Dam break analysis of Hirakud dam using HEC-RAS model

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ABSTRACT

Dam is indispensable hydraulic structure built across river for the aim of creating reservoir to store water. According to USACE Hydrologic Engineering Centre (HEC) Research Article 13, the failure of the dam was brought on by an earthquake, an extremely strong storm, a landslide, piping, damage to the structure, a failure of the foundation, an equipment malfunction, and sabotage. Regardless of the reason, almost all failures commence with a breach formation. In this study, dam break flow analysis for Hirakud dam was performed by assuming a hypothetical dam failure case using HEC-RAS. Data used in this study was obtained from Central Water Commission (CWC) and Shuttle Radar Topography Mission (SRTM). Study was carried out by using both Froehlich (2008) and Xu and Zhang (2009) regression models. The main objectives of this study are to route the flood wave movement, peak flood and maximum elevation of water surface due to hypothetical failure of Hirakud Dam at a distance of 0-100km in downstream and to assess the time of travel of dam break flow. Using dam geometric and elevation data the dam breach parameters are estimated in HEC-RAS as follows 1) Breach bottom width (final)=88m; 2) Breach side slope= 0.32 and 3) Breach developed time=14.50 hr.

Keywords: Dam Break Analysis, HEC-RAS, Breach parameters

1 Introduction

1.1. Importance and Background

The dam is one of the most important hydraulic structures to store excess water in the reservoir. Its main function is to release water for irrigation, and to moderate flood flows through river and thus, to protect the downstream areas from loss of life and property. Other important purposes include water supply, hydropower generation and navigation etc. Usually, the downstream and adjoining areas of a dam are highly cultivated due to high fertility of the flood plains and/or availability of water through canal networks and consequently become thickly populated because of various developmental activities (Chane, B., & Behailu, S. 2006).

Over 35000 big dams are now in use worldwide, and many more are being built, giving by the International Committee on Large Dams (ICOLD). In India, about 3000 major, medium and minor dams are in existence. There are many instances of dam break in India and abroad. Overtopping failure (inadequate spillways, improper use of road embankments, etc.), foundation failure (fault movement, settlement, etc.), and piping as well as seepage of embankment dams are the three main causes of dam collapse (Singh, V. P. 1996).

1.2 Review of Literature

1.2.1 Analytical Models

Studies to recognise the basic mechanism of dam break (lows (D13F) are very old and date back to the initial effort by Ritter in 1892. Ritter came up with an analytical solution to the hydrodynamic issue of an instantaneous dam-break in a horizontal, frictionless channel of a rectangular cross section. In Ritter's solution, both the reservoir and the channel were assumed to be infinite, and the channel downstream was assumed to be dry. The flow depth (h) and velocity, at any place downstream of the dam are functions of distance, time and reservoir water level. The flow depth and the discharge attained at the dam-site are constant in time and represent critical flow condition there. The shape of the free surface is a parabola, and the tip speed are twice that of the disturbance propagated upstream. Later, **Dressler (1952) and Whitham (1955)** expanded the study of DBF to include the influence of bed resistance and produced analytical equations for the wave-speed fronts and height. **Pohle (1952)** considered two-dimensional flow, in x and z direction Using Lagrange representation, he concluded that in the initial regime, the vertical acceleration is the predominant parameter. When the vertical acceleration is decreasing, the effect of channel cross-sectional geometry, bed friction and bed slope become more important, and the wave profile will then converge to one-dimensional analytical response.

If a wet-bed situation materialises in the future, **Stoker (1957)** expanded the Ritter solution. In terms of the starting depths upstream and downstream of the dam, he deduced analytical formulas for the surface profile. **Hunt (1982, 1987)** used analytical formulae that took into account reservoirs with limited lengths. Hunt's approach, however, was predicated on the idea of a kinematic wave.

1.2.2 Experimental Model

Escande et al. (1961) presented detailed results obtained from experimental studies using a 1.6 kin reservoir and J2 km long downstream reach with fixed bed. They presented. The front wave profile, due to the sudden failure of a dam for different conditions as well as the variation of front wave velocity with bed roughness, initial reservoir head and initial channel flow. One of the complete sets of laboratory data on dam break flows was collected at the U.S.A. Army Engineers, the Water-ways Experiment Station (WES,1960). **Rajor (1973)** presented results for dam break flows, obtained through experimental modelling of real valleys and of prismatic and non-prismatic channel.

