PREDICTION OF "LIPTOS" LIBDA DAM FAILURE AND BREACH'S PARAMETERS

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ABSTRACT

Behavior of dam break is very complicated process and it cannot be foreseen exactly. Many researchers have studied the problem of dam break by predicting the behavior and the propagation of a flood wave using different approximations.

In this study, Libda dam is taken as a case study. Collected data about Libda dam is included in the current work. Dam's break has been performed using a mathematical modeling as a tool to predict and simulate the dam break problem. Two major mathematical models have been employed namely, BREACH and FLDWAV for both predicting failure parameters, and simulation of flow routing after the break of the dam.

In this paper, hypothesizing a mode of failure, selecting or assuming parameters which directly affect the magnitude of resultant dam break flood such as breach dimensions and the time for breach development, were made. Piping failure is considered the most predictable one for Libda dam; also, the dam break causes a potential danger towards ancient roman town of Leptis Magna, since flooding occurs in the city for all assumed scenarios.

Keywords: Dam failure's behavior, Libda dam, piping failure.

INRODUCTION:

Floods due to failure of dams induce widespread damages to life and property owing to its high magnitude and in predictable sudden occurrence. Such flood is required to be simulated to determine the inundated area, flood depth and travel time of the flood waves so that adequate safety measures can be provided. The review of the past works revealed that dam break problem remains a topic of continued interest since Ritter (1892) attempted its first analytical solution for a horizontal frictionless rectangular channel.

The objective of this paper is to clarify the dam break problem, where Libda dam (Libya) is chosen as case study. The present work includes: prediction of Libda dam failure, and breach's parameters.

Although computation of dam break flood has been a topic of interest for more than hundred years, numerical simulation of dam break flow in relatively simple channels is found more often compared to real river flood simulation. Natural channels with steep slopes and wide flood plains offer numerous complexities and make the computation very challenging.

The review of the past works reveals that dam break problem remains a topic of continued interest since 1892 which started with simple cases such as rectangular frictionless channels by Ritter's, till date for mathematical simulation of dam break flood with complex channels and floodplains e. g., one Dimensional simulation models (Hicks F.E. et al (1997), Sanders B. F.(2001), two Dimensional simulation models Akanbi, A. A. et al (1988), Zhao D.H. et al (1996), Sharma, AK (1999), Zoppou .C. and Roberts S. (2000). Simulating dam break flood in natural channel is examined through conservative and neoconservative formulations of the unsteady flow in a hypothetical dam break situation occurring due to failure of a proposed dam on the Dibang River, a Himalayan tributary of the river Brahmaputra. The real non-prismatic channel of the river has been considered. The stability and accuracy of the numerical solutions for both conservative and non-conservative formulations are analyzed, and they are compared in terms of water depths in supercritical and mixed flow conditions with experimental data Bellos (1990), respectively. The numerical investigation includes first order Diffusive scheme, second order modified two-step predictor corrector F.D. scheme with both conservative and non-conservative formulations. According to local studies, two dams were studied using dam break analysis namely Derna dam (2001) and AL Qattara dam (2004).

CASE STUDY "DAM OF LIBDA"

Wadi Libda dam is located 135 km east of Tripoli, Libya, The dam site can be reached by passing 3.5 km long road, to the south from the Tripoli--Misurata highway. The mouth of the Wadi exists at approximately 3.0 km to the southeast from the town of Khoms, On the western part of Wadi Libda bed about 1 km from the Sea, the ancient Roman town of Leptis Magna is located. Figure (1).

