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Predicting the Breach Hydrograph Resulting Due to Hypothetical Failure of Haditha Dam

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ABSTRACT

A hydrologic model is developed to compute the reservoir outflow hydrograph resulting from a hypothetical failure of Haditha dam located across the Euphrates river in Iraq due to enemy attack. In this program, the reservoir routing is analyzed by the level pool method in which storage is a nonlinear function of discharge. The results indicate that the maximum reservoir outflow is (202547 m3/s) which occurred about three and half hours after dam failure, and the reservoir will be depleted after about 65 hours. The results of the hydrologic model are compared with available results reported by Macdonald and Monopolis (1984), and it is found that the hydrologic results situate within the limits of their study.

KEYWORDS: Breach hydrograph, Hypothetical failure, Haditha dam.

INTRODUCTION

Dams provide society with essential benefits such as water supply, flood control, recreation, hydropower and irrigation. However, catastrophic flooding occurs when a dam fails and the impounded water escapes through the breach into the downstream valley. Usually, the magnitude of the flow greatly exceeds all previous floods and the response time available for warning is much shorter than for precipitation-runoff floods (Fread, 1988).

Case studies show that dam failures may arise due to different reasons ranging from seepage, piping (internal erosion), overtopping due to insufficient spillway capacity and insufficient free board and to settlements due to slope slides on the upstream or downstream shells and liquefaction due to earthquakes. Regardless of the reason, almost all failures begin with a breach formation. Basically, breach is defined as the opening formed in the dam body that leads the dam to fail, and this phenomenon causes the concentrated water behind the dam to propagate towards downstream regions.

Hundreds of dam failures have occurred throughout the history. These failures have caused immense property and environmental damages and have taken thousands of lives. For example, South Fork Dam, also known as the Johnstown Dam in Pennsylvania/U.S.A. failed in May 1889. The dam was 36 years old when it failed. The earthfill dam was (21.9m) high and contained about (14. 2 million m^3) of water. The dam failed as a result of overtopping that occurred during a flood caused by a 25-year frequency storm. The failure resulted in about 2,209 deaths, the largest loss of lives among all U. S. dam failures. Nearly all of the fatalities occurred within the first (22.4 km) downstream from the dam. The world's most catastrophic dam failures occurred in August 1975 in the Zhumadian Prefecture of Henan Province in central China. A typhoon struck, causing reservoirs to swell. Banqiao Dam, (118 meters) high, and Shimantan Dam collapsed as did dozens of smaller dams. Millions of people lost their homes. The death toll estimates for these failures varied widely.

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Approximately 26,000 deaths occurred from drowning in the immediate aftermath of the dam collapses. There were as many as 230,000 deaths if those who died of consequent health epidemics and famine are included (Graham, 1999). However, in order to be able to assess the consequences of dam failure, simulation of the flood caused by a dam break is required to determine the inundated area, flood depth and travel time of the flood waves, so that adequate safety measures can be provided. Many studies have tried to model the flood wave resulting from dam failure. Flood routing methods have been applied to the reservoirs storage and to the outflows resulting from the failure. The purpose of this study is to develop a hydrologic model for predicting the reservoir outflow hydrograph resulting from the hypothetical failure of Haditha dam in Iraq.

RESERVOIR OUTFLOW

The two primary tasks in the analysis of a dam break are: the prediction of the reservoir outflow hydrograph and the routing of that hydrograph through the downstream valley. The determination of the reservoir outflow hydrograph can be further subdivided into two tasks (Wahl, 1988):

1- Predicting the breach characteristics (e.g., shape, depth, width, rate of breach formation).
2- Routing the reservoir outflow hydrograph through the breached and outlet structures.

Breach Formation

Concerning the breach simulation, it is possible to distinguish the following methods (Wurbs, 1987):

- Instantaneous complete removal of the dam.
- Instantaneous partial breach of the dam.
- Breach the growth of which is fixed with time.
- Breach the growth of which is predicted using an erosion model.

The first two methods may be appropriate, respectively, for concrete arch dams and for concrete gravity dams, but they seem to be too conservative and unrealistic for earth embankments. The assumption that the breach dimensions grow with time usually according to a linear law, appears more likely. However, the fourth method provides a more realistic representation of the erosion process. The assumption of a linear growth of width and depth of a gap is adopted in this study. In reality, when outflow is small, erosion is less than that supposed in the linear assumption. When the downstream water level becomes higher than the sill elevation submerged flow will occur. High downstream water levels reduce flow gradients thus diminishing the erosion capacity of the outflowing waters and consequently the erosion may be smaller than that supposed in the linear assumption .