Ritter predicted that the depth at the dam site would reach a constant value instantly. **Dressler (1954)** demonstrated this experimentally. The constant Ritter's value is reached in around nine non-dimensional time units.

All the above experimental studies are for straight channel reaches; however, the natural channels are seldom straight and meander in the channel alignment produce. Lateral gradients in the flow surface. **Miller and Chaudhry (1989)** presented the experimental results for dam break flows in meandering channels. **Memos (1983)** presented experimental results that in a partial dam failure (breach width less than valley width) three-dimensional effects are dominant during the first instance of the break. **Martin (1983)** presented the results of total dam break in a rectangular and in a channel havng divergent side wall (dry bed). Similar observations were also presented for convergent and divergent channels by **Townson and Al-Salihi (1989) and Bellos et al. (1992)**.

1.2.3 Numerical Models

The most practical instrument for a quick and thorough investigation of dam break flow is a numerical model. Generally, in case of a numerical model, the dam break flow is simulated by three consequential steps, i.e. (i) routing of the inflow hydrograph from the reservoir inlet to the dam site, (ii) dam break mechanism and (iii) routing of the dam break flow in the downstream side of channel.

All numerical models may be categorized, depending on the equations used to model the phenomenon, numerical scheme used to solve the equations, and implementation of different boundary physical conditions.

In most of the numerical models, available in literature, one dimensional St.Venant equations are used as the governing equations **(fennema & Chaudhry) 1987**, **Molls and Molls (1998), Fread 1988)**. One dimensional hydrostatic pressure distribution along a vertical plane is assumed according to the St. Venant equations. **Basco (1989)** pointed out limitations to the St.Venant equations in dam break flows analysis. In some studies, one-dimensional Boussinensq equations are applied to replicate the dam break flow **(Carmo et al. 1993)**. Two-dimensional St.Venent equations in x,y plane are used for dam break flow analysis by some researchers **(Fennema and Chaudhary 1990)**. Two dimensional Navier — Stokes equations in x and z plane to study dam break flows are also presented in literature.

1.3 Objectives and Scope of This Study

In this study the dam break flows resulting due to a hypothetical collapse of Hirakud Dam is analysed. The detail objectives of this study are:

i) To route flood wave movement due to hypothetical failure of Hirakud Dam, up to 100 km distance towards the downstream side of the Hirakud dam.
ii) To assess the time of travel of the dam break flow.
iii) To estimate peak flood and maximum water surface elevation at d/s of Hirakud dam at distance of 0-100km.

2 Methodology

2.1 HEC-RAS Model for Dam Break Analysis

The River Analysis System software from Hydrologic Engineering Centre is called HEC-RAS. With the help of this programme, we may calculate one-dimensional unsteady flow, steady flow, water quality assessments and sediment transport. HEC-RAS is made to be used interactively while doing many tasks. A graphical user interface (GUI), distinct analysis components, data storage and maintenance capabilities, visualisations, and reporting tools are all features of this system. Dam break study using the HEC-RAS model comprises forecasting the characteristics of the breach, the flood hydrograph, the time the flood wave will arrive, and the peak flow. The parameters are controllable by the user with HEC RAS. Both overtopping and pipeline failure breaches for concrete and earthen dams may be modelled using HEC-RAS. Unsteady flow equations are used to direct the ensuing flood wave downstream (Brunner, G. W. 1994).

2.2. Estimation of Dam Breach Characteristics using Regression Method

2.2.1. Xu and Zhang (2009) Regression Equation

The user of HEC-RAS has the option of using either the "User Entered Data" or the "Simplified Physical" breaching methodology. The User Entered Data approach necessitates the user entering every breach-related details. It is possible to enter correlations between velocity, breach widening and downcutting when using the Simplified Physical Breaching technique. (Xu and Zhang, 2009).