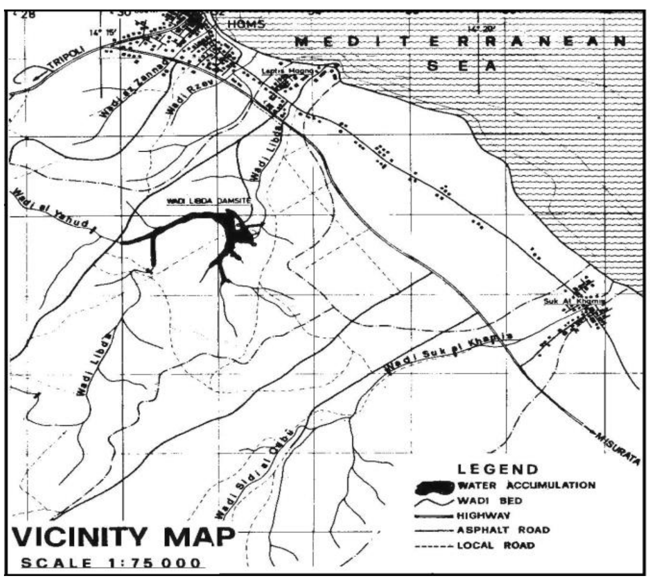


Fig. 1 Site of Wadi Libda Dam

It is an earthen storage dam. The crest width is 8.0 m. The carriageway is 6.0 m and has a one-sided cross fall of 1% towards the downstream slope. On the upstream side of the crest, a reinforced concrete parapet 1.20 m high is provided. The drainage system consisted of vertical and horizontal drainages. The vertical drainage was connected at the dam base to the horizontal drainage that was placed between the cutoff and the downstream dam toe. The horizontal drainage outfalls in the pipe drain at the downstream dam toe. Chute and side channel spillways were the two types of spillways that part of the dam.

Wadi Libda Project area covers about 4300 ha, the altitude ranges from 12 to 50 m above see mean level. The surveyed area is situated in the valleys of Wadi Libda. It is slightly hilly, sparsely afforested, severely eroded and cut by valleys. The bed of the Wadi and its tributaries are dry during the most suitable part of the year where water flows only in case of seasonal rains, which are often and can cause considerable erosion. Most of the rainfall occurs in winter (November-February) when evaporation is small. No rock falls or rockslides were observed in the region.

MAXIMAL FLOW DISCHARGES

Due to lack of measurements of water levels and discharges, indirect methods were used to determine maximum discharge and volumes of flood waves for different probabilities of occurrences (unit hydrograph, and Alekseev's formula). For both methods, data base on intensity and distribution of rainfall, as well as natural properties of the catchment area were used.
The dam was designed for an overflow of floods with probability of occurrence every 10 years plus 10% for safety and the spillway with the chute is designed for a capacity of: Normal conditions P = 0.1%; Q = 492 m^3/sec; W = 9.534x10^6 m^3, and at exceptional conditions; P = 0.01 and Q = 689m^3/sec.
Unit hydrograph method:

Probability of occurrence (P) %	0.01	0.1	1.0	10.0
Discharge (Q) m^3/sec	553	394	238	103
Volume of flood (W) m^3 x 10^6	10.78	7.47	4.39	1.95

Alekseev's Formula:

Probability of occurrence (P) %	0.01	0.1	1.0	10.0
Discharge (Q) m^3/sec	698	314	192	144
Volume of flood (W) m^3 x 10^6	12.84	6.42	4.37	3.50