Routing the Reservoir Outflow

The outflow hydrograph of the breached dam is computed either by level pool reservoir routing or dynamic routing with the breached dam outflow as an internal boundary condition between the upstream reservoir reach and the downstream river reach. Level pool routing is used for wide, flat reservoir surfaces with gradual changes in water surface elevation, while dynamic routing is needed for narrow valleys with significant water surface slope in the reservoir (Strum, 2001).

In this study, the reservoir routing system is analyzed by the level pool method in which storage is a nonlinear function of Q.

Level Pool Reservoir Routing

Usually, unsteady flow routing in reservoirs is approximated by a simple level pool routing technique based on the principle of mass conservation; i.e.,

$$\frac{\partial S}{\partial t} = I(t) - Q(t) \qquad \ldots (1)$$

in which inflow *(I)* and outflow *(Q)* are functions of time *(t)*, and storage (S) is a function of the water-surface elevation (h) which changes with time (t). The reservoir is assumed always to have a horizontal water surface throughout its length (Fread and Hsu, 1993).

Level pool routing is a procedure for calculating the outflow hydrograph from a reservoir with a horizontal water surface, given its inflow hydrograph and storage – outflow hydrograph (Chow et al., 1988).

The time horizon is broken into intervals of duration Δt, indexed by j; that is, t = 0, Δt, $2\Delta t$, …, $j\Delta t$, $(j + 1)\Delta t$, … and the continuity equation is integrated over each time interval. For the j-th time interval:

$$\int_{S_j}^{S_{j+1}} \partial S = \int_{j\Delta t}^{(j+1)\Delta t} I(t)\partial t - \int_{j\Delta t}^{(j+1)\Delta t} Q(t)\partial t \quad \ldots (2)$$

The inflow values at the beginning and at the end of the j-th time interval are I_j and I_{j+1}, respectively, and the corresponding values of the outflow are Q_j,Q_{j+1}. If the variation of inflow and outflow over the interval is approximately linear, the change in storage over interval S_j,S_{j+1} can be found by rewriting equation (2) as:

$$S_{j+1} - S_j = \frac{I_j + I_{j+1}}{2}\Delta t - \frac{Q_j + Q_{j+1}}{2}\Delta t \quad \ldots (3)$$

The values of I_j and I_{j+1} are known because they are prespecified. The values of Q_j and S_j are known at the j_{th} time interval from calculation during the previous time interval. Hence, the equation contains two unknown Q_{j+1} and $S_{j+1,}$ which are isolated by multiplying equation (3) through by $2/\Delta t$ and rearranging the result to produce:

$$\left[\frac{2S_{j+1}}{\Delta t} + Q_{J+1}\right] = \left(I_j + I_{j+1}\right) + \left[\frac{2S_j}{\Delta t} - Q_j\right] \quad \ldots (4)$$

In routing the flow through the time interval j, all the terms on the right side of equation (4) are known, and so the value of $(2S_{j+1}/\Delta t + Q_{j+1})$ can be computed. Reservoir outflow Q_{j+1} (breach outflow + spillway outflow + outlet work outflow) can be computed as a function of water surface elevation upstream and downstream of the dam using weir and orifice equations. When Q_{j+1} is determined, S_{j+1} can be obtained. Water surface elevation in the reservoir is computed by means of the relationship between storage and water surface elevation.

To set up the data required for the next time interval, the value of $(2S_{j+1}/\Delta t - Q_{j+1})$ is calculated by:

$$\left[\frac{2S_{j+1}}{\Delta t} - Q_{J+1}\right] = \left[\frac{2S_{j+1}}{\Delta t} + Q_{j+1}\right] - 2Q_{j+1} \quad \ldots (5)$$

THE CASE STUDY (HADITHA DAM)

