various distributions fitted to the same data result in different estimated values at the extrapolation range. There is no general agreement among hydrologists as to which of the various theoretical distributions available should be used for modelling the peak flood series at a site. The reason being that the hydrologists try to infer the probability distribution using the sample data which is subjected to sampling variability. The conclusions arrived regarding the correct distribution based on the given sample data is influenced by the extent to which the data satisfies the basic assumptions needed for flood frequency analysis and method of parameters estimation, model used and goodness of fit test adopted.

In the frequency analysis goodness of fit of various distributions is examined based on some statistical criteria. Most commonly adopted tests of goodness of fit are Chi-square and Kolmogorov-Smirnov tests and D-Index method. Chi-square and Kolmogorov–Smirnov tests are applied for checking the of fitting of probability distributions to the recorded data. Diagnostic test of D-index is used for theselection of a suitable distribution for estimation of Maximum Flood Discharge(MFD). The use of Chi-square and Kolmogorov- Smirnov tests are not being encouraged by hydrologists for testing the goodness of fit of the given data. The reason for this is the importance of the tails of the frequency distributions and the insensitivity of these statistical tests in the tails of the distribution i.e. these tests may tell us that the two different probability distributions fit the given data equally well, but they are unable to explain the reason for wide variations in the estimates of the same for higher recurrence interval.

Therefore, in this paper D- Index method has been described for the goodness of fit test. The distribution having the least D-index is identified as the better suited distribution for estimation of MFD (Vivekanandan, 1981). Once the suitability of the distribution is decided, the floods of different return periods may be estimated with the help of the sample data.

2.0 LITERATURE REVIEW

A large number of comparative studies of FFA implementations have been reported by Hosking et al.(1985); Kroll and Stedinger(1996); GREHYS(1996); Ouarda et al.(2006); Meshgi and Khalili(2009); Sankarasubramanian and Srinivasan(1999) etc. The comparison framework varies from one study to another, and based on Monte Carlo simulations, statistical tests, and graphical methods etc. Bobee et al.(1993) advocated a systematic approach to compare distributions used in flood frequency analysis. Abramowitz and Stegun(1972); Ahmad et al(1988); Arora & Singh(1989); Benjamin and Cornel(1972); Cunnane(1987, 1989); Greenwood et al(1979); Gumbel(1958); Hosking et al(1985); Jing et al(1989); Landwehr et al(1979a,b; NERC,1975); USGS(1982); and many others have carried out detailed studies about the application of Probability Distributions in Flood Frequency Analysis(FFA) and their parameter estimation methods.

Haktanir (1993) discussed an evaluation of various stream flow frequency distributions using annual peak data, the methods like Log-Normal Distribution, Gumbel Distribution, Pearson Type-III Distribution, Log-Pearson Type-III Distribution were applied to annual peaks series of stream flow data and it was found that the Gumbel Distribution and Log-Normal-III Distribution were better than the other distributions.

3.0 FITNESS OF THE FREQUENCY DISTRIBUTION

The Goodness of fit test is necessary to judge the suitability of the distribution. The result of frequency distribution depends on the length of the hydrological series. The flood frequency and the magnitude relation is important as the smaller flood events happen relatively frequently whereas the large floods occur rarely but cause the most damage. In India, a number of scholars have tried to validate different flood frequency distributions in different river basins. Most of the scholars have tried their best to give the best probability distribution through statistical tests and found different results. D-Index test is one of the most commonly used. Thus, flood frequency analysis of various rivers help in determining the best distribution for predicting the frequency of floods of a given magnitude.

Water Resources Council of the United State has suggested the D-Index method for comparison purposes for the fitness of the distribution. The distribution giving the minimum value of D-Index is considered as best fit distribution. The U.S.W.R.C. has also suggested the flood peak for the specified series of recurrence intervals 2, 5, 10, 15, 20, and 30 years for D-Index test. D- Index can be estimated with the help of formula given below:

$$\text{D-Index} = (1/\overline{X})\sum_{i=1}^{n}\left[X_{i(obs)} - X_{i(comp)}\right]$$

Where, $\overline{X}$ = mean of the observed series,
$X_{i(obs)}$ = value of the observed discharge data at the i^{th} recurrence interval, and
$X_{i(comp)}$ = value of the computed discharge data at i^{th} recurrence interval.

The distribution for which, D-Index gives the minimum value, is considered to be the best fit distribution. In this study, the fitness of the frequency distributions has been tested using D- index method. The flood frequency analyses have been carried out for the site by all eleven methods mentioned above.

The best fit distribution for the site under study was determined to use it for flood frequency analysis at a particular site in the further hydrological analysis. Many statisticians such as Haan(1977) and Yevjevich(1999) recommended that Standard Error(SE), Root Mean Square Error(RMSE) and standard bias(BIAS) can also be used for the comparison between fitted distributions, these measures were computed as :

$$\text{Standard Error} = \left[\frac{\sum_{i=1}^{n} \left[Q_o - Q_c \right]^2}{N-M} \right]^{0.5}$$

$$\text{RMSE} = \sum_{i=1}^{n} \left(\frac{Q_o - Q_c}{Q_o} \right)^2$$

$$\text{BIAS} = \sum_{i=1}^{n} \left(\frac{Q_o - Q_c}{Q_o} \right)$$

Where, N = Sample Size, Qo = Observed Discharge, Qc = Computed Discharge, M = The Number of Parameter Distribution.

4.0 DATA ANALYSIS AND DISCUSSION

A series of 80 years of flood discharges of Alaknanda River at Karnaprayag in Uttarakhand were used to carry out the flood frequency analysis by eleven statistical models as described earlier to estimate the flood magnitude for various return periods. Integrated computer software was used to compute the parameters of the distributions and peak flood. These parameters were estimated by method of moment. The software follows the calculations like mean, standard deviation, frequency factor and probable peak flood for various return periods and the magnitude of D-index has been computed which gives the decision about the suitability of distribution to be used for Maximum Flood Discharge(MFD) estimation. Predicted flood magnitudes and D-Index values have been shown in Table 1.0 and obtained values of Standard Error, Root Mean Square Error and Standard bias for judging the adequacy of fitted probability distribution have been shown in Table 2.0.

Table 1.0 Predicted flood magnitudes and value of D-Index

Name of methods/Recurrence interval in years	**Flood, Cumec**								**D-Index**
	2	**5**	**10**	**15**	**20**	**30**	**50**	**100**	
Normal Distribution	1736.2	2410.2	2762.8	2938.7	3053.9	3205.3	3381.4	3599.7	1.995
Log-Normal Distribution	1590.8	2248	2693.9	2948.4	3128	3380.8	3700.5	4139.2	1.568
Log-Normal –III Distribution	1585	2293.9	2774.9	3049.4	3243.1	3515.7	3860.6	4333.8	1.092

Gumbel Distribution	1608.5	2368.9	2872.3	3156.4	3355.2	3633.5	3980.3	4448.7	0.65
Pearson Type-III Distribution	1512	2231.3	2766	3079	3301.8	3617.1	4016.9	4564.6	0.843
Log-Pearson Type-III Distribution	1587	2260.6	2720.3	2983.5	3169.5	3431.8	3764.1	4221.2	1.391
Powel Distribution	1604.7	2312.4	2781.1	3045.4	3230.6	3489.1	3812.4	4248.4	1.168
Ven. T. Chow Distribution	1612.6	2348.9	2836.3	3111.4	3304	3573	3909.2	4362.8	0.861
Foaster Type-III Distribution	1502.2	2216.3	2757.6	3077.2	3305.4	3629.5	4042	4609.4	0.814
Stochastic Distribution	1269.7	2235.2	2965.6	3392.8	3696	4123.2	4661.5	5391.9	-0.68
Modified Stochastic Distribution	1403.7	2095.6	2619.1	2925.3	3142.5	3448.8	3834.5	4358	1.57
Wakeby Distribution	1543.2	2237.1	2765.3	3077.5	3300.4	3616.5	4018.3	4569.6	0.821

The perusal of Table 2.0 indicates that values of Standard error of Gumbel Distribution, Pearson Type–III, Wakeby Distribution, Foaster Type- III Distribution and Ven. T. Chow Distribution are less as compared to other methods. Similarly values of RMSE and BIAS are in range of 0.06 to 0.08 and 0.65 to 0.086 which is less as compared to other methods. Variation of errors obtained from various distributions have also been depicted in Fig. 1.0

Table 2.0 Computed Standard Error(SR), Root Mean Square Error(RMSE) and Standard bias(BIAS

Name of Methods	Standard Error cumec	RMSE	BIAS
NORMAL Distribution	282.55	0.24	0.58
LOG-NORMAL Distribution	211.24	0.12	0.53
LOG-NORMAL –III Distribution	184.19	0.08	0.31
GUMBEL Distribution	171.08	0.08	0.07
PEARSON Distribution	156.18	0.06	0.25
LOG-PEARSON Distribution	199.66	0.1	0.45
POWEL Distribution	194.83	0.1	0.32
VEN. T. CHOW Distribution	180.63	0.08	0.17
FOASTER TYPE-III Distribution	151.09	0.06	0.25
STOCASHTIC Distribution	126.56	0.11	-0.32
MODIFIED STOCASHTIC Distribution	185.66	0.1	0.66
WAKEBY Distribution	155.6	0.06	0.23

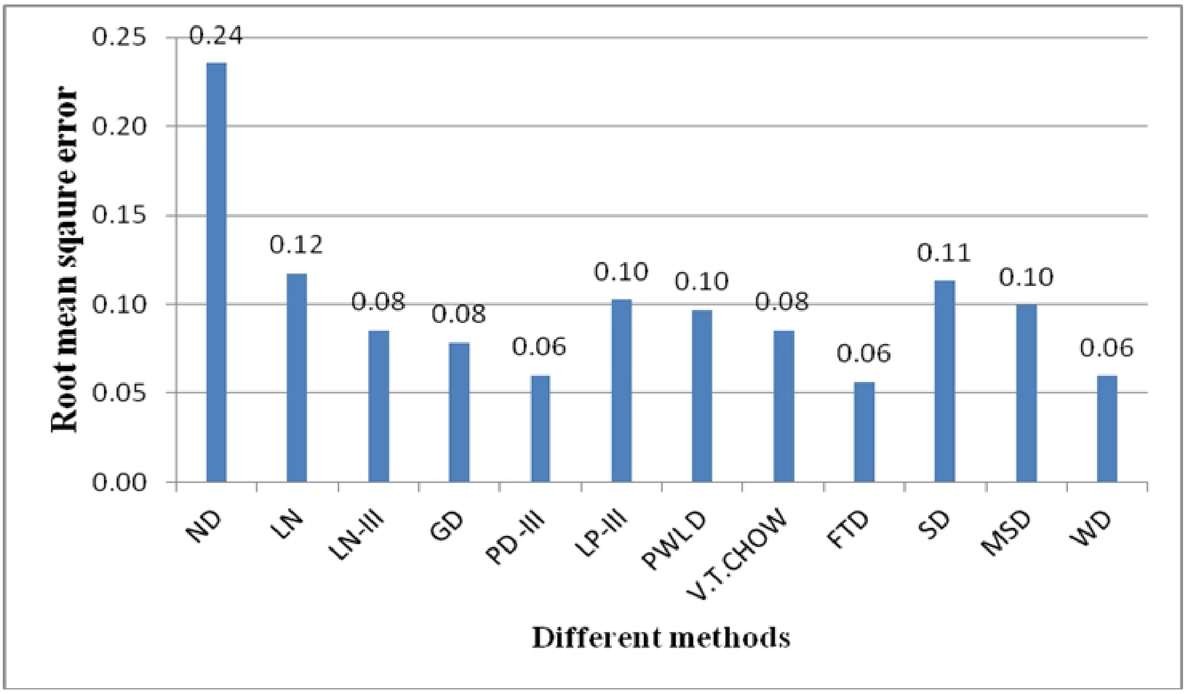

Fig 1.0 Evaluated RMSE values for all eleven methods

D-Index, computed for different methods have been summarised in Table 3.0. The perusal of Table 3 indicates that the Gumbel Distribution could be regarded as the best suitable method and could be used for flood frequency analysis for Alaknanda River at Karnaprayag in Uttarakhand. Moreover, Pearson Type III Distribution, Wakeby Distribution, Foaster Type-III Distribution was observed to be much considerable or better suited methods amongst other methods and could be adopted for flood frequency analysis. Fitted curves of distributions (accepted) with observed floods and different return periods have been depicted in Fig. 2.0.

