ORIGINAL PAPER

Dam breach analysis: a case of Gidabo dam, Southern Ethiopia

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Abstract

Gidabo dam provides flood control and irrigation water supply for sugarcane and rice cultivation. The dam has 25.8 m height and 335 m crest length with side ogee spillway to pass 10,000 years' flood. There were different economic developments downstream of the dam including irrigation command area and irrigation structures. Since Nature is full of uncertainties, it is likely that Gidabo dam can be subjected to sudden breaches due to the probable maximum flood. Therefore, dam breach analysis and flood inundation map preparation should be conducted. The Hydrologic Engineering Center's River Analysis System new version was used to analyze the dam breach for overtopping failure. River analysis system Mapper which is Geographic information system tool of Hydrologic Engineering Center's River Analysis and Geographic information system were used to develop flood inundation map. Dam breach parameters were calculated within the Geographic information system model by using beach parameter calculator tab. A two-dimensional unsteady flow simulation of the dam breach was performed by using the inflow hydrograph as upstream boundary condition. From the rainfall data analysis, the probable maximum precipitation resulting probable maximum flood was found 277 mm results a peak inflow of 6387 m^3/s. It was found that the breach bottom width was 143 m while the breach side slope (horizontal: vertical) was 1.4:1 and 2.7 h breach formation time. The peak breach outflow was found to be 15,848.85 m^3/s which covers 2050 hectares with maximum depth of 12.14 m.

Introduction

Dams are hydraulic structures used to store, control and divert water, impounding it behind the upstream side of dam in a reservoir for different purposes, like hydropower generation, water supply, irrigation, navigation and transportation, etc. Although dams have many advantages, the risk that may happen due to the failure still exists. Dams can have a risk to downstream communities and properties if not designed, operated and maintained properly (Hayimanot 2015). In India, the worst dam disaster occurred in Machhu II dam which was constructed to serve an irrigation

scheme. This dam failed because of excess flood, inadequate capacity of spillway and overtopping in August 1, 1979, an dam Kaddam also failed due to overtopping which resulted in 137.2 m of breach width on the left bank in August 1958 (Zagonjolli 2007).

The term dam breach analysis is usually related to the process of studying a dam failure phenomenon and analyzing the resulting consequences at downstream region. It deals with simulation of probable failure of existing dams and analyzing the resulting consequences (Pandya and Jitaji 2013). Dam breach modeling is typically done within a larger study context that develops inflow hydrographs from various frequency storms, evaluates project spillway adequacy, estimates breach parameters and performs routing and mapping of the resultant flood (Gee 2010).

Different organizations and researchers have contributed their findings in the analysis of dam breach and its consequence. They have derived regression equations based on data from historical dam failure events that are used in predicting the breach geometry. These include MacDonald and Langridge–Monopolis (1984) and Froehlich (1995).

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Developments of analytical models using the principle of hydraulics and sediment transport are also useful in simulating the breach process and downstream flooding (Leoul 2015).

Dam breach can be simulated with numerical models. Numerical models use computer program to model dam breach and are further classified as hydrologic and hydraulic models. The hydrologic models include Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) and Hydrologic Engineering Center's-1 (HEC-1) and hydraulic models include Hydrologic Engineering center hydraulic river analysis system (HEC-RAS), National Weather Service simple dam break (NWS SMPDBK) and Flood wave (FLDWAV) (Tadesse 2015).

Due to lack of understanding of hydrological parameters, capacity of the reservoir and spillway, the embankment dam breach is the most challenging in most countries in the world. Gidabo dam is rock fill dam found in Southern Ethiopia, which was constructed by Ethiopian water works construction Enterprise, in Ethiopia. There are different economic developments downstream of this dam. The purpose of this dam breach analysis study is to estimate the probable maximum flood (PMF) causing Gidabo dam to breach and illustrate how the flood wave propagates to the downstream of Gidabo dam. In the present analysis the HEC-RAS model is used for simulation of the flood wave caused by dam failure. This model is one of the most widely accepted models of its kind.

Studying of the Gidabo dam breach analysis has much significance. Some of these are the research result can be used to inform the flood-prone areal coverage to the downstream population if the dam breaches, to develop emergency action plan based on the flood inundation map, and the research finding will support other researchers to do analysis of similar dam breach.

Statement of the problem

Several problems cause dam failure like overtopping, piping, earthquake, land slide, etc. Due to these reasons, our world has experienced some catastrophic dam failures. Like the Banqiao dam which failed in August 8, 1975, it killed an estimate of 171,000 people and 11 million people lost their homes (Fish 2013). As dam breaches, residents, businesses areas, infrastructures, landowners, crops, etc. downstream of the dam will be affected by the flood. Therefore, it is important to analyze the causes and results of dam failure.

Gidabo dam is rock fill dam, which is constructed by Federal Water Works construction Enterprise of Ethiopia, to irrigate more than 14,000 ha of land. There are residential places on the downstream of the dam that can be affected by flood in case if breach. Spillway of the dam designed for 10,000 year's return period flood (Ministry of water,

irrigation and Irrigation of Ethiopia, 2008). Therefore, there is probability of occurrence of dam breach due to the probable maximum flood. In Ethiopia, such studies were not given attention which could be shown that more than 85% of the dam hasn't flood inundation map (Nugusa 2016). Gidabo dam is one of those, which has no such study, so it is critical issue to analyze the downstream damages caused by the dam breach and to set early warning for peoples living at the downstream of the dam, and downstream infrastructures that can be affected if the dam breach occurs.

Objective of the study

The general objective of this study was to predict the breach outflow hydrograph and rout through the downstream valley to prepare downstream flood inundation map for the flood-prone area.

Scope of the study

The scope of the study is limited to 4.1 km downstream of the Gidabo dam. In this study, the analysis is proposed on the prediction of breach outflow hydrograph for overtopping mode of failure. Among different dam breach modeling methods, the new HEC-RAS version 5.0.3 was used to estimate breach outflow and dam breach parameters. The flood inundation map has been prepared on RAS Mapper which is GIS tool of the HEC-RAS new version and on ARC-GIS.

Materials and methods

Description of the study area

Location

Gidabo dam irrigation project area is located in the Abaya-Chamo sub-basin of the Rift Valley Lakes Basin found in the southern part of Ethiopia in Oromiya and Southern regional States. The river is one of the most flow contributors to Lake Abaya. The dam site is approachable from Addis Ababa to Dilla town 360 km asphalt road and 17 km from Dilla town. The project area lies approximately between 6°20′ and 6°25′N and 38° 05′ and 38°10′E, at an average elevation of 1190 above mean sea level. The drain area up to Gidabo dam location is about 2543 square kilometer. The length of the river from the catchment boundary up to dam sites it about 79.6 km. The dam is constructed to irrigate total irrigable area of 15,000 hectares (Fig. 1).

The left bank of the command area of the project is located at Dibicha Laluncha kebele of Abaya woreda in Barona zone of Oromia Region, while the right bank command area is located at Abaya zuria kebele in Loka-Abaya

