DAM BREACH MODELING and INUDATION MAPPING: A CASE STUDY OF JEMA DAM, ABAY BASIN, ETHIOPIA

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Abstract
This Paper analyzed the probable failure of a dam under a set of pre-defined scenarios, within the framework of a case study, the case subject being the Jema dam located at Amhara Region of Ethiopia. A probable maximum flood of Gilgel Abay River (tributary of Jema River) has been computed using Hershfield (1961) and later modified (1965) is based on the general flood frequency equation by Chow (1961). Breach parameters prediction, peak outflow hydrograph were determined by HEC-RAS model based on available technical and geometric data. Different maps such as flood areal extent map, flood depth map and velocity map have been produced by HEC-GeoRAS. Probable maximum flood of Gilgel Abay River by Hershfield’s technique was found to be 1726.28 m³/s. The maximum breach discharge resulted from HEC-RAS model was 79,886.37 m³/sec and the maximum area inundated by this flood in downstream was found to be 41.6km². The areal maps show that the part of irrigation command area at right side is a farm land while on left side is Bikolo Abay Town and settlement villages to be prone to flooding. The depth and velocity of flood also depict that the downstream rural village near the river bank are under extreme hazard category. Therefore, a further study regarding the economic and life damage consequences in detail has to be done.

Key words: Dam Breach, Flood Inundation Mapping, HEC-RAS, HEC–GeoRAS.

INTRODUCTION

Ethiopia is situated in the horn of Africa, and is bordered by Sudan, Kenya, Somalia, Djibouti and Eritrea. The surface area is more than one million square kilometers and the country stretches from latitude 3° North to latitude 15° North of the equator and from 33° East to 48° East longitudes (MoWR, 2004). It has a large population of approximately 77.1 million people with an annual growth rate of 2.4% (FAO, 2008). Fortunately, Ethiopia is lucky in that it has got ample source of surface and subsurface water for which it is known as “The Water Tower of East Africa.”Moreover; the irrigation potential is estimated to be about 4.25 million hectares of which only 5.8% is irrigated. (Study carried out by International Water Management Institute). The dam break can be triggered by overtopping, piping or foundation failure depending on the type of dam, its constituent construction materials, hydrodynamic movements in the reservoir and/or on the dam, magnitude of inflow, performance of the dam and its appurtenance structures, operation of gates on times of peak floods and many more. Therefore, the dam break analysis should consider these factors. Jema dam is one of the embankment dams planned to impounding water for irrigation purpose. The dam is a rock fill type dam with impervious clay core at the middle covered by adjacent transition fill material and sand filters on both sides with a purpose of preventing finer materials from migration (WWDSE, 2010).
The safety of dam cannot be secured only with provision of sufficient discharging capacity of the spillway or adequate hydraulic design of the whole dam because still extreme cases like dam breaching should be expected. At downstream of Jema dam there are so many rural villages and bikolo abay town which are probably at risk and the command area of the project is also at downstream of the dam. Hence, the objective of this study is to model the dam breach process of Jema dam and map the downstream area to be inundated by the flood.

**MATERIALS and METHODS**

**Description of the dam and its location**
Jema Dam is located in Amhara National Regional State, West Gojam Zone, Mecha Woreda, about 527 kilometers from Addis Ababa. The dam axis is located in between the geographic grid ref. UTM E 37°10’58’, N 11°11’32” and E 37°11’11”, N 11°12’2”. The dam is rock fill type with crest length of 1022m and crest width of 10m. It has 70.9 m above the riverbed level, the dam crest level is at 2133.9 msl and the river bed level is at 2063 m.s.l., the upstream and downstream slopes of the dam are 1V:1.75H of 1V:1.5H respectively. The invert level of outlet is set at 2097.5 m.s.l and it is within the body of the dam. The reservoir is intended to accommodate a total of 124 million cubic meter of water.

**Materials Used and Data Collection**
The main materials used for this research are:

- HEC-RAS model for dam break simulation and unsteady flood routing.
- HEC-GeoRAS tools to obtain modeling reach and floodplain cross-sectional geometry and for inundation mapping.
- HEC-GeoHMS to delineate the watershed.

Daily rainfall data, Stream flow data of Gilgel Abay and Koga River, Salient features of Jema dam, Reservoir area-elevation-volume curves, Design report of the project and Hydrological data (full PMF inflow hydrograph) were collected from National Meteorological Agency, Ministry of Water, Irrigation and Energy and from Ethiopian Construction Design and Supervision Works Corporation.

