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ASSESSING THE RISK OF THE ASWAN HIGH DAM BREACHING

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ABSTRACT

The present paper assesses the risk of the Aswan High Dam breaching, numerically. Literature, in the field of dam breaching, was reviewed. The Aswan High Dam was visited. Site data, so as measurements data, were collected and analyzed. A suitable numerical model was chosen and was selected to be implemented. Scenarios were designed. The expected impacts of the Aswan High Dam failure that might result from overtopping or piping was simulated and the obtained results were analyzed. Outflow hydrographs due to the failure mode were obtained and analyzed. A risk assessment to the dam breaching was achieved. The results of this investigation could be further applied and could assist decision makers to set a plan to confront the risks of the Aswan High Dam failure.

يهدف هذا البحث إلى تقييم المخاطر الناتجة عن حدوث إنهيار للسد العالى بأسوان بإستخدام نموذج رياضى. تم مراجعة الدراسات السابقة والخاصة بإنهيار السدود، كما تم زيارة السد العالى بأسوان وتجميع البيانات وعمل القياسات اللازمة للدراسة وتحليلها. تم إختيار النموذج العددى المناسب وتطبيقه وتصميم السيناريوهات لتوقع شكل ونوع الانهيار والتنبؤ بكمية المياه الخارجة من السد وتم تحليل النتائج، حيث أنه من الممكن أن تستخدم نتائج هذا البحث فى تقييم مستوى المخاطر الناتجة عن حدوث إنهيار للسد العالى ومساعدة متخذى القرار فى وضع خطة للطوارىء وتحسين نظام مراقبة السدود.

Keywords: Dam Breach, Outflow hydrograph, Aswan High Dam, Simulation Model

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1. INTROUDUCTION

The Aswan High Dam (AHD), was built in 1968 to protect Egypt against flood and draught of the Nile River. It also secures a sustainable supply of water demands in the Nile River. The location of the Aswan High Dam is 6.50 km south of the Aswan Old Dam (AOD). This location was considered as the most suitable and appropriate location due to the relative narrowness of the course of the Nile.

The AHD is a rock-fill dam with a length of 3830 m of which 520 m are within the river channel and the rest is in the shape of two wings at both sides of the river. The length of the right wing is 2520 m, while the left wing is 780 m. The dam width at the bottom of the river bed is 980 m, and 40 m at the crest. The height of the dam above the river bed is 111 m. The bulk volume of materials used in building the AHD is about 43 million cubic meter, (MCM) which is about 17 times the size of the great Giza pyramid, "Cheops". The body of the dam is constructed of granite blocks, sand and clay, in the midst of which is a clay core to prevent seepage of water. The core is connected at the upstream part with a horizontal blanket of clay for the same purpose. Figure (1) shows the cross section of the dam and its materials.

Since the Nile bed, on which the dam was built, consists of sedimentary deposits, it was provided with a vertical injected curtain extending 170 m under the main core until it reaches the solid impermeable layer. The injected curtain has been built of special materials like Aswan clay and other chemical materials to prevent the seepage of water. The width of the injected curtain is 40 m under the main core, and decreased until it reaches 5 m at the point where it meets with the solid layer.

The core was penetrated by three galleries, constructed with reinforced concrete. During construction, the galleries were used in completing the vertical injected curtain, while they are being used now for inspection and maintenance purposes. Various measuring devices have been installed in these galleries to measure vertical and horizontal movements, pore pressure in clay and seepage, if any. The dam is provided before the end of its toe with a row of vertical relief wells to drain the water which may seep through the dam. **Abdel Azim Abul-Atta, [1]**.

The AHD formed a large artificial lake of 500 km length, with an average width of 12 km. the surface area of the lake is 6000 km^2. It is considered one of the largest man-made lakes in the world. Its maximum capacity, which mounts to 162 Billion Cubic meter, (BCM) is divided into three parts as follows:

a) Dead storage capacity of 31.6 BCM up to 147 meter above Mean Sea Level, and designed for the silt deposition over 500 years. **b)** Live or working storage capacity between the levels 147, and 175 m above MSL, mounting to 90.4 BCM, which guarantees the annual requirements of water.

c) Flood control capacity of 40 BCM between levels 175 and 182 m above MSL. Figure (2) shows the Nasser Lake elevation-storage curve.

2. LITERATURE REVIEW

Literature was reviewed in the field of dam breaching. From the reviewed literature, it was clear that several researchers dealt with dam breaching, worldwide, while the Aswan High Dam did not gain any interest from researchers.

Due to the importance of the dam in the Egyptian lives, this study was initiated in order to assess the breaching risks.

3. SITE VISITS

The Aswan High Dam area was thus visited in order to accumulate data about the dam itself and inspect the dam location so as the neighboring zones.

4. FIELD SO AS MEASUREMENTS DATA ACCUMULATION AND ANALYSIS

Also, field, so as, measurements data was collected from different sources. These data described the water levels in the Nasser Lake and the inflows so as the outflows of the dam. Also, metrological data were collected. All these data were analyzed in order to perceive an insight to the physical properties of the breaching process. The analyzed data were a guide in the design of the simulated scenarios.

5. Numerical Model

A numerical model is needed to predict the outflow hydrograph, and breach characteristics due to dam failure. Therefore, the available numerical models that could simulate breaching were inspected.

In the present study, the HR-Breach Model was chosen to be implemented in order to simulate the expected breach development of the AHD. Numerical models could be considered as the most widely applied technique to solve mathematical expressions that describe any physical phenomena. Those models are mainly classified by number of spatial dimensions over which variables are permitted to provide much more detailed results than others. However, collection of adequate and reliable field data is highly required to fulfill the model calibration and verification which lead to successful application.

5.a. Model Description

InfoWorks-RS incorporates parts of the HR-BREACH model developed by HR Wallingford. The HR BREACH model is a 1D model that can simulate the failure of homogeneous or composite embankment dams by overtopping or piping. It is a tool based on knowledge and experience gained from many failures of dams in the world.

The HR BREACH model takes into account the soil mechanics principles in the breaching process and is based upon the principles of hydraulics and sediment transport. A simple probabilistic approach is also incorporated to represent the uncertainties of the process. The effect of plain grass, and riprap as protective layers was also incorporated into the model. The model predicts the outflow hydrograph from a breached embankment dam. It also simulates the erosion processes involved in the breaching process and predicts the growth of the breach in the longitudinal and the transverse directions.

A detailed description of the model is given in, **Mohamed, et al. [5]**. It is worth to mention that, in most of the model test cases, the model has showed a better performance than other breach models that were used to model similar cases.

5.b. Basic Equations

The model calculates outflow and sediment discharge. It can also route the water and sediment flows through the downstream channel and floodplains. To simulate the flow over the crest and on the downstream face of the dam, the 1D Saint-Venant equations are used. The model is based on the principles of hydraulics, sediment transport, soil mechanics, the geometric properties of the dam, and the reservoir characteristics. The governing equations are used to describe the flow are full Saint-Venant equations for unsteady flow, and momentum correction coefficient.