Regression equations for the parameters of the breach have been created by several researchers. Data for earthen dams, earthen dams with impermeable rockfill and core dams were used to develop these equations. Concrete dams and earthen dams with concrete cores are not directly affected by these calculations. Numerous equations that are used to estimate breach parameters are described in the report by Wahl (1998). The regression equations created using the breach dimensions and failure time in the USBR report (Wahl, 1998) are:

- Froehlich (1995a)
- Froehlich (2008)
- MacDonald and Langridge-Monopolis (1984)
- Von Thun and Gillette (1990)
- Xu and Zhang (2009)

2.2.2. Froehlich (2008) Regression Equation

Dr. Froehlich revised his breach equations in 2008 considering the inclusion of fresh information. Dr. Froehlich used 74 earthen, zoned earthen, earthen with a core wall (i.e., clay), and rockfill data sets to create a set of equations to calculate the average breach width, side slopes, and failure time. The below ranges comprised the data that Froehlich utilised for his regression analysis:

- Volume of water at breach time: 0.0139 – 660.0 m^3 x 106 (11.3 - 535,000 acre-feet) (with 86% < 25.0 m^3 x 106, and 82% < 15.0 m^3 x 10^6)
- Dam Height: 3.05 – 92.96 m (10 – 305 ft) (with 93% < 30 m, and 81% < 15 m)

Froehlich's regression equations for average breach width and failure time are:

$$B_{ave} = 0.1803 K_0 V_w^{0.32} h_b^{0.19}$$

$$t_f = 0.00254 V_w^{0.53} h_b^{-0.9}$$

Where B_{ave} = average breach width (m); h_b= final breach height (m); V_w= reservoir volume during failure time (m^3); g= acceleration due to gravity; t_f= breach formation time (sec); and K_o= constant (1.3 for overtopping failures, 1.0 for piping)

Froehlich's 2008 paper states that the average side slopes should be:

1.0 H:1V, in case of overtopping failures

0.7 H:1V or else, (i.e., piping/seepage) (Froehlich 2008).

2.2.3. Xu and Zhang (2009) Regression Equation

182 earth dams and rock-fill dams from China and the United States were included in the database created by Drs. Xu and Zang, with over half of the dams being taller than 15 meters. Their final equations, however, are based on a much smaller percentage of these dams due to a lack of data. Their report provides information on 75 dams, including concrete fronted, core wall, zoned-filled, and homogenous earth fill dams. Only 28 dams were employed in their equation for the timing of collapse, even though they included 45 dam failures in their final equation for the average breach width. This regression equation is applicable for the range of dames mentioned below:

Dam Height: 3.2 – 92.96 m

Volume of water at breach time: 0.105 – 660.0 x 10^6 m^3

(a) Average breach width (B_{avg})

The Average breach width (B_{avg}) is given by the Xu and Zhang (2009) is

$$\frac{B_{avg}}{h_b} = 0.787 \left(\frac{h_d}{h_r}\right)^{0.133} \left(\frac{v_w{}^{1/3}}{h_w}\right)^{0.652} e^{B_3}$$

Where, B_{avg} = breach width (average) (m); h_b= the final breach's height (m); h_d= height of dam (m); V_w= volume of reservoir at time of failure (m^3); h_w= height of water above the breach bottom elevation during time of breach (m); hr=15m is used as a standard height to distinguish between large and small dams; B_3= b_3+b_4+b_5 , a coefficient that depends on the characteristics of the dam; b_3= 0.226, 0.026, and 0.041 for homogeneous/zoned-fill dams, concrete faced dams, and core walls, respectively; b_4= 0.149 and -0.389 for overtopping and seepage/piping, respectively; b_5= 0.391, 0.14, and 0.291 for low, medium, and high dam erodibility, respectively.

(b) ***Height of the Breach (h_b)*** high the equation for the breach top width is given by Xu and Zhang (2009) is

$$\frac{B_T}{h_b} = 1.062\left(\frac{h_d}{h_r}\right)^{0.092}\left(\frac{{V_w}^{1/3}}{h_w}\right)^{0.508} e^{B_2}$$

where: B_T= breach top width (m); B_2= b_3+b_4+b_5 a coefficient that depends on the characteristics of the dam; b_3=0.089, 0.088, and 0.061 for dams with homogeneous/zoned-fill dams, concrete faced dams, and core walls, respectively; b_4= 0.239 and 0.299 for seepage/piping and overtopping, respectively; b_5= 0.289, 0.062, and 0.411 for low, medium, and high dam erodibility, respectively.