**Data Analysis**
PMF hydrograph of jema river at dam was estimated by determine first a Probable Maximum Precipitation (PMP) and unit hydrograph. Watershed of Gilgele abay was delineated by HEC GeoHMS to determine the area. Dam breach analysis is then done to predict the breach size and estimate the outflow hydrograph using HEC RAS. Having determined the outflow magnitude, the flood is again routed through the downstream channel and flood plain using HEC-RAS. Results of hydraulic flow simulation are exported into HEC-GeoRAS to produce flood inundation maps in order to determine the flood prone area, so that flood early warning system can be set and emergency action plans can be made.

**Hydrology Analysis**
In order to determine PMF hydrograph, a Probable Maximum Precipitation (PMP) is initially calculated using observed daily rainfall data from stations located nearby to the watershed. A total of six rain gauge stations located around the periphery of the catchment are considered as a data source for the study. The data obtained for all station was 30 years and all stations are outside of the catchment. The inverse-distance weighting method is used to fill the missed daily rainfall data. The data were tested for its independence, stationarity, homogeneity, outlier and trend.

The PMF hydrograph of Jema dam were directly taken from project report of designer organization and scaled up by 1.47 in order to breach the dam. Such type of failure may occur in such circumstances as in basin wide extreme hydrological event and the PMF may be underestimated by different reasons. The PMF of Gilgele abay river was computed by convoluting a Probable Maximum Precipitation (PMP) on a representative Unit Hydrograph (UH) of the watershed. The statistical approach suggested by Hershfield (1961) can be used to estimate PMP and rainfall of different return periods, (WMO, 2009).

\[
X_T = X_{av} + K \sigma x 
\]  
\[
K = \frac{(X_1 - X_{n-1})}{\sigma n-1} 
\]

Where
- \(X_T\) is the event (magnitude) at return period of \(T\) years
- \(X_{av}\) the mean of the sample data
\( \sigma \) the standard deviation and

\( K \) =the frequency factor, which depends upon the frequency distribution representing the sample series.

\( X_1, X_{n-1} \) and \( \sigma_{n-1} \) are the highest, mean and standard deviation respectively excluding the \( X_1 \) value from the series.

In a survey of more than 2700 stations over world, \( K_m \) values as calculated from the above equation vary from less than 3 to 14.5. Hershfield adopted the highest value rounded to 15 for estimating PMP,

\[ PMP = X_n + 15 \sigma_n \] (3)

Depth of rainfall at different return period was determined in using the general flood frequency equation Chow (1961), \( k \) value was adopted 15(Table 2).

The synthetic unit hydrograph of Snyder was used to determine \( Q_p \) and \( T_p \) from watershed characteristics. Having peak discharge and lag time from Snyder, the unit hydrograph was estimated from the synthetic dimensionless hydrograph.

The SCS dimensionless hydrograph is a synthetic unit hydrograph in which the discharge is expressed by the ratio of discharge \( q \) to peak discharge \( q_p \) and the time by the ratio of time \( t \) to the time of rise of the unit hydrograph, \( T_p \). Convoluting of PMP and UH was made to obtained PMF hydrograph and to have a complete PMF a base flow has to be added in the Direct Runoff Hydrograph (DRH).

**HEC-RAS Model and HEC-GeoRAS**

The dam break tool in HEC-RAS can simulate the breach of an inline structure such as dam, or a lateral structure such as a levee. The latest versions of the HEC-RAS model include algorithms to model both overtopping and piping breaches (HEC, 2010). Dam break simulations are performed with HEC-RAS for dam safety studies as well as for flood damage analyses for situations that involve possible dam breaches (D. Michael Gee, 2010). Two scenarios were created using HEC-RAS, the first scenario is the Probable Maximum Flood (PMF) dam breach and the second is the Sunny day dam breach. The HEC-RAS breach computations include both overtopping and piping failure modes.

HEC-GeoRAS is an ArcGIS extension developed by the HEC. It assists in creating data sets in GIS to extract information essential for hydraulic modeling. After steady or unsteady flow simulation, HEC-RAS results can be exported for processing in the GIS by GeoRAS.

**Equations to Predict Dam Break Parameters**

The breach width (b), breach height (hb) and breach time (Tf) have a great influence on the forecast of the outflow and the flooded area downstream of the dam.

**Von Thun and Gillette (1990):**

The Von Thun and Gillette equation for average breach width is:

\[ B_{ave} = 2.5hw + C_b \] (5)

Where: \( B_{ave} = \) average breach width (meters)

\( hw = \) depth of water above the bottom of the breach (m)

\( C_b = \) coefficient, which is a function of reservoir size

**Froehlich (2008):**

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

1H: 1V for overtopping failures
0.7H: 1V otherwise (i.e. piping/seepage)

Froehlich has suggested the peak flow as follows;

\[ Q = 0.607V_w^{0.295}h_w^{1.24} \] (6)

**Analysis Scenarios**

The analysis considers: Gilgile Abay River 32 km from jema dam to downstream.