Table 3.0 D-Index Goodness Fit for Peak Flood Data

Name of Methods	D-Index	Decision
NORMAL Distribution	1.996	Rejected
LOG-NORMAL Distribution	1.569	Rejected
LOG-NORMAL Distribution	1.093	Rejected
GUMBEL Distribution	0.651	**Accepted (Best)**
PEARSON Distribution	0.844	**Accepted**
LOG-PEARSON Distribution	1.391	Rejected
POWEL Distribution	1.169	Rejected
VEN. T. CHOW Distribution	0.861	**Accepted**
FOASTER TYPE-III Distribution	0.814	**Accepted**
STOCASHTIC Distribution	-0.681	Rejected
MODIFIED STOCASHTIC Distribution	1.57	Rejected
WAKEBY Distribution	0.821	**Accepted**

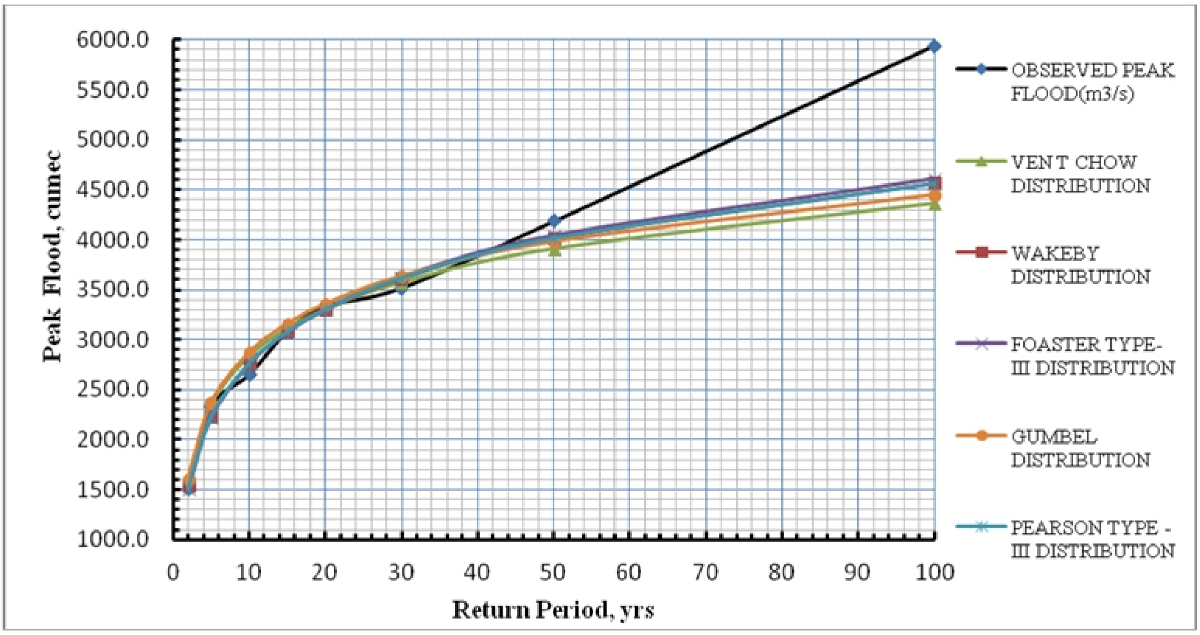

Fig. 2.0 Variation of Distributions with Observed Floods and Different Return Periods

5.0 CONCLUSIONS

The paper describes briefly the study carried out for estimation of Peak Flood or Maximum Flood Discharge (MFD) with the help of integrated software for Flood Frequency Method, based on eleven probability distributions, for Alaknanda River. The following conclusions have been drawn from the study:

(i) For the return period of 2, 5, 10, 15, 20, 30, 50 and 100 years, it was found that the estimated Maximum Flood Discharge (MFD) by Gumbel Distribution, Pearson Type – III Distribution, Foaster Type -III Distribution are higher as compared to other methods.

(ii) The study presents the selection of suitable distribution evaluated by GOF using D-Index test. On the basis of analysis of the data and comparison of Standard Error, Root Mean Square Error and BIAS of eleven methods, it can be concluded that the Gumbel Distribution is the best suited method for Flood Frequency Analysis for the flood data of Alaknanda River. It can also be concluded that Wakeby Distribution, Pearson Type–III Distribution, Foaster Type-III Distribution, and Ven T. Chow Distribution can also be adopted for flood frequency analysis.

(iii) The study suggested that the MFD values for different return periods computed by Gumbel Distribution could be considered as the design parameter for planning and design of water resource structures on Alaknanda River.

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Tsunami Run-Up On Sloping Continental Shelfs

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A B S T R A C T

The frequency of occurrences of tsunami has increased in the recent past and its disastrous impact on society is very high. So it is necessary to understand and estimate the tsunami characteristics with better accuracy. Numerical modelling is the most adopted method for tsunami simulation and understanding, due to the scale of the problem. It can also be noted that tsunami is governed by shallow water equations (SWE). Although, the initiation of tsunami is expected to take place in the deeper waters, it may have significant amplification while traversing over the slopes. In the present study, an attempt is made to understand the effect of continental slope on the propagation and run-up of tsunami. To carry out this investigation and to get a preliminary understanding, a one dimensional numerical model study is carried out using shallow water equations. The study is carried out by considering rectangular solitary wave. In this study, various continental slopes available along the Indian coast were considered. The amplification or attenuation of the tsunami characteristics over these cross-sections is studied. It was observed that the tsunami run-up is altered with varying continental slope.

Keywords: Tsunami; shallow water equation; Crank-Nicolson method; continental slope; run-up

1. INTRODUCTION

Tsunamis generate in the deep ocean and propagate towards the shore (Li and Raichlen 2001). Tsunami is a shallow water wave, which undergoes deformation due to reducing bathymetry. The speed of tsunami reduces, resulting in increase of wave height in the process of their propagation towards the shore (Kowalik et al. 2006; Sriram et al. 2006). After reaching the shore, tsunamis can break and travel over land for large distances causing severe property damage and loss of life. It is necessary to understand the effect of near shore bathymetry on tsunami characteristics and prediction of run-up (Behera et al. 2011), which will help in adopting appropriate measures to reduce the property damage and loss of life. In the past, various studies have been carried out to investigate the tsunami run-up on different beach slopes that suggested empirical formulae for the computation of tsunami run-up. Gjevik and Pedersen (1983) developed a numerical model based on a Lagrangian description for studying run-up of long water waves on beach slopes varying from 0° to 45°.

Synolakis (1987) studied the run-up of solitary wave on plane beaches by conducting various laboratory experiments over different beach slopes varying from 2° to 45°. The author observed that run-up variation is different for breaking and non-breaking waves. Grilli et al. (1997) had reported that waves will not break on the beach with slope steeper than 1:4.7. They have also concluded that shoaling rate decreases for steeper slopes greater than 1:15 and for moderately steep slopes, waves may break very close to the shore line. Madsen and Fuhrman (2008) observed that the impact of run-up on flat beaches (with slopes of the order of 1:100) is much higher compared to that on steep beaches (with slopes of the order of 1:15). The reflection coefficient decreases when run-up height and wave steepness increase (Gedik et al. 2011).

There were numerous studies carried out on behavior of tsunami along the coast. However, most of the studies were based on the analysis of solitary wave propagation over different beach profiles and the subsequent run-up. The literatures suggest that there is a need to study the effect of continental slope also on tsunami run-up, which might play a significant role in tsunami characteristics in the nearshore and over land region. Thus, a preliminary 1D numerical study is carried out to investigate the effect of continental slope on tsunami run-up.

2. NUMERICAL MODEL

Considering the large horizontal spanning of tsunami propagation in comparison to its vertical scale, it can be considered in 2D. The preliminary investigation to understand the effect of continental slope can be carried out using a 1D numerical model. Tsunami being a shallow water wave can be simulated using shallow water equations.

2.1 Governing equations and boundary conditions

In the present study, a numerical model is developed using 1D shallow water equations (SWE). The equations are solved by using Crank-Nicolson finite difference method. The continuity and momentum equations used are given by

$$\frac{\partial \eta}{\partial t}+\frac{\partial q}{\partial x}=0 \tag{1}$$

$$\frac{\partial q}{\partial t}+\frac{\partial (uq)}{\partial x}=-\frac{H}{\rho}\frac{\partial p_a}{\partial x}-gH\frac{\partial \eta}{\partial x}+\frac{\tau_b}{\rho H} \tag{2}$$

$$\tau_b = \rho k_b H^{-2} |q| q \tag{3}$$

where,   is sea surface elevation, H is total water depth (still water depth from mean sea level (d) + sea surface elevation ()), u is average velocity of the water particle, q is the flow discharge in the x direction (uH), g is acceleration due to gravity, ρ is density of sea water, τ_b is bottom stress and k_b is dimensionless friction coefficient. The value of friction coefficient (k_b) vary from 1.0×10^{-3} to 3.0×10^{-3} (Dotsenko 1998) and $k_b = 2.0 \times 10^{-3}$ is considered in the present study. Here, P_a is the atmospheric pressure, i.e. equal to 0. Thus, the modified momentum equation can be written as,

$$\frac{\partial q}{\partial t} + \frac{\partial (uq)}{\partial x} = -gH\frac{\partial \eta}{\partial x} + \frac{\tau_b}{\rho H} \tag{4}$$

In the present study, the shoreline boundary is assumed as an abrupt end of the coast with finite water depth and no flow condition is applied. The open boundary is defined by using radiation condition given by (Flather 1976). Velocity of tsunami at the open boundary is given by

$$u = -\sqrt{\frac{g}{H}}(\eta) \tag{5}$$

2.2 Validation of the Numerical model

The developed model is validated with Kowalik et al. (2006) who have studied the tide-tsunami interaction for Gulf of Alaska region. A numerical model domain and initial tsunami profile, same as of Kowalik et al. (2006) is considered for the study (fig.1). The simulation was carried out and tsunami elevation profiles were compared with the results of Kowalik et al. (2006) and are found to be in excellent agreement (fig.2).