2.c. Breach Morphology

In this model, the breach shape is controlled by two common assumptions. The first mechanism assumes an initial rectangular shape. The following relationship governs the width of the breach:

$$B_o = B_r\, y \qquad (1)$$

where:

B_o: width of the breach [L]

B_r: factor based on the optimum hydraulic efficiency [-]

y: depth of flow in the breach [L]

The parameter B_r is a factor based on the optimum channel hydraulic efficiency and it has a value of 2.0 for overtopping failures and equals to 1.0 for piping failure. For the failure of man-made dams, the model assumes critical depth at the entrance of the breach channel. Whilst for the failure of natural dams, (e.g. landslides) the water depth in the breach channel is assumed to be the normal uniform depth rather than the critical depth. This is due to the relatively long breach channel length compared to man-made dams.

The second mechanism is derived from the stability of soil slopes. The initial rectangular shaped channel changes to a trapezoidal channel when the sides of the breach channel collapse, as shown in Figure (3).

This change from rectangular shape to trapezoidal shape forms an angle α, with the vertical. The collapse occurs when the depth of the breach cut reaches a critical depth, H', that can be expressed as follows:

$$H' = \frac{4c\cos\phi\sin\theta}{\gamma_s\,[1-\cos(\theta-\phi)]} \qquad (2)$$

where:

c: soil cohesion [ML^{-1} T^{-2}]

φ: internal angle of friction [^{o}]

γ_s: soil specific weight [ML^{-2} T^{-2}]

θ: breach channel side slope

[^{o}]: angle with the horizontal before failure

Then the width could be calculated using equation (1), assuming that, B_o is the bottom width.

In order to determine the flow through the breach and the flow depths and velocities in the model, the boundary and initial conditions need to be defined. The initial condition is the initial water level in the reservoir. The upstream boundary condition can be either a flow - time or a head - time boundary for the reservoir. Three types of downstream boundary could be used in the model. These are head-time boundary, rating curve, and downstream valley cross section.

5.d. Reservoir Hydraulics

To compute the outflow from the dam, the reservoir level has to be calculated. If a head -time boundary is used then this level could be obtained readily from the boundary data by linear interpolation. If a flow-time boundary is used then this level could be calculated iteratively by solving the following equation:

$$\frac{dV}{dt} = Q_i - Q_o \qquad (3)$$

where:

d_V/d_t: rate of change of the water volume of the reservoir [L^{3}S^{-1}]

Q_i : inflow rate to the reservoir [L^{3}S^{-1}]

Q_o: outflow rate from the reservoir [L^{3}S^{-1}]

The inflow rate to the reservoir could be calculated from the upstream boundary conditions. The outflow rate of the reservoir consists of the flow thorough the breach (Q_b), the flow over the crest of the embankment (Q_w), and the flow through any outlets or spillways.

5.e. Hydraulics of Flow through the Breach

For an overtopping failure, the reservoir water level must exceed the top of the dam before any erosion occurs. Erosion is assumed to occur only along the downstream face of the dam. The flow over the dam crest (Q_w) and through the breach (Q_b), see Figure (4), can be computed using the broad crested weir formula. The equations used in the model to compute these two components are as follows:

$Q_w = C_d (L - B_b) H_w^{3/2}$ (4)

$Q_b = C_d B_b H_b^{3/2}$ (5)

where:

Q: discharge $[L^3 S^{-1}]$

L: crest width [L]

B_b: breach width [L]

C_d: discharge coefficient [-]

H_b: total head over the breach [L]

H_w: total head over the crest [L]

The values of the flow are corrected if the flow is submerged. Usually this condition occurs after the reservoir water level has receded and there is no flow over the crest. It is thus likely that the value of the flow through the breach is affected.

The steady non-uniform flow equation has been used to compute the water depths, velocities, and energy slope on the downstream slope of the dam. The effect of the downstream steep slope has been included in the computations of the flow depths and velocities

For a piping failure, it could be concluded that piping starts when water flows through the embankment body through cracks, cavities, the interface with the buried structures, and/or less compacted layer in a concentrated leak. Water can then remove soil particles until a tunnel is formed through the body of the dam. Water continues to flow through the pipe and erodes more material. As the pipe grows parts of the material forming the downstream face above the pipe become unstable and eventually fall into the pipe and are washed out by water. The erosion of the pipe and slumping of the material continue until the force of the water pressure in the dam exceeds the strength of the material above the pipe or the material above the pipe falls under its own weight. The slumped material is also washed away by the flowing water leaving an opening in the dam body. Then water flows through this opening in a similar way to that described in the overtopping failure mechanism, as shown in Figure (5). The flow into the pipe is calculated by the orifice flow formula as follows:

$$Q_b = A \sqrt{\frac{2g(H - H_P)}{h_L}} \quad \textbf{(6)}$$

where:

Q_b: flow through the pipe $[L^3/S]$

g: gravitational acceleration $[L/S^2]$

A: pipe cross section area $[L^2]$

H: water level in the dam [L]

H_p: pipe centre line elevation [L]

h_L: losses due to friction and contraction [-]

$$h_L = (0.05 + \frac{fL}{D}) \quad \textbf{(7)}$$

where:

D: pipe diameter [L]

L: pipe length [L]

f : Darcy friction factor [-]

5.f. Limitations

The HR BREACH model has the following limitations:

- Composite dams in the model is simulated as only two layers (i.e. outer and core layer)
- Limited selection of erosion formulae
- Assuming uniform erosion along the sides and bottom of the breach

6. Model schematization

To set up the HR BREACH model, it is necessary to define the network of nodes and branches. The network presents the upstream boundary, (Inflow hydrograph), the storage area, (Nasser Lake Lake), the spill unit, (Aswan High Dam), and the downstream boundary, (water levels). The network is shown in Figure (6).

7. Input Data

The boundary conditions were specified to the model. Also the dam data, reservoir data and simulation period were given to the model.

7.a. Boundary Conditions

The developed model is based on the unsteady flow, and it is necessary to provide boundary conditions at the upstream, and the downstream for the model. The following conditions were used in the present investigation:

Up-stream conditions: was applied as the annual incoming flow to Nasser Lake. The inflow was entered to the model in m^3/s. These discharge values were incorporated as time series.

Down-stream conditions: was assumed to be the water level downstream the Aswan High Dam, (upstream the Aswan Old Dam). It varied according to the studied scenario.

7.b. Reservoir Data

Nasser Lake data was entered to the model as a stage-area curve, or water level of reservoir versus its cross section area.

7.c. Dam Data

The dam data were provided to the model as the dam geometry and material properties. The data comprised crest level, length, and width, foundation level, and downstream and upstream slopes. The dam material properties comprised median diameter (d_{50}), porosity, dry unit weight, friction angle, cohesion, shear and tension strength, and Manning coefficient.

7.d. Simulation Period

In the model, One year was considered as a simulation period during each failure scenario. One year was selected by trial and error to be suitable to the annual inflow to Nasser Lake.

8. DAM BREAK MODEL SCENARIOS

For dam break studies, three scenarios were designed to represent three hydrological conditions for the initiation of a dam failure:

- A dam failure could occur under "fair-weather" conditions, i.e. under normal operation condition, due to structural failure, piping under the dam, or any unmanageable external causes.
- A dam failure could also occur under external flow conditions due to climatic events (tropical storms, severe rainfall events) when the critical design water levels are exceeded and structural instability or dam erosion occur.