Fig. 1 Location map of the study
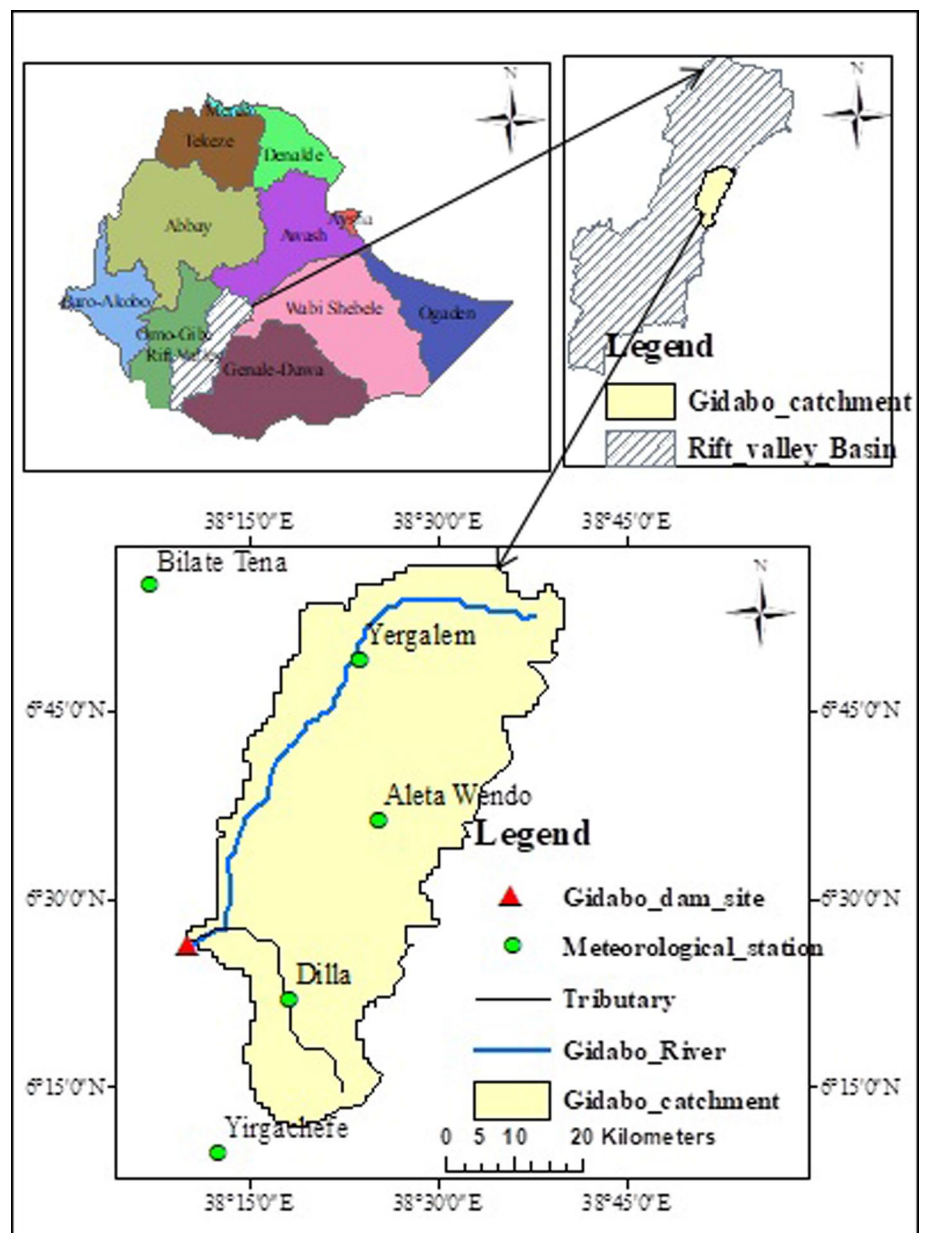
woreda the East Abaya Lake and is very close to Dure and Gola marshes. Urban and rural population at Dibicha Laluncha kebele is about 3057 and 808 accounting to about 79.1% and 20.9%, respectively, whereas urban and rural population at Abaya Zuria kebele is about 2172 and 353 accounting to about 86.0% and 14.0%, respectively. The people in the command as well as in the reservoir area are rural areas.

Satellite image of the study area downstream of the dam was obtained from Google earth, and it was geo-referenced by using Arc-GIS. The geo-referenced satellite image of the study area downstream of Gidabo dam is shown in Fig. 2.

Climate, soil and land use

According to MoWIR (2008a, b), average annual rainfall of the catchment was 1303 mm. Average minimum temperature varies from 10.24 to 12.32 °C, and average maximum temperatures vary from 25.88 to 30.52 °C.

The major soil types of the catchment are chromic luvisols, eutric nitosols, chromic vertisols, eutric cambisols, calcaric fluvisols, calcic cambisols, dystric nitosols, orthic acrisols pellic vertisols, orthic luvisols From these, chromic vertisols type of soil is the most dominant covering about 525 square kilometer (MoWIE 2010).
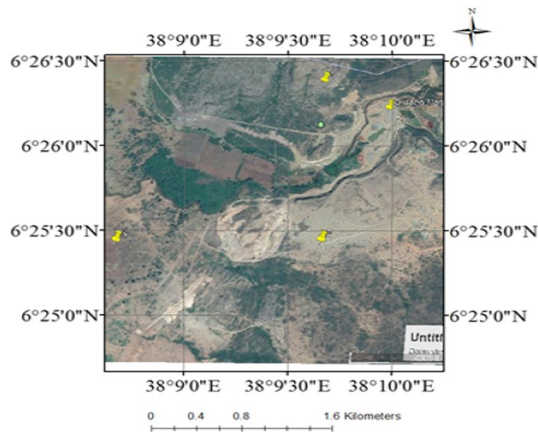
Fig. 2 Satellite image of the study area downstream of Gidabo dam

The land cover of the catchment is dominated by intensively cultivated, moderately cultivated, shrub land, forest and marshland (MoWIE 2010).

Hydrology

Gidabo catchment is measured at the main stream of Gidabo River at Aposto and two other locations on the upstream tributary rivers Kolla and Bedessa tributaries which are gauged close to Aleta Wondo and at Dilla, respectively. These three stations constitute slightly higher than one third of the total catchment area (37%) of Gidabo at dam site (close to) but contribute about 75% of the flows. Gidabo at Aposto alone contributes more than 45% of the flow while its area is only one fourth. Mean discharge of Gidabo River at Aposto gauging station is 17.35 m³/s, 5.04 m³/s at Bedessa, 6.82 m³/s at Aleta Wendo and 42.25 m³/s at Meissa near the dam site. The total mean annual flow volume at the dam site is About 570 Mega cubic meter. Almost one fourth of the flow is contributed during the two peak flow seasons of May and October. Almost 50% of the flow occurs at the dam site during

May, August, September and October (Ethiopian Water Works Design & Supervision Enterprise, 2008).

Hydro-geology

In the project area, Gidabo River is found to be deeper and groundwater availability is more promising around the command and reservoir area though the quality is doubtful. Groundwater level will rise on application of irrigation, seems inevitable and requires further detailed studies. Geologically, it is mainly composed of volcanic products out poured in tertiary quaternary period; these are pyroclastic fall deposits, alkali basalt, rhyolite, volcanic centers compositionally acidic. Additionally, soil units that comprise weathering products of volcanic materials all along the slopes and lacustrine deposit on the flood plain chiefly at the command area characterize Gidabo catchment (WWDSE 2008).

Geomorphology

The geomorphology of Gidabo catchment contains land forms like low mountains, plain with hills, high mountains, low hills, moderate hills and irregular plains. The land from near the dam site is irregular plain surrounded by high Mountains. At the proposed dam site, the river flows through a narrow gorge and the site is extremely suitable for construction of rock fill dam from topographical and geological consideration of the area.

Characteristics of Gidabo dam

The Gidabo is 25.8 m height zoned rock fill dam with 335.52 m crest length (Fig. 3). The storage capacity of the reservoir at maximum water is 102.4 mm³. The upstream area submerged during maximum water level is 1003 ha. The spillway type for Gidabo dam is side ogee weir type constructed with crest level of 1219.5 m a.m.s.l. The spillway

Fig. 3 Typical cross section of Gidabo dam (WWDSE, 2008)
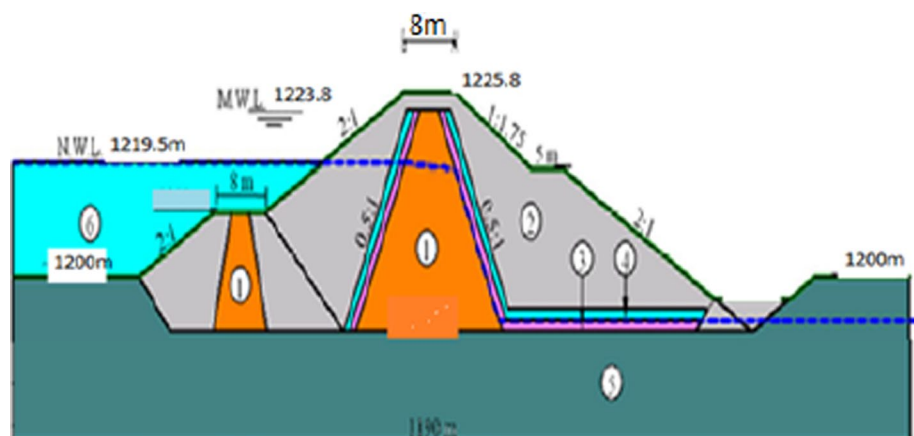
of the dam was constructed in flood susceptible area to pass 10,000 years' flood. The total effective crest length of the spillway is 70 m with discharge capacity of 1417 m³/s. Figure 3 shows the physical characteristics of Gidabo dam obtained from feasibility study of Gidabo Irrigation project.