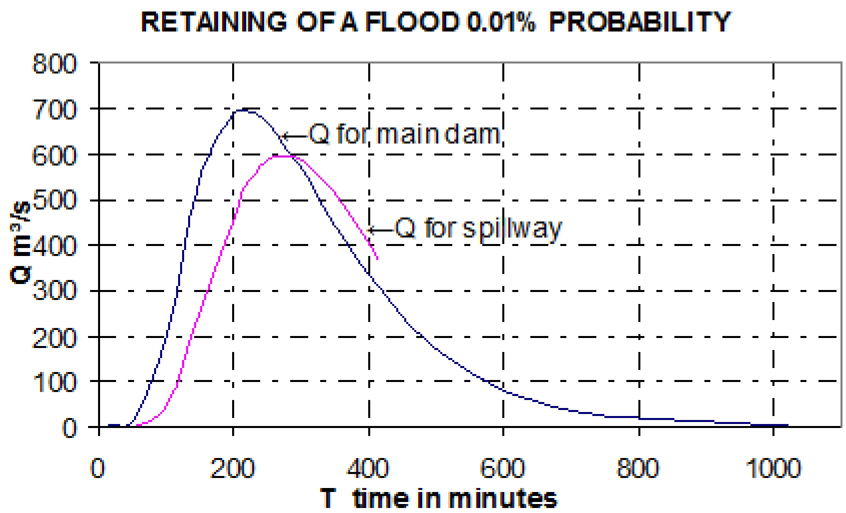


Fig. 2 Flood hydrograph for main dam and spillway 0.01% probability (Final Report-1982)

MAXIMAL FLOOD WAVES

Maximal flood waves are given by Alekseev's formula is 25 to 30% higher than the values obtained based on unit hydrograph.

Although both methods are affected by the problem of insufficient recording and particularly of lack of data on the intensity of precipitations, however, higher values for discharges do not influence relatively the constructional costs, but give higher security, particularly in a situation when note a single measurement maximal floodwater was done. For this case of study, the dam was assumed to be under case of Class (I) which states that the failure of the dam or any structure could result in the loss of human life.

BREACH PREDICTION

Dam's spillway was designed to accept maximum probable flood discharge of 689 m³/s, that corresponds to a 29.0 m height of water behind dam, this elevation is much smaller than the top elevation of dam (38.83m) which makes the overtopping impossible, and piping failure is considered. The shape and size of failure are unknown. Froehlich's equations were used to predict the breach parameters

$$b = 9.\times 0.7\left[V_r \times h_d\right]^{0.25} \tag{1}$$

$$\tau = (0.59 \times V_r^{0.47})/(h_d^{0.9}) \tag{2}$$

Where:
b = final breach bottom width (ft.) τ = time of failure (hr.) V_r = volume of reservoir (Acre-ft.) h_d = height of the water behind dam (ft.). From the geometry of the dam and reservoir, volume of reservoir corresponding to different elevations is used to get time of failure and final breach bottom width

Table 1 Prediction using Froehlich's equations

h_d (m)	V_r (10^3 m^3)	τ (hr.)	b (m)
22	60×10^3	2.0	26.66
23	380×10^3	4.57	24.90
24	475×10^3	4.89	45.85

BREACH MATHEMATICAL MODEL

BREACH mathematical model was used to simulate piping failures for different piping elevations and constant inflow to the reservoir equals 698m^3/s. Several runs were made, each with certain initial piping elevation, figures (3 and 4).

H_b = initial elevation of piping center (m), Q_P= peak outflow through the breach (m3/s), h_{bm} = final elevation of breach's bottom (m), τ = time from starting breach to the final breach formation - time of failure (hr.), b = final breach bottom width (m), Z = final breach side slope (m/m)

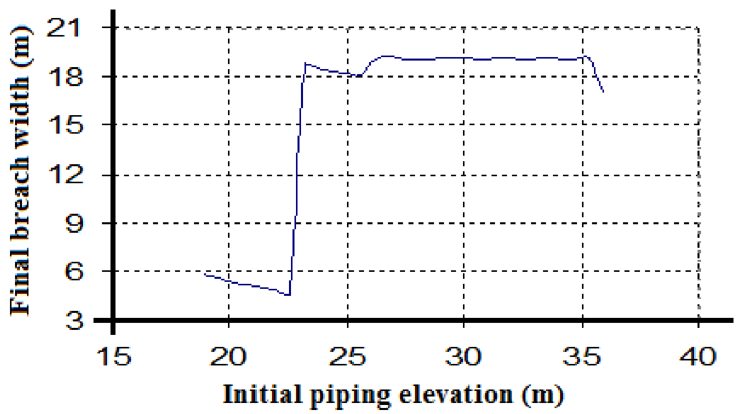
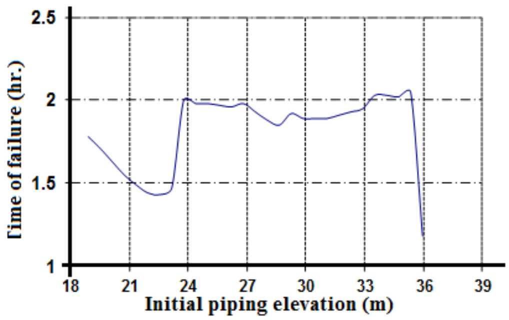


Fig. 3 Initial piping elevation and breach width.