Here are the scenarios set for analysis:

**Jema Dam Piping Break with Gilgel abay mean flow (Scenario1)**

The outlet of the dam is through the body of the dam .2097.5m elevation is assumed susceptible point of start for piping.

**Jema Dam Overtopping Break with Gilgel abay PMF flow (Scenario2)**

Jema Dam has been tested for overtopping failure applying a PMF inflow while the reservoir is at full condition. For the second scenarios, a catastrophic full PMF inflow hydrograph was used as the inflow hydrographs at the upstream end (i.e. Jema dam) of the modeling reach. The value of the lateral mean inflow of Gilgel Abay River were obtained from the daily observed flow data. The annual average flow was found 132.2m³/s. annual average average flow of Jema River at the dam found to be 38.1m³/s.
Downstream Flow Routing and Flood Mapping

The Outflow hydrograph resulted from the breaching dam is routed through the downstream river channel and flood plain and different maps of the resulting water surface extent, water depth and velocity are produced. The models HEC-RAS and HEC-GeoRAS are used integrally. The geo-spatial data used for hydraulic computation in HEC-RAS has been used a DEM with 12.5x12.5m resolution.

The breach outflow discharge is routed through downstream channel and flood plain using the model HEC-RAS. Manning roughness coefficients for various open channel surfaces (Chow, 1988) was assessed. Based on the above justifications the researcher adopts a roughness value between 0.035-0.09. For unsteady flow analysis, upstream boundary conditions are typically flow hydrographs. The downstream boundary conditions are input as a normal depth with friction slope of 0.00172. After the unsteady flow simulation is done on HEC-RAS and a reasonable water surface profile is obtained, results are exported into HEC-GeoRAS for flood mapping.

RESULTS and DISCUSSION

Hydrology

Hydrology study was conducted to estimate the probable maximum flood (PMF) of Gilgel abay river by Hershfield’s estimation technique. Using hershfield method peak discharge of 1726.28m³/s was obtained for Gilgel abay (Figure 1.)

Simulation in HEC-RAS Model

The modeling domain was tested for a set of scenarios under overtopping and piping failures. These scenarios were simulated using breach parameter results from two types of estimation methods namely; Froehlich (2008) and Von Thun and Gillete (1990).

Jema Dam Piping Break with Gilgel Abay Mean Flow (Scenario 1)

In all cases a fictional failure is assumed to occur on 01/1/2025, 00:00hr. piping failure mode was assumed to occur during sunny day.According to this analysis the whole process i.e., the dam breach and peak flood at final cross section in the downstream regions has taken 5 hours (Figure 2).

Jema Dam Overtopping Break with Gilgel Abay PMF (Scenario 2)

In all cases a fictional failure is assumed to occur on 01/1/2025, 00:00hr. PMF in Gilgel Abay and the burst flood from the dam has resulted the following outflow hydrograph. The whole process i.e., the PMF event beginning upstream of the dam, the dam breach and peak flood at final cross section in the downstream regions has taken 37 hours (Figure 3).

Breach Outflow Verification

Overtopping outflow verification

Peak overtopping outflows obtained were compared to the envelope curve as a taste of reasonableness as shown below in Figure 4 VonThun and Gillete (1990), and Froehlich (2008)’s method are found to have 31%, 17% error respectively in relative to the envelope result. However this envelope curve was developed from only fourteen data sets, and may not be a true upper bound of peak flow versus hydraulic depth. Among the three estimation methods of overtopping results in all scenario Von thun and Gillete(1990) has resulted the maximum peak flow of 79886.37m³/s and flood height of 28.9 m and Froehlich(2008) resulted maximum outflow of 50956.07m³/s for piping mode of failure.

Flood Inundation Mapping

An extended area downstream of the dam would be affected as a result of Jema dam failure. The flood inundation map for the entire study area for Jema dam failure is presented for overtopping type failure and for piping type failure. Von Thun and Gillete (1990) and Froehlich (2008) method of parameter estimation has resulted in the highest discharge values for overtopping and piping modes of failure respectively. Depth map, velocity map and area map for overtopping failure was prepared for the worst scenario (Scenario2) using Von Thun and Gillete (1990)’s result, for piping failure the maps were prepared by Froehlich (2008)’s and the result is shown in the following consecutive figures.