Conceptual framework of the study

See Fig. 4.

Data collection

To achieve this study, different data were collected from different sources. Some of these data were rainfall data, river cross-sectional data downstream of Gidabo dam, terrain data, and land use/land cover data of the catchment, reservoir characteristics and dam characteristics.

The rainfall data were collected from National Meteorological Agency for metrological stations with in and around the catchment. The cross-sectional data were collected from detailed survey of the Gidabo River downstream of the dam site up 4.1 km. The land use/land covers of the catchment, digital elevation model 30 m resolution and soil data were collected from GIS department of Ministry

of Water, Irrigation and Electricity. Land cover data of the area downstream of the dam were collected from the field visit survey and verified it from the relevant literature Chow (1959), whereas the reservoir and dam characteristics were obtained from the study and design of Gidabo irrigation project final feasibility report.

Meteorological data

Meteorological data for selected rain gauge stations were collected from national meteorology agency. Six stations with thirty years' daily rainfall data (1986–2015) were collected within and around the catchment. Table 1 presents the longitude, latitude and elevation of the six selected stations.

Data analysis

Filling missing rainfall data

The data series obtained from the stations contains enormous missing data inside the record. Different techniques have been employed to fill the data gap. When there are concurrent data in the surrounding stations, the Inverse Distance Square method technique is used to fill the missing
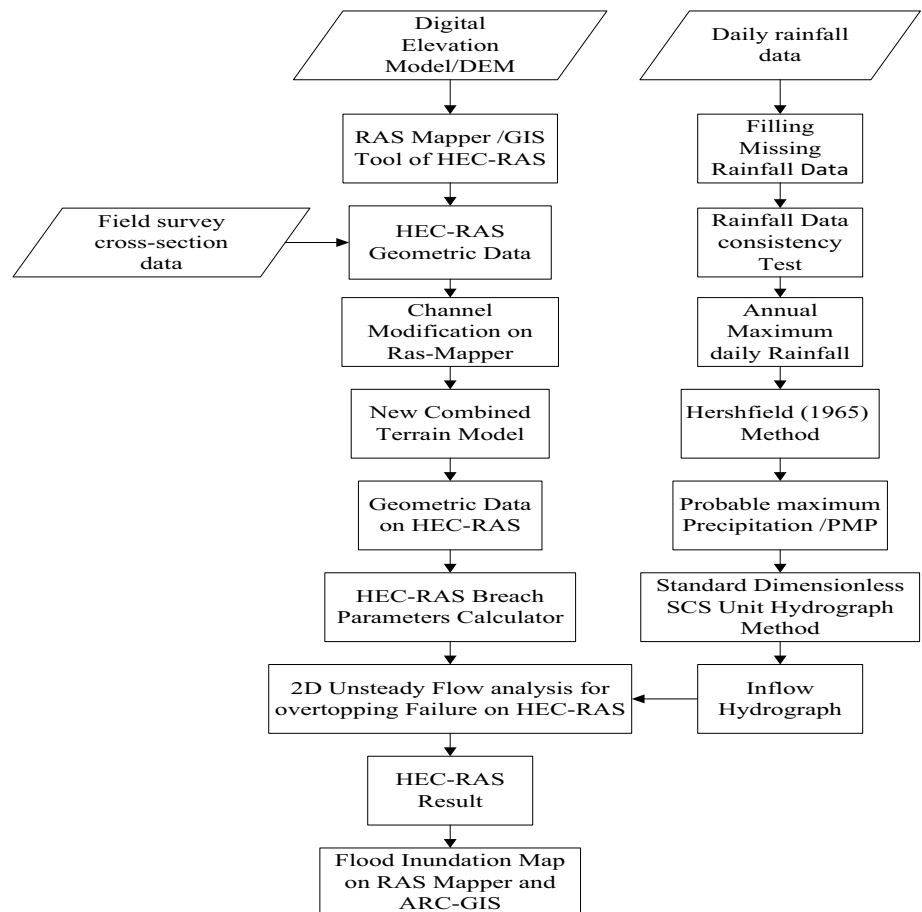
Fig. 4 Conceptual frame work

Table 1 Location of metrological stations with in and around the catchment

Station name	Latitude (°)	Longitude (°)	Elevation (m)
Bilate Tena	6.91667	38.1167	1496
Dilla	6.3667	38.3	1579
FisehaGenet	6.0667	38.1833	2240
Yergalem	6.81683	38.3927	1786
Yirga Chefe	6.15067	38.202	1856
Aleta Wendo	6.60338	38.4181	1947

data using at least three stations. The equation of inverse distance method is stated below.

$$P_x = \sum_{i=1}^{n} \frac{P_i / D_i^2}{1 / D_i^2} = \frac{\frac{P_1}{D_1^2} + \frac{P_2}{D_2^2} + \cdots + \frac{P_n}{D_n^2}}{\frac{1}{D_1^2} + \frac{1}{D_2^2} + \cdots + \frac{1}{D_n^2}} \quad (1)$$

where P_x is an index station that contains the missing data, P_i's are the concurrent value of rainfall at the surrounding stations and D_i's are the distance between P_x and the surrounding station.

Checking consistency of rainfall stations

A consistent record is the one where the characteristics of the record have not changed with time. Adjusting for gage consistency involves the estimation of an effect rather than a missing value. For this study, the consistency of selected rainfall stations was checked by double mass curve analysis. Adjustment of inconsistent record was employed using the following equation.

$$P_{cx} = \frac{P_x M_c}{M_a} \quad (2)$$

where P_{cx} = corrected precipitation at any time period, P_x = original recorded precipitation at time period, M_c = corrected slope of the double mass curve and M_a = original slope of the double mass curve.

Estimation of the probable maximum precipitation (PMP)

According to World Meteorological Organization (1986), PMP which helps for calculating Probable maximum flood is the largest depth of precipitation for a given duration that is physically possible over a particular area and geographical location at a certain time of the year. World Meteorological Organization (1986) suggested Statistical procedures for estimating probable maximum precipitation wherever sufficient precipitation data are available. Therefore, Hershfield (1965) statistical approach was used

to estimate probable maximum precipitation. The general procedure of the method is presented below:

1. Extraction of maximum annual daily rainfall for 30 years from the daily rainfall data for Dilla, Yergalem and Aleta Wendao rainfall stations stations.
2. Calculating PMP of the individual stations by using Hershfield (1965) formula.

$$X_{PMP} = X + \sigma K_m \quad (3)$$

where X_{PMP} = PMP estimate for a station, X = mean of the annual extreme series, σ = standard deviation of the annual extreme series and K_m = Frequency factor for PMP.

Frequency factor 'K_m' is obtained by Eq. 4

$$K_m = \frac{X_{max} - X_{n-1}}{\sigma_{n-1}} \quad (4)$$

where X_{max} = largest value of the annual extreme series, X_{n-1} = mean of the annual maximum series omitting the largest value from the series, σ_{n-1} = standard deviation of the annual extreme series omitting the largest value from the series.

3. Determine area PMP by using Theisen's polygon method using the formula:

$$Areal_{PMP} = \frac{A_1 PMP_1 + A_2 PMP_2 + \cdots + A_n PMP_n}{A_{Total}} \quad (5)$$

where PMP_1, PMP_2 and PMP_n are probable maximum precipitation at stations 1, 2 and n, respectively, and A_1, A_2 and A_n are Thiessen polygon areas of stations 1, 2 and n, respectively.

4. Hershfield formula for PMP is valid only for area less than 25 km^2. For areas greater than 25 km^2 area, reduction factor must be applied. For Ethiopia, area reduction factor is not calculated. For large watershed, > 1000 km^2, area reduction factor lower than 0.6 has been used in East Africa (Watkins and Fiddes 1984). For this study, 0.59 is used as a reduction factor.

