DAM BREAK ANALYSIS USING HEC-RAS

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ABSTRACT

The consequence of dam failure could be catastrophic depending on the magnitude of the failure and downstream properties. The overall objective was to estimate the dam break outflow hydrograph, route the dam break hydrograph through the downstream valley and floodplain and then estimate the inundation levels. It is therefore required to carry out dam break analysis that will help in devising appropriate management strategies and mitigate the impact effect of such disaster. The dam break analysis of the University of Ilorin dam was studied with HEC-RAS model in conjunction with Geographic Information System (GIS) and HEC-GeoRAS. HEC-GeoRAS was used to extract geometric information from a digital terrain model and then imported into HEC-RAS. The Steady and unsteady flow simulation of the dam break event was performed using HEC-RAS and results were mapped using GIS. The result showed that area immediately after the bridge (about 456.66-607.40 m downstream from the inline structure) and the tail end of the river reach (about 1.3 to1.6 km downstream from the inline structure) will experience greater impact of the flood for the maximum water surface profile from flood inundation. The largest flow area covered between the dam and the bridge is 1,873.8 m^2 at river station 1464 (RS 1464) having a top width of 332.88 m while the largest flow area covered after the bridge is 2,597.2 m^2 at river station 1023 (RS 1023) with a top width of 468.38 m. However, the study reveals that University reception ground, the bridge, the faculty of Agriculture farm practical plot and the area of land used for plant nursery would be affected by the flooding at the worst flow condition (i.e. Maximum water surface profile). However, the University reception ground, the bridge, the faculty of Agriculture farm practical plot, and the area of land used for plant nursery would be affected by the flooding at the worst flow condition. This result provides an insight into emergency preparedness and disaster management plan the University of Ilorin can put in place.

KEYWORDS: Dam break, HEC-RAC, hydrograph, hydrology, runoff, watersheds

1. INTRODUCTION

Dams are hydraulic structures of impervious material constructed across a river to create a reservoir on its upstream side for impounding water for various utilization. Dams can be classified in some ways but most usually are based on function, structure, and design. The classification is based on the construction type and material gravity, arch, embankment, and buttress (Ray *et al.,* 1992) while those classified based on function are storage, detention, debris, and coffer dams. Floods resulting from the failure of constructed dams have produced some of the most devastating disasters in the last two centuries (Adewale *et al.*, 2010; Adegbola and Jolayemi, 2012; Balogun and Ganiyu, 2017). When dams fail, property damage is certain, but the loss of life can vary dramatically with the extent of the inundation area, the size of the population at risk, and the amount of warning time available (Wahl, 1998). The inundation area and the warning time depend mainly on the dam break scenario and dam breach parameters like the breach size, shape, and timing. As the breach opening gets larger and the time to total failure gets shorter, the peak outflow gets larger. Hydraulics, hydrodynamics, hydrology, sediment transport mechanics, and geotechnical aspects are all involved in breach formation and dam failure (USACE, 1998).

Dam failure results from both external force and internal erosion (Cederwall, 2005). Dam failure warrants dam break modeling which assesses the flood hydrograph of discharge from the dam breach due to the propagation of flood waves along with their time of occurrence. A dam break may result in a flood wave up to tens of meters deep traveling along a valley at quite high speeds. The impact of such a wave on developed areas can be sufficient to destroy infrastructure such as roads, railways, bridges, and buildings. Dam failure can lead to the inevitable loss of life if warning and evacuation are not possible. Additional features of such extreme flooding include the movement of a large number of sediments and debris along with the risk of distributing pollutants from any sources such as chemical works or mines in the flood risk area. Though there has been great advancement in design methodologies, failure of dams and water retaining structures can still occur. International Commission of Large Dams (ICOLD) defined dam failure as the "collapse or movement of part of the dam or its foundations so that the dam cannot retain water". The term dam breaks analysis usually relates to the process of studying a dam failure phenomenon and analyzing the resulting consequences in the downstream region. This generally deals with the simulation of assumed failure for existing dams and analyzing the resulting consequences (Purvang and Thakor, 2013). In the most simplistic approach, the outflow hydrograph must be predicted, route the flood through the downstream valley to determine inundation levels, flow conditions, and flood arrival times, and then use that information to estimate the consequences (Tony, 1998). A breach is defined as the opening formed in the dam body that leads the dam to fail and this phenomenon causes concentrated water behind the dam to propagate toward downstream regions (Xiong, 2011).

Mac Donald and Langridge-Monopolis(1984) related the breach time to failure to the volume of eroded material. The volume of eroded material was related to the breach formation factor, defined as the product of the outflow volume times the initial depth of water above the breach bottom. Froehlich (1987) developed non-dimensional prediction equations for estimating average breach width, average side-slope factor, and breach formation time. Froehlich's predictions were based on the characteristics of the dam, including reservoir volume, depth of water above the breach bottom, breach depth, length of an embankment at the dam crest, breach bottom width, and empirical coefficient that account for overtopping against non-overtopping failures. 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. Reservoirs with long narrow pools will exhibit greater water surface upstream of the dam than reservoirs that are wide and short. Therefore, the most accurate modeling technique to capture pool elevations and outflows of long narrow reservoirs is full dynamic wave (unsteady flow) routing (Gary, 2014).

Mo *et al*. (2023) selected Chengbi River Reservoir as the research object, HEC-RAS as the simulation software, unsteady flow differential equations, and one-dimensional Saint-Venant equations as the control equations, and it used the four-point implicit finite difference method for discrete solution. The dam-break flood evolution was simulated under three boundary conditions thus: full breach, 1/2 breach, and 1/3 breach. From the dam site section to the Tianzhou hydrological station section, the peak discharge decay rates of the three schemes are 78%, 77%, and 67%, respectively. The water level decay rates of the three schemes are 47%, 36%, and 30%, respectively. A 1 m increase in the bursting water level elevation increases the peak flow by approximately 7%, and the highest water level in front of the dam by 1 m and delays the peak time by 1.5 hr on average. In addition, the preliminary inundation extent for Baise City is obtained. The authors recommend the analysis results can provide a fundamental basis for good control as well as a reference for flood disaster management.

Balogun and Ganiyu (2017) considered a hypothetical dam break on Asa Dam located in Ilorin, Nigeria was analyzed using the United States Army Corps of Engineers (USACE), Hydrologic Engineering Center's River Analysis System (HEC-RAS) computer model. The unsteady flow simulation was performed using geometric data obtained from Digital Terrain Model (DTM) with a 100-year, 24-hour flow event. The HEC-RAS was used in concert with HEC-GeoRAS to assess the flood hazard along the Asa River channel starting from the dam axis and approximately 12 km downstream because of the dam break. The highest discharge Q (1913.66 m^3/s) and the highest peak stage (277.35 m) just below the dam were produced with a breach width of 130.86 m and a time of failure of 1.45 hours. The outcome of the analysis showed that in the event of such failure of the Asa dam, some areas which include industrial and residential sections along the river channel are at very high risk of being inundated due to the significant difference in the value of the produced water surface elevation and existing ground elevation affecting thousands of people living along the channel's immediate vicinity

Manmohan *et al*. (2020) carried out a hypothetical dam break of the Meja dam using the HEC-RAS model with river geometry derived from DEM. HEC- RAS was applied for combined flood routing and flood level forecasting. The details of water surface elevations, depth of flood, flood arrival time, and velocity of flood wave at different locations downstream give an idea of the extent of flooding. The outcome of the modeling showed that in the event of failure, some parts including residential, agricultural, and industrial areas were identified to have a very high risk of being inundated due to the significant difference in the value of water surface elevation and ground elevation. The simulation results are essential for characterizing and reducing negative effects that occurred in the downstream area. The development of an emergency action plan requires an exact estimation of the inundation level and the arrival time of flood waves at the downstream point, and this makes the result significant for policy maker.

Seepage, piping, overtopping, earthquake, landslide, and foundation failure or sabotage are major sources of dam failures (Wu, 2011; Brunner, 2014; Yasin *et al.,* 2023) while the main causes of dam failure are overtopping and piping (Amini *et al*., 2017; Li *et al*., 2021; Yasin *et al.,* 2023). It can be deduced from the above discourse that accurate prediction of breach parameters is necessary to make reliable estimates of outflow and resulting downstream inundation near the dam. Some recent rainfall events that left the dam almost toppled are a serious concern because the implication of a dam break would be colossal to the university considering the infrastructure downstream, staff, and students. Therefore, there is a need to carry out a dam break analysis that will help in devising appropriate management strategies. The aim of this study was to analyze the dam break event and disaster management plan for the University of Ilorin dam while the study scope is limited to the university.