A dam break event might occur due to overtopping The HR BREACH model predicts the outflow hydrograph from a breached embankment dam. The HR BREACH model gives the outflow hydrograph due to the expected failure of the Aswan High Dam. This hydrograph could be used as a flood model to put emergency plans that might help the decision makers.

9.BREACH ASSUMPTIONS

All dam break studies rely on a number of assumptions in order to define the unknown conditions which are required to perform calculations. These assumptions ensure the results from different simulations to be comparable.

The failures due to overtopping and piping are treated here. The AHD failure by overtopping is obtained when the Nasser Lake Lake water level rises and exceeds the dam crest level causing a flow over the AHD that is not designed for such condition. The assumptions used for breach formation by overtopping depend on the possibility of sabotage that lead to initial breach. The initial breach is developed as soon as the water level reaches the lowest crest level of the AHD.

The AHD failure by piping is obtained when a defect occurs in the vertical curtain, the core, horizontal filter, or/and vertical relief wells that at the toe of the AHD. Also, the piping may be formed due to any destructive action at the galleries. The assumptions used for the initial breach due to any act of sabotage are based on the Egyptian Ministry of Defense.

The power plant of the AHD was built on the right bank with water intake embedded in solid rock. A failure of this structure is not likely to occur and is not considered for the simulation in the present study.

10. DAM BREAK MODEL SCENARIOS

The collected data were incorporated in the designing process of the breaching scenarios. Ten (10) dam break scenarios were designed and are summarized in table (1). These scenarios were simulated.

In first step, the dimensions of potential breach are determined for each failure case based on the assumptions of the initial breach, as well as the inflow hydrograph, and the Nasser Lake content.

The inflow conditions, (upstream boundary of dam break model) are considered to be maximum, average, and minimum flood hydrographs. The head water level at the AHD is set to be at maximum operational level (182.00 m), or normal operational level (175.00), as shown in table (1). Figures (7), and (8) show the inflow hydrographs and downstream water level of the AHD.

11. RISK ASSESSEMENT

This section will assess the risk resulting from the model results due to the simulation of the two modes of breaching (i.e. Overtopping and piping breaching failure modes).

11.a. Overtopping Failure Mode

Regarding the overtopping failure mode (i.e. scenarios 1 to 6) considered a breach in the rock-fill of the dam from the crest to the bottom. The breach characteristics are given in table (2). For the first scenario, the inflow hydrograph considered the hydrograph of year 2002/2003, the initial breach is assumed as 10.0 m width and 21.0 m depth in the rock-fill part of the AHD, The breach developed progressively in 42 hours and reached a depth of 27 m, and a width of 444.0 m, at level 169.0 m+ MSL, (bottom level of the breach). The peak flow reached was 11068.82 m^3/s. Figure (9), shows the outflow hydrograph for scenario 1. The water level of Nasser Lake decreased from level 175.0 m+ MSL, to 169.72 m+ MSL, as shown in Figure (10).

For the second scenario, the inflow hydrograph considered the hydrograph of year 1999/2000, the initial breach is assumed as 10.0 m width and 21.0 m depth in the rock-fill part of the AHD, The breach developed progressively in 51.50 hours and reached a depth of 28.5 m, and a width of 444.0 m, at level 169.0 m+ MSL, (bottom level of the breach). The peak flow was 15473.22 m^3/s. Figure (9), shows the outflow hydrograph for scenario 2. The water level of Nasser Lake decreased from level 175.0 m+ MSL, to 168.19 m+ MSL, as shown in figure (10).

For the third scenario, the inflow hydrograph considered the hydrograph of year 1964/1965, the initial breach was assumed to be 10.0 m width and 21.0 m depth in the rock-fill part of the AHD. The breach developed progressively in 70.5 hours and reached a depth of 32.5 m, and a width of 444.0 m, at level 169.0 m+ MSL, (bottom level of the breach). The peak flow was 29569.42 m^3/s. Figure (9), shows the outflow hydrograph for scenario 3. The water level of Nasser Lake decreased from level 175.0 m+ MSL, to 164.79 m+ MSL, as shown in figure (10).

For the fourth scenario, the inflow hydrograph considered the hydrograph of year 2002/2003, the initial breach is assumed as 10.0 m width, and 14.0 m depth in the rock-fill part of the AHD, The breach developed progressively in 76 hours and reached a depth of 61.80 m, and a width of 666.30 m, at level 134.20 m+ MSL, (bottom level of the breach). The peak flow was 374309.84 m^3/s. Figure

(11), shows the outflow hydrograph for scenario 4. The water level of Nasser Lake decreased from level 182.0 m+ MSL, to 134.50 m+ MSL, as shown in Figure (12).

For the fifth scenario, the inflow hydrograph considered the hydrograph of year 1999/2000, the initial breach is assumed as 10.0 m width, and 14.0 m depth in the rock-fill part of the AHD. The breach developed progressively in 83 hours and reached a depth of 62.10 m, and a width of 666.30 m, at level 133.90 m+ MSL, (bottom level of the breach). The peak flow was 377957.19 m^3/s. Figure (11), shows the outflow hydrograph for scenario 5. The water level of Nasser Lake decreased from level 182.0 m+ MSL, to 134.06 m+ MSL, as shown in Figure (12).

For the sixth scenario, the inflow hydrograph i considered the hydrograph of year 1964/1965, the initial breach is assumed as 10.0 m width, and 14.0 m depth in the rock-fill part of the AHD, The breach developed progressively in 95 hours and reached a depth of 63.04 m, and a width of 666.30 m, at level 132.96 m+ MSL, (bottom level of the breach), The peak flow was 389009.69 m^3/s. Figure (11), shows the outflow hydrograph for scenario 6. The water level of Nasser Lake decreased from level 182.0 m+ MSL, to 133.89 m+ MSL, as shown in Figure (12).

11.b. Piping Failure mode

Scenarios from 7 to 10, considered a breach in the rock-fill dam by piping. The breach properties are given in table (3) and the outflow hydrographs for each scenario are shown in figures (13), and (14).

In cases of piping failure, for the seventh and eighth scenarios, the inflow hydrograph considered the hydrograph of year 1964/1965 as the maximum flood hydrograph. The initial piping diameter is assumed as d50 of the material of the AHD and equal 300 mm, and pipe level is 135 m+ MSL.The breach developed progressively in 3965 hour, for scenario 8, and in 1109.50 hour in scenario 9. The peak flow for scenarios 7 and 8 were 11297.7 and 11905.8 m^3/s, respectively.

For the ninth and tenth scenarios, the inflow hydrograph considered the hydrograph of year 1964/1965 as the maximum flood hydrograph. The initial piping diameter is 6.0 m, and pipe level is 127 m+ MSL. The breach developed progressively in 395, 192.5 hour, respectively. The peak flow for scenarios 9 and 10 were 9245.4 and 30666.1 m^3/s, respectively.

12. CONCLUSIONS

The maximum peak outflow of the AHD failure is 389009.69 m^3/s. This was in case of overtopping failure when, the inflow of hydrograph of year 1964/1965 was considered. The Nasser Lake contents is 162.3 BCM, the assumed initial breach_is 10.0 m width, and 14.0 m depth in the rock-fill part of the AHD. The breach developed progressively in 30 minutes and reached a depth of 63.04 m, and a width of 666.30 m, at level 133.89 m+ MSL,

The minimum water level of Nasser Lake was 133.89 m+ MSL.