Inflow hydrograph

In dam breach analysis, inflow hydrograph is required as upstream boundary condition. In this study, the inflow hydrograph of Gidabo dam site was computed by using the standard dimensionless SCS unit hydrograph for the computed PMP. The standard SCS-CN method is based on the following relationship between rainfall, P (mm), and runoff, Q (mm) (Schulze et al. 1992).

$$Q = \frac{(P - I_a)^2}{(P - I_a) + s} \qquad (6)$$

where I_a (mm) is the initial abstraction including surface storage, interception and infiltration prior to runoff, mm S (mm) is potential maximum retention after runoff begins, which varies with antecedent soil moisture, and other variables can be estimated as:

The relationship between I_a and S was developed from experimental catchment area data. It removes the necessity for estimating I_a for common usage. The empirical relationship used in the SCS runoff equation is:

$$I_a = 0.2S \qquad (7)$$

Substituting $0.2S$ for I_a in Eq. 6, the SCS rainfall–runoff equation becomes:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (8)$$

S (mm) can be calculated as:

$$S = \frac{25400}{CN} - 254 \qquad (9)$$

where CN is the runoff curve number

In Soil Conservation Service (SCS) method, curve number determination is the most important task. CN is a dimensionless catchment parameter ranging from 0 to 100. The SCS has developed standard tables of curve number values as functions of catchment land use/land cover conditions and hydrologic soil group.

For a catchment with sub-areas that have different soil types and land use, a composite curve number CN_c is determined by weighting the curve number values for the different sub-areas in proportion to the land area associated with each:

$$CN_c = \frac{CN_1 A_1 + CN_2 A_2 + CN_3 A_3 + \dots CN_i A_i}{A_1 + A_2 + \dots + A_n} \qquad (10)$$

where CN_i is the curve number of the sub-area i, A_i is the area of the sub-area i and n is the total number of sub-areas.

Parameters of the inflow hydrograph To determine the inflow hydrograph of the catchment, the drainage network above the dam site has been delineated from the 30 m by 30 m DEM using Arc Hydro tool in the GIS. The GIS processing phase includes derivation of the important morphological characteristics that is used to derive the maximum time of flow concentration (t_c), the longest flow length (L), the centroid flow length (L_c) and the average slope.

Reliability of Gidabo dam The designers of hydraulic structures have to consider reliability of water project over n-years of planning period. The n-year reliability (R_n) is simply the probability of no flood event for the first n-years. According to Thomas (1984), reliability for T-year return period flood and n-year planning period can be expressed as:

$$R_n = \left(1 - \frac{1}{T}\right)^n \qquad (11)$$

Gidabo dam is constructed for planning period (n) of 50 years and return period (T) of 10,000; therefore, the reliability (R_n) will be 99.50%.

Dam breach parameters estimation

One of the capabilities of the HEC-RAS new version is the possibility to calculate the breach parameters inside the software. For this study, the breach parameters were calculated by using HEC-RAS breach parameter calculator. The methods are discussed below.

1. Froehlich (1995): Froehlich utilized 63 earthen, zoned earthen, earthen with clay core wall (i.e., clay) and rock fill data sets to develop as set of equation to predict average breach with, side slopes and failure time. Froehlich's (1995) regression equations for average breach width and failure time are:

$$B_{ave} = 0.1803 K_o V_w^{0.32} h_b^{0.19} \qquad (12)$$

$$T_f = 0.00254 V_w^{0.53} h_b^{-0.9} \qquad (13)$$

where B_{ave} = average breach width (m), K_o = constant (1.4 for over topping failure, 1.0 for piping), V_w = reservoir volume at the time of failure (m³/s), h_b = height of the final breach (m), T_f = breach formation time (h).

Froehlich's 1995 paper states that average side slopes should be 1.4h:1v overtopping failure and 0.9h:1v piping failure,

where h = horizontal, v = vertical.

2. Froehlich (2008): Dr. Froehlich utilized 74 earthen, zoned earthen, earthen with a core wall (i.e., clay) and rock fill data sets to develop asset of equations to predict average breach width side slopes and failure time. Froehlich's (2008) regression equations for average breach width and failure time are:

$$B_{ave} = 0.27 K_o V_w^{0.32} h_b^{0.04} \qquad (14)$$

$$T_f = 63.2 \sqrt{V_w / h_b^2 g} \qquad (15)$$

where B_{ave} = average breach width (m), K_o = constant (1.3 for over topping failure, 1.0 for piping), V_w = reservoir volume at the time of failure (cubic meter), h_b = height of the final breach (m), T_f = breach formation time (h), g = acceleration due to gravity (g = 9.81).

Froehlich's 2008 paper states that average side slopes should be 1h:1v overtopping failure and 0.7h: 1v for piping failure piping failure, where h = horizontal, v = vertical.

3. Macdonald and Langridge–Monopolis (1984): MacDonald and Langridge–Monopolis utilized 42 data sets (predominantly earth fill dam, earth fill dam with clay core and rock fill dams) to develop relationship for what they call "Breach Formation Factor." The following equations show the volume of material eroded and breach formation time for earth fill dams.

$$V_{eroded} = 0.0261(V_{out}h_w)^{0.769} \tag{16}$$

$$T_f = 0.0179V_{eroded}^{0.364} \tag{17}$$

where V_{eroded} = volume of material eroded from the dam embankment (cubic meter), V_{out} = volume of water that passes through the breach (cubic meter), h_W = depth of water above the bottom of the breach (m), T_f = breach formation time (h).

According to State of Washington (1992), the bottom breach width can be calculated as

$$W_b = \frac{V_{eroded} - (CZ_b + (h_bZ_bZ_3)/3)h_b^2}{h_b(C + h_bZ_3/2)} \tag{18}$$

where W_b = bottom width of the breach (m), h_b = height from the top of the dam to bottom of breach (m) $Z_3 = Z1 + Z2$ Z_1 = average slope ($Z1$:1) of the upstream face of dam, Z_2 = average slope ($Z2$:1) of the downstream face of dam, Z_b = side slopes of the breach (Zb:1), 0.5 for the MacDonald method.

MacDonald and Langridge–Monopolis stated that the breach should be trapezoidal with side slopes of 0.5h:1v, where: h = horizontal, v = vertical

Hydraulic modeling using HEC-RAS

Terrain model

One of the major problems of hydraulic modeling is that the terrain data does not often include the actual terrain underneath the water surface of the channel region. The RAS Mapper of HEC-RAS 5.0.3 new version can be used to create a terrain model of the channel region from the HEC-RAS cross section and the cross-sectional interpolation surface (USACE 2016). For this study, cross section of the river taken from field survey was used to modify the original terrain. From the modified terrain maximum and minimum bed elevation, Gidabo River was found 1203.81 and 1189 m above mean sea level, respectively.

Geometric data on HEC-RAS

In this study, the storage area is being used to represent the reservoir pool. The hydraulic connection between the storage area and the two-dimensional (2D) flow area is used to model the dam. The 2D flow area is being used to model the hydraulics of the flow downstream of the dam.

Storage area/reservoir Storage areas are like a region in which the water can be diverted into or from. For this study, Gidabo dam reservoir was modeled as storage area. The basic data required for storage area are elevation versus volume data obtained from the design document (Fig. 5).

Storage area/2D area connection Storage area/2D area connection is a used to link the storage area (reservoir) and the 2D flow area. For this study, Gidabo dam was modeled as storage area/2D area connection. The basic information required for SA/2D area connection includes breach plan data and embankment profile. In addition to

Fig. 5 Elevation–volume curve of Gidabo dam
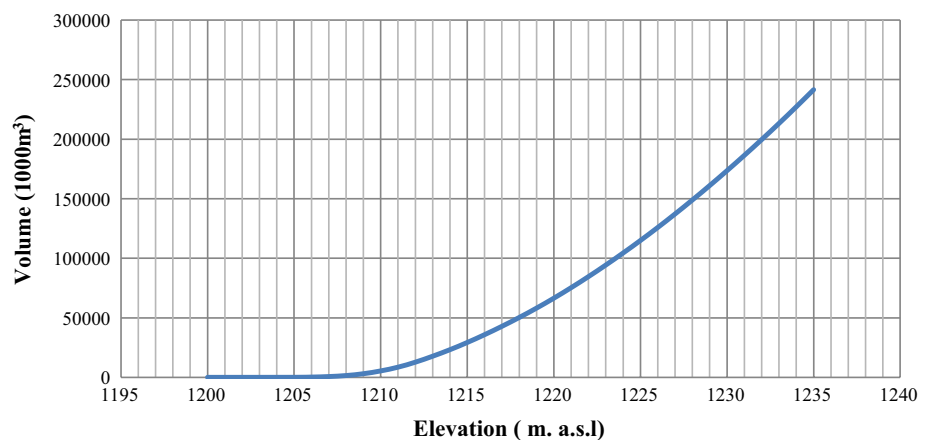
the dam profile failure location, failure mode, full breach formation time, Trigger mechanism, Breach parameters data have to be filled for the SA/2D area connection editor. The embankment profile of the dam is shown in Fig. 6.