(c) ***Breach side slopes (Z)***

The breach bed slope is expressed as (Xu and Zhang, 2009)

$$\text{Side slope } (Z) = \frac{B_T - B_{avg}}{h_b}$$

(d) Breach Formation time (t_b)

$$\frac{t_b}{t_u} = 0.304\left(\frac{h_d}{h_r}\right)^{0.707}\left(\frac{{V_w}^{1/3}}{h_w}\right)^{1.228} e^{B_5}$$

Where t_f= breach formation time (hr); t_u= 1 hr (unit duration); V_w= reservoir volume at time of failure (m^3); h_w= water level above the breach's bottom elevation at the time of the breach. (m); hr=15m is used as a standard height to distinguish between large and small dams; h_d= height of the dam (m); B_5=b_3+b_4+b_5 a coefficient that depends on the characteristics of the dam; b_3= 0.189, 0.674, and 0.327 for dams with homogeneous/zoned-fill dams, concrete faced dams, and core walls, respectively; b_4= 0.611and 0.579 for seepage/piping and overtopping, respectively; b_5=0.579, 0.564, and 1.205 for low, medium, and high dam erodibility, respectively (Xu and Zhang 2009).

3 Study Area and Data Collection

3.1 The Mahanadi River Basin System

One of the main rivers in the nation, the Mahanadi River flows east and empties into the Bay of Bengal. In terms of its water potential and ability to cause floods, it is in second place to the Godavari among the Peninsular rivers. The river's source is located 6 kilometres away in Madhya Pradesh's Raipur district, in the Pharsiya hamlet close to Nagri town. The river's total length, from its head to where it empties into the sea, is 851 kilometres, of which 357 kilometres are in Madhya Pradesh and 494 kilometres are in Orissa. The Seonath, the Koonk, the Hasdeo, the Mand, the /IBS the Ong, and the Tel river are the river's main tributaries.A basin neap of the Mahanadi river system showing the details is given in Fig. 1. Total catchment area of Mahanadi River basin is 1,41,720 square km and at Hirakud it is 83400 square km. It covers four states namely Madhya Pradesh, Orissa, Maharashtra and Bihar, having the distribution of 73138 sq.Km, 65770 sq.Km., 238 sq.Km and 634 sq.km, of drainage area respectively (Kumar and Reddy, 2006).

3.2 Hirakud Dam

Hirakud dam consists of 4800 m long main dam of earth, concrete and masonry portion being flanked by earthen dykes in both left and right sides. The maximum height of the dam in concrete portion is 61 m and that in earth portion is 59.44 m. The provision of 64 sluices of size (3.66 x 6.2) in controlled by vertical slide gates operated from on operation gallery size 3.35 in x 6.10 m along the body of the dam is a special feature of this project. There are 21 crest bays in the left spillway having 15.54 wide and 6.1 m high radial gates (Dhal et al., 2006).

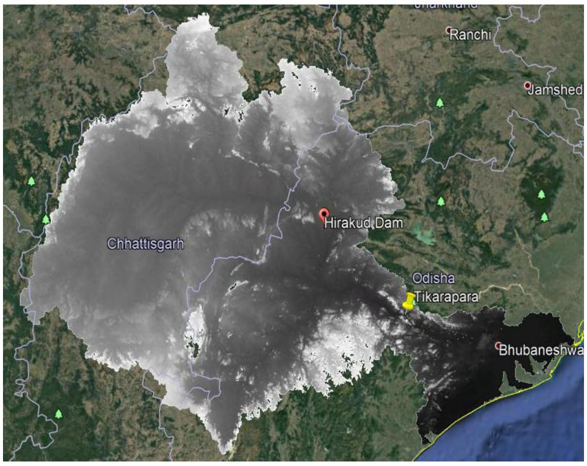

Fig. 1: Study Area showing in Google Earth

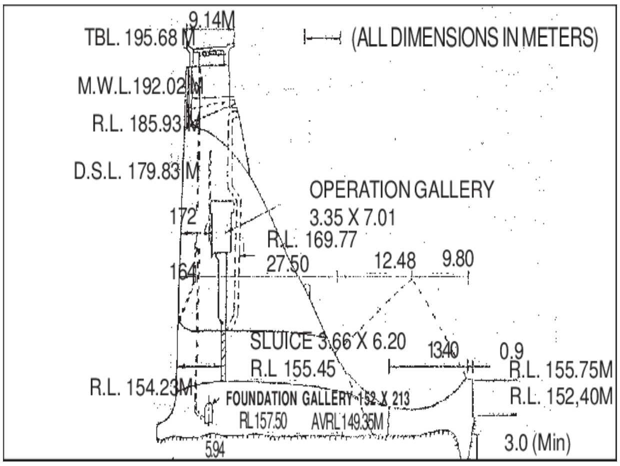

Fig.2: Hirakud dam spillway section (Source: Dhal et al., 2006)