The inflow hydrographs had low effect on the AHD failure in comparison to Nasser Lake effect.

13. RECOMMENDATIONS

A Flood Model should further use these outflow hydrographs to simulate the Nile River downstream the AHD, inundations hazards, and risk analysis to assess the impact of such a catastrophic event on the inhabited areas or any urban which might be in the path of the flood surge waves. Future studies might assist engineers in designing protective and measures to assure the safety of the affected areas, and to develop emergency evacuation procedures.

14. LIST OF ABBREVIATIONS

1D	: One dimensional,
AHD	: Aswan High Dam,
AOD	: Old Aswan Dam.
BCM	: Billion Cubic Meters,
d_{50}	: Median grain size of sediment,
MCM	: Million Cubic Meters, and
MSL	: Mean Sea Level.

15. ACKNOWLEDGMENTS

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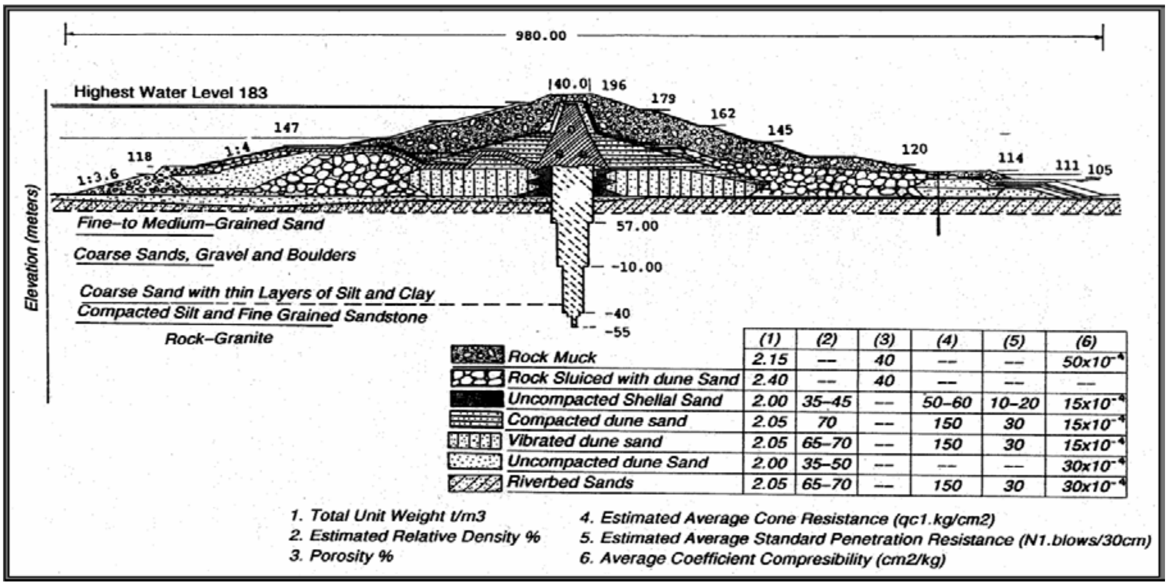

Figure (1): The Aswan High Dam Cross section, Abdel Azim Abul-Atta, [1]

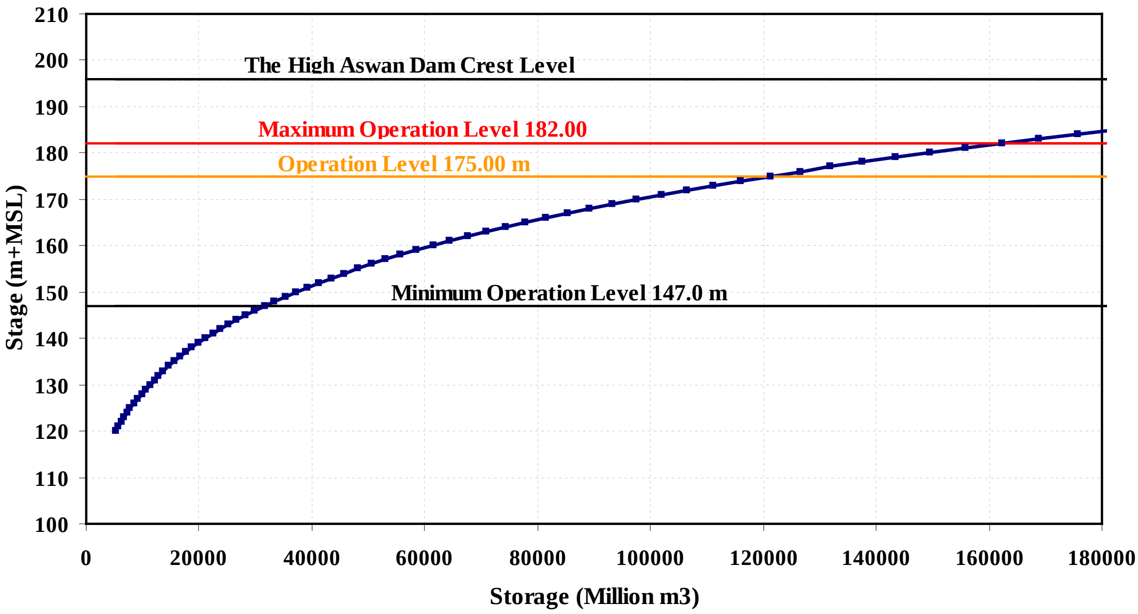

Figure (2): Nasser Lake Elevation-storage curve

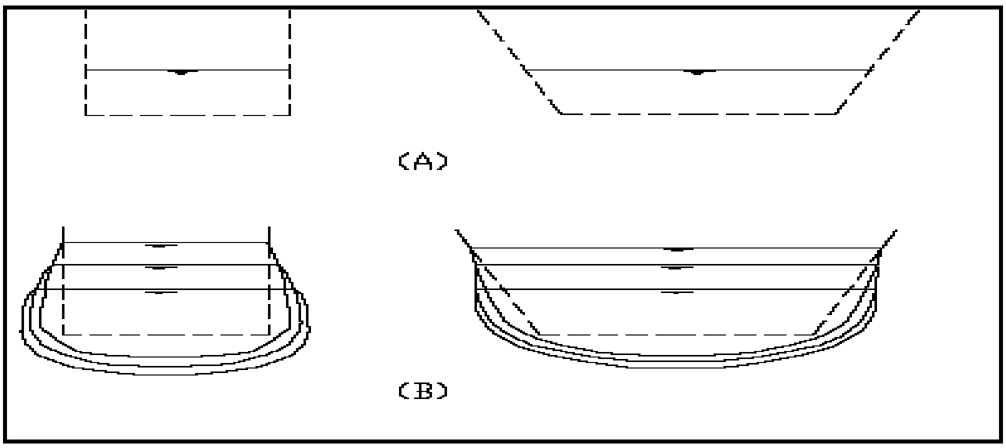

Figure (3): a. initial breach shape

b. Hypothetical breach shape, M. A. Mohamed, and et. al [5]