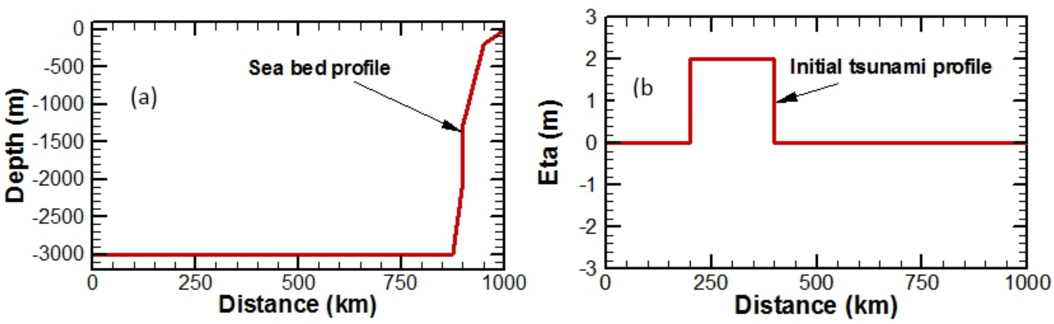

Figure 1 (a) Bathymetry profile of the model domain (b) Initial tsunami profile

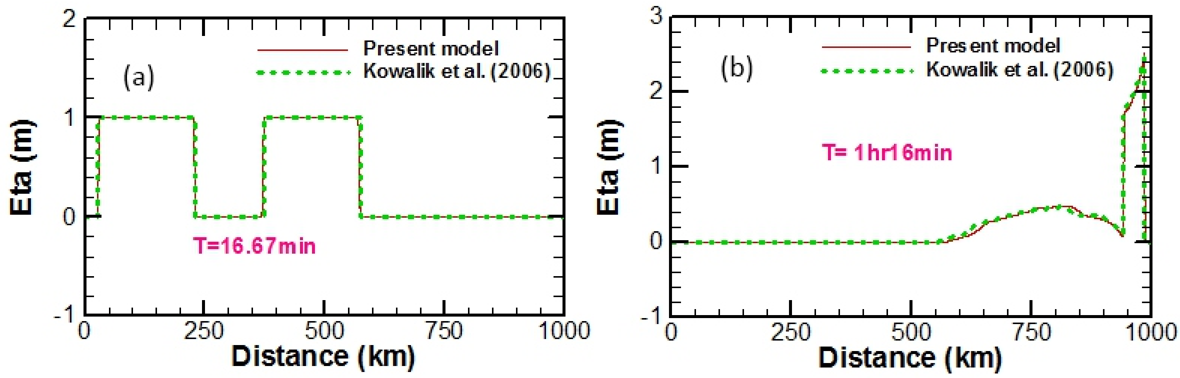

Figure 2 Comparison of tsunami profiles from present model with Kowalik et al. (2006) at different time instants

3. NUMERICAL MODEL DOMAIN

The study is carried out by considering a rectangular solitary wave profile to represent the initial tsunami perturbation. Different continental slopes available along the Indian coast are considered for the study. The slopes varying from 1:75 to 1:0.1 are considered in this study for the overall understanding. The transformation in tsunami height is studied while tsunami is propagating over different continental slopes and reaching continental shelfs of different depths. The simulations were carried out for four different continental shelf depths (50 m, 75 m, 100 m and 200m).

The present study is carried out by considering a domain of 1200 km long channel having constant water depth of 3 km up to 875 km. The domain and bathymetry profile used for the present study is shown in fig.3. An initial tsunami profile of height 2 m ($2H_0$) and length 200 km spanning between 200 km and 400 km is considered in the study (fig.3). Tsunami elevations are recorded at three locations P1, P2 and P3 as shown in fig.3. The first location is placed in deep ocean (3 km depth) at P1 (800 km from the open boundary) to record the incident tsunami profile. The second is placed at P2 (1100 km from the open boundary) which is on the continental shelf to observe the transformations in the tsunami profile due to continental slope. The third location is placed at shore (P3) to observe the tsunami run-up characteristics at the shore.

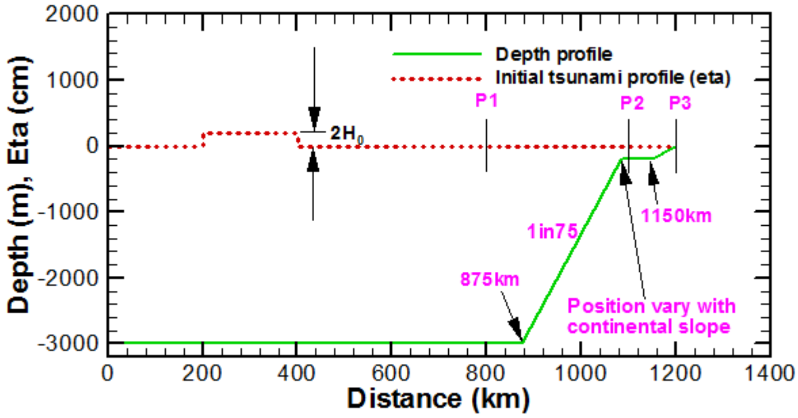

Figure 3. Typical initial tsunami and bathymetry profile considered in the study

4. SIMULATION AND RESULTS

The simulation was initiated by releasing the initial tsunami perturbation, that splits in to two waves and each wave propagates on opposite directions. The wave that propagates towards the open (radiation) boundary moves out of the domain without any reflection. The tsunami that propagates towards the shore undergoes various transformations in its wave height and wave length along the channel. The transformation of the tsunami characteristics occurs mainly due to continental slope and nearshore slope. The simulations were carried out for different slopes and continental shelf depth combinations. The tsunami height (H_0) is measured at probe P1 and is considered as incident wave height to ride over the continental slope. The tsunami height (H_s) is measured at probe location P2 after shoaling over the continental slope.

The tsunami height on the continental shelf is analysed and presented in terms of normalized tsunami height (H_s/H_0) as shown in fig.4. The results show that the tsunami height on continental shelf is lower for steeper slope in comparison to flatter slope. It is also observed that for a lower continental shelf depth, the amplification in tsunami height is higher. This is mainly driven by the depth. The celerity reduces in lower water depths, then corresponding tsunami length is reduced but tsunami height is amplified. Thus, continental shelf with higher water depth will encounter comparatively lower tsunami height. Thus, the offshore structures located on continental shelf with high depth are prone to higher tsunami height.

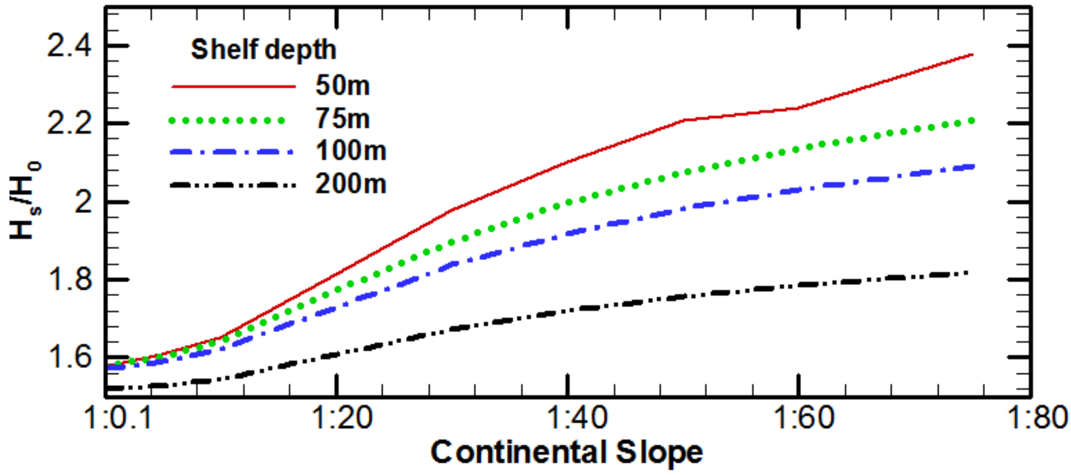

Figure 4. Normalized tsunami heights on continental shelf for different continental shelf depths

Although, the tsunami height in the continental shelf has been obtained, the resulting tsunami run-up (R_s) at the shore is more important to the coastal community. The R_s values are recorded at probe location P3 and analysed for 50 m, 75 m, 100 m and 200 m depth continental shelf and the results are shown in fig.5. The tsunami run-up at the shore is analysed and presented in terms of normalized tsunami run-up (R_s/H_0). It was observed that for deeper continental shelf, the momentum transmitted to the shelf is higher and the energy dissipation on corresponding nearshore slope is higher. Similarly, for shallow continental shelf less energy is transmitted and less energy is dissipated on the nearshore slope. The above process results in higher tsunami run-up for deeper continental shelf and steeper continental slope, whereas lower tsunami run-up for deeper continental shelf and flatter continental slope.

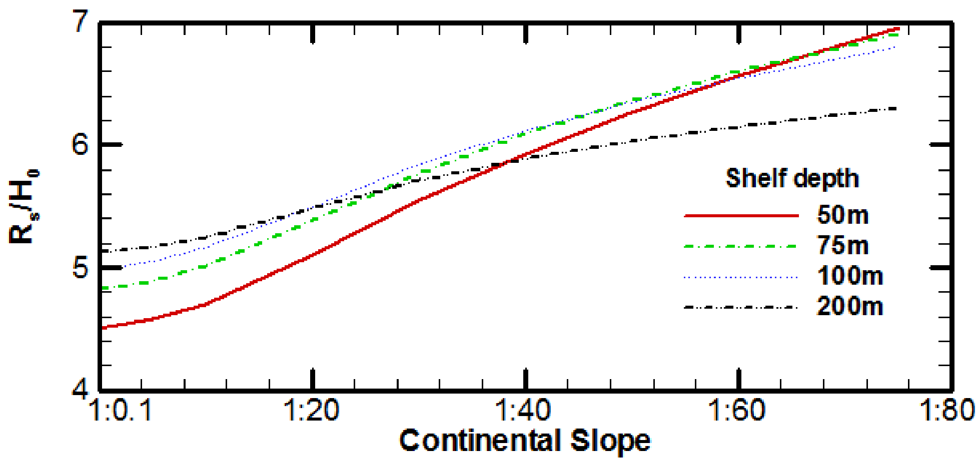

Figure 5. Normalized maximum run-up for various continental slope and shelf depths

5. CONCLUSIONS

In the present study, a 1D numerical model was developed using shallow water equation to study the effect of continental slopes on tsunami characteristics. The model was validated with Kowalik et al. (2006) with good agreement. This model was applied to study the effect of typical continental slopes present along the Indian coast on tsunami run-up. A systematic study was carried out for better understanding of the effect of slopes on tsunami with continental slopes varying from 1:0.1 to 1:75. The study suggests that tsunami height on the continental shelf increases for flatter continental slopes and shallow continental shelf depths. This increase in tsunami height due to decrease in continental shelf depth is not significant for steeper continental slopes. However, for flatter continental slopes a decrease in continental shelf depth leads to a significant increase in tsunami height. The percentage increase in tsunami height for decreasing continental shelf depth from 200 m to 50 m is found to be 31%.

The tsunami run-up at the shore is analysed and presented in terms of normalized tsunami run-up (R_s/H_0). It was observed that for deeper continental shelf, the momentum transmitted to the shelf is higher and the energy dissipation on corresponding nearshore slope is higher. Similarly, for shallow continental shelf less energy is transmitted and less energy is dissipated on the nearshore slope. The above process results in higher tsunami run-up for deeper continental shelf and steeper continental slope, whereas lower tsunami run-up for deeper continental shelf and flatter continental slope. However, the present study can be extended for 2D investigation to better understand the physical processes involved in tsunami propagation and transformation.

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A 2-D Hydrodynamic Model for Urban Flood Plain of Surat City, India

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A B S T R A C T

The Surat city, situated on the bank of the Tapi River, has encountered severe recent floods in years 1994, 1998 and 2006. The flood of year 2006 with a peak discharge of about 25780 m3/s from UkaiDam, inundated about 78% urban area of Surat city. The frequent flooding in Surat City needed suitable hydrodynamic model which may be useful in forecasting of the flood in future.Previously, one dimensional (1D) hydrodynamic model (Timbadiya et. al. 2014a) and coupled 1D-2D hydrodynamic model (Timbadiya et. al. 2014b) were developed for the said river. The two dimensional (2D) hydrodynamic models can predict more accurately flow depths and inundation areas in the flood plains as they represent better flow field in the flood plain. As, such models are, invariably, time intensive, and can be made more efficient due to advent of computational facilities in the recent past. In the present study, a two dimensional hydrodynamic model, at 25 m resolution, has been developed using MIKE 21 while simulating the flood of year 2006 and taking flood hydrograph from Ukai Dam and tidal levels at sea as upstream and downstream boundary conditions respectively. The model has been calibrated for river and floodplain roughness using data on flood and land use land cover of the flood plain. The simulated peak water levels in floodplain are compared with corresponding observed values in different residential zones of Surat city. The RMSE (Root mean square error) of highly inundated area West, North and Central zones are 0.67 m, 0.0.62 m and 0.44 m respectively. The developed model would be useful in floodplain delineation, flood hazard mapping and flood forecasting of Surat city in future.