Haditha dam is a multi-purpose hydro-development designed to control the Euphrates river flow in interests of irrigation, electric power generation and for partial accumulation of extreme Euphrates river inflows into Haditha reservoir. Haditha dam was constructed on the Euphrates river in the middle west of Iraq 7 km upstream from Haditha town. The project generates (660 MW) of electrical power aside from performing its flood control function. Central and southern parts of Iraq get the benefit of irrigation water from its reservoir. Haditha dam was conceived in the late 1960s but construction did not begin until 1977. The Haditha dam's embankment was designed by the Soviet Union's Ministry of Energy and its power station was designed and constructed by Yugoslavian firms. It was conceived as a multi-purpose dam that would generate hydroelectric power, regulate the flow of the Euphrates and provide water for irrigation. Construction lasted between 1977 and 1987 and was a joint undertaking by the Soviet Union and Iraq. The dam consists of an asphaltic concrete cutoff wall at its core, followed by mealy detrital dolomites and a mixture of sand and gravel. These materials were chosen because they are readily available near the construction site. This core is protected by a reinforced concrete slab revetment on the upstream side of the dam and a rock-mass revetment on the downstream side. Flood control at Haditha dam is accomplished with a spillway integrated in the powerhouse and equipped with six radial gates of 16m width and 13m height each. Sill level of the spillway is at an elevation of 134m a. s. 1. and the top elevation of the radial gates in closed position is at 147m a. s. l., which is the normal height of water level. The storage volume of the reservoir at this elevation is 8.2×10^9 m^3, whereas at the maximum design reservoir level of 150m a. s. l. it is approximately 9.7 x 10^9 m^3 of water to be stored in the reservoir (Swiss Consultants, 1985 a).

Since Haditha city lies between Al-Qaim near the west border of Iraq, which is considered an entry point, and Baghdad, Haditha became a center for fighting after Haditha dam and the surrounding areas were initially secured by U. S. troops in April 2003 as part of the invasion of Iraq. Also, Haditha dam is considered the second-largest hydroelectric contributor to the power system in Iraq after the Mosul dam, and therefore an attack on the dam would result in severely flooded towns along the Euphrates downstream from Haditha, as well as eliminating an important source of electricity and affecting the functioning of the country's electrical grid. Therefore, in this study, the failure of Haditha dam by an enemy attack is considered the most probable failure.

SELECTED DAM – BREAK ASSUMPTIONS

As explained above, the failure of Haditha dam is supposed to be by an enemy attack. In this study, MK 84 penetration bomb shall be considered. Such a bomb could create a crater of a depth of up to 16 m in earth dam or rock fill or of up to 4m in solid concrete (Swiss Consultants, 1985 a). The spillway and powerhouse section forms a part of the dam. The power house block is located in the center of the valley. All gates, doors, trapdoors, …etc are weak points where attacks could occur. Flooding of the power house could cause a lot of damage, but the concrete structure would definitely resist the erosion forces of water. The side wall on the other hand is too solid to be destroyed and initiate an erosion of the embankment dam, therefore it is assumed that the attack will be on the embankment to the left side of the powerhouse. Due to the sandy–silty nature of the fill, the erosion will proceed quickly vertically down to the valley bottom and sidewards to the powerhouse block on one side and on the other side at least to the point where the foundation level rises significantly. On the left side, the valley bottom is at an elevation of 100m a. s. l. and on the right side the valley bottom is at an elevation of 105m a. s. l. The lateral washout rate for channel sides is assumed to be about (2.6m/min), the size of the breach is calculated for each time step by assuming a linear growth of width and depth from the initial values until the final depth within the given breach time. This assumption is crude, however, although the erosion capability of the outflowing waters increases with the size of the breach, the dam volume to be eroded increases as well. It is assumed that these factors compensate to a linear behavior. In reality, it is to be expected that in the very first phase, when the outflow is still small, erosion is less than supposed in the linear assumption. In the following phase, a rapid development of the breach may be expected, while in the final phase erosion may be smaller since the high downstream water levels reduce flow gradients, thus diminishing the erosion capacity of the outflowing waters. The assumption of linear growth of width and depth of the gap that is adopted in this study is considered as an approximation to the real process.

A possible scenario is given hereafter and breach formation is assumed to consist of four phases:

Phase (1)

The first phase starts at the enemy attack with MK84 penetration bombs. A channel of trapezoidal cross-section with (86.5m) top width at the top of the left embankment and (22.5m) bottom width is assumed to be formed within about 3 minutes. The side slope of the breach is assumed 1V: 2H. Typically $0 \leq H \leq 2$ (Chow et al., 1988).