2. MATERIALS AND METHODS

2.1 Introduction

This study involves field and desk work. The fieldwork involves carrying out reconnaissance surveys to have first-hand information about the site condition, identification of physical features, and susceptible failures. The downstream valley cross-section data were collected on the field using the total station and Global Positioning System (GPS) with the help of a skilled surveyor. The hydrological features of the dam to be determined are the catchment area, and the location of the catchment both the longitude and the latitude. Desk work involves the simulation of dam break events using hydraulic models; HEC-RAS.

2.2. Study area

The University of Ilorin dam was commissioned on the 25th of May 2007 primarily for water supply to the University community. The dam is located on Latitude $8^0 28' 0.76''$N and Longitude $4^0 39' 57.02''$E on the Oyun River. River Oyun originates at an elevation of 490 m amsl, close to Ila-

Orangun, and flows in an approximate north-west direction for about 80km before joining river Asa (Okeola, 2009). The impounded water serves as the main source of water for the University campus and a major source for irrigating the sugarcane fields. It has a capacity of 1.8 million m^3 and an embankment crest length of 178m with a maximum embankment height of 10.3m. The upstream has a slope of 1:3 while the downstream has a slope of 1: $2\frac{1}{2}$. The University of Ilorin dam is a zoned earth-fill embankment with an Ogee-shaped concrete spillway. There is ongoing physical infrastructure development which is putting more pressure on the water supply facility.

2.3. Methodological Tools

Three key tools adopted in this study are Google Earth Software, Geographic Information System (GIS), and Hydrologic Engineering Center's River Analysis System (HEC-RAS), and are briefly discussed:

2.3.1 Google earth software

Google Earth is a geo-browser that accesses satellite and aerial imagery, ocean bathymetry, and other geographic data over the internet to represent the earth as a three-dimensional globe. It provides search capabilities and the ability to pan, zoom, rotate, and tilt the view of the earth. This software was used to take the satellite imagery of the study area and locate points of about 100m intervals to 1.6 km on the downstream river valley for easy location during the actual field survey.

2.3.2. Geographic information system (GIS)

The Geographic information system (GIS) is an integrating tool that is employed by various fields utilizing spatial analysis techniques for the capture, storage, retrieval at will, manipulating, analyzing, modeling, and displaying of geographically referenced data in solving a complex human-related problem. The GIS can show different kinds of data on a map which enables users to easily view, analyze and understand patterns and relationships. The real strength of GIS is the use of spatial and statistical methods to analyze attributes and geographic information. The result of the analysis can be derivative information, interpolated information, or prioritized information. The GIS was used in post-processing applications in this study. The output data from flood routing models are formatted into GIS-compatible data and projected on maps or aerial photographs which are of high importance in the emergency action plan.

2.3.3. Hydrologic engineering center's river analysis system (HEC-RAS)

HEC-RAS is a one-dimensional steady-flow hydraulic model designed to aid hydraulic engineers in channel flow analysis and floodplain determination. The results of the model can be applied in floodplain management and flood insurance studies. HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking and multi-user network environment. The system is comprised of a geographical user interface (GUI), separate hydraulic analysis components, data storage, and management capabilities, graphics, and reporting facilities. The HEC-RAS system contains four one-dimensional river analysis components for (1) steady flow water surface profile computations; (2) unsteady flow simulation; (3) movable boundary sediment transport computations; (4) water quality analysis. A key element is that all four components use a common geometric data representation and common geometric and hydraulic computation routines.

2.4 Dam Break Theoretical Concepts

2.4.1 Dam break scenario

Dam break analyses usually involve two classes of failure scenarios commonly referred to as sunny day failure and rainy day failure and the latter was adopted for the study. The rainy day failure examines a scenario when the reservoir is full and there is an extreme inflow into the reservoir, as well as in the adjoining downstream catchments. This assessment is a hypothetical one that focuses on the potential downstream consequences regardless of the failure type. The rainy day failure results would be used in an emergency action plan since this failure scenario indicates the largest extent of inundation likely from dam failure for warning and evacuation in a flood situation. HEC-RAS is a one-dimensional river hydraulics model commonly used in steady-flow and unsteady-flow water surface profile computations via a network of open channels (Leoul and Kasshun, 2019).

2.4.2 Modeling process

Mathematical modeling of dam break floods can be carried out either by one- or two-dimensional analyses. According to Lin *et al*. (2021), Chaudhuri *et al.* (2020) and Yu *et al.* (2020) dam breach simulations were challenging due to the accuracy of extreme flood estimates and time step definition. Therefore, varieties of multidimensional hydrodynamic models have been designed to simulate extreme events by evaluating flood timing and inundation areas. HECRAS, DAMBRK, FLO-2D, and MIKE are examples of such models. This study adopted a one-dimensional analysis that is generally accepted when the valley is long and narrow and when the flood wave characteristics over a large distance from the dam are of main interest as typical for the University of Ilorin reservoir. It gives information about the magnitude of the flood, variations of these with time and velocity of flow through breach can be had in the direction of flow. The modeling process approximates a physical phenomenon through which the physical phenomenon and its effects can be studied. Thus, dam break modeling has inherent approximations through assumptions. The foremost assumptions are in the hydrodynamics equations derived based on the following assumptions:

1. The water is incompressible and homogenous.
2. The bottom slope is small.
3. The wavelengths are large compared to the water depth. This ensures that the flow everywhere can be regarded as having a direction parallel to the bottom.
4. The flow is sub-critical i.e when the specific energy increases with an increase in potential energy.

The other assumptions are associated with the breach parameters, especially, breach width and breach depth, which have a great impact on flood peak and arrival times. The essence of dam break modeling is hydrodynamic modeling which involves finding the solution of two partial differential equations known as Barre De Saint Venant (Brunner and CEIWR-HE, 2010). The equations are:

a) Conservation of mass

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - \mathrm{q} = 0 \tag{1}$$

b) Conservation of momentum

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2 /A)}{\partial x} + \mathrm{gA}\left(\frac{\partial h}{\partial X} - S_0 + S_f\right) = 0 \tag{2}$$

where:

Q = discharge (m^3/s)
A = active flow rate (m/s)
h = water surface elevation (m)
q = lateral outflow (m^3)
x = distance along the waterway (m)
t = time (hr)
S_f = friction slope
S_0 = bed slope (S)
g = gravitational acceleration (Nm^2/Kg^{-2})

There are different approaches for dam break analysis. This includes comparative analysis, empirical analysis, physically based models, and parametric models. The empirical method and parametric approach were adopted. These approaches are advanced and efficient. However, the empirical method is used to predict time to failure, breach width, and peak breach discharges. The four most widely used and accepted empirically derived equations for predicting breach parameters are, (i) MacDonald and langridge-Monopolis, (ii) USBR, (iii) Von Thun and Gillette, and (iv) Froelich (Tony *et al.*, 2002). These methods have a reasonably good correlation when comparing predicted values to actual observed values. Reasonable values for the breach size and development time along with feasible breach geometry are needed to make a realistic estimate of the outflow hydrograph. Therefore, the estimation of the breach parameters yields a significant source of uncertainty in the results and turn downstream inundation extents. Froelich Equations 5, 10, and 13 are adopted to calculate the breach width, failure time, and peak discharge respectively.

2.4.2.1 Breach width

This refers, depending on each model, to the top, lower or average width of the breach. It would be generated using any of Equations 3 – 6 (Tony, 2002).

$$B_{avg}(\mathrm{m}) = 2.5h_W + C_b \tag{3}$$

where:

B_{avg} = average breach width (m)
h_W= the depth of water at the dam at the time of failure (m), and
C_b =Coefficient, which is a function of reservoir size

$$B_{avg}(m) = 0.47K_0\,(S)^{0.25} \tag{4}$$

where:

B_{avg} = the non-dimensional average width (m),
S = dimensionless storage and
K_0 = 1.4 for overtopping else 1

$$\mathrm{B}\ (\mathrm{m}) = 0.1803K_0\,V_W^{0.32}h_b^{0.19} \tag{5}$$

where:

V_W= reservoir water volume at the time of failure (m^3) and
h_b= distance from the breach to the bottom of the final breach (m)

$$\mathrm{B}\ (\mathrm{m}) = 3h_W \tag{6}$$

where:

h_W = height measured from the initial reservoir water level to the breach bottom elevation This is assumed to be the stream bed elevation at the toe of the dam in meters.