1. INTRODUCTION

Flood is the most expensive and devastating natural hazard, and continues to be a problem in many parts of the world. The India is the second most flood affected country after Bangladesh and having one fifth of the global death counts due to floods (Kadam and Sen, 2012).The flood forecasting is essential to provide warnings to the people to evacuate areas threatened by the floods, and help water management personnel to operate flood control structures, along and across the channel. The, flood forecasting forms a very important and relatively inexpensive non-structural flood control measure for mitigating the flood.

The 1D Saint Venant equations are employed to compute the water levels and flow rates at different sections of the streams by solving the same through finite difference, finite volume and finite element methods. Several hydrodynamic models such as MIKE-11, ISIS, SOBEK (1D), HEC-RAS are extensively used for 1D hydrodynamic flow simulation. Tibadiya et al. (2012) developed 1D model for water level prediction in the lower Tapi River. The 1D model is not accurately predict water level when it goes beyond the either bank levels. The 1D model are not effective when flow crosses the either banks which commonly occurs during the flood event. The efficacy of 1D model give rises to the need 2D hydrodynamic model which accurately predict the flood water level in flood plain due to its ability to present flood flow physically.

To resolve the said limitations, in the present study, an attempt has been made to develop 2D hydrodynamic model for lower Tapi River to accurately predict flood water levels in coastal urban flood plain of Surat City. The study area,development of model, data collection, hydraulics of flow and its calibration are discussed in the paper along with the results and discussions at the end.

2. STUDY AREA AND DATA COLLECTION

The Tapi River originates from Multai in Betul district of Madhya Pradesh, and flows towards west for a distance of about 724 km to join Arabian Sea at Hazira about 20 km downstream of the Surat city (see Fig. 1). The Tapi River basin accounted for 65145 km2. It is distributed in Madhya Pradesh (9804 km2), Maharashtra (51504 km2) and Gujarat (3837km2) states. The lower Tapi River from Gidhade to Arabian Sea is fairly flat (Jain et al. 2007). The tidal water enters in Tapi River from sea up to Singanpur weir.

The Surat district in South Gujarat is having 4418 km2 area, out of which, fast growing Surat city itself is spread in about 326 km2 area. The Surat city is situated on the banks of the lower Tapi River, about 20 km upstream of river mouth at Hazira/Dumas. The Surat had experienced several heavy floods, during the last century. The highest flood of magnitude 42500m3/s was experienced in August 1968 after which flood embankments were constructed along both the banks from Nehru Bridge to Amroli Bridge in the Surat City for safe passage of flood water. Subsequent to construction of Ukai dam in 1972, the regulated floods were released. Notable floods of the recent past of magnitude 14866m3/s, 19057 m3/s and 25780m3/s were experienced in years 1994, 1998 and 2006 respectively.

The bathymetry of river, details of weir and flood plain were collected from SMC while the flow and tidal data were collected from Surat Irrigation Circle (SIC) and State Water Data Centre (SWDC).

The methodology adopted in development of 2D hydrodynamic model using aforesaid data is described in subsequent topics.

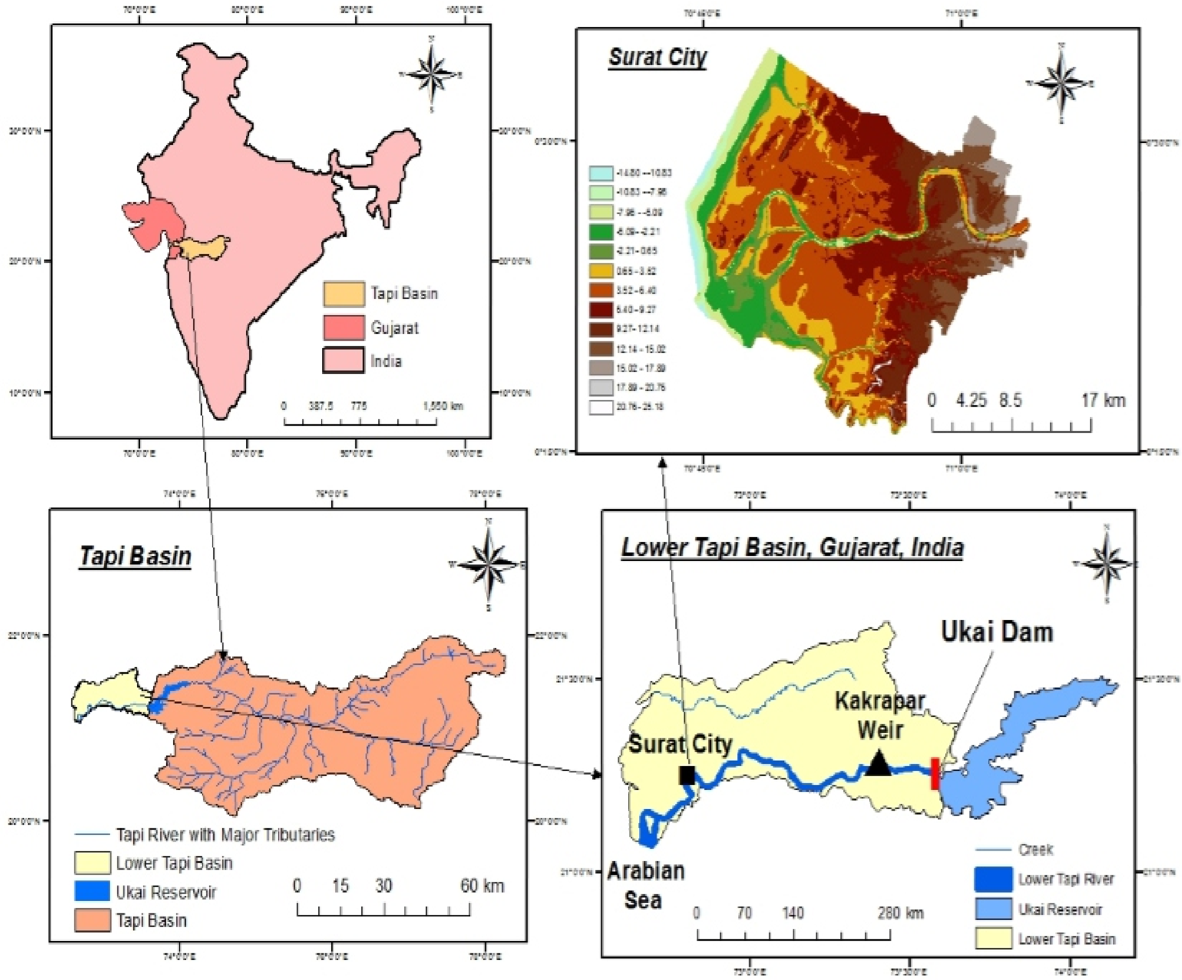

Fig. 1 Index map of Tapi basin

3. METHODOLOGY

The flow chart includingmethodology adopted in present study in development of 2D hydrodynamic model is shown in Fig. 2. Firstly, the 1D hydrodynamic model is developed and calibrated for the lower Tapi River, and the hydraulic results at the chainage 85 km were calculated, and with the help of these results the downstream boundary conditions were given to 2D hydrodynamic model. The developed model was validated on the basis of highest water levels in the flood plain at different point of the Surat City.

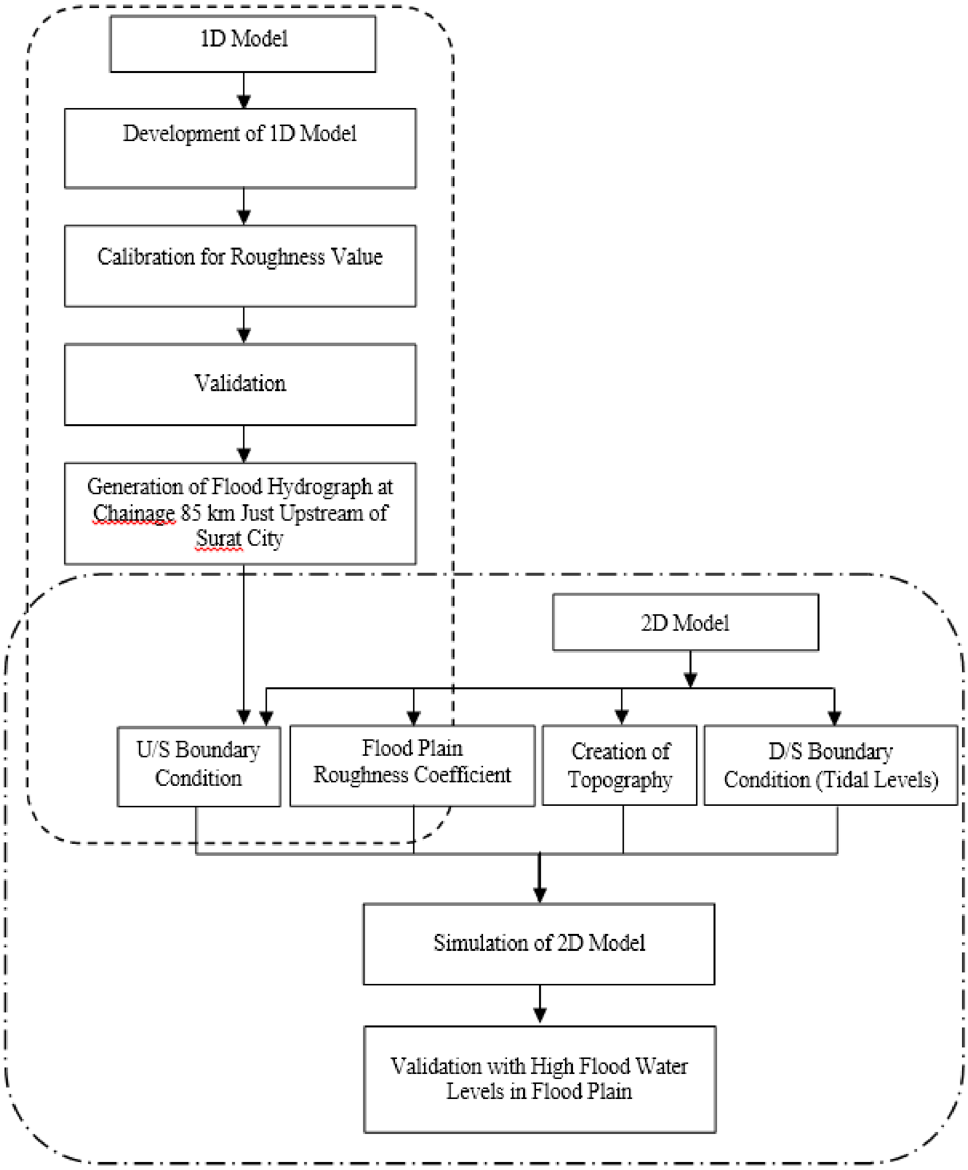

Fig. 2 Methodology adopted in development 2D hydrodynamic

4. DEVELOPMENT OF 1D MODEL

The MIKE 11 HD, a one dimensional hydrodynamic model has been used for the simulation of unsteady non uniform flow conditions of floods of year 2006, which uses the six-point Abbott-Ionescu finite difference scheme for the solution of one dimensional hyperbolic Saint-Venant equations. The 1D hydrodynamic model MIKE 11 have been calibrated for flood of year 1998, and, subsequently, validated for the floods of year 2003 and 2006. The calibrated MIKE 11 model is used to create the upstream boundary (in terms of flood hydrograph) for 2D model for Surat city and its outskirts (chainage 85km) for the flood of year 2006.

5. DEVELOPMENT OF 2D MODEL

The MIKE 21 flow model is applicable to simulate of the hydraulics of 2D free-surface flows based on numerical solution of the depth-averaged equations that describe the conservation of mass and momentum in two horizontal dimensions (Timbadiya et al., 2014b; DHI 2009). The 2D hydrodynamic model has been, subsequently, developed from a digital terrain model (DTM) constructed from topographic information available at 1 m contour intervals.

The 25×25 m resolution topography of the Surat city and its outskirt area, has been developed from the physically surveyed contour data of 1 m interval, and considered as the basic file input for MIKE 21 in the .dfs2 file format. For unsteady simulation, the discharge hydrograph at upstream and water levels as downstream boundary conditions are invariably being considered. The discharge from the Ukai reservoir was routed with the help of 1D model and discharge hydrograph at chainage 85 km was calculated, which has been used as an upstream hydrograph of the 2D model.