Figure 3 show that final breach width reached a nearly constant value when the initial piping elevation was greater than 25m, with max. Breach width of 19.4m. Figure 4 shows, for 18.9 m elevation with time of failure is 1.78 hr., then curve decreased to 1.44 hr. for 22.0 m elevation, and after elevation 24.0m the time of failure is approximately constant, curve drops and time of failure is decreased as initial elevation increased, last point on the curve is of time equals 1.18 hr. at elevation of 35.96 m.



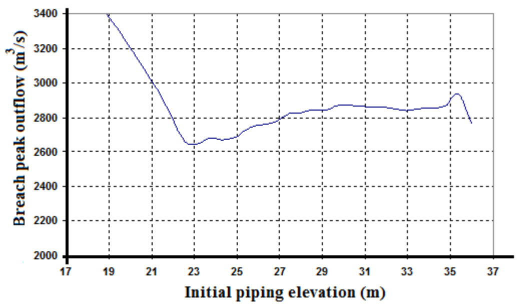


Fig. 4 Initial piping elevation flow through breach

Fig. 5 Initial piping elevation and peak and time of failure

Figure 5 shows the relation between initial piping elevations and breach peak outflows with an average of 2934.13 m^3/s and depths between 25m to 35m.

ADOPTING BREACH'S PARAMETERS

From the proceeding trends, obtained values can help in confining the ranges from which parameters are adopted. For the time of failure, the minimum obtained value from BREACH model runs is 1.43hr, and the maximum time's 3.6hr, minimum value is close to lower recommended limit within FLDWAV manual (0.10 hr), where maximum value is less than that obtained from Froehlich's equations (2.0 hr.). For the final breach bottom width, the minimum value from BREACH model is equal to 9.51m, and maximum value is equal to 19.26m. Values obtained from Froehlich's equations are more than the actual geometry of dam at bottom.

From BREACH model runs the following parameters are adopted, minimum time of failure = 1.2 hr. Average time of failure=2.6 hr. Maximum time of failure = 4.6 hr. Minimum final bottom width = 7.0 m. Average final bottom width =13.50 m. Maximum final bottom width =20.0 m. Breach side slope (shape factor) is equal to 0, 1, or 1.3. Final bottom elevation of breach = 17.40m (this elevation is near the dam's bottom and that produce breach with maximum height).

FLDWAV APPLICATION

The FLDWAV model will route the outflow hydrograph through the channel valley and determine the water-surface elevations, discharges, maximum velocities, and travel times along the routing reach. Hydrographs and peak flow and water-surface elevation profiles are generated using this model. Many parameters are used to define variant routing conditions; these parameters are defined in 86 data groups.

UPSTREAM BOUNDARY

The upstream boundary condition is normally a discharge hydrograph. The upstream boundary location must be identified for each dynamic river. This location should be far enough upstream (specified at each case of study) where the influence of downstream backwater conditions is not felt (Sylvester, 2002). In current study, the inflow hydrograph at Wadi Libda Dam is shifted to the upstream location of the watercourse.

DOWNSTREAM BOUNDARY

The downstream boundary condition on the main river must be reproducible in the forecast model. A typical downstream boundary condition in FLDWAV is the generated rating curve. Under some backwater condition (e.g., backwater from a downstream river), the rating curve is not adequate to represent the stage-discharge relationship, therefore, the downstream boundary is moved far enough away until it has no influence on the last point of interest. The final reach may be either a fictitious reach manipulated to produce the best results at the last point of interest.