2.4.2.2 Failure time

The time of failure is the duration of time between the first breaching of the upstream face of the dam until the breach is fully formed and this is expressed in Equations 7- 12.

$t_f\,(hr) = \frac{B_{avg}}{4h_W}$ (7)
$t_f\,(hr) = \frac{B_{avg}}{(4h_W + 61)}$ (8)
$t_f\,(hr) = 0.79(S)^{0.47}$ (9)
$t_f\,(hr) = 0.000254\, V_W^{0.53}$ (10)
$t_f\,(hr) = 0.011B$ (11)
$t_f\,(hr) = 0.0179(V_{er})^{0.364}$ (12)

where:
B is in meters
t_f = breach formation time in hours
V_W= reservoir water volume at the time of failure (m^3)
S = dimensionless storage
h_W= the depth of water at the dam at the time of failure (m),
V_{er} = volume of eroded embankment material.

2.4.2.3. Peak discharge

The peak discharge is the maximum discharge expected because of the breach and this is expressed with Equations 13 and 14.

$Q_p = 0.607\, V_W^{0.295} h_w^{1.24}$ (13)
$Q_p = 1.175\, (V_w H_w)^{0.41}$ (14)

where:
Q_p = Peak outflow in cubic meters per second (m^3/s)
V_w = The total quantity of stored water at failure (m^3) and
H_w= the hydraulic height of water directly at the reservoir before the breach, measured from the bottom of the final breach (m).

3. RESULTS AND DISCUSSION

3.1. Estimation of Dam Breach Parameters

The breaching mechanism of a dam is described by dam breach parameters and represented by breach width (b), breach height (h), side slope (s), time of failure (t_f), and peak discharges (Q_p). Dam breach can be specified by trapezoidal, rectangular, or triangular shape. Linear breaching and erosion-based breaching are both relevant, but the latter contains a high degree of uncertainty. The trapezoidal formation with linear formation mechanism is adopted for dam-break modeling, based on the assumption that the dam's breach linearly with time. Where; H = height of the dam; BR = Average breach width and Z = Slope. Using the Froelich equations and considering Figure 1, The results for the Breach Width, Failure time, and Peak discharge are 26.25 m, 0.53 hr and 48,892.8 m^3/s respectively.

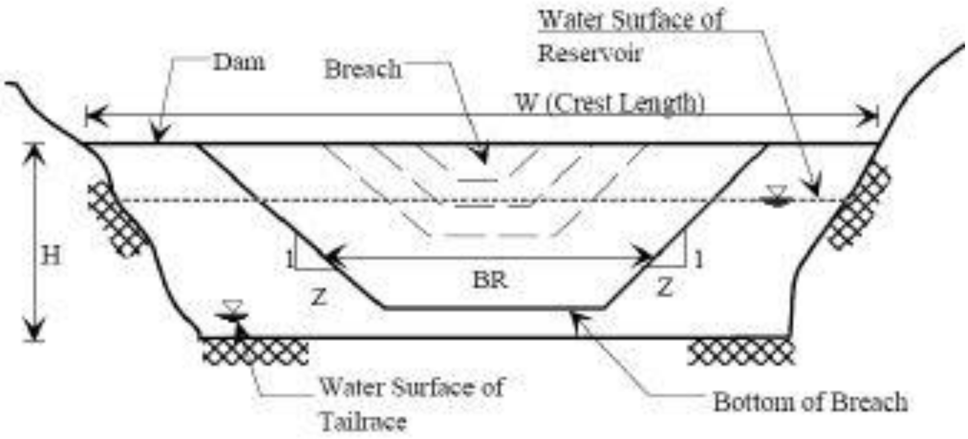


Figure 1: Definition sketch of breach parameters

3.2 Hydraulic Analysis

The satellite imagery of the reservoir and located points at 100m intervals to 1.6 km on the downstream river valley are shown in Figures 2 and 3 respectively using Google Earth. HEC-RAC was used to carry out the steady and unsteady flow simulation for the dam break events using streamflow data from a previous study by Sule *et al.* (2011). The geometric data were input (Figure 4) and the window displayed the University of Ilorin lake, inline structure, bridge, boundaries, and cross sections along the river reach. The steady flow simulation was carried out for a 40-year profile with a discharge of 19.34 m^3/s. Figure 5 shows the profile plot which is the elevation in meters against the main channel distance in meters along the considered river channel. Also shown are the Energy grade line, water surface, and the critical water surface at the end of the unsteady simulation window. Figure 6 shows the final rating curve after the unsteady flow simulation in which it is shown clearly that the water surface elevation increases with an increase in discharge. Figures 7 and 8 show the stage and flow hydrographs at the initial (River Station 1624) and final (River Station 60) stages of the simulation respectively.