The final shape of the assumed channel is the same as if one assumes that a breach section is formed at the top of the embankment with a top width of (2.6m), a side slope of 1V:2H and an initial depth of (0.48m). Then the channel sides erode at a rate of (2.6m/min) and the channel bottom erodes at a rate of (0.48m/min). The estimated time required to perform the same channel with the aforementioned dimensions is approximately 33 minutes.

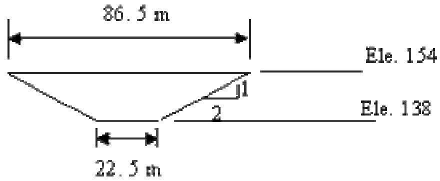

Figure 1: End of phase (1)

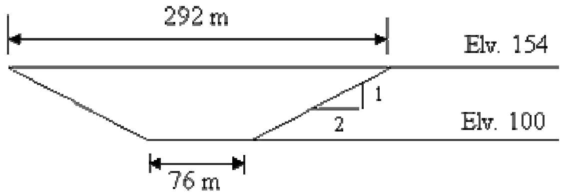

Figure 2: End of phase (2)

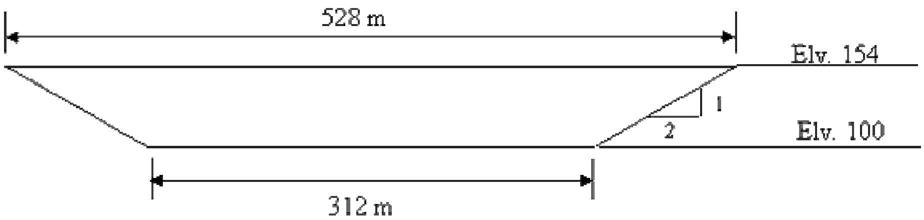

Figure 3: End of phase (3)

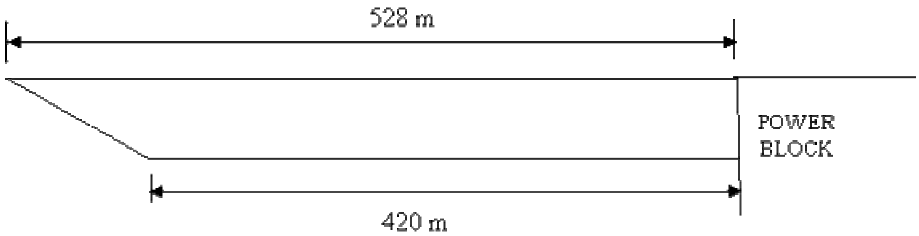

Figure 4: End of Phase (4)

Phase (2)

Once a first breach is opened and a larger quantity of water starts to flow out of the reservoir, the enlargement of the breach will proceed quickly. During this phase, the channel sides erode at a rate of (2.6m/min) and the channel bottom erodes at a rate of (0.48m/min). The bottom of the breach will be lowered until it reaches the valley bottom (an elevation of 100m a.s.l). Further down cutting is prevented since the river bed is less erodible. This phase is assumed to last 79 minutes.

Phase (3)

In the third phase, the breach will extend to both sides by the flowing water. The breach is enlarged laterally at the same rate of (2.6m/min) and side slope of 1V:2H until the left side of the channel reaches the left valley side, and no further lateral growth of the left breach side occurs. This phase is assumed to last 90.7 minutes.

Phase (4)

During this phase, which lasts about 41.5 minutes, the lateral erosion will proceed sidewards to the powerhouse block on the right side, as mentioned above. The block itself will not fail. At the end of this phase, the breach reaches its final shape, which will be limited at the bottom by the surrounding valley bottom elevation and to the sides by the powerhouse block and by the left valley side.

As a summary, in this scenario, it was assumed that almost the complete dam to the left of the power house block will erode. The final shape of the breach has a bottom width of 420m and a side angle of about 26.50 at the left valley side and a vertical delimitation at the powerhouse block. It is assumed that the material is eroded down to an elevation of 100.00m a.s.l. The breach development time is approximately three and half hours.

The sketch of this scenario is presented in Figures (1) through (4).

DESCRIPTION OF THE ELEMENT OF THE HYDROLOGIC MODEL

A hydrologic model requires the determination of the following:

The Initial Conditions for the Reservoir before the Failure

a- Initial elevation of the reservoir before failure (146m).

b- Initial inflow to the reservoir before failure (795m^3/s).

c- Initial outflow from the reservoir before failure (502m^3/s).