CROSS SECTIONS

Cross sections may be of regular or irregular geometrical shape. Each cross section is described by tabular values of channel top width (B_i) and water surface elevation (h_i) which constitute a piece-wise linear relationship.

The start is assumed at the most downstream point (the intersection of the Wadi course with sea 0+00), and it increase as the measurement goes upstream, last cross section location has a distance of 04+617km it is the most upstream location where the main dam is located at same distance.
Twelve locations are chosen to represent the course geometry. Cross sections of 12 locations are obtained from contour map.

FLDWAV model is a wet one (there must be certain minimum flow, it cannot be routed dry), so two additional top-width/elevation pairs were added at each cross section to make sure that all sections have at least small amount of flow (wetted section).

MANNING ROUGHNESS COEFFICIENT (N)

The roughness coefficient (n) can be evaluated directly by discharge and stage measurements for a known cross-section and slope. However, this information is rarely available, and it is necessary to rely on documented values obtained from similar channels (Chadwick and Morfett, 1998). For natural channels the estimates are likely to be rather less accurate. In such cases, a suitably conservative value is normally adopted.

The Manning roughness is user-defined for each channel reach between user-specified cross-sections. The Manning n is user-specified as a function of either stage or discharge according to a piece-wise linear relationship with both n and the independent variable (h or Q) user-specified in FLDWAV in tabular form. Linear interpolation is used to obtain n for values of h or Q intermediate to the tabular values (Fred and Lewis, 1998).

RUNNING FLDWAV OPERATIONALLY

In all data files, global groups defining the system in whole are common, different simulations are done by changing values in data groups, which are defining breach parameters, or roughness. Default tolerances are used, discharge tolerances are equated to 2.831m^3/s, where stage tolerances are equated to 0.003m, weighting factors in finite difference technique are equated to 0.6 as recommended by FLDWAV manual. Minimum 10m% is assumed as the base flow before failure (note that this value is ignored gradually within the model as the failure initiated and stopped when the breach reaches one quarter of its final size).
Each failure scenario is formed from running the model with a unique combination of several parameters, all scenarios have been assumed to occur with the maximum inflow of constant value

(698m3/s), initial water surface elevation in the reservoir (behind dam) is at the crest elevation of dam's spillway (35.5m), and failure defined to occur as elevation of water reach (36m). In the following paragraphs these scenarios are described.

SIMULATION OF STEADY STATE

Scenario's runs are performed to simulate steady flows running in the watercourse for different roughness coefficients. This simulation represents the steady flow conditions occurred due to the maximum discharge flow at the most upstream location without the occurrence of dam failure. Dam failure is prevented to occur by defining the required elevation to initiate the breach formation is high enough (30.0m). The consequence of steady state occurrence is as following: since the inflow discharge into the reservoir is more than that the minimum base flow, that will cause water surface elevation to be raised (the stored water is increased). But as water surface is rising, more water head will be available over the spillway's crest which resulting in increasing the overflow discharge through the spillway. This process will continue until balance between the inflow and outflows exist, and flows and stages in each location are constants.

Several runs are done each with constant roughness value. Stages at twelve different locations are recorded. The effects of changing watercourse roughness on stages at the twelve locations are represented by histograms, which are drawn in Figure 6.