Figure 2: Satellite imagery of the study area

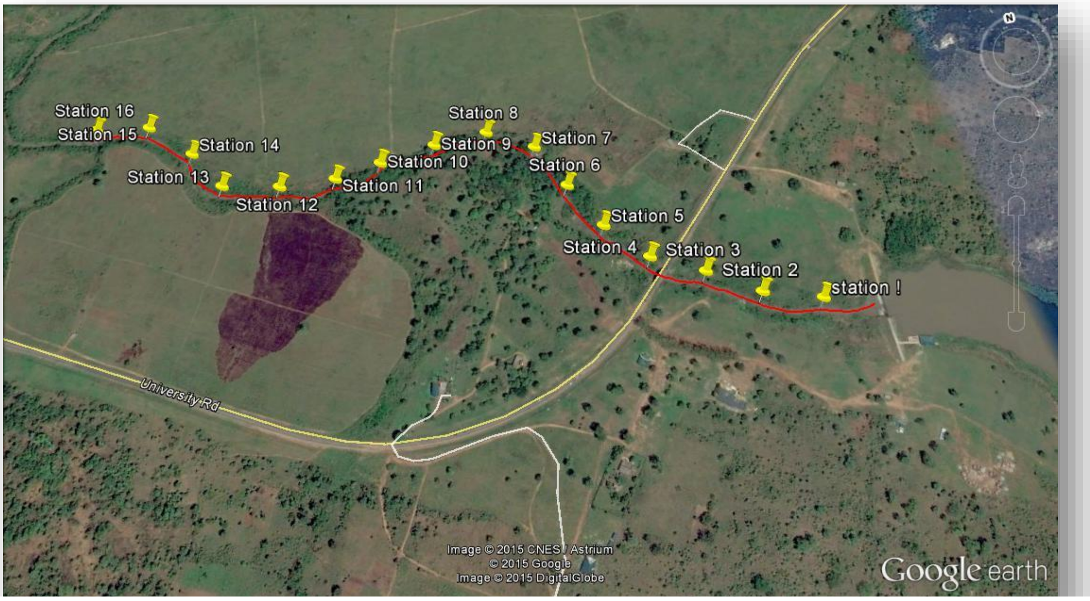


Figure 3: The study area showing stations downstream

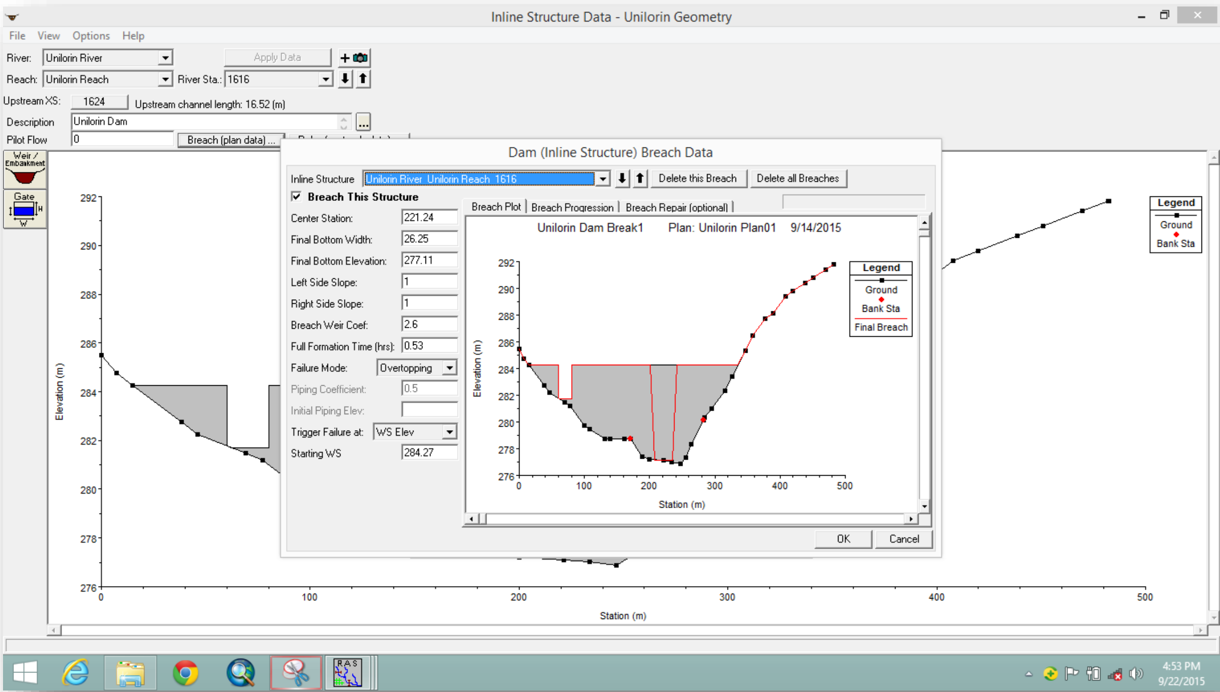


Figure 4: Dam breach data input window

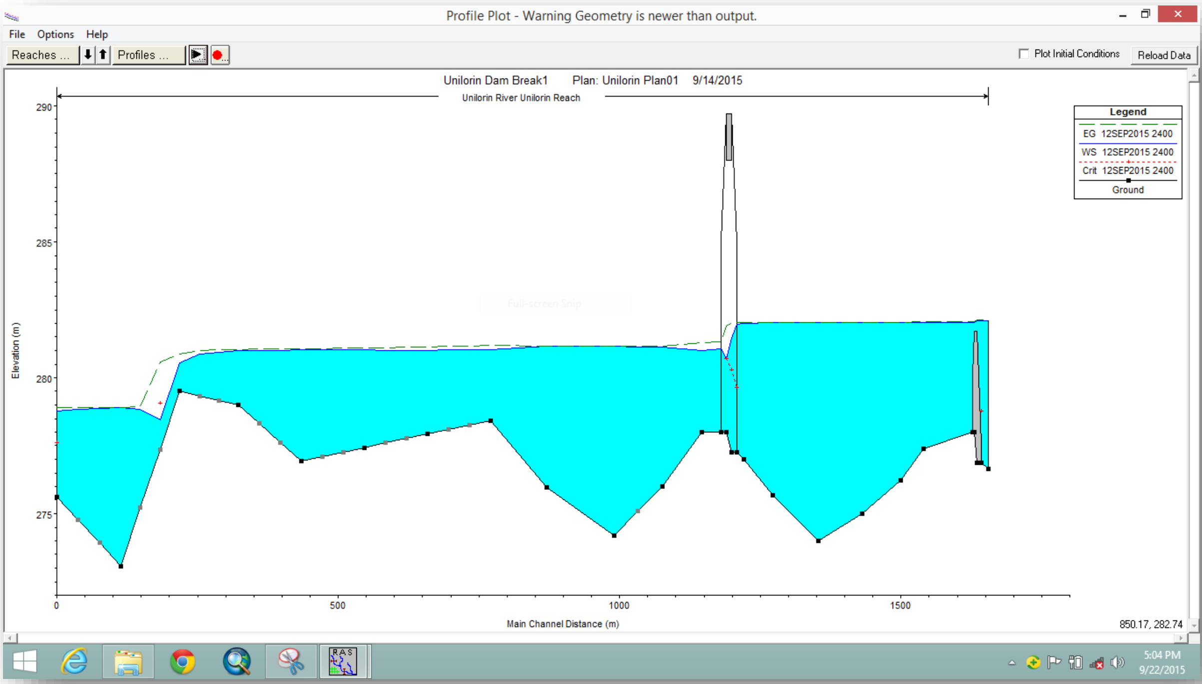


Figure 5: Profile plot

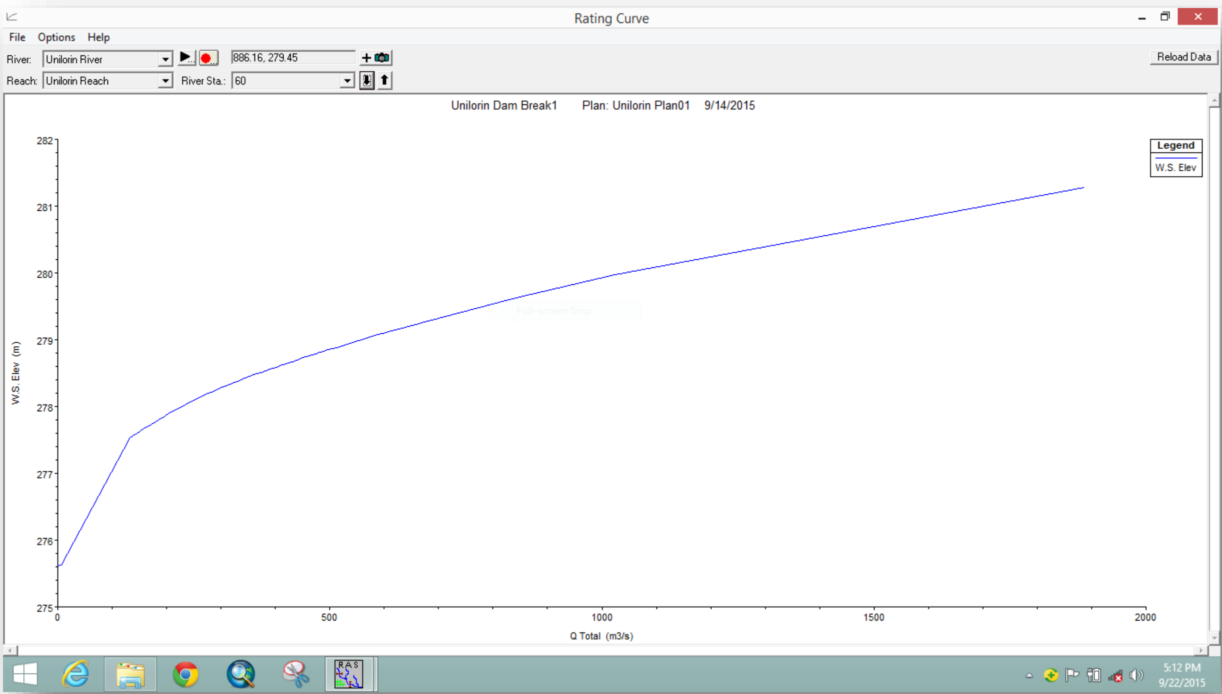


Figure 6: Rating curve

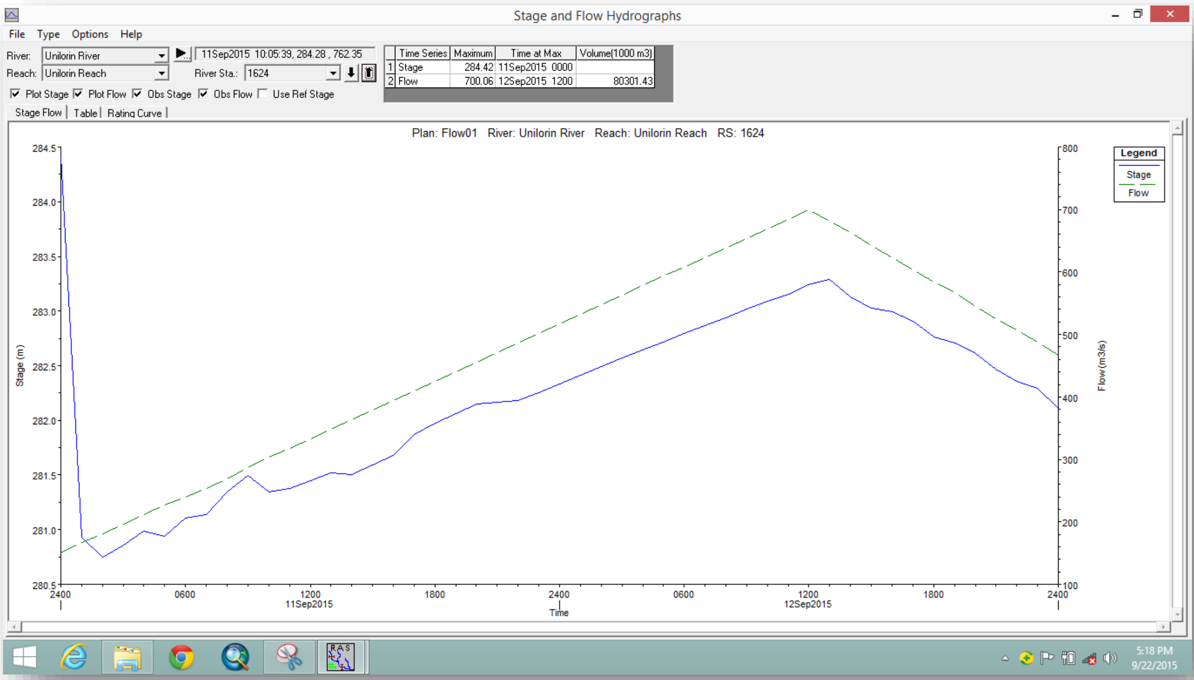


Figure 7: Stage and flow hydrograph for river station 1624

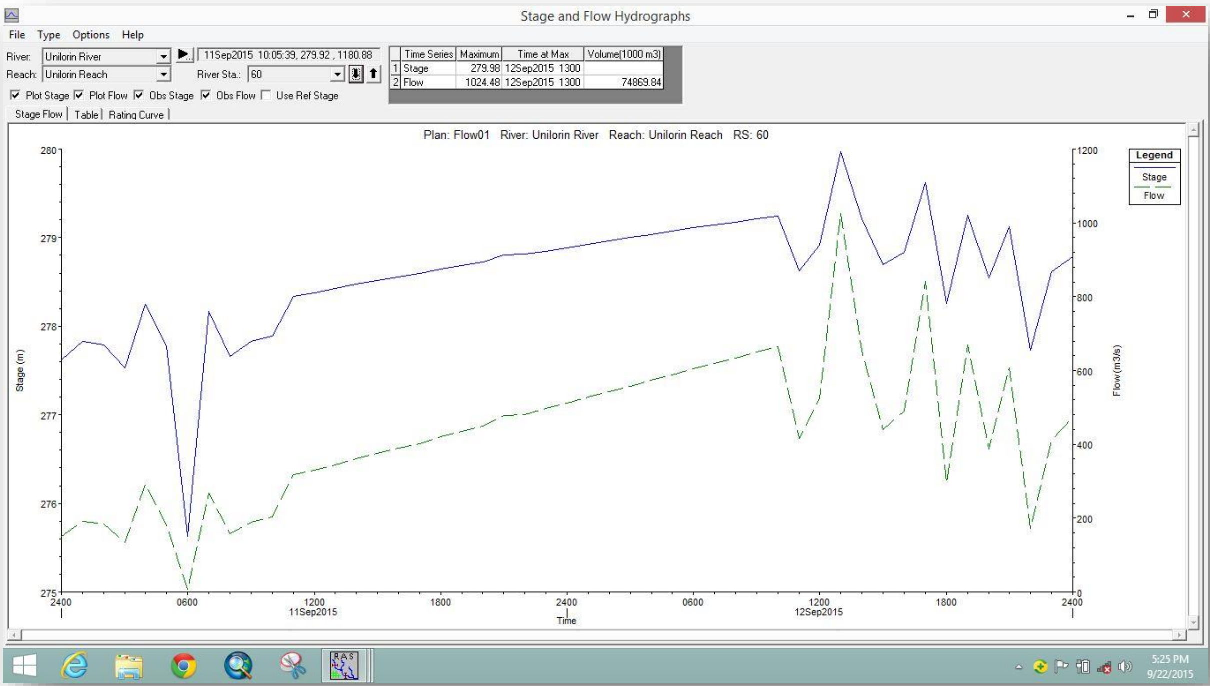


Figure 8: Stage and flow hydrograph for river station 60

The stage and flow hydrographs show the relationship between the stage, flow, and time. It can be deduced from Figure 7 that the flow increases with time until it reaches its peak at an hour mark of 12:00, at a flow rate of 700 m^3/s. Maximum stage of 284.42m was reached at an hour mark of 00:00 and the volume of water at the peak discharge was 80,286.01 x 10^3 m^3, while for Figure 8, the stage and flow hydrographs show that the flow reaches its peak at an hour mark of 13:00, at a flow rate of 1024.48 m^3/s. The maximum stage of 279.98m was reached at an hour mark of 13:00 and the volume of water at the peak discharge was 74869.84 x $10^3 m^3$. The X-Y-Z Perspective plot as shown in Figure 9 shows a 3-dimensional plot of multiple cross-sections within the river reach while Figure 10 indicates the velocity profile.

It can be deduced from the Figure that the flow in the channel has the highest velocity of 4.42 m/s. The profile output table in Figure 11 shows values at different river stations for the total discharge (Q_{Total}), Minimum channel elevation (Min ch El), calculated water surface from energy equation (W.S. Elev), Critical water surface elevation (Crit W.S) i.e water surface corresponding to the minimum energy on energy versus depth curve, Energy grade line for given water surface elevation (E.G. Elev), Slope of the energy grade line at a cross-section (E.G Slope), Average velocity of flow in the main channel, Total area of a cross-section of active flow, top width of the wetted cross-section and Froude's number for the main channel.

It can be deduced that the area i.e., river station 1607 immediately after the dam will experience a greater impact of the flooding with a water surface as high as 283.22 m because of the dam breach and a high discharge of 683.43 m^3/s, a velocity of 0.75 m/s and a flow area of 1071.80 m^2. Another area prone to destruction is River Station 173 having the highest discharge of 3672.73 m^3/s, a velocity of 3.87 m/s and a flow area of 1310.60 m^2. River station 60 which is the last cross-section has a discharge of 1886.34 m^3/s, the velocity of 2.55 m/s and a flow area of 955.20 m^2.