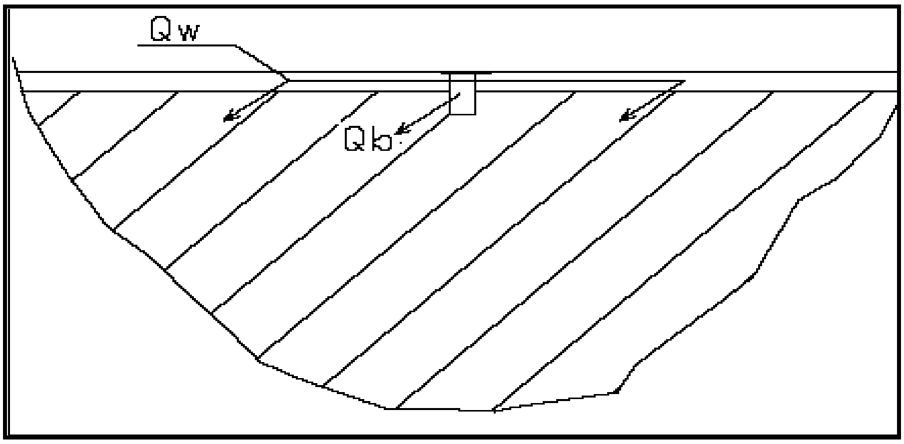

Figure (4): Flow computations through the breach and over the dam, M. A. Mohamed, and et. al [5

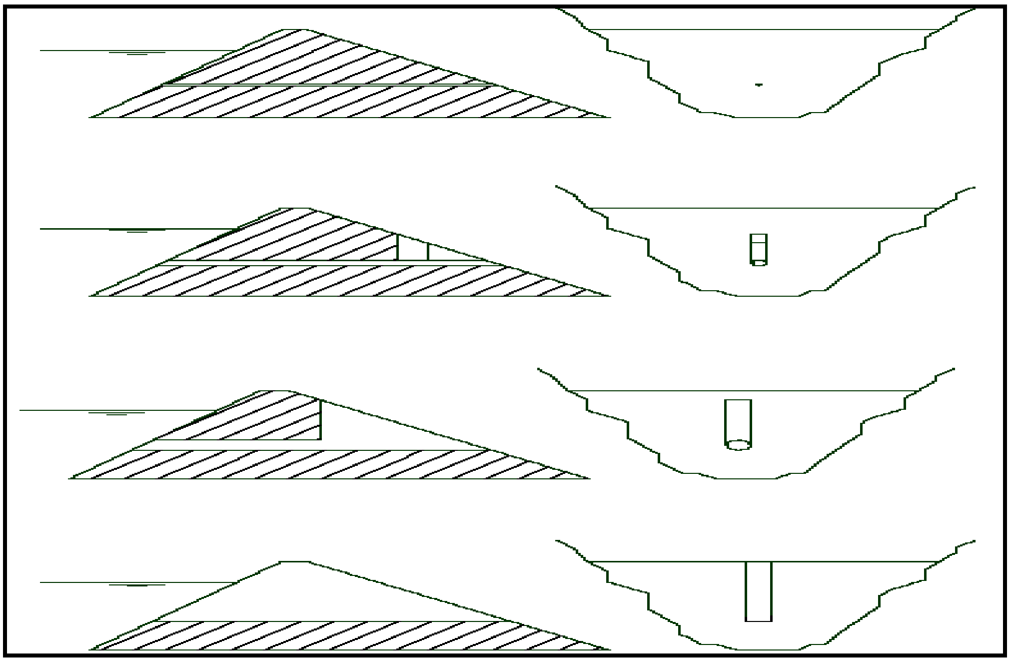

Figure (5): Mechanism of piping failure, M. A. Mohamed, and et. al [5

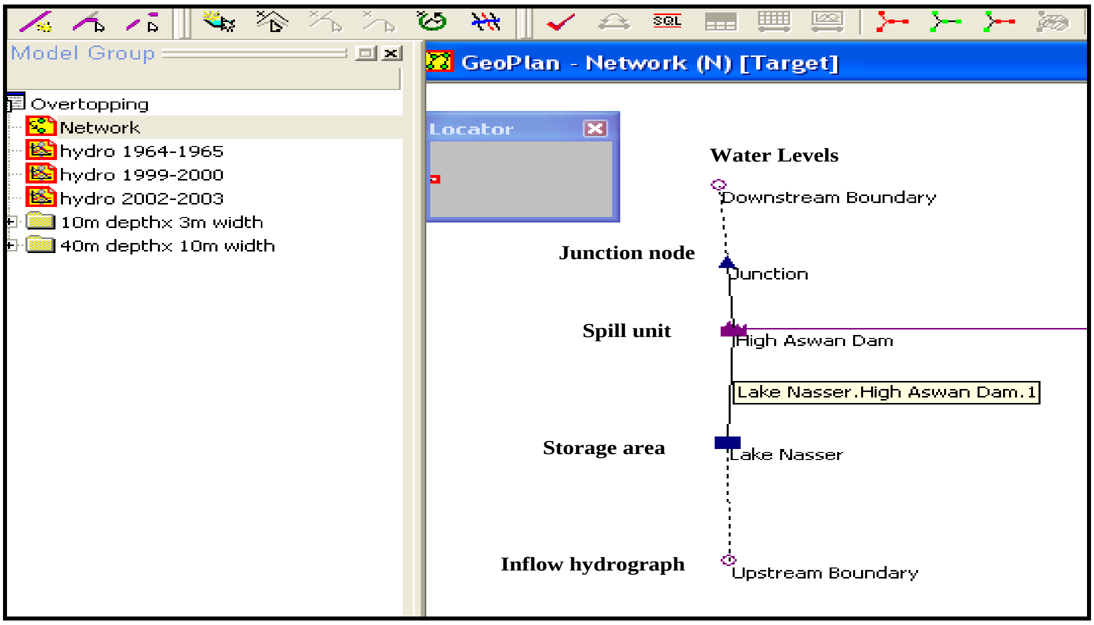

Figure (6): The network at the HR BREACH model

Table (1): Designed scenarios of the Aswan High Dam Breach

No	Failure Mode	Initial Breach (m)	Flood type / year	Flood inflow (Billion m^3/year)	Nasser Lake Level (m)	Nasser Lake Contents (Billion m^3)
1	Overtopping	10.0 m x 21.0 m	Minimum, *Year (2002-2003)*	41.79	Normal, (175.00)	121.3
2		10.0 m x 21.0 m	Average, *Year (1999-2000)*	81.45	Normal, (175.00)	121.3
3		10.0 m x 21.0 m	Maximum, *Year (1964-1965)*	119.08	Normal, (175.00)	121.3
4		10.0 m x 14.0 m	Minimum, *Year (2002-2003)*	41.79	Maximum, (182.00)	162.3
5		10.0 m x 14.0 m	Average, *Year (1999-2000)*	81.45	Maximum, (182.00)	162.3
6		10.0 m x 14.0 m	Maximum, *Year (1964-1965)*	119.08	Maximum, (182.00)	162.3
7	Piping	0.3 m Pipe diameter	Maximum, *Year (1964-1965)*	119.08	Normal, (175.00)	121.3
8		0.3 m Pipe diameter	Maximum, *Year (1964-1965)*	119.08	Maximum, (182.00)	162.3
9		6.0 m Pipe diameter	Maximum, *Year (1964-1965)*	119.08	Normal, (175.00)	121.3
10		6.0 m Pipe diameter	Maximum, *Year (1964-1965)*	119.08	Maximum, (182.00)	162.3