3.3. Data Collection

Geometric details of Hirakud Dam, Spillway rating curve, Surface area-elevation data of Hirakud reservoir is collected from Chief Engineer, Designs & Dam Safety, Bhubaneswar. Daily stage-discharge from year 1970-2010 entering Hirakud reservoir and cross-section data at d/s of Hirakud dam is collected from Central Water Commission (CWC) website <http://www.india-wris.nrsc.gov.in/>. Digital elevation Model (DEM) 90m90m resolution of the study region is obtained from the Shuttle Radar Topography Mission (SRTM) website (<http://srtm.csi.cgiar.org/srtmdata/>) to obtain elevation at necessary spots and to obtain cross-section data of the river channel sections in the study area. <http://srtm.csi.cgiar.org/srtmdata/>.

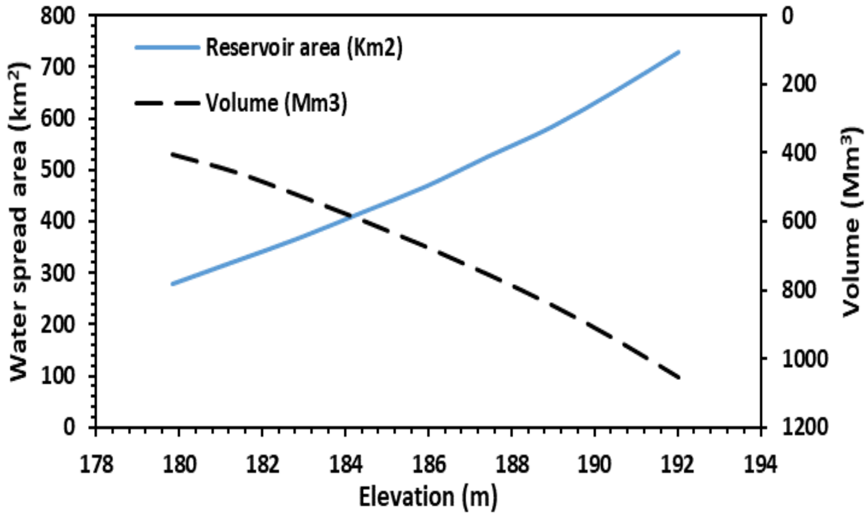

Fig. 3: Area-Elevation-volume curve of Hirakud reservoir

4 Results and Discussion

4.1. Input data in HEC-RAS

Terrain map is used from DEM to give the elevation at various points in HEC-RAS as shown in Fig. 3. The Hirakud reservoir behind the dam is considered as storage area. Flow area and dam section is prepared in HEC-RAS.