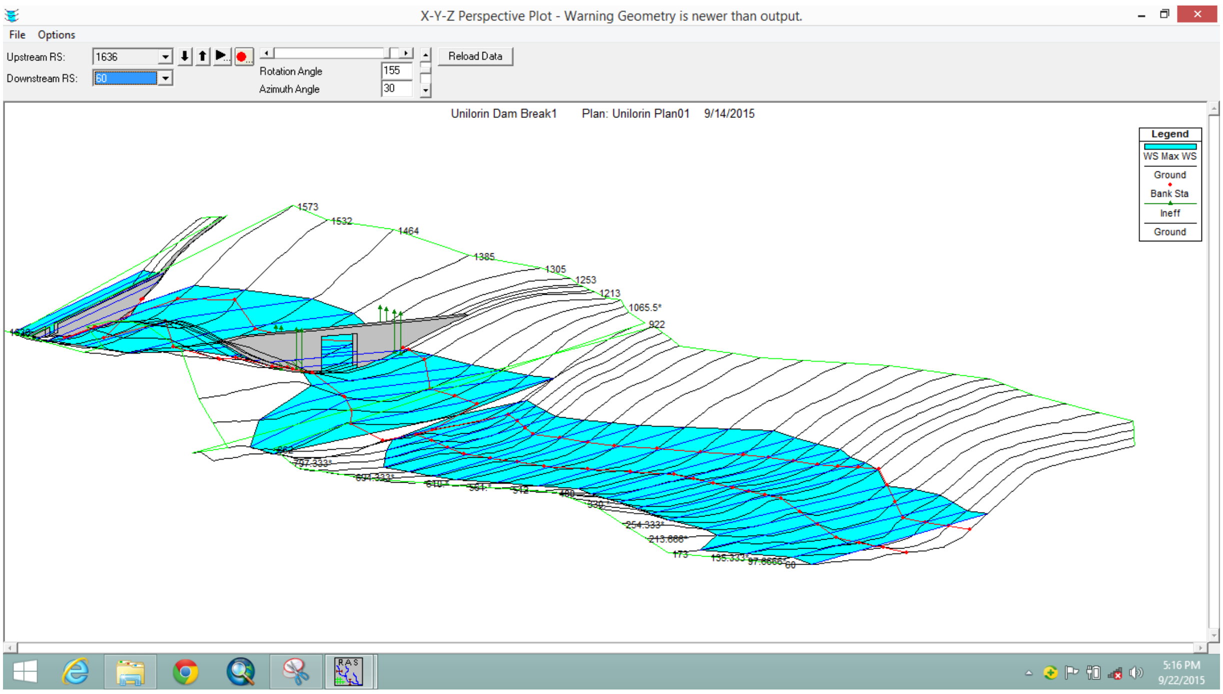


Figure 9: X-Y-Z Perspective plot for maximum water surface

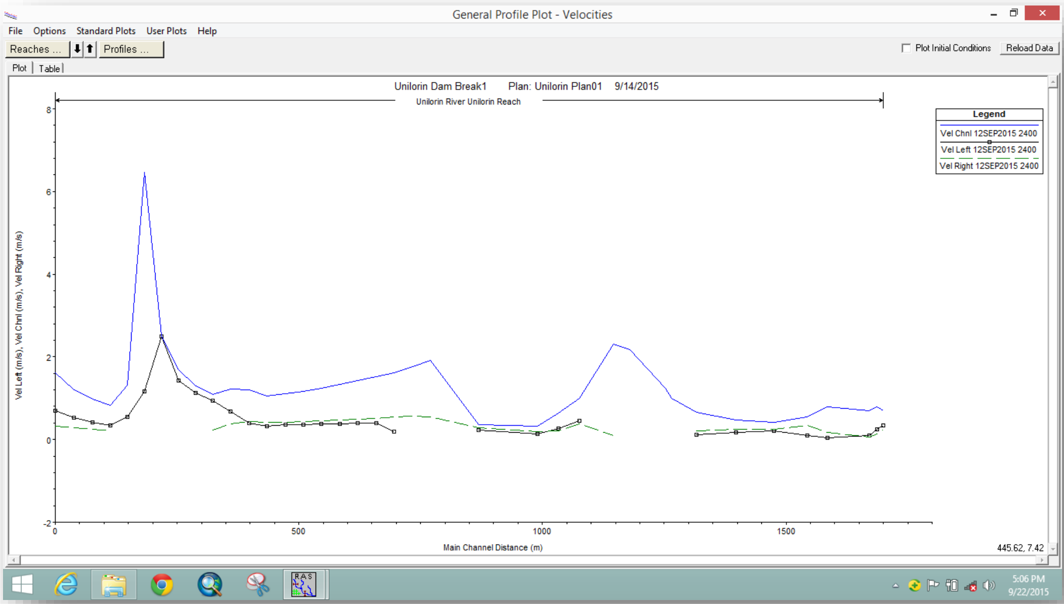
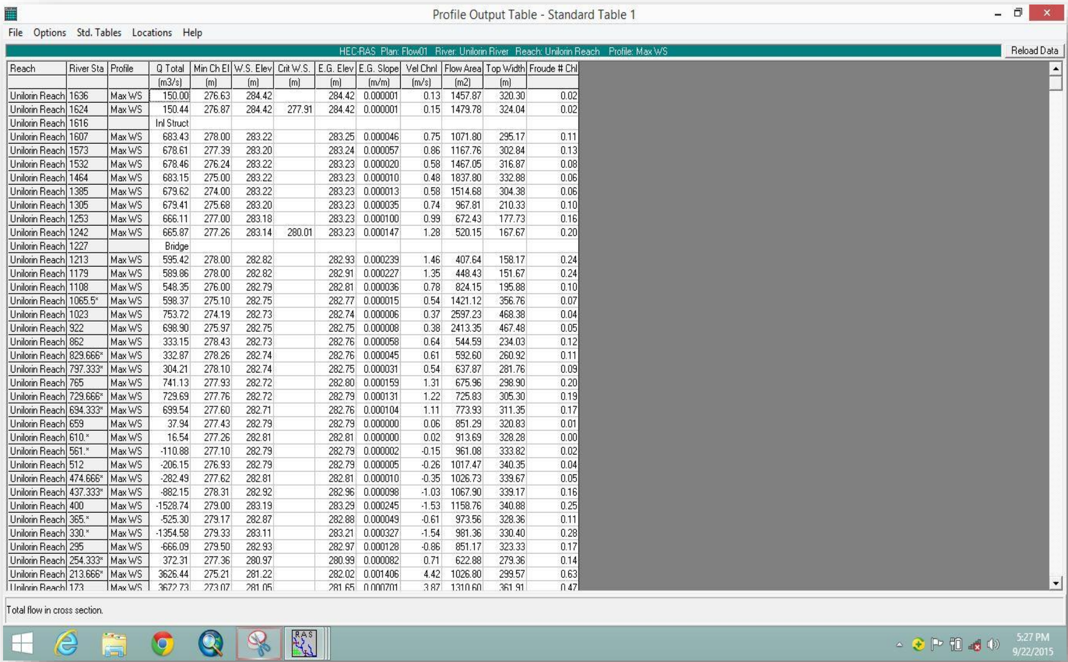


Figure 10: Velocity profile


Profile Output Table - Standard Table 1

HEC-RAS Plan: Flow01 River: Unilorin River Reach: Unilorin Reach Profile: Max WS

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
Unilorin Reach	1636	Max WS	150.00	276.63	284.42		284.42	0.000001	0.13	1457.87	320.30	0.02
Unilorin Reach	1624	Max WS	150.44	276.87	284.42	277.91	284.42	0.000001	0.15	1479.78	324.04	0.02
Unilorin Reach	1616		Inl Struct									
Unilorin Reach	1607	Max WS	683.43	278.00	283.22		283.25	0.000046	0.75	1071.80	295.17	0.11
Unilorin Reach	1573	Max WS	678.61	277.39	283.20		283.24	0.000057	0.86	1167.76	302.84	0.13
Unilorin Reach	1532	Max WS	678.46	276.24	283.22		283.23	0.000020	0.58	1467.05	316.87	0.08
Unilorin Reach	1464	Max WS	683.15	275.00	283.22		283.23	0.000010	0.48	1837.80	332.88	0.06
Unilorin Reach	1385	Max WS	679.62	274.00	283.22		283.23	0.000013	0.58	1514.68	304.38	0.06
Unilorin Reach	1305	Max WS	679.41	275.68	283.20		283.23	0.000035	0.74	967.81	210.33	0.10
Unilorin Reach	1253	Max WS	666.11	277.00	283.18		283.23	0.000100	0.99	672.43	177.73	0.16
Unilorin Reach	1242	Max WS	665.87	277.26	283.14	280.01	283.23	0.000147	1.28	520.15	167.67	0.20
Unilorin Reach	1227		Bridge									
Unilorin Reach	1213	Max WS	595.42	278.00	282.82		282.93	0.000239	1.46	407.64	158.17	0.24
Unilorin Reach	1179	Max WS	589.86	278.00	282.82		282.91	0.000227	1.35	448.43	151.67	0.24
Unilorin Reach	1108	Max WS	548.35	276.00	282.79		282.81	0.000036	0.78	824.15	195.88	0.10
Unilorin Reach	1065.5*	Max WS	598.37	275.10	282.75		282.77	0.000015	0.54	1421.12	356.76	0.07
Unilorin Reach	1023	Max WS	753.72	274.19	282.73		282.74	0.000006	0.37	2597.23	468.38	0.04
Unilorin Reach	922	Max WS	698.90	275.97	282.75		282.75	0.000008	0.38	2413.35	467.48	0.05
Unilorin Reach	862	Max WS	333.15	278.43	282.73		282.76	0.000058	0.64	544.59	234.03	0.12
Unilorin Reach	829.666*	Max WS	332.87	278.26	282.74		282.76	0.000045	0.61	592.60	260.92	0.11
Unilorin Reach	797.333*	Max WS	304.21	278.10	282.74		282.75	0.000031	0.54	637.87	281.76	0.09
Unilorin Reach	765	Max WS	741.13	277.93	282.72		282.80	0.000159	1.31	675.96	298.90	0.20
Unilorin Reach	729.666*	Max WS	729.69	277.76	282.72		282.79	0.000131	1.22	725.83	305.30	0.19
Unilorin Reach	694.333*	Max WS	699.54	277.60	282.71		282.76	0.000104	1.11	773.93	311.35	0.17
Unilorin Reach	659	Max WS	37.94	277.43	282.79		282.79	0.000000	0.06	851.29	320.83	0.01
Unilorin Reach	610.*	Max WS	16.54	277.26	282.81		282.81	0.000000	0.02	913.69	328.28	0.00
Unilorin Reach	561.*	Max WS	-110.88	277.10	282.79		282.79	0.000002	-0.15	961.08	333.82	0.02
Unilorin Reach	512	Max WS	-206.15	276.93	282.79		282.79	0.000005	-0.26	1017.47	340.35	0.04
Unilorin Reach	474.666*	Max WS	-282.49	277.62	282.81		282.81	0.000010	-0.35	1026.73	339.67	0.05
Unilorin Reach	437.333*	Max WS	-882.15	278.31	282.92		282.96	0.000098	-1.03	1067.90	339.17	0.16
Unilorin Reach	400	Max WS	-1528.74	279.00	283.19		283.29	0.000245	-1.53	1158.76	340.88	0.25
Unilorin Reach	365.*	Max WS	-525.30	279.17	282.87		282.88	0.000049	-0.61	973.56	328.36	0.11
Unilorin Reach	330.*	Max WS	-1354.58	279.33	283.11		283.21	0.000327	-1.54	981.36	330.40	0.28
Unilorin Reach	295	Max WS	-666.09	279.50	282.93		282.97	0.000128	-0.86	851.17	323.33	0.17
Unilorin Reach	254.333*	Max WS	372.31	277.36	280.97		280.99	0.000082	0.71	622.88	279.36	0.14
Unilorin Reach	213.666*	Max WS	3626.44	275.21	281.22		282.02	0.001406	4.42	1026.80	299.57	0.63
Unilorin Reach	173	Max WS	3672.73	273.07	281.05		281.65	0.000701	3.87	1310.60	361.91	0.47

Total flow in cross section.