In development of the 1D hydrodynamic model, the channel section was calibrated and best performance of model was obtained for Manning's 'n' as 0.03. Hence, the Manning's n for the river portion has been taken as 0.03. The floodplain roughness value 'n' were taken as 0.14 from the previous study reported by Timbadiya et al. (2014b).

6. FLOOD SIMULATIONS

The 2D hydrodynamic model was used to simulate the flood of year 2006 from August 06, 2006, (time: 06:00:00) to August 10, 2006, (time: 24:00:00) for total of 114 hours, with simulation time step as 2 sec and the results were saved at 30 minutes interval. The performance of the calibrated model has been assessed while comparing the simulated water levels with corresponding observed water levels.

7. VALIDATION

The model is validated by comparing the simulated high flood depths with the corresponding observed data for the flood of 2006 in different locations in all seven zones of Surat City. The observed high flood depths at different locations for the flood event of year 2006 in all seven zones of Surat City, were available from the Surat Municipal Corporation (SMC).

The results obtained through simulations are compared with the observed data in the flood plain. The observed peak water depths were compared with the corresponding simulated peak water depths in all zones of Surat City. Total 598 no. of data are used in the said comparison and presented in Fig. 3. The performance of the model in all seven zones are satisfactory as 98.33% of the data points are lying within ±95% prediction lines.

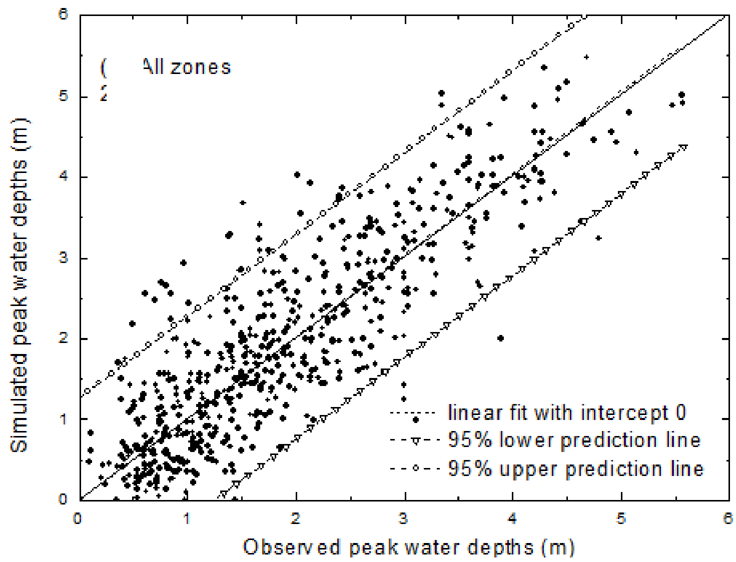

Fig. 3. Scatter plot of highest water depth in all zones of Surat City

The observed peak water depths has been compared quantitatively with the simulated peak water depths by calculating the RMSE. The values of RMSE for all the zones of Surat City is presented in Table 1. The RMSE (m) is defined as

$$RMSE = \sqrt{\frac{\sum_i^N (y_o - y_s)^2}{N}} \quad (1)$$

Here, y_0 = observed peak water depth (m),
y_s = simulated peak water depth (m),
N = number of observations.

Table 1 Performance of 2D model for highest flood levels in Surat for flood of year 2006

Zone	RMSE (m)
Central Zone	0.44
East Zone	0.5
North Zone	0.62
South-East Zone	0.86
South-West Zone	0.68
South Zone	2
West Zone	0.67
Weighted value	0.82

The performance of the model in all the zones are satisfactory except the south zone. The reason for such deviation in accuracy of the model in south zone is the bathymetry. The Kankada creek travels from south zone to south west zones and in actual flooding situations it takes the flood water from the south zone and meets the Arabian Sea. As the basic contour files of the geometry is not available in the south west zone, because of which model creates extra ponding in the south zone. So as to increase the accuracy of the model in south zone, the basic survey of the creek is required.

8. CONCLUSION

The successful development of 2D hydrodynamic model of the lower Tapi River at 25 m resolution has been presented for urban coastal flood plain of Surat City. The developed 2D model is more accurate in presenting the floodplain than 1D and 1D-2D coupled hydrodynamic models. The calibrated values of Manning's 'n' value is 0.03 for the lower Tapi River. The performance of 2D model has been assessed by plotting the scatter plot of observed and predicted levels, and total 98.30% data points are lying within ±95% prediction line. The highest affected zones of Surat City due to flood of year 2006, Central zone, West Zone and North zone has RMSE value for peak flood depth as 0.44 m, 0.62 m and 0.67 m respectively. The inferior results in the south zone is due to the basic topographic data error. The developed model can be used to predict the flood levels at various locations in the flood plain for the future flood events, and can be useful for the development of the flood hazard maps of the Surat City which forms future scope of present study.

ACKNOWLEDGEMENT

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Study of Wind and Water Forces on The Displacement of The Offshore Structures Using CFD

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A B S T R A C T

In recent future the need of offshore wind farms and oil platform has increased and it is assumed that in 2030 world's 10% energy would be generated through offshore wind farms. As offshore structures have reached to greater depths, engineers and designers are confronted with new problems such as water depth, weather conditions and ocean currents. So design of offshore structures has been one of the toughest engineering problem.As the displacement is the key parameter for the design of offshore structures and displacement of offshore structures is mainly governed by water and wind forces. In the present paper the variation in the displacement of monotower offshore structure with different water depth and wind speed has been evaluated using CFD simulations through COMSOL.In the present study Fluid Structure Interaction (FSI) module have been used to model the interaction between the exterior boundary of the tower and ocean water. Arbitrary Lagrangian-Eulerian (ALE) technique has been adopted for discretizing into finite elements as it have additive advantage of computational mesh inside the domains moving arbitrarily to optimize the shapes of elements, while the mesh on the boundaries and interfaces of the domains can move along with materials to precisely track the boundaries and interfaces of a multi-material system.

Keywords:Offshore structures, CFD Analysis, Displacement based design, Fluid Structure Interaction, Arbitrary Langrangian- Eulerian approach

1.0 INTRODUCTION

The oceans present a unique set of environmental conditions that dominate the methods, equipment, support, and procedures to be employed in offshore structural designing. Many literature articles (Holand et al. 2004, Perez and Lamas 2013) have addressed the extreme environmental events and adverse exposures as they affect design, still there is need to evaluate the impact of the combination of wind and water waves including Fluid Surface Interaction (FSI) at the outer perimeter of the offshore structure(Chen and Yu2009).

Fluid structure Interaction (FSI) is one of the emerging fields in numerical simulation and calculation of multiphysics problems (Paidoussis 2004;Kock and Olson 1991; Zienkiewicz and Bettess 1978; Kanok-Nukulchai and Tam 1999). FSI becomes crucial in capturing large deformations in offshore structures and these deformations are caused by the vortex induced loads (Sharif and Sudharsan 2015). FSI imposes various challenges due to its complexity in nature of interaction between a fluid and solid geometries and requirements of computational resources (Sandberg 1995). In this paper, FSI analysis is conducted on a cylindrical member which may be considered as a leg of an offshore platform. The FSI analysis is performed using the commercial package. It illustrates how fluid flow can deform structures and how to solve for the flow in a continuously deforming geometry using the arbitrary Lagrangian-Eulerian (ALE) technique (Ortega et al. 2013).

2. MODELLING

The model geometry consists of a horizontal flow channel in the middle of which is an obstacle, a narrow vertical structure. The fluid flows from left to right, except where the obstacle forces it into a narrow path in the upper part of the channel, and it imposes a force on the structure's walls resulting from the viscous drag and fluid pressure. The structure, being made of a deformable material, bends under the applied load. Consequently, the fluid flow also follows a new path, so solving the flow in the original geometry would generate incorrect results.

Total of 100 m height of domain is considered in which structure height is 45 m. Structure is modelled in 2D with fixed base and width of 5 m. The fluid is a water-like substance with a density ρ = 1000 kg/m3 and dynamic viscosity η = 0.001 Pa-s. To demonstrate the desired techniques, assume the structure consists of a flexible material like that of steel with a density ρ = 7850 kg/m3 and Young's modulus E = 200 kPa.

Boundary Conditions

(i) At the outflow (right-hand boundary), the condition is $p = 0$.

(ii) At left boundary inlet wind velocity linearly varying with height is givenv = U* y/H where U is considered as 33.33 m/sec as max velocity.

(iii) On the solid (Non-deformable) walls, no-slip conditions are imposed, $u = 0$, $v = 0$ (fixed constraint).

In parametric study three different water heights are considered to determine the impact of water height on the displacement and stress generated in the structure. Three models with water height of 0, 20, 45 meters are considered in the present study.

3. ARBITRARY LAGRANGIAN-EULERIAN (ALE)

Arbitrary Lagrangian-Eulerian (ALE) is a finite element formulation in which the computational system is not a prior fixed in space (e.g. Eulerian based finite element formulations) or attached to material (e.g. Lagrangian based finite element formulations). ALE-based finite element simulations can alleviate many of the drawbacks that the traditional Lagrangian-based and Eulerian-based finite element simulations have.

When using the ALE technique in engineering simulations, the computational mesh inside the domains can move arbitrarily to optimize the shapes of elements, while the mesh on the boundaries and interfaces of the domains can move along with materials to precisely track the boundaries and interfaces of a multi-material system.

ALE-based finite element formulations can reduce to either Lagrangian based finite element formulations by equating mesh motion to material motion or Eulerian-based finite element formulations by fixing mesh in space. Therefore, one finite element code can be used to perform comprehensive engineering simulations, including heat transfer, fluid flow, fluid-structure interactions and metal-manufacturing.

The ALE method handles the dynamics of the deforming geometry and the moving boundaries with a moving grid. COMSOL Multiphysics computes new mesh coordinates on the channel area based on the movement of the structure's boundaries and mesh smoothing. The Navier-Stokes equations that solve the flow are formulated for these moving coordinates.

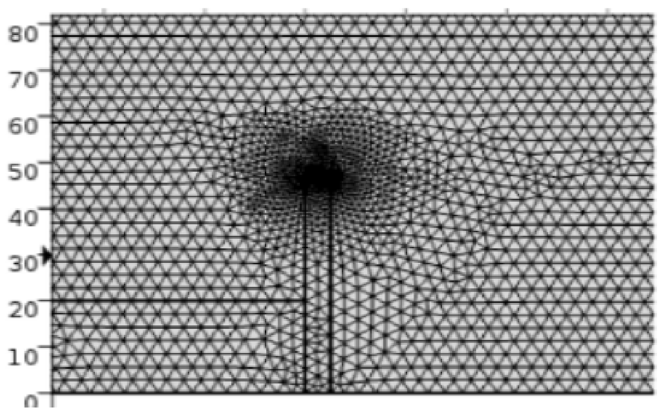

Figure 1 Structured mesh for 2D domain(triangular Meshing)

4. GOVERNING EQUATIONS

4.1 Fluid Flow Equations

The fluid flow in the channel is described by the incompressible Navier-Stokes equations for the velocity field, u = (u, v), and the pressure, p, in the spatial (deformed) moving coordinate system:

$$\rho \frac{\partial u}{\partial t} - \nabla[-pI + \eta\{\nabla u + (\nabla u^T)\}] + \rho((u - u_m).\nabla)u = F \quad (1)$$

$$-\nabla . u = 0 \quad (2)$$

Where**I** is the unit diagonal matrix, and **F** is the volume force affecting the fluid. The model neglects gravitation and other volume forces affecting the fluid, so **F** = **0**.