The Inflow to the Reservoir during the Time of Simulation

It was assumed that the mean daily inflow to the Haditha reservoir during the time of simulation is (2799m^3/s) in May 1988.

The Outlet Works, Spillway and Breach Flow Models

It was assumed that at the onset of failure the spillway gates can't be opened because of a malfunctioning mechanism, an accident or a repair work, and that-simultaneously-no discharge through the bottom outlets and turbines will be possible. Therefore, all discharge facilities are closed and their flows don’t need to be modelled. The breach (weir) is defined by its sill elevation and width, both given as functions of time. Weir outflow is calculated according to the Jacobi formula, where the parameter M is given as input. According to the shape of the sill, M varies between 1. 21 and 1. 62 (Swiss Consultants, 1985c).

$$Q = \frac{2}{3} * \mu_1 * \sqrt{2g} * h_0^{1.5} * B \qquad \ldots (6)$$

Where:

$h_0 = H_0$ - Zw

H_0 = Upstream water level

Z_w = Sill elevation

B = Average width of overfall

g = Gravity acceleration (9. 81 m/s^2)
Q = Discharge
μ_1 = Discharge coefficient = 1. 0607 * $M^{-1.5}$

The Reservoir Elevation – Storage Relationship

The relationship between storage and elevation is very important in the dam break study. No fixed formula is found in literature for this relationship. From the scatter diagram of the storage – elevation data of reservoirs, the best fit equation may be a polynomial, logarithmic, exponential or power equation. Elevation – storage field data of Haditha reservoir can be seen in Table (1). A statistical analysis program SPSS (Statistical Package for Social Sciences) is used to find the best fit equation for the elevation – storage data of Haditha reservoir. This equation is:

$$S = exp\left(11.962 - \frac{1445.9}{Elv}\right) \qquad \dots (7)$$

where:
S = Volume of water stored at the specified elevation (Elv) in billion m^3.
Elv = Water level in the reservoir in m a.s.l.

The coefficient of determination for equation (7) is (R^2= 0. 998).

Equation (7) does not give zero volume at an elevation of 100m a.s.l. Therefore, for the dam break analysis, a linear equation is used for the first two points (Al-Ghazali, 2009):

$$S = \frac{0.77}{19}(Elv - 100) \qquad \dots (8)$$

MODEL RESULTS

The computed reservoir hydrograph of Haditha dam is shown in Fig. (5). The maximum reservoir outflow is (202547m^3/s) which occurred about three and half hours after dam failure. The Figure shows that after about 65 hours from the dam failure, the whole volume of the reservoir will be going out to the river reach, indicating that the reservoir was depleted at the end of simulation time. At (966) minutes after the dam failure, the reservoir water surface elevation was 119m a. s. l. For times beyond (966) minutes, Eq. (8) was used instead of Eq. (7) because the application of Eq. (7) ends for an elevation of (119), and that is why the abrupt change in the trend of outflow hydrograph is seen for this period (Al-Ghazali, 1999).

Table 1. The elevation – storage field data of Haditha dam

Elevation (m a.s.1.)	Storage (billion m^3)
100	0
119	0. 77
122	1. 17
135. 84	3. 97
140	5. 26
143	6. 4
144	7
145. 87	7. 64
146. 4	7. 96
147	8. 3
150	9. 7

Macdonald and Langridge-Monopolis (1984) developed best-fit and envelope curves for peak outflow from breached earth fill dams as a function of the breach formation factor. These curves were used to verify reasonable results from breach simulations carried out using breach parameters predicted from the breach formation factor (Wahl, 1998).

By using the graphs presented by Macdonald and Monopolis to check the breach development time and peak outflow, it is found that :
1- Maximum breach development time = 5. 5 hr.
2- Peak outflow = 240550 m^3/s.

It is assumed that the breach development time will be equal to (3.5 hours), which is less than the maximum breach development time (5.5 hours). The computed peak outflow is (202547m^3/s) and is also less than the peak outflow (240550m^3/s). Therefore, the results of the hydrologic model situate within the limits of the study of Macdonald and Monopolis.