Figure 11: Profile output table

3.4 Inundation Mapping

Figure 12 shows water surface extent (blue points), bank points, cross-section cut lines, river streamline, boundary polygon, and storage area with their various keys shown on the table of contents at the left corner of the image. Figures 13, 14, 15, and 16 show the depth, shear stress, stream power, and velocity layers respectively for the maximum water surface profile from the dam break simulation carried out in HEC-RAS. The depth is represented in Figure 13 using blue color. The deeper the color the higher the depth with the deepest part having a value of 9.22 m. The shear stress is represented in Figure 14 using purple color, the deeper the color the higher the shear stress as shown in the image. The maximum water surface has the highest shear stress value of 69.80 N/m^2 at the tail end of the river reach considered and the lowest value of 0. The stream power is represented in Figure 15 using a beryl green color, the deeper the color the higher the stream power as shown in the image. The maximum water surface has the highest stream power of 308.69 watts at the tail end of the river reach considered. The velocity is represented in Figure 16 using the mars red color, the deeper the color the higher the velocity as shown in the image. The maximum water surface has the highest velocity of 4.42 m/s at the tail end of the river reach considered. Figure 17 shows the flood inundation map projected on Google Earth aids hazard assessment. It can be seen from the figure that the areas with high depth include the area immediately after the dam, after the bridge, and at the tail end of the river reach. The areas with positive results (where water the surface is higher than the terrain elevation) are included in the floodplain area (inundation depth grid) and the areas with negative results are considered as dry.

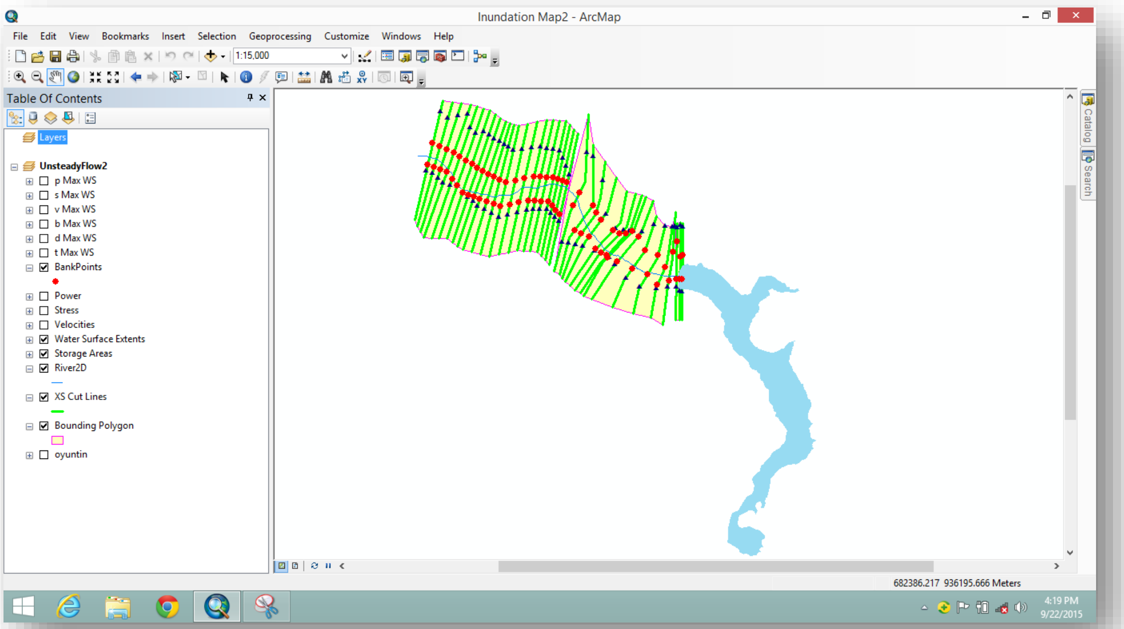


Figure 12: Inundation map showing layers

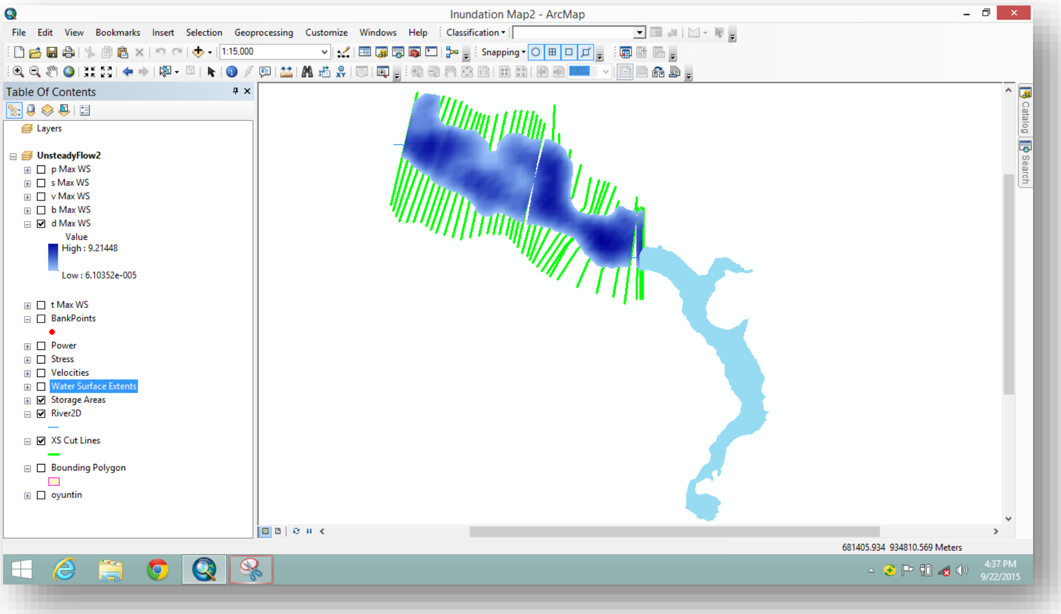


Figure 13: Inundation map showing depth layer for maximum water surface profile

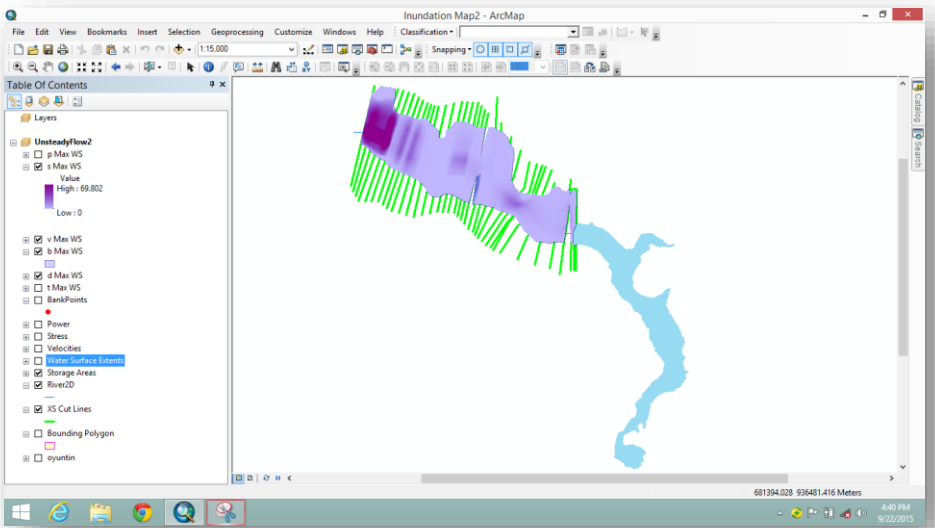


Figure. 14: Inundation map showing shear stress layer for maximum water surface profile