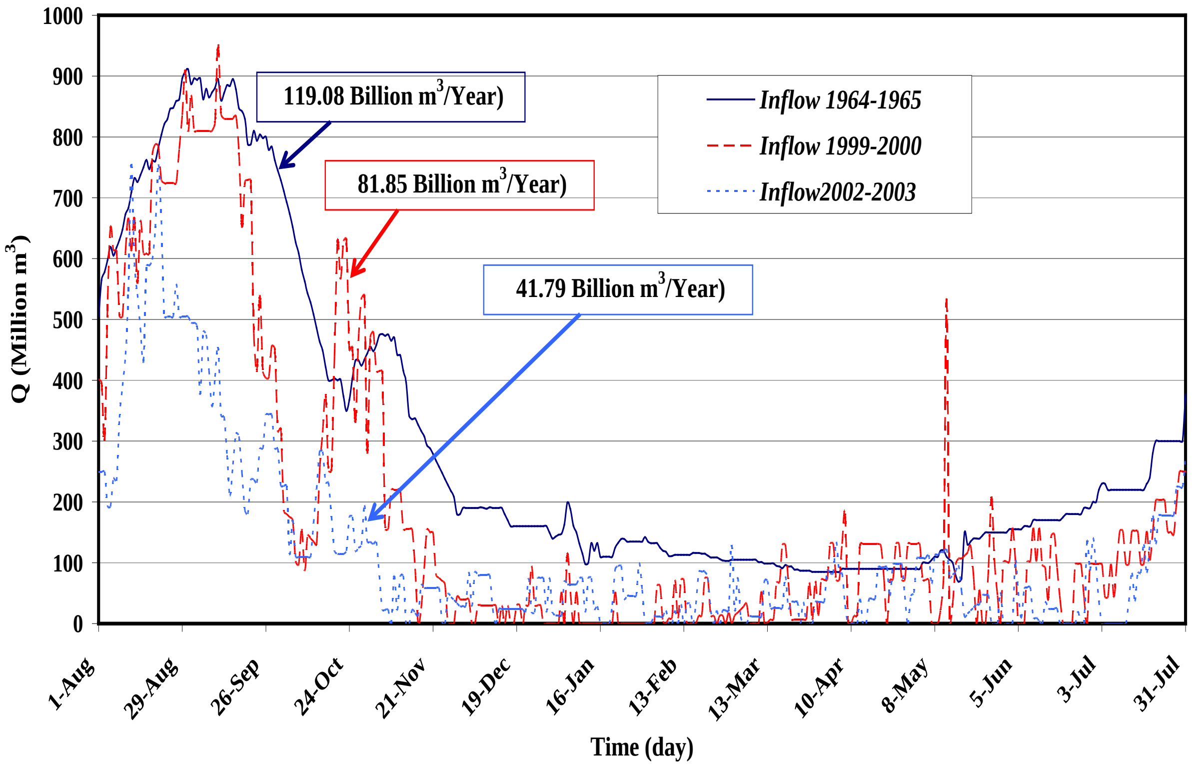

Figure (7): Inflow hydrographs of the Nasser Lake

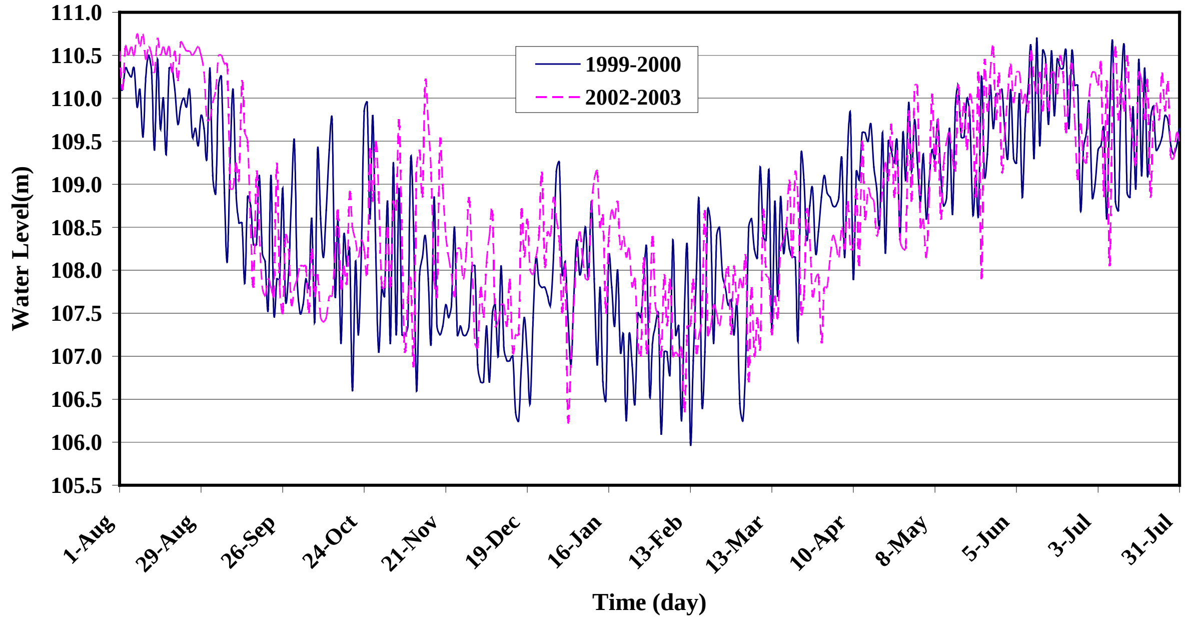

Figure (8): Downstream water level of the AHD

Table (2): Overtopping breach Characteristics

Scenario No.	**Failure Mode**	**Peak Outflow** (m^3/s)	**Breach depth** (m)	**Breach width** (m)	**Lake water Level** (m + MSL)	**Formation time** (hours)
1	**Overtopping**	11068.82	26.28	333.30	169.72	42.00
2		15473.22	27.81	380.60	168.19	51.50
3		29569.42	31.21	444.55	164.79	70.50
4		374309.84	61.50	490.50	134.50	76.00
5		377957.19	61.94	580.00	134.06	83.00
6		389009.69	62.11	666.30	133.89	95.00