2D flow area Two-dimensional flow areas (2D flow areas) region of a model in which the flow through that region will be computed with HEC-RAS two-dimensional computation algorithm. 2D flow areas are defined by laying out a polygon that represents the outer boundary of 2D flow area. Then, the user must define the computational mesh. For this study, the area downstream of Gidabo dam was modeled as 2D flow area with a computational mesh size of 30 m*30 m as shown in Fig. 7. The mesh contains 185,253 grids with maximum, minimum and average grid area of 1631 m^2, 688 m^2 and 901 m^2, respectively.
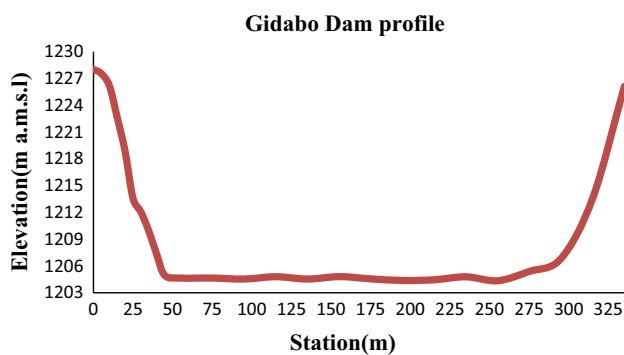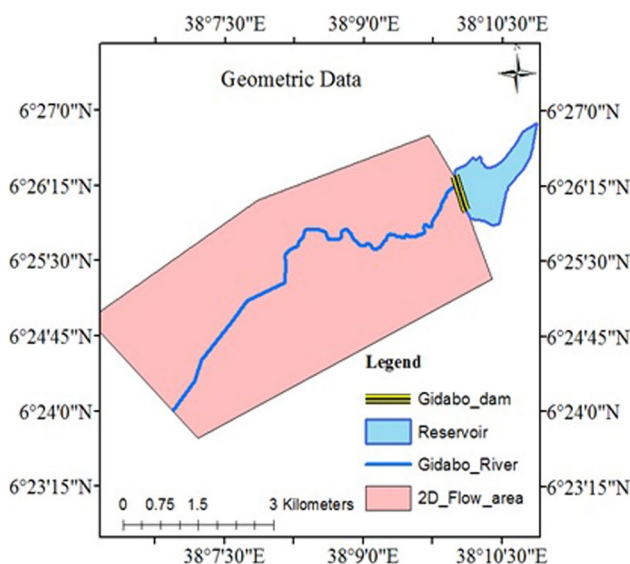

Fig. 6 Gidabo dam profile

Fig. 7 HEC-RAS geometric data on Arc map

Dam breach flood routing

Routing of the breach outflow hydrograph downstream to evaluate the potential consequence of dam failure is the major and main task in modeling dam breach and downstream risk analysis USACE (2010). HEC-RAS can be used to route an inflow flood hydrograph through a reservoir either with one dimensional unsteady flow routing or two- dimensional unsteady flow routing. Full unsteady flow routing (one or two-dimension) will be more accurate for both with and without breach scenarios. The unsteady flow routing method can capture the water surface slope through the pool as the inflowing hydrograph arrives, as well as the change in water surface slope that occurs during a breach of the dam. The governing equations for dam breach analysis are mass conservation (continuity) equation and diffusion wave form of momentum equation (Sisay 2016). For this study, two-dimensional full unsteady flow routing (diffusion wave equation) was used for routing the flood through the reservoir and the downstream which is embedded in HEC-RAS.

The unsteady mass conservation equation (continuity equation) is expressed as:

$$\frac{\partial H}{\partial t} + \frac{\partial(hu)}{\partial x} + \frac{\partial(hv)}{\partial y} + q = 0 \tag{19}$$

where t is the time, u and v are the velocity components in the x- and y-directions, respectively, q is a source or flux term, H is the water surface elevation and h is the water depth (HEC-RAS version 5.0.3 hydraulic reference, 2016). In vector form, the continuity equation takes the form

$$\frac{\partial H}{\partial t} + \nabla \cdot hV + q = 0$$

where $V = (u, v)$ is the velocity vector, the deferential operator del (∇) is the vector and the partial derivatives operator is given by $\nabla = (\partial/\partial x, \partial/\partial y)$.

The diffusion wave form of momentum equation for 2D unsteady flow condition is also expressed as:

$$\Delta V = \frac{-(R(H))^{2/3}}{n} \frac{\nabla H}{|\nabla H|^{1/2}} \tag{20}$$

where ΔV is the velocity vector; R is the hydraulic radius; ∇H is the surface elevation gradient; and n is the manning's n (HEC-RAS version 5.0.3 hydraulic reference, 2016).

Diffusion wave equation when the velocity is determined by the balance between bar tropic pressure gradient and bottom friction, the diffusion wave form of momentum equation (20) can be used in place of the full momentum equation and the corresponding system of equation can be in fact be simplified to one equation model. Direct substitution of diffusion wave equation (20) in continuity

equation (19) yields the classic differential form of diffusion wave approximation equation which is sated as:

$$\frac{\partial H}{\partial T} - \nabla.\beta\nabla H + q = 0 \tag{21}$$

where $\beta = \frac{(R(H)^{5/3}}{n|\nabla H|^{1/2}}$

In reservoir routing, the outflow is a maximum when the storage is a maximum. The outflow is dependent on the height of water (h) above the crest of the spillway. Since this study focuses on overtopping mode of failure, the HEC-RAS model requires weir profile including spillway, weir length (m), weir coefficient (C), weir shape and initial water surface elevation (1219.5 m a.m.s.l) as input to compute the out flow. The outflow can be computed as:

$$Q = CLh^{3/2} \tag{22}$$

where Q = discharge (m³/s), L = weir length (m), L = 335.52 m, h = height of water above the weir crest in (m) computed by the HEC-RAS model, C = coefficient which is (2.6–3.3) for clay core embankment dams as suggested by Brunner (2014), for this study C = 2.95.

Manning's roughness coefficient

Roughness coefficients represent the resistance to flow and flood pains. Manning's n coefficient is used in the manning equation to calculate discharge in the open channel and 2D flow area Chow et al. (1988). One of the capabilities of the HEC-RAS 5.0.3 new version is that it is possible to create spatially varying Manning's n value by overlying Google satellite image and field survey data to define the land cover for the 2D flow area; finally, it is possible to assign Manning's n value for each land cover. The assigned value of the Manning's n was based on Chow (1959) for different land cover. For this study, the land cover polygons for 2D flow areas are created based on field collected data and Google satellite image (Fig. 8).

Unsteady flow analysis

Once all of the geometric data are entered into HEC-RAS, the required unsteady flow data must be entered to perform the unsteady flood simulation. The data required for unsteady flow analysis include upstream boundary condition, downstream boundary condition and initial condition. For this study, inflow hydrograph into the reservoir
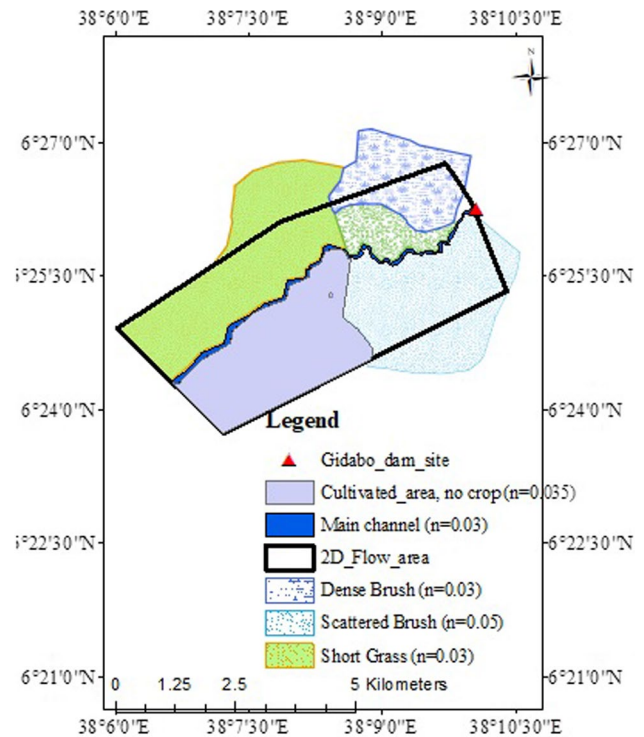

Fig. 8 Manning's roughness value for study area downstream of the Gidabo dam

was used as upstream boundary condition, whereas normal water level was used as initial condition.