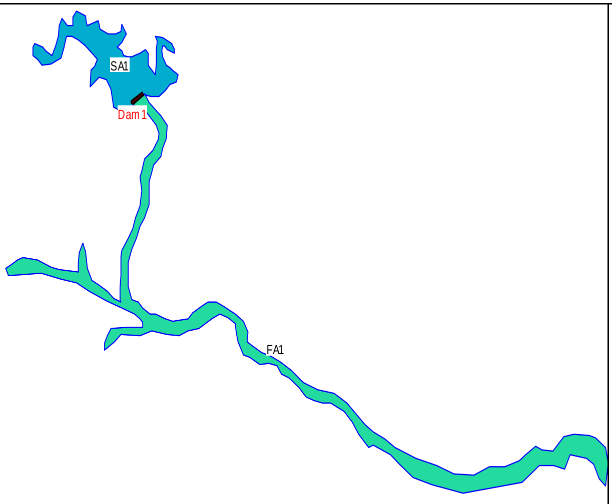

Fig. 4: Mapping of storage area, flow area and dam in HEC-RAS

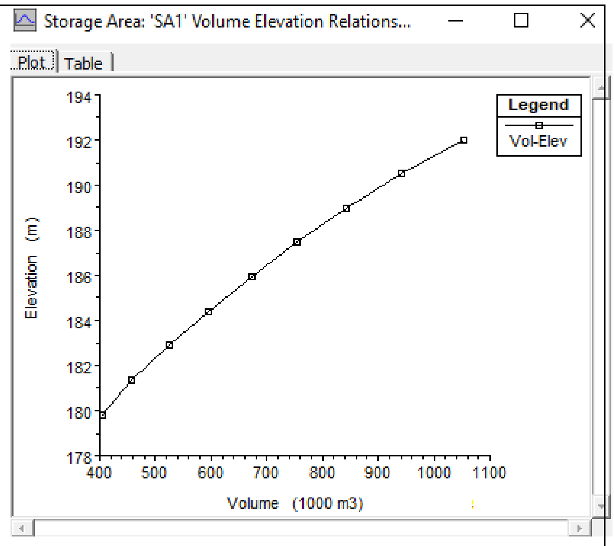

Fig. 5: Elevation-volume curve in HEC-RAS

4.2. Dam Breach Parameter Estimation

Herein the Xu and Zhang (2009) regression model is used to estimate parameters of dam breach. Xu and Zhang (2009) are the latest model among all regression models. Using dam geometric and elevation data the dam breach parameters are estimated in HEC-RAS as follows (Fig.4):

Breach bottom width (final)=88m,

Breach side slope= 0.32,

Breach developed time=14.50 hr.

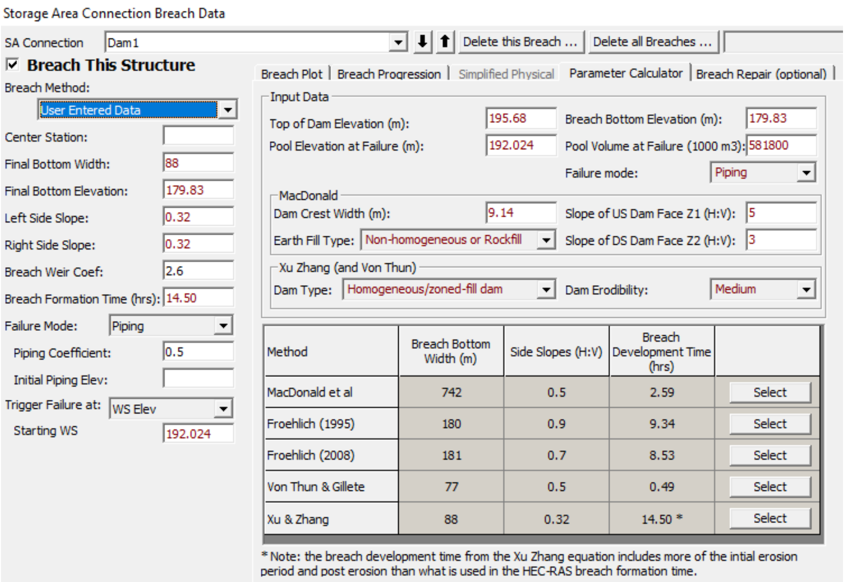

Fig. 6: Dam breach parameter estimation in HEC-RAS

4.3. Flood Routing Result During Dam Break

4.3.1. Using Froehlich (2008) Regression Model

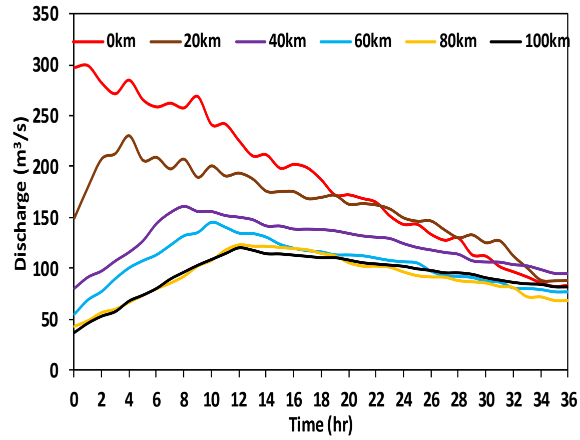

Fig.7: Discharge hydrographs obtained in HEC-RAS at different d/s location (0-100km) due to dam failure using Froehlich (2008) regression equation.

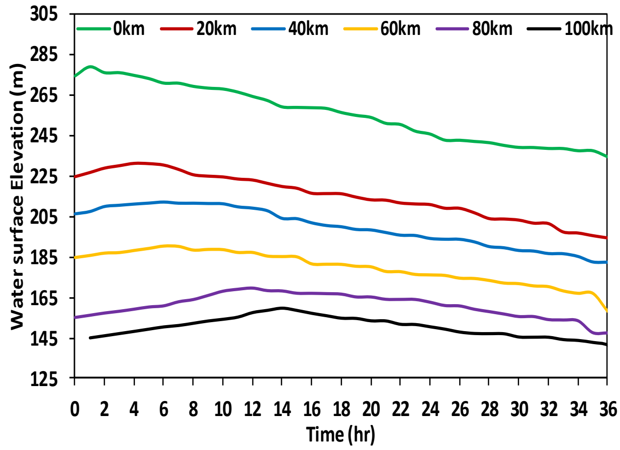

Fig.8: Stage hydrographs obtained in HEC-RAS at different d/s location (0-100km) due to dam failure using Froehlich (2008) regression model.