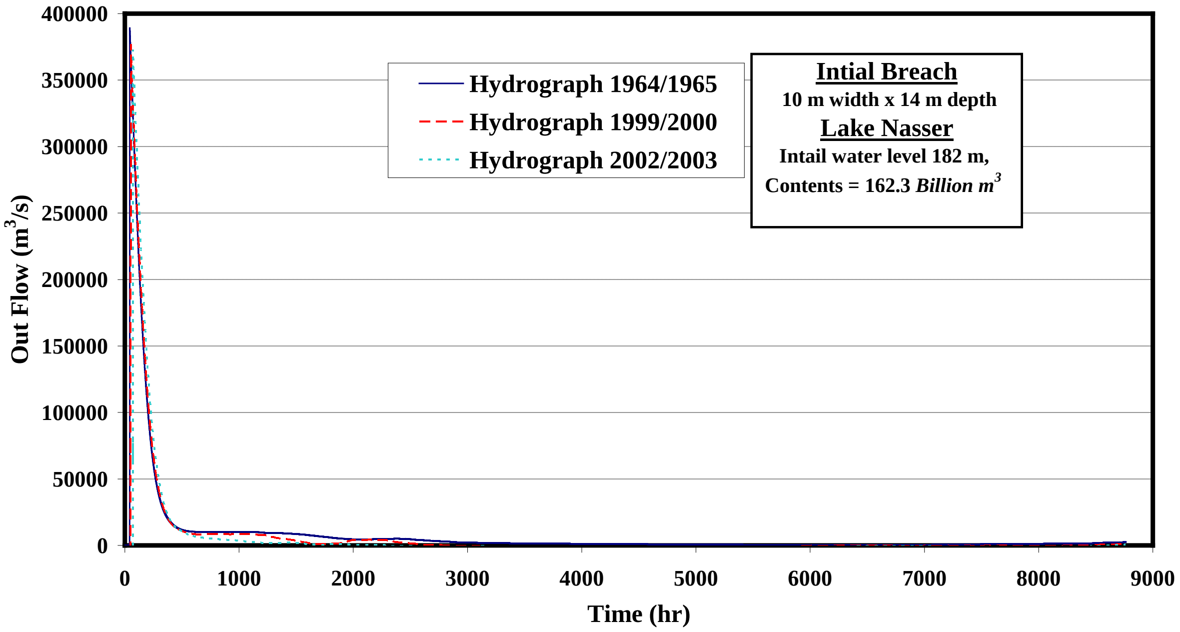

Figure (9): Outflow hydrograph of scenarios No. 1, 2, and 3

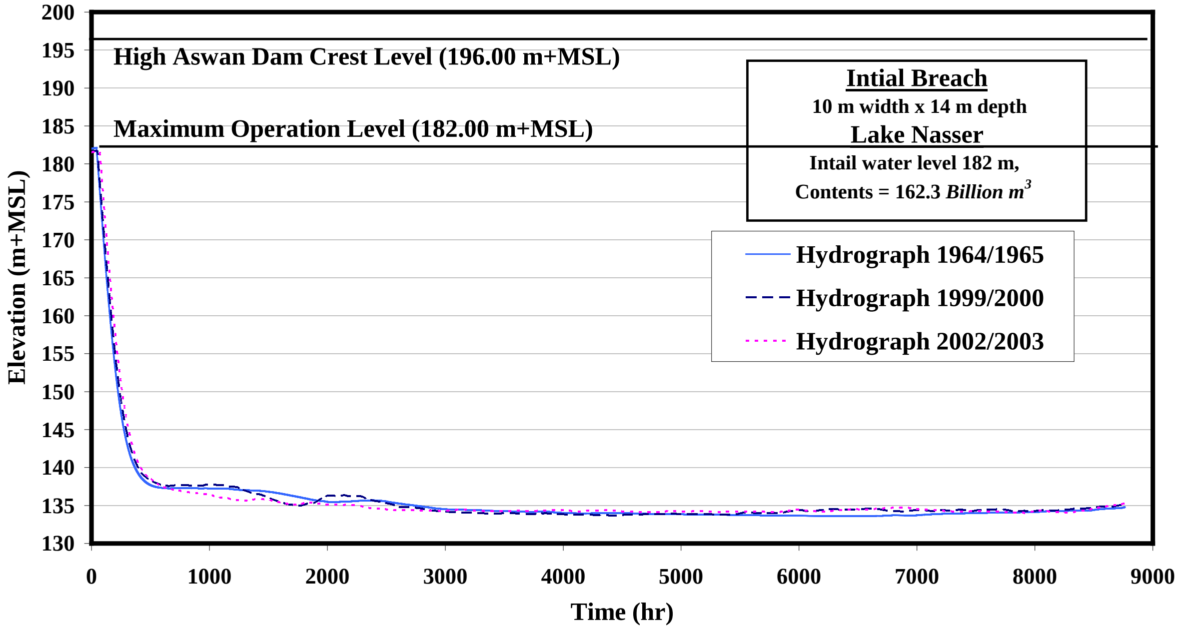

Figure (10): Water level of the Nasser Lake for scenarios No. 1, 2, and 3

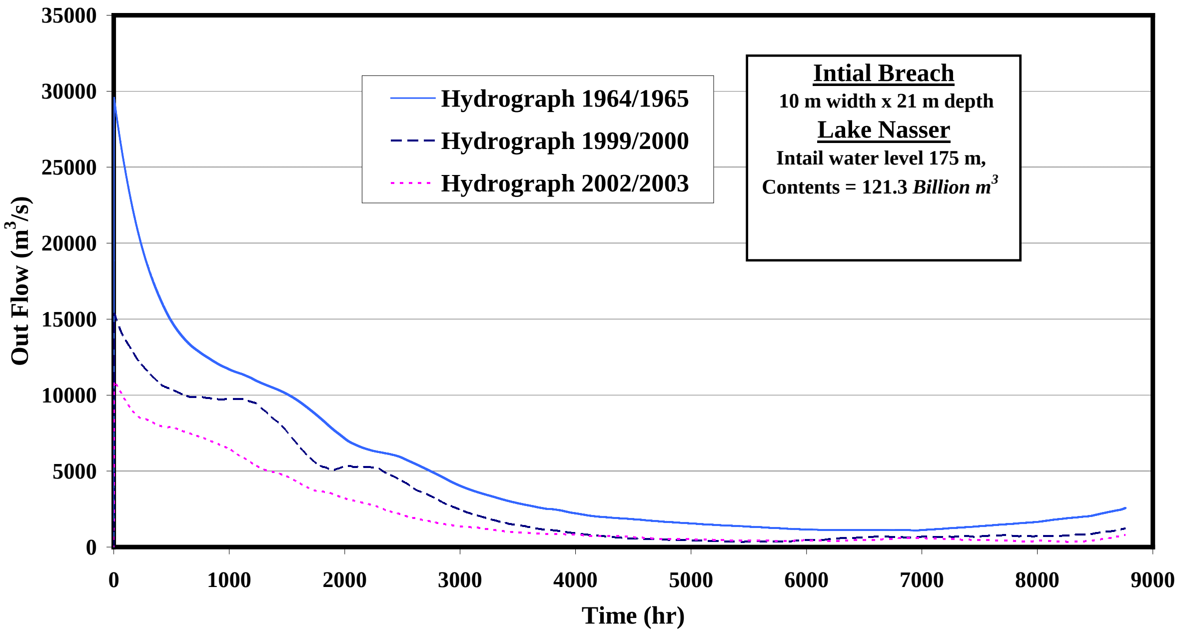

Figure (11): Outflow hydrograph of scenarios No. 4, 5, and 6

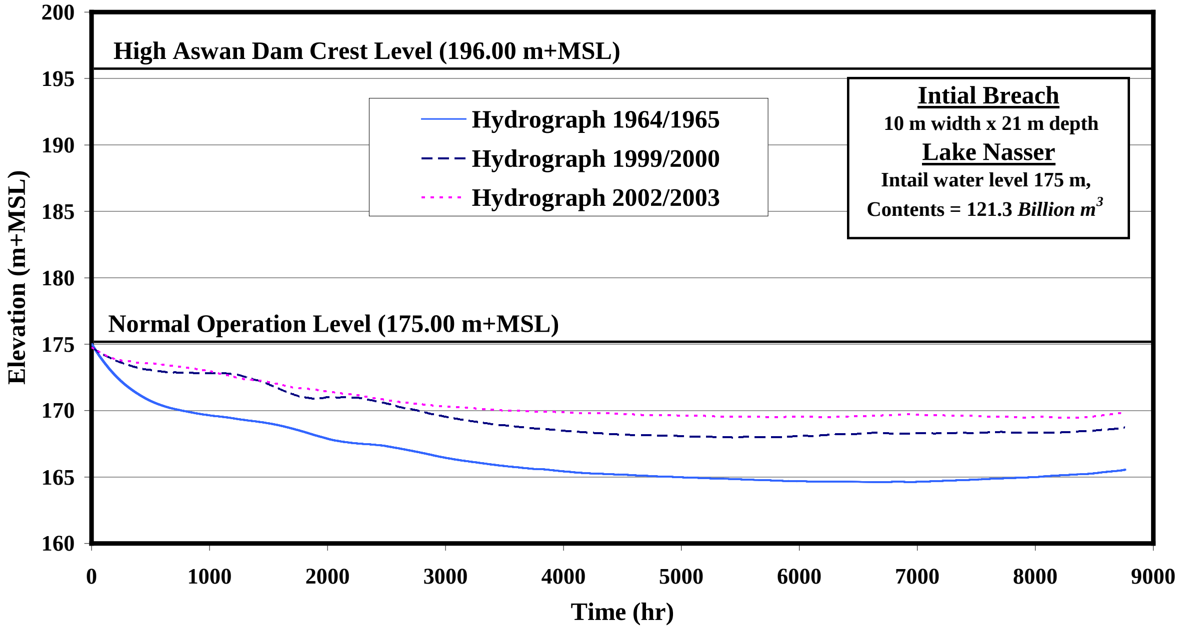

Figure (12): Water level of Nasser Lake for scenarios No. 4, 5, and 6