Peak breach outflow

For this study, the peak breach outflow was computed using HEC-RAS program for Froehlich (1995), Froehlich (2008) and Macdonald and Langridge–Monopolis (1984) methods. Once a breach hydrograph is computed in HEC-RAS, the computed peak flow from the model can be compared with envelope curves of historically failed dams to select the appropriate method. To do this, hydraulic depth should be calculated in feet using the breach parameter information by the formula:
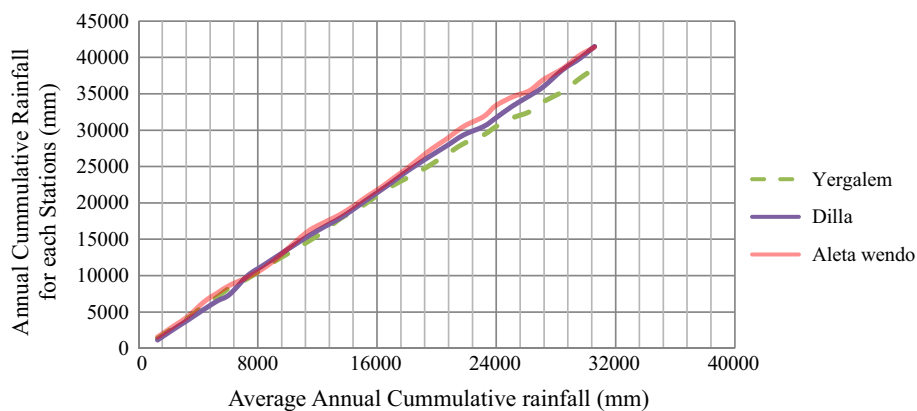
$$\text{Hydraulic (D)} = \frac{\text{Wetted area(A)}}{\text{Top width (T)}} \tag{23}$$

According HEC-RAS dam breach study (USACE 2014), training document peak envelop curve developed from historically failed dams has power trend with equation:

$$Q_B = 75D^{1.85} \tag{24}$$

where Q_B is the peak outflow in feet cubic per second and D is the hydraulic depth in feet.

Fig. 9 Consistency test of rainfall data

Flood inundation mapping

For this study, the flood inundation map was prepared by exporting the depth as tiff file into ARC-GIS. The flood inundation map contains all the necessary elements like Map collar information, base map, and inundation polygon and inundation elevation.

Results and discussion

Data consistency

After filling missed rainfall data, consistency of recorded rainfall data was checked. To test consistency of rainfall data, double mass curve technique was used and applied for Dilla, Aleta Wendo and Yiragalem stations. As observed from Fig. 9, there is no clear shift or change in slope for all stations; hence, the rainfall data at all stations were found consistent.

Probable maximum precipitation (PMP)

As mentioned in the methodology part statistical technique, Hershfield (1965) formula was used to estimate the probable maximum precipitation (PMP) of each rainfall stations from annual maximum daily rainfall of the rainfall stations within the study area. Figure 10 shows Theissen's polygon with rainfall station, and Table 2 presents PMP for each rainfall stations with area enclosed by their polygon.

Finally, the average depth of probable maximum precipitation calculated was 277 mm by applying 0.59 area reduction factor. This areal PMP was used for the computation of inflow hydrograph.

Inflow hydrograph

From the study, the computed composite inflow hydrograph resulting from triangular hydrographs at dam site by using
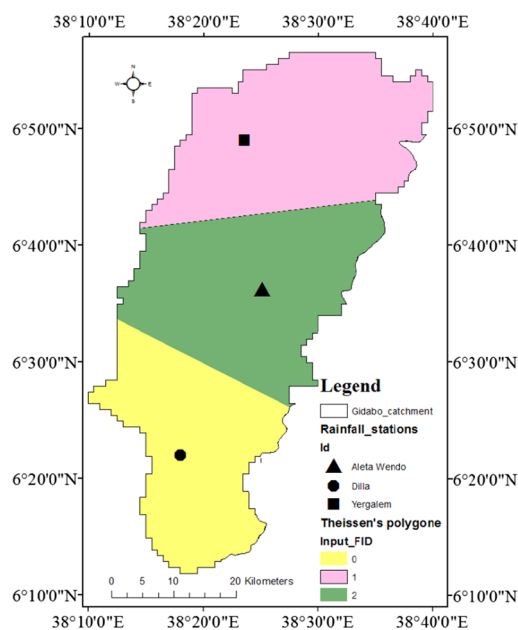

Fig. 10 Theissen's polygon with rainfall Stations

Table 2 PMP of rainfall stations

Name of the station	PMP (mm)	Polygon area (km²)
Dilla	438.114	872
Yirgalem	510.132	815
Aleta Wendo	441.529	856

standard dimensionless SCS unit hydrograph method has a peak flow of 6387 m³/s as shown in Fig. 8. As obtained from final feasibility report, Gidabo dam spillway was designed to pass 10,000 years' flood with peak flow of 1797.38 m³/s. Therefore, the peak of the computed hydrograph has lager magnitude than the 10,000 years' flood; as a result, there is probability of occurrence of overtopping failure.
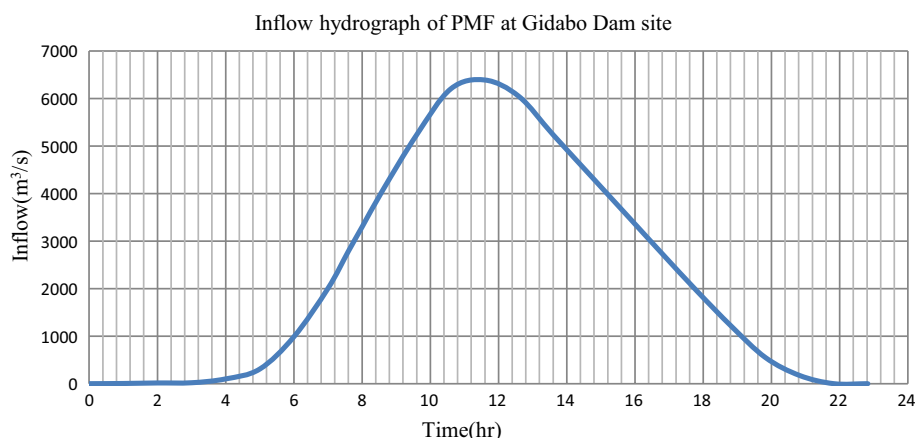
Fig. 11 Inflow hydrograph at dam site

Inflow hydrograph of PMF at Gidabo Dam site

Table 3 Breach parameters for overtopping failure mode

Method	Breach bottom width (m)	Breach side slopes (H:V)	Breach top width (m)	Breach formation time (h)
Froehlich (1995)	143	1.4	179.84	2.57
Froehlich (2008)	127	1	178.6	2.35
MacDonald and Langridge–Monopolis (1984)	322	0.5	425.2	2.14

Furthermore, the computed inflow hydrograph will be used as upstream boundary condition for the 2D unsteady flow simulations in HEC-RAS (Fig. 11).

Dam breach

Breach parameters

Estimating the dam breach parameters is one of the most important things that have to be performed before 2D unsteady simulation analysis with HEC-RAS for overtopping failure was done. For this study, the dam breach parameters were estimated by using Froehlich (1995, 2008) and Macdonald and Langridge–Monopolis methods on HEC-RAS breach parameter calculator tab. As mentioned from the methodology part, the breach shape was assumed to be trapezoidal. Table 3 presents summary of the results obtained from the HEC-RAS breach parameter calculator tab for overtopping failure mode.