4.3.2. Using Xu and Zhang (2009) regression model

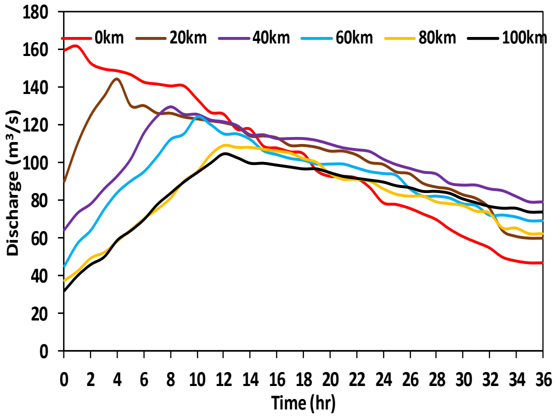

Fig.9: Discharge hydrographs obtained in HEC-RAS at different d/s location (0-100km) due to dam failure using Xu and Zhang (2009) regression model.

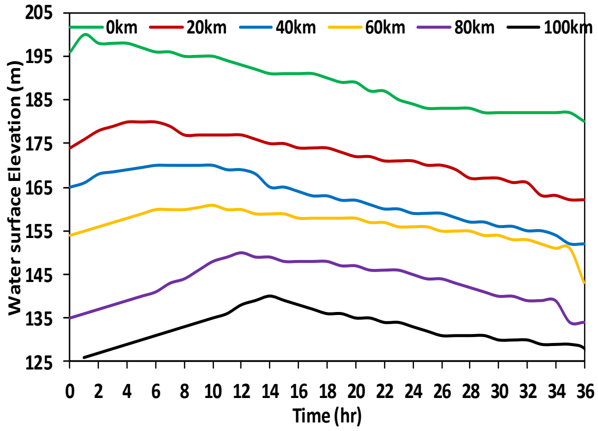

Fig. 10: Stage hydrographs obtained in HEC-RAS at different d/s location (0-100km) due to dam failure using Xu and Zhang (2009) regression model.

Table 2: Summary of the dam break results obtained in HEC-RAS using Froehlich (2008) regression model

Distance from dam (Km)	Q_P (1000 m^3/s)	Time to peak discharge (T_p) (hr)	Maximum water surface elevation (m)	Time to peak maximum water surface elevation	Maximum Velocity (m/s)
0	300	1	279	1	13.19
20	231	4	231	4	12.87
40	161	8	212	8	12.34
60	145	10	190	10	11.89
80	123	12	170	12	11.31
100	120	13	160	13	11.03

Table 3: An overview of the dam break findings from the HEC-RAS using Xu and Zhang (2009) regression model

Distance from dam (Km)	Q_P (1000 m^3/s)	Time to peak discharge (T_p) (hr)	Maximum water surface elevation (m)	Time to peak maximum water surface elevation	Maximum Velocity (m/s)
0	162.5	1	200.5	1	10.58
20	144.7	4	180.3	4	10.31
40	130.4	8	170.0	8	10.11
60	124.9	10	161.07	10	9.88
80	109.0	12	150.23	12	9.45
100	105.2	13	140	13	9.09

Table 4: Average values of the dam break results obtained in HEC-RAS using Froehlich (2008) and Xu and Zhang (2009) regression model

Distance from dam	Q_P (1000	Time to peak discharge	Maximum water surface	Time to peak maximum water	Maximum Velocity

(Km)	m^3/s)	(T_p) (hr)	elevation (m)	surface elevation	(m/s)
0	230.99	1	239.27	1	11.89
20	187.34	4	205.75	4	11.59
40	145.74	8	190.87	8	11.23
60	134.65	10	175.72	10	10.89
80	116.14	12	159.83	12	10.38
100	112.39	13	149.82	13	10.06