The depth varies from 56m within the gorge at immediate downstream of the dam to 11m at downstream end of the study area. This is because the cross-section at upstream end is narrow and deep whereas it goes wide and shallow towards downstream end. Hence, the flood is dispersed on the flood plain gaining small depth. The velocity is high along the river channel since it is a well formed water route with less obstacles and lower value of Manning’s coefficient when compared to the adjacent flood.
plain. Velocity maps are also useful in understanding how fast a flood front approach to a specific area. Largest area in the downstream is inundated by this scenario. The flood inundates large command area especially at the confluence of Jema River.

According to FEMA (2013) classification all area near the river bank is under extreme hazard Flood Severity Category i.e. product of depth and velocity is under categories of greater than 2.5, for example the rural village 10 km from the dam at confluence of small tributaries with Jema river and Bikolo Abay town are under extreme hazard category. The depth of flood in the rural village 10km from the dam ranges from 7 to 11m. The downstream side of right command area is flooded with a maximum depth of 11m whereas the left side farm and resettlement villages are inundated with average depth of 10m. The average velocity within the river channel is 3.5m/s whereas it decreases gradually up to 0.5m/s going both to left and right extremities. The velocity at any desired location can be read from Figure 8.

Velocity maps are also useful in understanding how fast a flood front approach to a specific area and this in turn helps in preparing Flood Emergency Action Plans. The velocity of the flood over the entire study area ranges from 0 to 13m/s.

In the left side of the river many rural village and farms are inundated. And finally after Gilgel Abay River with mean flow is join Jema River the subsequent area is Bikol Abay town and the map shows that the town is inundated. Generally, According to FEMA (2013) classification all area near the river bank is under extreme hazard Flood Severity Category i.e. product of depth and velocity is under categories of greater than 2.5.

CONCLUSIONS

The severity level of a dam breach depends on the size of reservoir, the incoming flood, the method implied to forecast breach and the settlement magnitude and types on the downstream reaches. In general scenario one i.e. piping modes of failures are obviously found to result in lowest values of breach parameters when compared with overtopping results. Von Thun and Gillette (1990) method of breach estimation has resulted in a peak flow of 79886.37m/s, during overtopping.

For worst scenario the flood depth, velocity and area map were prepared and in all cases part of rural villages, command area and Bikolo Abay town both side of river were inundated with considerable depth and velocity. The area where at joining of tributary and Bikolo Abay town were found to be more prone by the flood. Based on dam hazard potential classification Jema dam is categorized under high hazard dam. The dam breach by von thun and Gillete (1990) evacuate all water in the reservoir within one and half day. Total area of 41.6km² in the downstream inundate by this method. The dam breach modeling and flood maps results in this study can adequately inform the user the extents of hydrological induced dam break and the subsequent flood.

Based on the result obtained, herein there are invaluable suggestion:

- Cross-section measurements data at required River station were not available at dam owner (MoWIE) and design (WWDSE) offices. To fill this data gap cross-section geometry were generated from low resolution DEM (12.5mx12.5m details) by using HEC-GeoRAS software. There would, however, be further need to improve the generated cross-sections by taking cross-section measurements on reach or by generating cross-sectional geometry from high resolution DEM for the study area. This is because cross-sectional data generated from low resolution DEM (12.5m) might not be of sufficient span-width or maximum elevation required for the analyses. The topographic maps should be produced by actual topographic surveying or generated from finer DEM if possible.
- The results of this study can be re-checked using different dam breach and flood models with higher accuracy and strong computational algorithms, such as models considering unsteady flow and 1D-2D hybrid models.
- Further study should be continued and emergency action plan (EAP), shall be prepared in order to evacuate people potentially at risk in case the dam fails.
- The area below the dam is relatively flat and people living on this area, especially near the river banks, shall be resettled at significantly far place.
- Flood protection dykes should be designed and implemented to protect Bikolo Abay town.
- Any water resource and infrastructure development in downstream should consider the result of this thesis breach maps.
- Effective dam monitoring system should be in place.
- Further study on economic and life damage consequences should be undertaken in detail.
- Finally, I would like to recommend both the dam owner (Ministry of water, irrigation and Energy) and the designer (Water works design and supervision enterprise) to give special attention to the Dam break analysis and make a detail investigation by using the latest dam break software.
REFERENCES

Fread 1988 Fread, D.BREACH: An Erosion Model for Earthen Dam Failures. Silver Spring:
ICOLD European club, 2012. working group on dam safety. [Report].
L.M. Zhang, M. Peng and Y. Xu (2010), “Assessing Risks of Breaching of Earth Dams and Natural Landslide Dams”, Indian Geotechnical Conference from December 16-18, 2010, Department of Civil and Environmental Engineering, the Hong Kong University of Science and Technology, Hong Kong.
Organization.
WMO No.168, Guid to hydrological practice, fifth edition 1994, Geneva