From the above table, HEC-RAS breach parameter calculator result breach top width calculated by MacDonald and Langridge–Monopolis (1984) method was found 425.2 m. Since the crest length of Gidabo dam is 335 m, MacDonald and Langridge–Monopolis (1984) method can be ignored for further two-dimensional unsteady flow simulations. Therefore, one of the two remaining methods will be used for unsteady flow simulation by comparing the methods with the peak envelop curve. It is also observed that the breach size is related to the breach formation time. The larger the
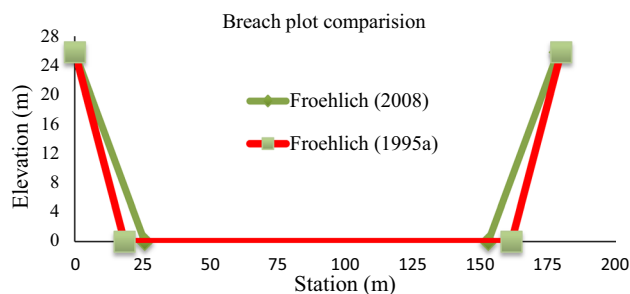

Breach plot comparision

Fig. 12 Breach plot Comparison

breach sizes, the large the breach formation time. For compression purpose, the breach parameter can be plotted on the same chart. From the figure, it was clearly observed that the breach size for Froehlich (1995) method is larger than Froehlich (2008) method. The breach plots for the two methods plotted on spreadsheet scatter plot are shown in Fig. 12.

Breach outflow hydrograph and envelop curve

Unsteady flow simulation of overtopping failure in HEC-RAS requires PMF inflow hydrograph as an upstream boundary condition. Overtopping failure occurs when the flood due to the PMF inflow passes over the embankment. After performing 2D unsteady flow simulation, the peak breach outflow from Gidabo dam on HEC-RAS was found 15,848.85 m³/s and 15,194.9 m³/s for Froehlich (1995) and
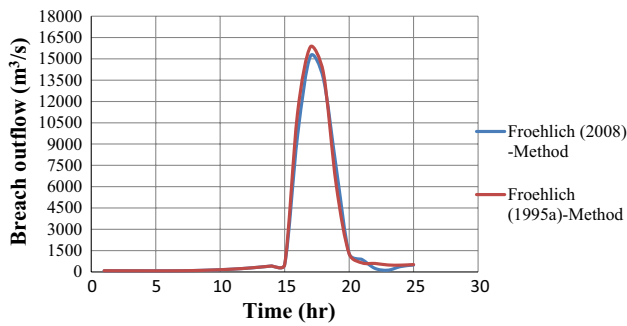
Fig. 13 Comparison of breach outflow hydrographs for overtopping failure

Table 4 Breach peak outflow and hydraulic depth

Method	Hydraulic depth (m)	Hydraulic depth (ft.)	Q_B (m^3/s)	Q_B (ft^3/s)
Froehlich (1995)	18.29	60	4173.29	147,368.10
Froehlich (2008)	17.37	56.98	3819.25	112,618.47

Froehlich (2008) methods, respectively. Like breach plot compassion, it is also important to compare the outflow for the two methods by plotting on the same chart. Figure 13 shows the breach outflow hydrograph for Froehlich (1995) and Froehlich (2008). As observed from the hydrograph, higher magnitude of peak flow was obtained on Froehlich (1995) method.

Once a breach hydrograph is computed in HEC-RAS, the model peak outflow of the two methods was compared to envelop curve of historical failure to select the appropriate method. To do this, the hydraulic depth (D) which is the ratio of wetted area to the top width was calculated in feet. After the computation of the hydraulic depth then the peak outflow (Q_B) was computed as a function of hydraulic depth by using peak envelop equation. The resulting hydraulic depth and outflow was found (18.29 m, 4173.29 m^3/s) and (17.37 m, 3819.25 m^3/s) for Froehlich (1995) and Froehlich (2008) methods, respectively. These values were converted into feet and cubic feet per second as summarized in Table 4.

In order to select the appropriate method from the two methods, the hydraulic depth in (feet) and peak flow in (ft^3/s)
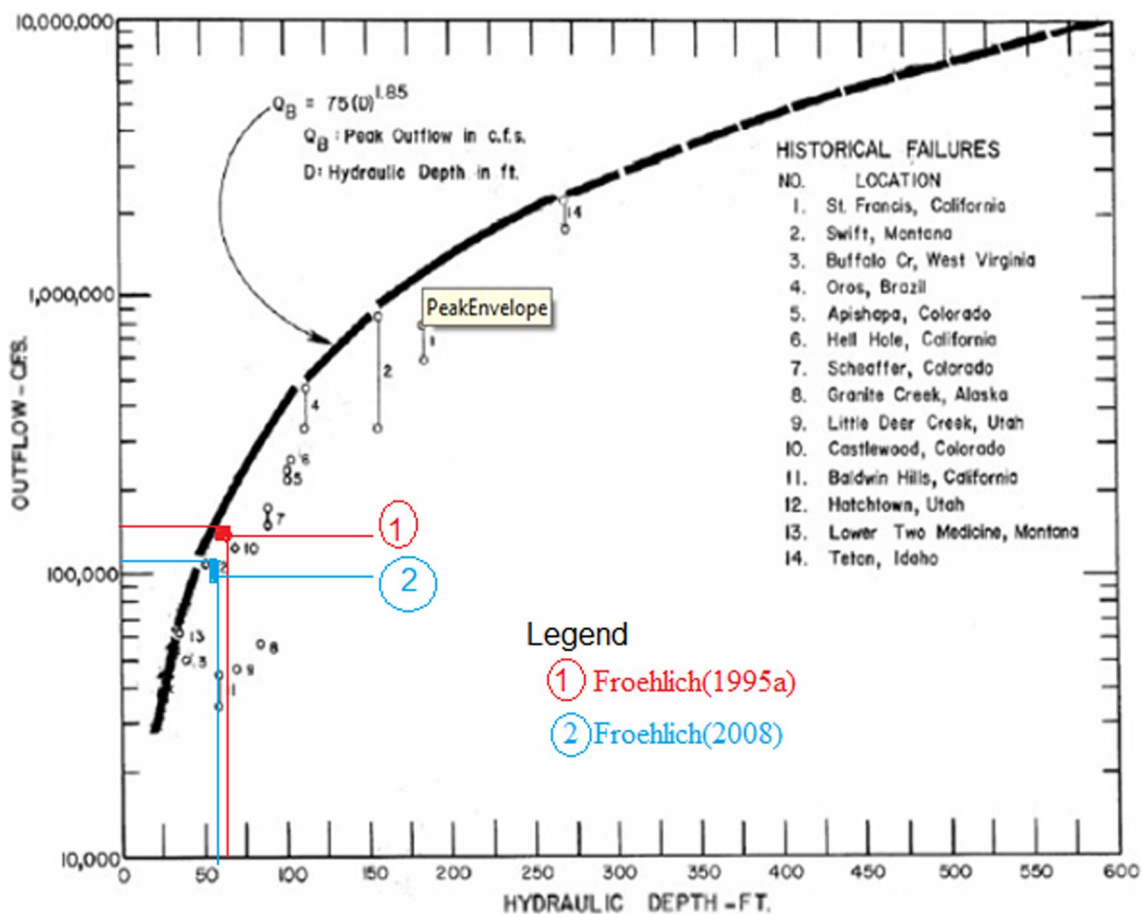

Fig. 14 Envelope curve from historically failed dams

were plotted on develop curve. As a result, Froehlich (1995) method was found appropriate method (see Fig. 14) method, for 2D unsteady flow simulation. Similar studies were done by Nugusa (2016), for Fincha dam and Leoul (2015), for Kesem Kebena dam found in Ethiopia. Both studies analyze dam breach using HEC-RAS model and compare the result with envelop curve. Another study was done by Sisay (2016), for Melka Wakana dam found in Ethiopia. The study uses HEC-RAS to analyze the dam beach and found that Froehlich (1995) is the suitable method for unsteady flow simulation.