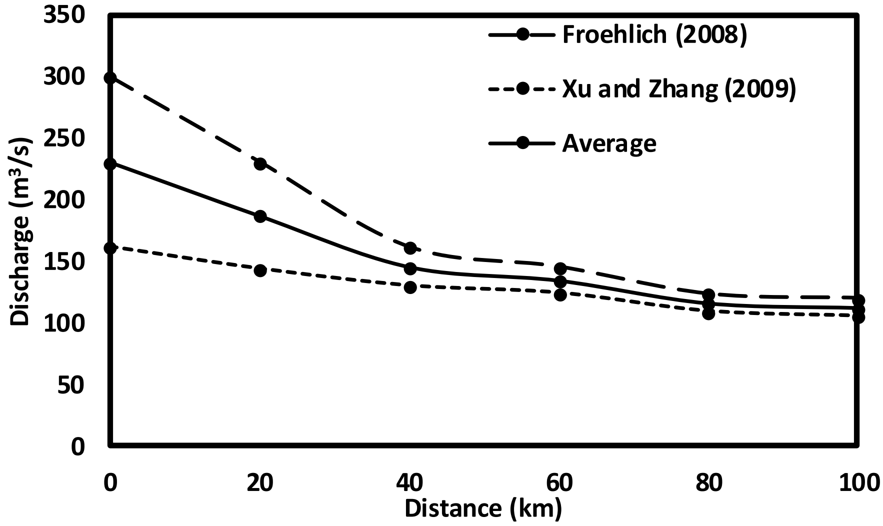

Fig. 11: Comparative plot of peak discharge obtained in HEC-RAS using Froehlich (2008) and Xu and Zhang (2009) regression models.

Initially, the discharge estimated using Froehlich (2008) regression model is comparatively more than estimated using Xu and Zhang (2009) regression model but while reaching 100 km distance is almost coming to equal.

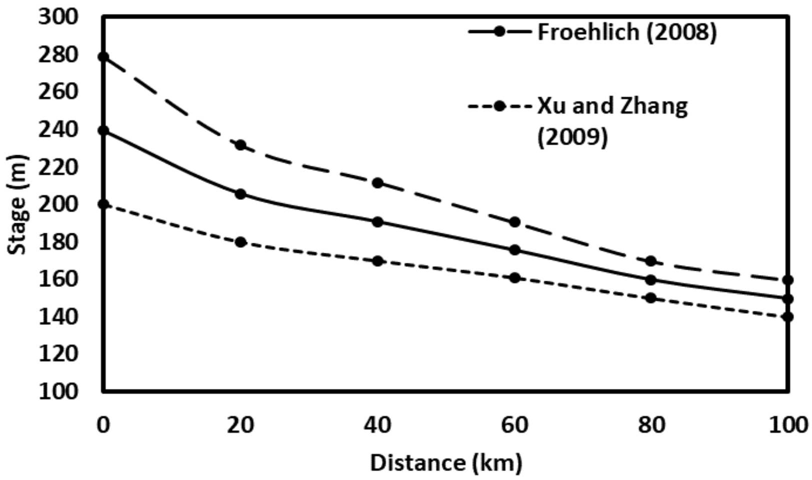

Fig. 12: Comparative plot of the peak stage obtained in HEC-RAS using Froehlich (2008) and Xu and Zhang (2009) regression models.

Initially, the stage estimated using Froehlich's (2008) regression model is comparatively more than estimated using Xu and Zhang (2009) regression model.

5 Conclusion

5.1. Conclusion

In this study, dam break flow analysis for Hirakud dam was performed assuming a hypothetical dam failure case using HEC-RAS. Data required for the above study was obtained from various sources. Study was carried out using both Froehlich (2008) and Xu and Zhang (2009) regression model. Conclusions derived from the present study are given below:

(i) The discharge and stage estimated using Froehlich (2008) regression model is comparatively more than estimated using Xu and Zhang (2009) regression model.

(ii) The average peak discharge at dam site is found to be 230990 cumecs, 187.34 cumecs, 145.74 cumecs, 134.65 cumecs, 116.14 cumecs, and 112.39 cumecs at 0km, 20km, 40km, 60km, 80km, and 100km, respectively.

(iii) The maximum water levels at the dam site is 239.27 m, 205.75m, 190.87m, 175.72m, 159.83m, 149.82m at 0km, 20km, 40km, 60km, 80km, and 100km, respectively.

5.2. Future Scope of Research

a) Sensitive analysis of bed roughness, breach width, breach slope and breach time on water surface elevation and discharge can be studied.
b) Through study in 2-Dimensional direction is required.

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