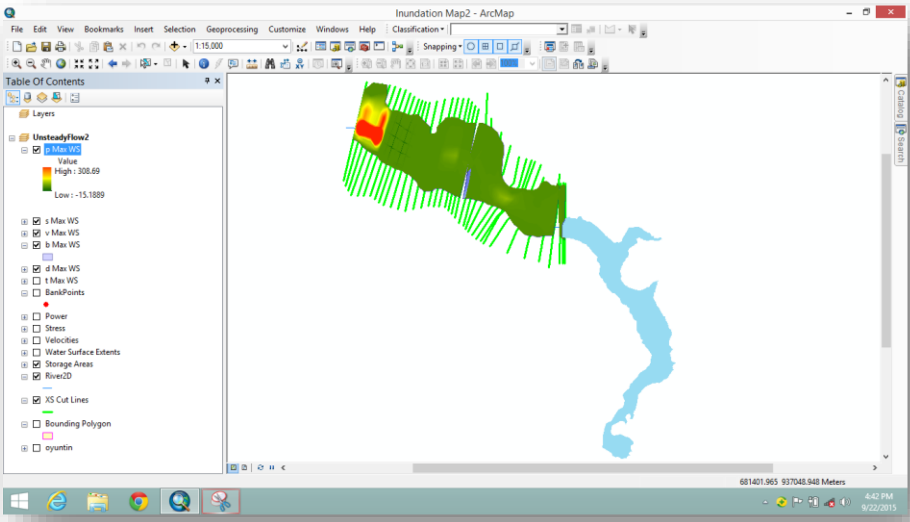


Figure 15: Inundation map showing stream power layer for maximum water surface profile

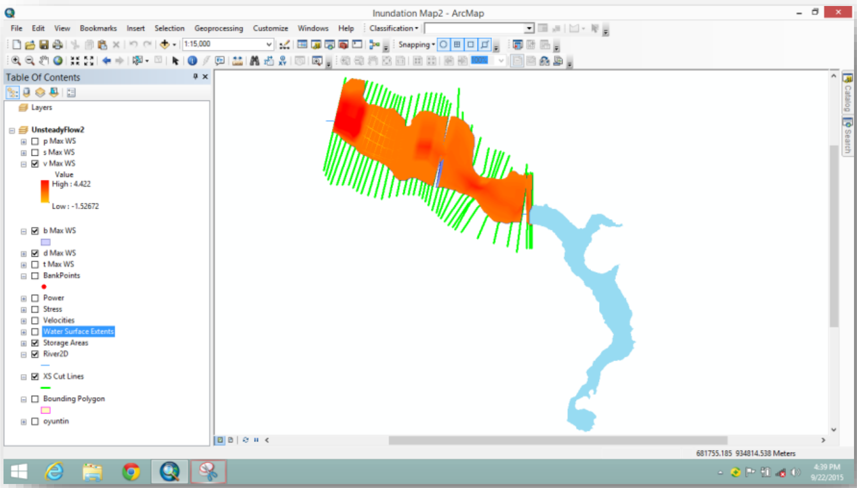


Figure 16: Inundation map showing velocity layer for maximum water surface profile

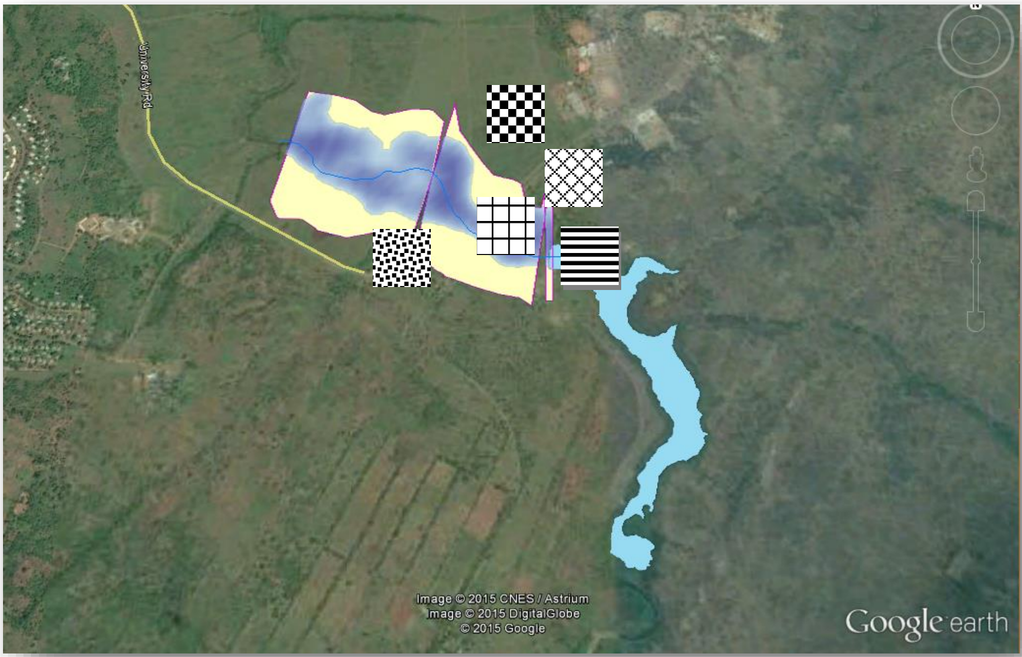
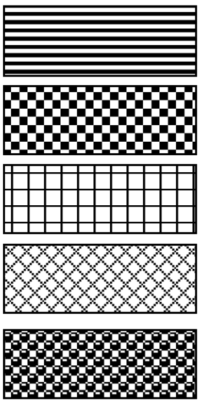


Figure 17: Flood Inundation map projected on google earth

KEY



The University Reception Ground

The University Fishery

The Faculty of Agriculture Farm Practical Plot

The University Fire Service Unit

Land Used for Plant Nursery

3.5. Hazard Assessment

The vulnerability may be defined as the extent to which a community, structure, service, or geographic area is likely to be damaged or disrupted by the impact of a particular hazard, on account of their nature, construction, and proximity to hazardous terrains or a disaster-prone area. The following are the vulnerable properties identified at the dam downstream:

1. The University Fire Service unit
2. The University reception ground
3. The University Fishery
4. The bridge
5. The faculty of Agriculture farm practical plot
6. An area of land used for a plant nursery and
7. At least 50 people downstream mainly securities and non-teaching staff at a point in time

It was identified from the inundation mapping that only the University reception ground in Figure 18 (roughly 200 m downstream from the inline structure), the bridge (roughly 346.0 m downstream from the inline structure), the faculty of agriculture farm practical plot in Figure 19 (roughly 662.97 m downstream from the inline structure), and the area of land used for plant nursery (roughly 616.09 m-1.1 km downstream from the inline structure) would be affected by the flood. If eventually, the dam fails, from the analysis and flood inundation mapping carried out the hazard rating would be high since 50 lives and school properties would be affected.

Figure 18: The University of Ilorin reception ground plots

Figure 19: The faculty of agriculture practical plots

5. CONCLUSION

In this study, a dam failure analysis was performed by using HEC-RAS on university of Ilorin dam. Event of occurrence of dam failure is a colossal loss of lives, properties, and academic productivity. Due to apprehensiveness of the authors to the probable dam failure of the university of Ilorin considering occasional impounding water near overtopping the embankment, this research was carried out. The dam break was carried out with the following methodological available tools; Google Earth, GIS, GPS, and HEC-RAC. The result showed that area immediately after the bridge (about 456.66-607.40 m downstream from the inline structure) and the tail end of the river reach (about 1.3 to1.6km downstream from the inline structure) will experience greater impact of the flood for the maximum water surface profile from flood inundation. The largest flow area covered between the dam and the bridge is 1,873.8 m^2 at river station 1464 (RS 1464) having a top width of 332.88 m while the largest flow area covered after the bridge is 2,597.2 m^2 at river station 1023 (RS 1023) with a top width of 468.38 m. However, the study reveals that University reception ground, the bridge, the faculty of Agriculture farm practical plot and the area of land used for plant nursery would be affected by the flooding at the worst flow condition (i.e. Maximum water surface profile). This result provide insight into appropriate emergency preparedness and disaster management plan. The first of this is safety programs in term of annual comprehensive checking and monitoring of the dam. The adaptation of technological warning siren should be placed. Finally future research on this dam should consider climate change effect and multiple scenarios on the dam break analysis and utilization of other software to carry out dam break.

Conflict of Interest

The corresponding author states that there is no conflict of interest on behalf of the other author.

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