Table (3): Piping breach Characteristics

Scenario No.	**Failure Mode**	**Nasser Lake Level** (m+ MSL)	**Peak Outflow** (m^3/s)	**Formation time** (hour)
7	**Piping**	175.00	11297.7	3965.00
8		182.00	11905.8	1109.50
9		175.00	9245.4	395.00
10		182.00	30666.1	192.50

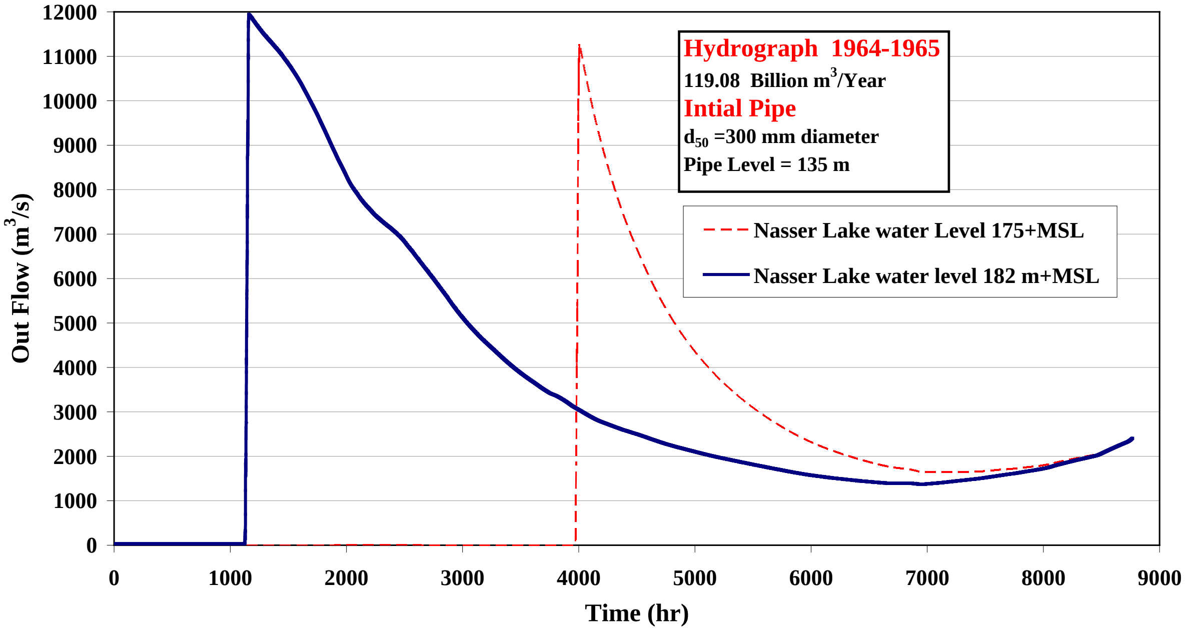

Figure (13): Outflow hydrograph of scenario No. 7, and 8

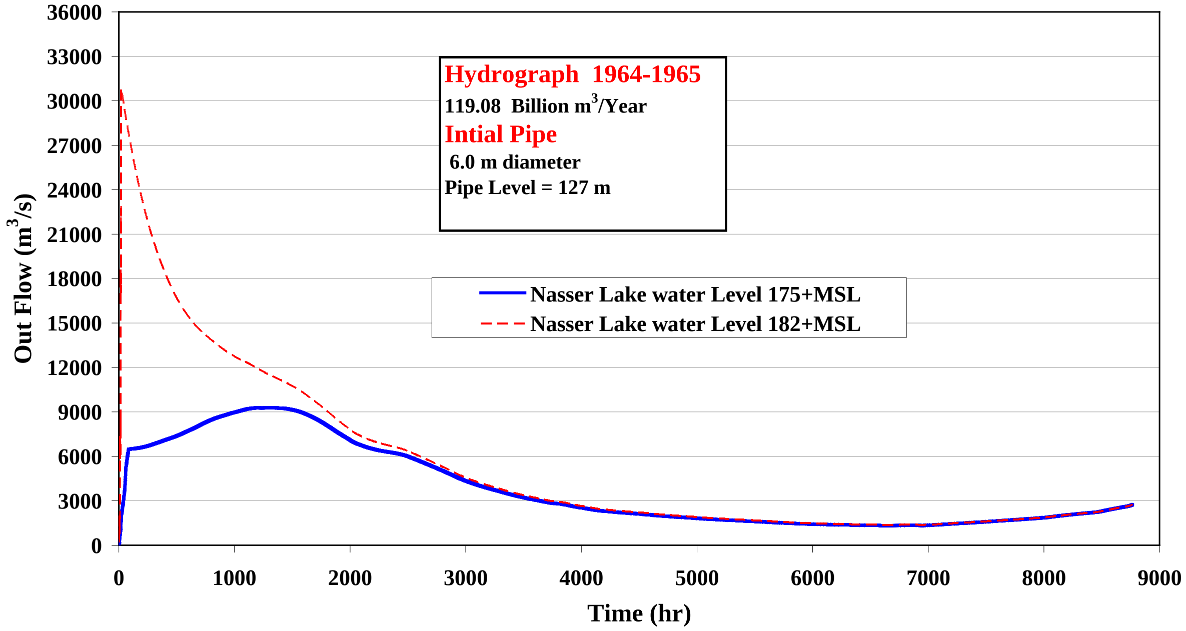

Figure (14): Outflow hydrograph of scenario No. 9, and 10