Flood inundation map

Flood inundation boundary map

The flood inundation map boundary provides a description of the areal extent of flooding which would be produced by the dam breach. For this study, flood inundation boundary map of the flood-prone area was prepared by exporting inundation boundary result from the RAS Mapper to ARC-GIS as shape file. The total areal coverage of the flood was found 2050 hectares as read from the attribute table of ARC-GIS for inundation polygon. Figure 15 presents flood inundation boundary map of the area. From the map, it was obtained that the flood inundation boundary was within Dibichala Luniche kebele (*local name*) found in Oromia regional state.
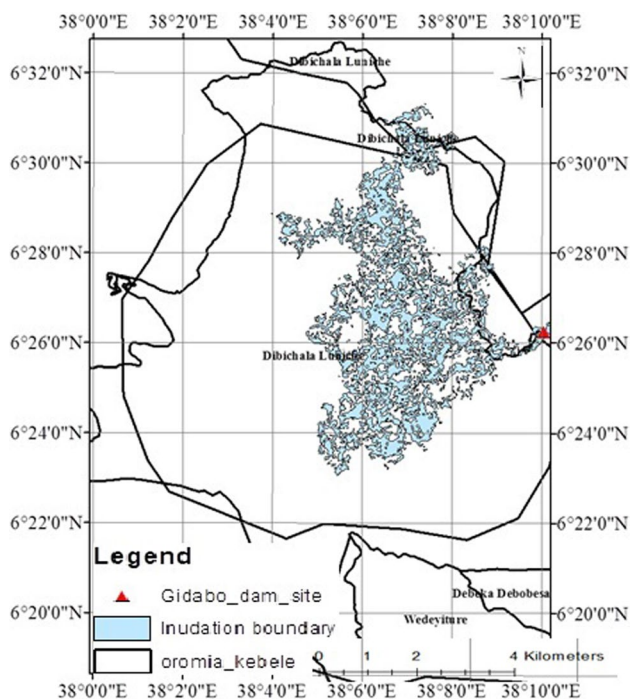

Fig. 15 Flood inundation map

Flood inundation depth

In flood inundation mapping, accurate prediction of inundation levels at the places where there is infrastructure and population at risk is important task. For this study, inundation depth map was prepared by exporting inundation depth Geo-tiff file (raster) from RAS Mapper to ARC-GIS and overlie it to shape file. Figure 16 shows flood inundation depth variation at different part of the flood plain area. The minimum and the maximum inundation depth downstream of the dam was found 0.0019 m and 12.14 m, respectively. From the figure, it was observed that most of the flood plain areas were covered by a flood depth of water within the range of 0.001 m to 6 m. This flood depth can cause severe damage to the peoples settle downstream of the dam and the irrigation command area.

Conclusion

Generally, the aim of this study was to analyze the breach of Gidabo embankment dam and mapping of the breach outflow flood downstream of the dam. For analysis, different types of data have been gathered with the current recommended literature and models. From the hydrological data analysis, the probable maximum precipitation (PMP) was found 277 mm. The inflow hydrograph resulting from the PMP was computed using standard dimensionless SCS method. The peak flow for the computed inflow hydrograph was 6387 m^3/s. The river hydraulics model HEC-RAS 5.0.3 two-dimensional modeling has been used to compute dam breach parameters, breach out flow and flood wave propagation resulting from dam failure. The Dam breach has been simulated for overtopping failure, with a breach bottom width 143 m and breach formation time of 2.57 h. The simulated results on HEC-RAS reached peak breach outflow of 15,848.85 m^3/s. Finally, the flood inundation map was prepared by exporting water depth and water surface elevation tiff raster file from RAS Mapper to ARC-GIS and overlies it to shapes file to visualize coverage of the flood affected area due to the breach. The areal extent of the flood due to the breach downstream of the dam was found 2050 hectares with maximum flood depth of 12.14 m.

Recommendation

Gidabo dam irrigation project is under completion stage. In this study, 2D unsteady flow routing technique was used to carry out dam breach analysis. From the study, it was found that the dam will overtop due to probable maximum flood; as a result, the dam may be subjected to overtopping failure. In order to prevent the dam from failure, every stake holder should participate. Some of the measures to
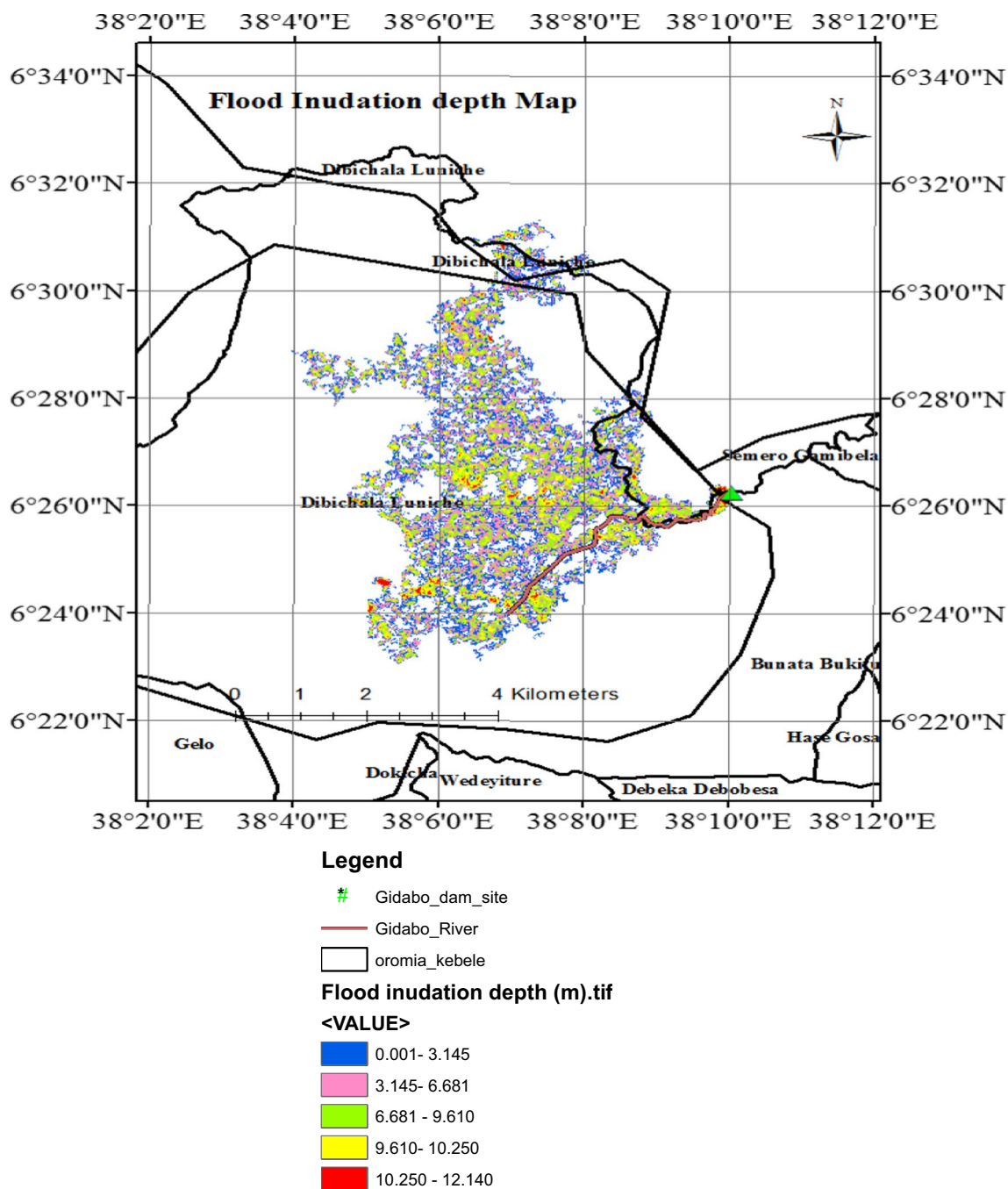
Fig. 16 Flood inundation depth map

prevent the dam from failure may be facilitate a forestation of the upstream catchment to reduce runoff and sediment, provide emergency spillway by conducting detail hydrological and geological investigation, increase freeboard, provide erosion protection on downstream slope by placing riprap or other appropriate materials. In addition to these, the flood-affected areas should be free of permanent development structures, and it better to resettle the peoples leaving living downstream of the dam by finding another settlement area.

In this study, digital elevation model 30 m * 30 m resolution was used for unsteady flow simulation. This is because of the availability DEM at the time of study. The researcher would like to recommend other researchers to use high resolution DEM to conduct similar studies.

Finally, the researchers would like to recommend that researchers should give special attention to the Dam breach

analysis and make a detail investigation by using the latest dam breach software's like FLO-2D and HAZUS-MH. Flo-2D, which is more stable model than other while running 2D unsteady flow state simulation and may requires less adjustments after initial setup and simulation. On the other hand HAZUS-MH (natural hazard analysis tool -multi hazard), which provides additional means to estimate potential economic damage due to the breach event. The software uses GIS technology to estimate physical, economic and social impact flood disaster.

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