4.2 Structural Mechanics Equations

The structural deformations are solved for using an elastic formulation and a nonlinear geometry formulation to allow large deformations.
The Structure is fixed to the bottom of the fluid channel. All other object boundaries experience a load from the fluid, given by where n is the normal vector to the boundary. This load represents a sum of pressure and viscous forces. We could see from equations that fluid flow equations give us force exerted by the fluid and the same forces are used to calculate the strain in the solid structure.
The Structure is fixed to the bottom of the fluid channel. All other object boundaries experience a load from the fluid, given by

$$F_X = - n . (- pI + \eta (\nabla u + (\nabla u)^X)) \quad (3)$$

Where**n** is the normal vector to the boundary. This load represents a sum of pressure and viscous forces

5. COUPLED SOLVER

It uses MUMPS solver, two segregated solutions, segregated solution 1, and segregated solution 2. Segregated 1 solution comprises of spatial coordinates and displacement Segregated 2 Solution comprises of Velocity field and pressure.

6. RESULTS

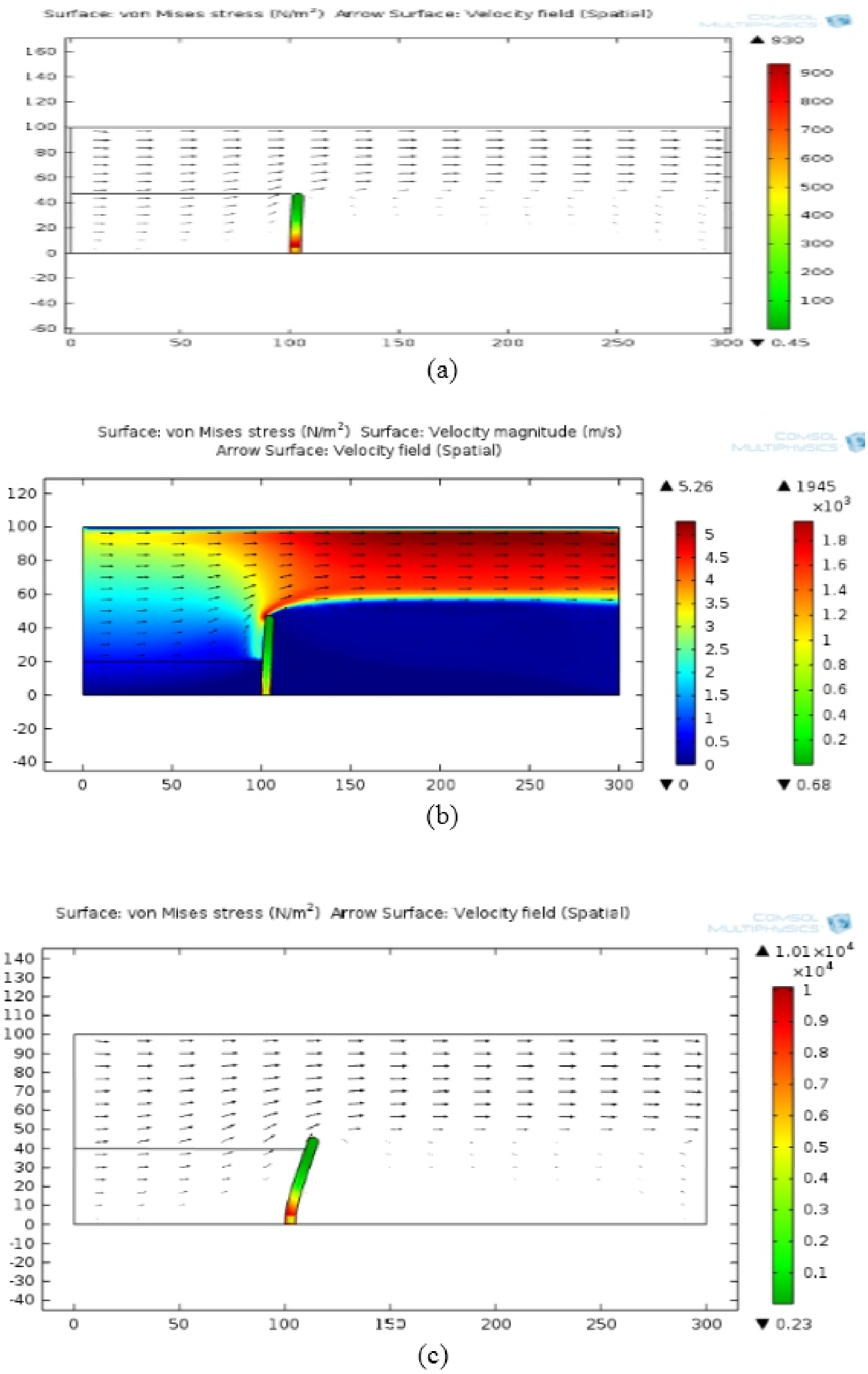

Figure 2 Maximum surface stress in structure for water height of 0, 20 and 45 meters

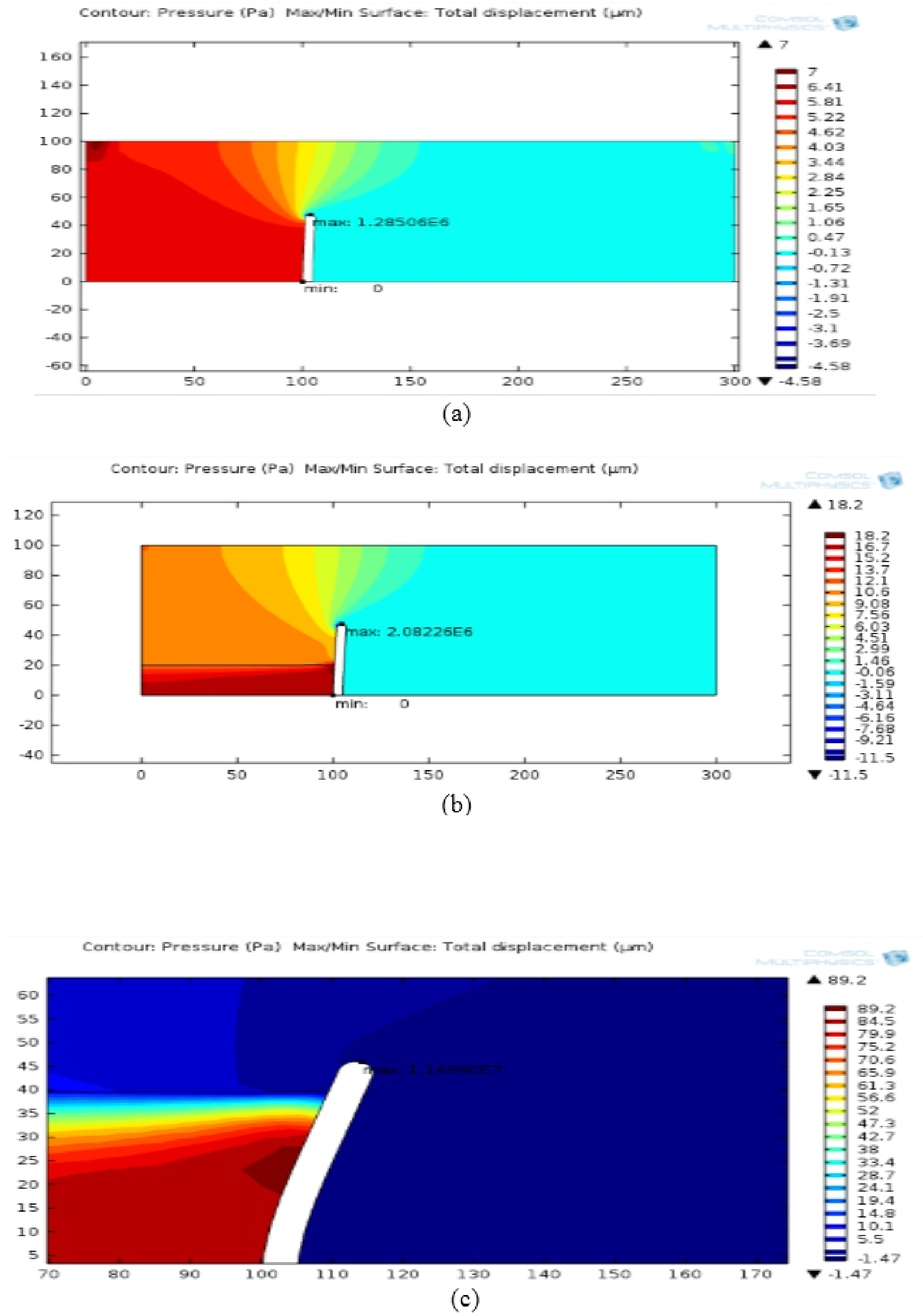

Figure 3 Displacement of structure for water height of 0, 20 and 45 meters

Table 1 Parametric Study

Height of water	Displacement of Structure	Max Surface stress(KN/m2)
0	1.28 m	0.93
20 m	2.08 m	1945
45 m	11.69 m	101000

Table 1 represents the variation of displacement and maximum surface stress in the offshore structure with change in height of water. Figure 2 represents the location of the maximum stress in the structure and Figure 3 shows the displacement and deflected shape of structure under action of load.

7. CONCLUSION

This study shows how the stresses the offshore structure varies with change of water depth and also impact of water height on the displacement of the structure. It is also seen that there is huge increment in the displacement and stress as water height rises from 0 meter to 20 meter and then to 45 meter, it's mainly because of increase in the water forces and water forces become much more dominant as height of water rises.

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Flood Mitigation Plan For Failure Of Reservoir Embankment Using MIKE FLOOD

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ABSTRACT

Protection of the public lives and properties from the consequences of dam failures has become important as population have concentrated in areas vulnerable to dam break disasters. Due to unfavourable site condition, artificial reservoir are developed for storage and supply of water for some intended use. These reservoirs are generally huge in size, impounds significant amount of water and imposes flood risk in its downstream reaches which were otherwise safer. The treatment of reservoir embankment failure is similar to the dam failure. The organizations responsible for the safety of the structure plan for preventive measures in case of failures so that damages to the lives and properties of the population living in the downstream area may be minimized. The prior assessment of extent, magnitude and time of flooding due to failure of dam/ embankment are the important input in planning for flood preparedness measures. Further, these inputs help in issuing flood warning to the downstream population at the time of failure. However, it is quite difficult to conduct analysis and determine the warning time and extent of inundation at the time of disaster. Therefore, pre-determination of these parameters is done by simulating a hypothetical dam break/ embankment failure situation. The paper discusses the case study of failure of embankment of a man made reservoir near a major industrial set up in Karnataka. The study envisages the identification of various scenarios of flooding, estimation of breach parameters, modelling of dam break flood and its routing in the downstream reach to compute the maximum flood inundation and its time of occurrences.

Key words: dam break flood, MIKE FLOOD, emergency action plan

INTRODUCTION

Failure of dam is a serious concern because of loss of life and property. Even, it may damage more than what envisaged at design stage due to facilitation of settlement and developmental activities downstream of dam as the resources and the sense of security from flooding has increased. The dam break flood is different from the storm generated flood in the sense that it hardly gives any response time for emergency action. The sudden and uncontrolled release of stored water, generally coinciding with the catastrophic climatological events raises the flood magnitude to a very high level and thus the water spreads to a wider area in the downstream causing the losses much more as compared to normal rainfall generated floods. Further, with the increase in extremist actions over the objects of national and economic interest, the emergency action plans are prerequisite. Such plans consist of the maps showing the flood inundations under various scenarios of dam failures. This helps in estimating the potential damage. The amount and extent of flooding and occurrence of their time are also included. The threatened settlement, property, and other infrastructures, in addition to emergency rescue plan, communication links etc. are the important constituents of the EAP. The hypothetical dam break analysis is carried out to predict potential flood damage and to prepare emergency action plan (EAP) in advance. Such analysis is mandatory not only for the existing and old dams but also for the proposed dams. (MoEF, 2015).