Appendix

Table 1. Adjustment of Parameters Mean and Standard Deviation as per WMO 2009

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Adjusted values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Statistical Parameters for Observed Data</td>
<td></td>
</tr>
<tr>
<td>Mean (n)</td>
<td>59.7</td>
</tr>
<tr>
<td>Mean (n-m)</td>
<td>61.7</td>
</tr>
<tr>
<td>SD (n), n-number of data</td>
<td>8.9</td>
</tr>
<tr>
<td>SD(n-m), m=1</td>
<td>8.8</td>
</tr>
<tr>
<td>Adjustment of Mean and SD for max. Observed event</td>
<td></td>
</tr>
<tr>
<td>Mean (n)-Adjustment factor</td>
<td>1.03</td>
</tr>
<tr>
<td>SD (n)-Adjustment factor</td>
<td>1.1</td>
</tr>
<tr>
<td>Adjustment for Fixed Observational Interval (i.e. conversion of 1-day RF to 24-hr RF values) Recommended</td>
<td></td>
</tr>
<tr>
<td>Mean (n) -Adjustment factor</td>
<td>1.13</td>
</tr>
<tr>
<td>SD (n)-Adjustment factor</td>
<td>1.13</td>
</tr>
<tr>
<td>Adjustment of Mean and Standard Deviation for Sample Size</td>
<td></td>
</tr>
<tr>
<td>Mean (n) -Adjustment factor</td>
<td>1.01</td>
</tr>
<tr>
<td>SD (n) -Adjustment factor</td>
<td>1.04</td>
</tr>
<tr>
<td>Final Adjusted Values of Mean (Xn) and Standard Deviation (Sn)</td>
<td></td>
</tr>
<tr>
<td>Mean (n)</td>
<td>70.2</td>
</tr>
<tr>
<td>SD (n)</td>
<td>11.6</td>
</tr>
</tbody>
</table>

\[
Fr = 1 - 0.02 D^{-0.33} A^{0.5} \quad \text{........................................................... (4)}
\]

Where: D = duration in hours (in our case D=24hrs), A = drainage area in sq.km

Table 2. Point and Areal Rainfall of the Catchment

<table>
<thead>
<tr>
<th>Point and Areal rainfall of the catchment</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Return period</td>
<td>Depth of PRF(mm)</td>
</tr>
<tr>
<td>PMP</td>
<td>243.5</td>
</tr>
</tbody>
</table>
Figure 1. PMF hydrograph of Gilgel Abay River by Hershfield

Figure 2. Maximum breach outflow hydrographs at selected stations, piping by Froehlich (2008)
Figure 3. Maximum flow hydrographs at selected points, overtopping by Von thun and Gillete

Figure 4. Verification of overtopping outflows using experienced outflow rates envelope
Frohelich (2008) method: 66,328.79 m³/s or 2,342,379.33 ft³/s
Vonthun and Gillete method: 79,886.37 m³/s or 2,821,160.79 ft³/s
Hydraulic depth: 232.6 ft

Figure 5. Flood Depth Map by Von Thun and Gillete (1990) Scenario Two
Figure 6. Velocity Map by Von Thun and Gillette (1990) Scenario Two

Legend
- Main Road From Addis Ababa to Bahirdar
- Jema River
- Jema right side downstream tributary
- Jema left side downstream tributary
- Gilgel Abay River
- Jema Dam site
- Jema reservoir
- Bikolo Abay Town
- Max Flood Induration Boundary Area Map
- Jema command area

Velocity Map
VALUE
0 - 0.649431386
0.649431386 - 1.594058856
1.594058857 - 2.479647109
2.479647109 - 3.365235362
3.365235363 - 4.309862832
4.309862833 - 5.313529519
5.313529520 - 6.43527464
6.435274641 - 7.734137412
7.734137413 - 9.564351315
9.564351316 - 15.05500031

Variations in Velocity Map
- Velocity range from 0 to 15.05500031
- 8 Kilometers scale

 command area

Max Flood Induration Boundary Area Map
By Von Thun and Gillette

0 1 2 4 6 8 Kilometers
Figure 7. Flood Inundation Area Map by Von Thun and Gillete (1990) Scenario Two

Figure 8. Flood Depth Map by Froehlich (2008) Scenario One
Figure 9. Velocity Map by Froehlich (2008) Scenario One

Figure 10. Inundation Area Map by Froehlich (2008) Scenario One