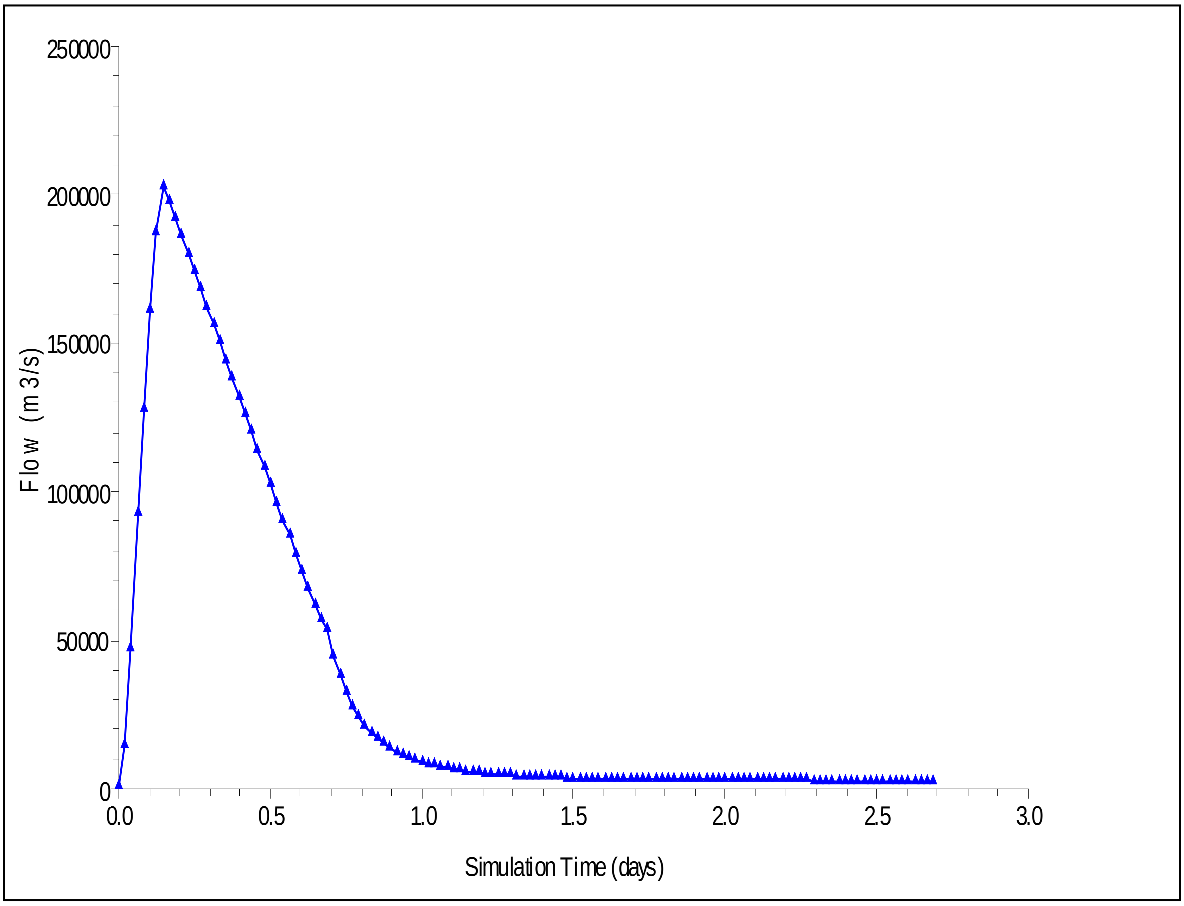

Figure 5: Reservoir outflow hydrograph

CONCLUSIONS

In this study, the calculations of the breach hydrograph are based on the assumption that within three and half hours after an enemy attack on Haditha dam almost the complete dam to the left of the powerhouse block will be eroded. The size of the breach is calculated for each time step by assuming a linear growth of width and depth from the initial values until the final depth within the given breach time. The first phase of breach is assumed to be a trapezoidal channel with 86.5m top width and 22.5m bottom width and the final breach has a bottom width of 420m and a side angle of about 26.50 at the left valley side and a vertical delimitation at the powerhouse block. It is assumed that the material is eroded down to an elevation of 100.00m a.s.l. The model results indicated that the maximum reservoir outflow is (202547m^3/s), which occurred about three and half hours after dam failure, and the reservoir will be depleted after about 65 hours.

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