A complete dam break analysis involves a balanced consideration of hydrological, hydraulic, environmental, and geotechnical and structural parameters pertaining to dam and downstream flood plains. The description of time dependent flood wave propagation, downstream of a breached dam is extremely complex. It is a function of site specific parameters including reservoir characteristics and breaching characteristics of dam. Further, the movement of flood wave across the flood plain will be governed by another range of determinants, many are difficult to replicate in a mathematical model; e.g. dynamic variation in terrain and surfaces, influence of land use change etc. The dam break flood simulations are carried out by developing various scenarios leading to dam failure in addition to re-evaluating the probable maximum flood (PMF), the most general cause of dam failure in natural circumstances. The insufficient capacity of the spillways may be incapable to pass the updated estimate of PMF causing dam failure due to overtop. Another likely reasons for dam failure includes earthquake induced structural failure, equipment failure, criminal action, sabotage etc. (CWC, 2006, EAP).

With the advancement in computational fluid dynamics (CFD) and availability of new tools and techniques like; geographical information system (GIS) and satellite images, some of the difficulties have been successfully accounted for. Several experimental, analytical, and numerical models have been developed to carry out dam break analysis. Computer-aided numerical models such as DAMBRK (Fread, 1988), SMPDBK (Wetmore and Fread, 1991), CADAMBRK (Liong et al., 1991), NWS FLDWAV (Fread, 1993), HEC RAS (USACE, 2006), BOSS DAMBRK (Kho, 2009), and MIKE 11 (DHI, 2004) have been widely used successfully across the world due to their high computational speed and efficiency. Majority of dam break analysis studies have been carried out using 1D model. The NWS DAMBRK model has been used for dam break analysis of Barna dam, Madhya Pradesh, India (NIH, 1997), Ghodahoda project Odisha India (NIH, 2000), and the proposed dam on Yamuna river, India (Lodhi and Agrawal, 2012). BOSS DAMBRK model was used to study the dam break analysis of proposed dam on the Gerugu river Malaysia (Kho et al., 2009). HEC RAS model has been used for dam break analysis of Oros Dam Brazil (Gee, 2008), Danjiangkou and Yahekou dam failures in the Han river

China (Minglong and Jayawardena, 2008), and Foster Joseph Sayers Dam in Center Coutry PA, USA (Xiong, 2011). SMPDBK model has been used for dam break analysis of Foster Joseph Sayers Dam in Center Coutry PA, USA (Shahraki et al., 2012). MIKE 11 model has been used for dam break analysis of Bichom and Tenga dam (Husain and Rai, 2000), Buffalo Creek Dam, North Carolina, USA (Tingsanchali and Chinnarasri, 2001), Indra Sagar and Omkareshwar project, India (Pillai et al., 2012), Hirakud dam, India (Mohite et al., 2014). 1D models, though simple to use and provide information on bulk flow characteristics, fail to provide detailed information regarding the flow field. Hence, attempts have been made to model the 2D nature of floodplain flow. A 1D approach is used to describe breach growth and breach flow and a 2D approach is used to predict flood propagation in the inundated areas. In this paper, a case study of failure of the embankment of the raw water storage reservoir near a major industrial set up in Karnataka has been discussed. The treatment of reservoir embankment failure is similar to the dam failure. The prior assessment of extent, magnitude and time of flooding due to failure of dam/ embankment are the important input in planning for flood preparedness measures. Further, these inputs help in issuing flood warning to the downstream population at the time of failure. The study envisages the identification of various scenarios of flooding, estimation of breach parameters, modelling of dam break flood and its routing in the downstream reach to compute the maximum flood inundation and its time of occurrences. The failure of embankment for raw water reservoir has been simulated in MIKE-11 while the downstream inundation mapping has been carried out in MIKE FLOOD that dynamically links two independent software packages; MIKE 11 (1D) and MIKE 21 (2D). MIKE 11 solves the Saint-Venant equations by means of a finite difference scheme. Breaches can be modeled by means of a "dam break" structure. Breach growth can be described by time series for breach width, crest level and side slope. The "classic" version of MIKE 21 uses a rectangular grid and solves the shallow water equations by means of a finite difference scheme. For downstream reach, the floodplain is represented through bathymetry of 10 m grid size. The maximum flood level and its time of occurrence and flood warning time have been estimated at important locations. The extent of maximum inundation for various cases of flooding has also been computed.

STUDY AREA

A thermal power plant is proposed at proposed in Basavana Bagevadi taluk of Vijaypura (earlier Bijapur) district in Karnataka, a southern state of India on the right bank of Hire Hall Nala, a small tributary of Krishna river. The water requirement for the plant will be sourced from Almatti dam at Krishna river through a pipeline at a distance of about 18 km from the plant site. Accordingly, a raw water reservoir spreading over the area of 336 acre has been planned where the water will be stored for further use of the plant. The reservoir with ponding capacity of 10.5 MCM at full reservoir level of

575.5 m would be created by construction of embankments all along. Hire Halla Nala, the natural drainage in the area flowing along the north east embankment of the raw water reservoir as shown in . The elevation within the reservoir area varies from RL 575 m to RL 560 m. From surveyed river cross section, the average slope of the drain between reservoir site to the confluence of Almatti dam is computed as 2.78 m/km. The important settlement in the downstream reach adjoining the drain Hire Halla Nala includes Golsangi (RL 551.3 m), Budni (RL 549.6 m), Biraldinni (RL 544.9 m), Hunsihal (RL 528 m), Gudadinni (RL 537.5 m) and Vandal (RL 532 m). Bijapur districts can be classified into semi-arid region, with extreme summers. Average temperature variation of the area lies between 20° C to 42° C. Coldest month are in December and January.

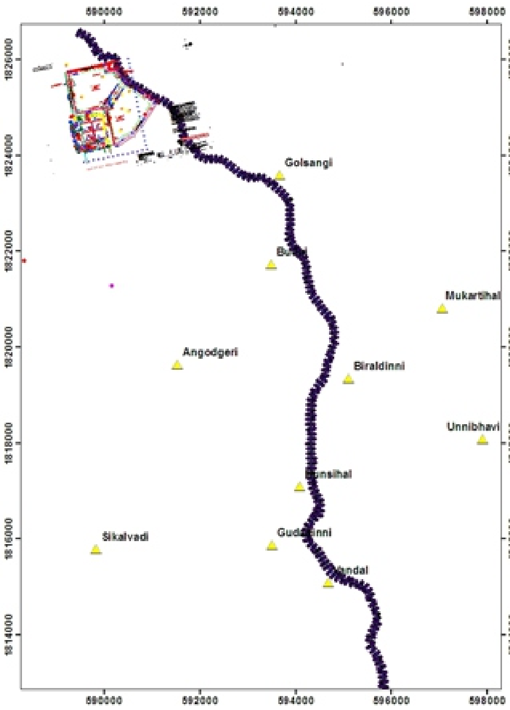


Figure 1 Raw water reservoir and Hire Halla Nala upto Almatti reservoir

(CGWB, 2008). Average rainfall of the Bijapur district is 578 mm. The maximum rainfall received in the month of September followed by the month of October. Highest rainfall reordered in a single day is 149.2 mm in the month of September and minimum rainfall was 3.4 mm in the month of February. Normal rainy days in Bijapur district, varies from 36.5 to 39.5 days. (CGWB, 2008). South west monsoon is the principle rainy season which contribute 80% of the total rainfall, while 12% of the rainfall received in post monsoon period. Summer season contribute 7 % of the total annual rainfall whereas winter season (January to February) contributes less than 1% of the total rainfall (KSPCB, 2012). The ground elevation at the reservoir site varies from RL 560 m to RL 570 m and the top of the dam is RL 577 m. Hence, the height of embankment is varying from 7.5 m to 17.5 Further, the capacity of the reservoir at full reservoir level (FRL) of RL 575 m is computed as 10.5 MCM. Hence based on criteria of gross storage and static height at FRL, the dam is classified as small dam for which the inflow design flood for safety of dam is specified as 100-year return period flood (IS: 11223 – 1985). CWPRS, 2013 has carried out the flood study and estimated 100-year return period flood which has been used in this study. The dam details like type of dam, construction materials and type, design details of dam and reservoir, bed level of river at dam site, height of top of dam etc. are obtained from the project authority. The material for embankment is compacted earth. The top level of embankment is RL 577.0 m while the FRL of the reservoir is RL 575 m. The elevation capacity curve for the reservoir has been computed from the contours data for reservoir area.

The contour map of the reservoir area is survey data using ArcGIS. The cross section data for Hire Halla Nala in a stretch of 17.3 km at an interval of 100 m is used in the study. The digital elevation model (DEM) of the site has been developed using the field survey data, SOI toposheets contours and spot levels. All these elevation data are imported in ARCGIS in UTM coordinate and of the area at 30 m grid size is generated and shown in Figure 2.

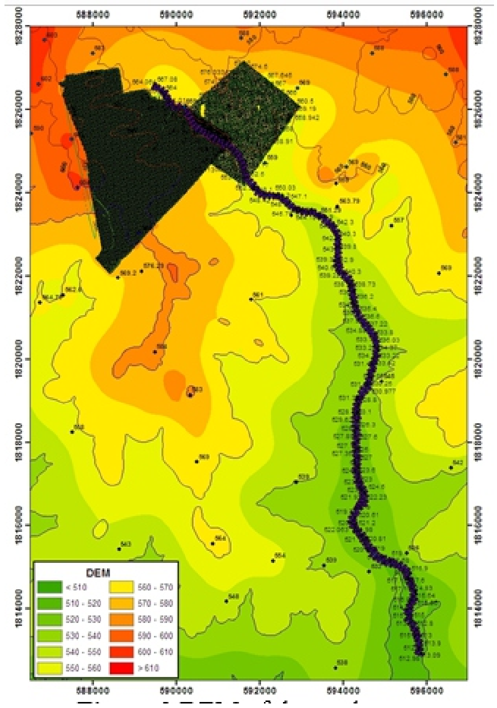


Figure 2 DEM of the study area

METHODOLOGY

The outflow from the embankment breach and the rainfall induced flow in Hire Halla Nala has been modelled in MIKE 11. The flood plain is represented through gridded DEM and used in MIKE 21 model which simulates the overland flow and spill from the river bank into floodplain and vice versa. The MIKE 21 has been dynamically linked to the MIKE 11 model, into a single package called MIKE FLOOD is widely used for flood inundation studies. (Sanders, 2007; Chatterjee et al., 2008; Patro et al., 2009 and Pramanik et al, 2010). The governing equations in MIKE 11 are 1-D (one-dimensional) and shallow water type, which are the modifications of basic Saint-Venant equations. These are transformed to a set of implicit finite difference equations, and solved using double sweep algorithm (Abbot and Ionescu, 1967). In the present setup the time step is kept very low as 10 seconds because the dx value is larger, i.e., 200 m. The simulation has been performed for 3 days period starting from January 1, 2015, 08:00 hours to January 4, 2015, 12:00 hours. These dates are hypothetical. The bathymetry is generated from 30 m grid DEM. The manning's roughness for the floodplain is specified as 0.05 considering the conservative values for cultivated matured field crops (IS 2912:2004). The computational time step (Δt) is set to lower value of 2 seconds for different simulations. The lateral link have been used to couple the MIKE 11 model with MIKE 21 along the Hire Halla Nala. The standard links have been used to link the breach location in the embankment of raw water reservoir with MIKE 21 grid.

DAM BREAK FLOOD ANALYSIS

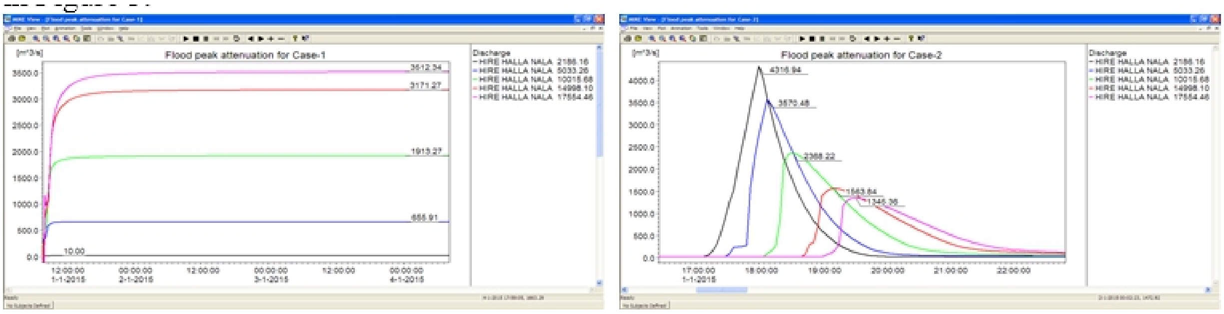
The Hire Halla Nala from raw water reservoir to the confluence of Almatti reservoir, a stretch of about 17.6 km, has been simulated in MIKE 11. A separate branch has been created for reservoir portion and the breach location on the embankment is defined at its mid point. The embankment fails when raw water reservoir is at FRL of 577 m and downstream of Hire Hala Nala is at constant water level of 519.6 m (i.e. FRL of Almatti dam). The breach starts at FRL i.e. RL 577 m and develop in trapezoidal shape and the bottom of the section come down to the lowest bed level of RL 559.5 in a duration of one hour. The breach parameters are defined in Table 1.

Table 1 Description of breach parameters

Breach Time (sec)	**Breach Width (m)**	**Breach Level (MSL)**	**Breach Slope**
0	0	577	1
1800	35	570	1
3600	46	560	1
6566400	46	559.5	1

The breach outflow is computed using NWS DAMBRK equation The flow simulation starts at 1/1/2015 8:00:00 PM while the breach starts to develop at 1/1/2015 5:00:00 PM to coincide with the peak of 100 year return period flood. The time of breach is considered as 1 hour i.e. the duration of start of breach to its complete development to final breach size. The river flow is simulated upto 1/4/2015 12:00:00 PM so that the entire water outflow from the reservoir and maximum downstream inundation is computed.

Various cases of flooding are considered in the study including pre-project and post project scenario. In the pre-project scenario (Case-1), only design flood (100-year return period flood) is considered in Hire Halla Nala. In post project case, two flooding scenario have been considered; firstly (Case-2), isolated flooding from embankment breach and secondly (Case-3), combination of flooding due to embankment failure and design flood in Hire Halla Nala. The breach failure of embankment in all cases of simulation is considered under FRL and the downstream boundary condition at Almatti dam is FRL of Almatti dam i.e. 519.6 m. The attenuation of flood hydrograph at various downstream sections are shown in Figure 3.



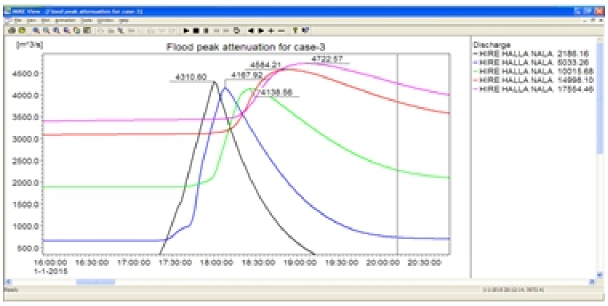


Figure 3 Hydrographs at various d/s reach for flooding scenario of Case-1, Case-2 and Case-3.

SENSITIVITY OF BREACH PARAMETERS

The selection of breach parameters are based on the experiences from historical dam failure events and the technical guidelines based on scientific studies provided by various agencies of international repute. The uncertainty in breach parameter estimates are due to several reason including different construction techniques and workmanship and other unforeseen conditions. The flood peak due to dam failure is highly impacted by breach parameters and is influenced by the combination of various breach parameters. Hence, the sensitivity of individual breach parameter are carried out Sensitivity analysis for three breach parameters namely; breach width, side-slope of breach and time of breach has been carried out. In the sensitivity analysis, breach outflow is computed by varying one parameter at a time while keeping others parameters as constant. The sensitivity of breach width has been carried out for breach width of 50 m, 70 m and 100 m. Similarly, the sensitivity of breach time has been evaluated by varying the time of breach to 0.5 hr, 1 hr and 2 hr. The sensitivity of various breach parameters are shown in Figure 4.

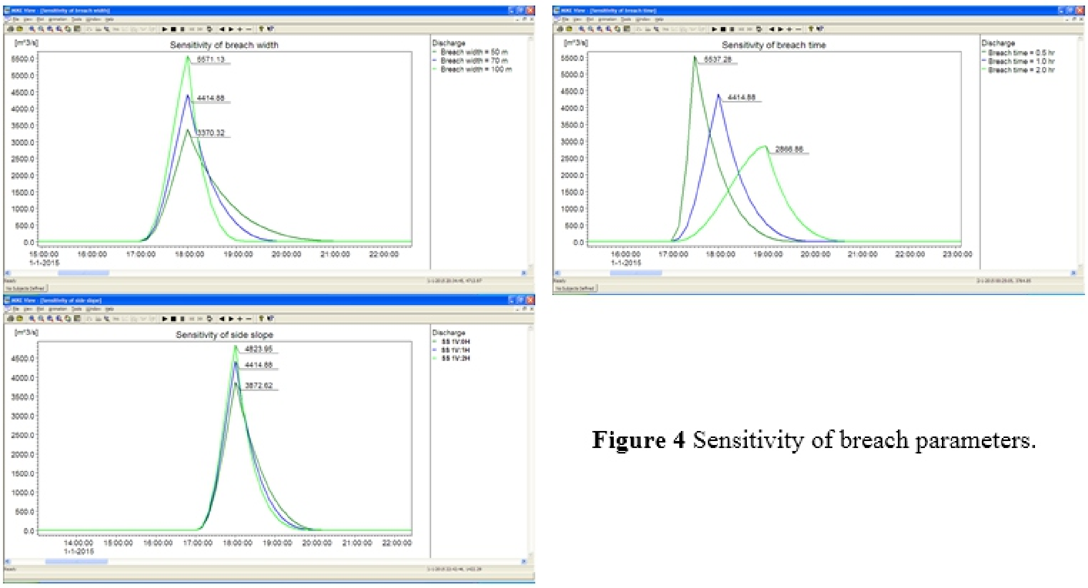


Figure 4 Sensitivity of breach parameters.

MAXIMUM FLOOD LEVEL

In MIKE FLOOD the computation of water depth is carried out in grided form, the maximum flood level at important locations have been extracted for various cases of flooding. The important locations in the downstream reach of the raw water reservoir are Golsangi, Budni, Biralnandini, Hunsihal, Gudadinni and Vandal at which the maximum flood elevation (msl) has been computed for various cases of flooding as shown in Table 2. The time of occurrence of maximum flood level is shown within bracket. For Case-1 in which the flooding due to design basis flood in the catchment has been simulated, the maximum flooding time is computed since impingement of design flood in the Hire Halla Nala. The maximum flooding for Case-2 (when flooding due to dam break is simulated) and Case-3 (combination of Case-1 and Case-2) is computed since inception of breach. Further, the table also shows the maximum inundation for various cases of flooding. The maximum flood elevation for combined flooding due to failure of raw water embankment and design basis catchment flooding is 548.54 m near Golsangi while it inundates an area of 977 ha.

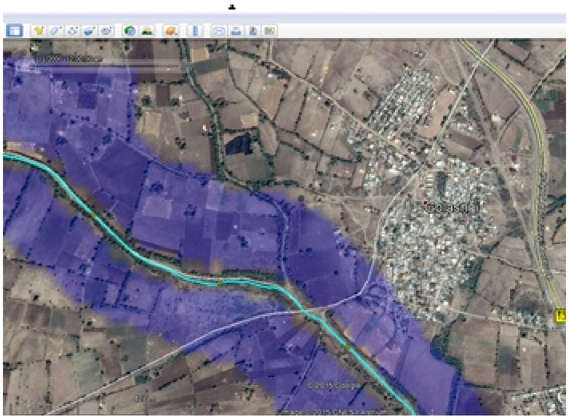
Table 2 Inundation statistics for various cases of flooding at important locations

Particulars	**Maximum inundation area**	**Maximum Flood Elevation (m) near important locations**					
Important locations	**(ha)**	Golsangi	Budni	Biralnandini	Hunsihal	Gudadinni	Vandal
Case-1	644	547.57 (1 hr 16 m)	541.34 (1 hr 31 m)	534.95 (1 hr 42 m)	527.35 (2 hr 10 m)	523.72 (2 hr 26 m)	521.5
Case-2	785	547.14 (1 hr 11 m)	541.34 (1 hr 18 m)	535.09 (1 hr 34 m)	529.31 (1 hr 52 m)	526.23 (2 hr 5 m)	524.25
Case-3	977	548.54 (1 hr 10 m)	542.7 (1 hr 17 m)	536.75 (1 hr 29 m)	530.29 (1 hr 42 m)	527.29 (1 hr 54 m)	525.32

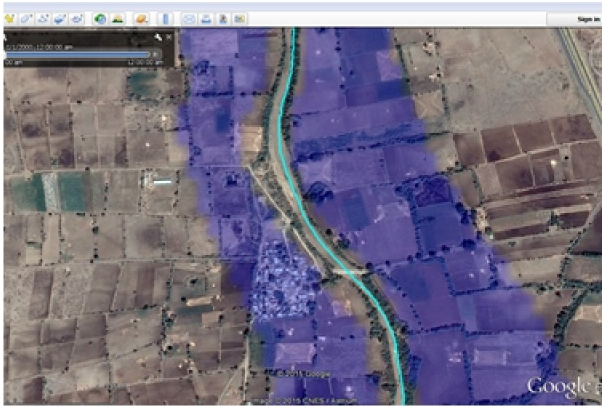
(Time in bracket shows the time of arrival of peak flood near important locations)

DISASTER MANAGEMENT PLAN

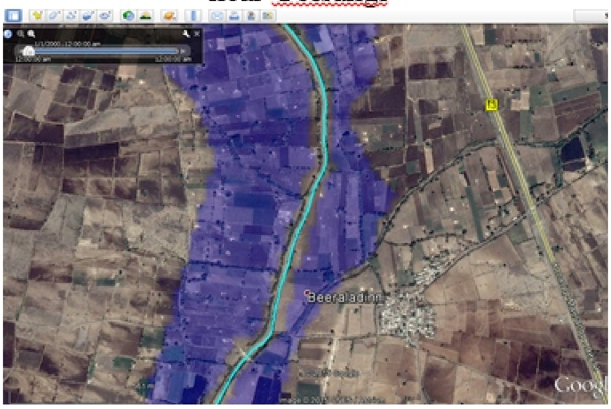
The maximum flood level, extent of inundation and time of flood are important input for preparing emergency action plan during disaster. Table 2 shows that the maximum flooding is due to Case-3. The maximum inundation extent for Case-3 is exported to Google Earth which where detailed description of area is available. The overlaid inundation map will identify the obstruction in movement during emergency. The inundation maps are zoomed around the important locations and the extent of inundation along with the road network can be visualised. Figure 5 shows the inundation near Golsangi, Budni, Biralnandini and Hunshihal. The emergency exits from these areas to nearest highway and railway line and other elevated areas can be clearly identified. The figure shows the elevated exit roads (free from inundation) are approachable to all the affected areas and can be reached within few minutes while the minimum warning time of 1 hour is available for all cases of flooding scenario (Table 2). Hence, a suitable flood warning dissemination mechanism like siren along Hire Hala Nala may work adequately. Further, these settlements may be protected from flooding by construction of flood wall along these locations. The maximum flood level at these locations can be obtained from Table 2 which will define the top of flood wall/ embankment with prescribed free board.



near Golsangi



near Budni



near Biralnandini

near Hunsihal

Figure 5: Maximum inundation for Case-3.

CONCLUSIONS

In this study, three cases of flooding scenario has been simulated; (i) design basis flooding, (ii) flooding due to dam breach and (iii) combination of (i) and (ii). The maximum flood inundation for Case-1, Case-2 and Case-3 are 644 ha, 785 ha and 977 ha respectively. Case-3 is the severe most flooding scenario causing maximum inundation. The flood inundation maps are exported and overlaid in Google Earth where detailed transport networks and other features can be visualised. These maps, in turn, can be used for preparation of emergency exits during flood disaster.

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