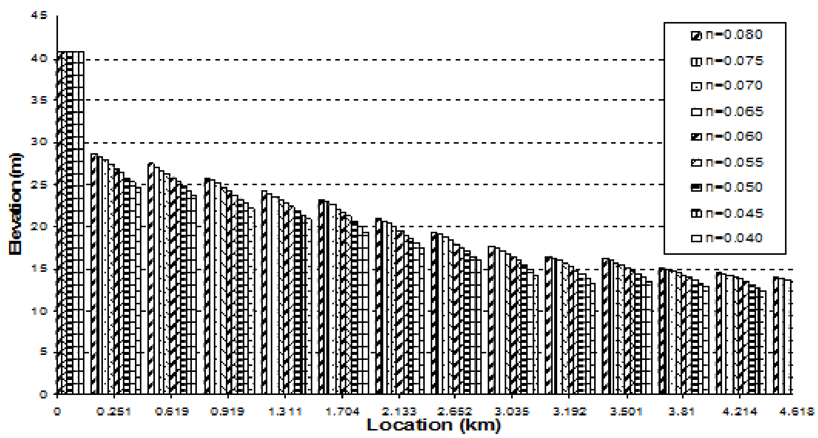


Fig.6 The variation of elevation for different roughness

Figure 6 shows that the variations in the water surface elevation at the same location using different roughness coefficients are very small at dam location section but the variations are increased in other sections. It is shown also that water surface elevations increase as the values of Manning roughness coefficients are increased.

Figure 7 and 8 illustrate the hydrograph at dam's location and at different sections. It's clear that with different roughness coefficients, the elevation of water surfaces at the location of the dam stayed nearly constant and for other sections varied.

For the downstream reaches, flooding occurred at some locations where water surface raised more than the maximum channel depth, therefore improving the channel geometry by cutting, straightening or deepen channel training course is required.

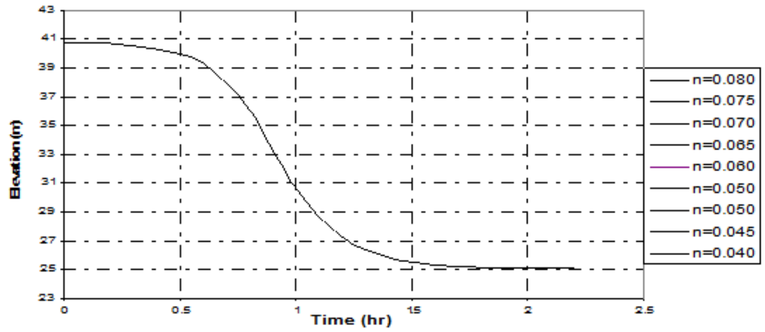


Fig. 7 Hydrograph at dam's location

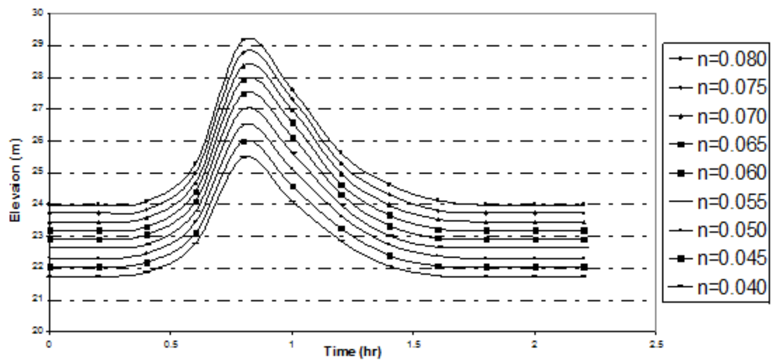


Fig. 8 Hydrograph at second section

INITIAL PIPING ELEVATION EFFECT

Dam failure is considered to occur with different initial piping elevations. The effect of changing initial piping elevation on peak breach outflows and its time of occurrence is studied. Three Manning roughness coefficients (n = 0.050, 0.060, and 0.070) are used in performing all scenario's runs of this study. Three main simulation cases are considered, each with constant Manning coefficient and with average breach parameters (τ= 2.6 hr., b = 13.5m, and Z = 0.64). Peak breach outflows with its time of occurrence for each run are recorded and tabulated in Table 2

Table 2 Peak breach outflows and time of occurrence For different initial piping elevations

h_p **(ft)**	h_p **(m)**	**n=0.05**		**n=0.06**		**n=0.07**	
		τ(hr)	**Q**(m^3/s)	τ(hr)	**Q**(m^3/s)	τ(hr)	**Q(m^3/s)**
62	18.8976	1.78	3395.851	1.95	3125.395	2.12	2900.138
66	20.1168	1.63	3180.987	1.77	2971.164	1.92	2792.805
70	21.336	1.49	2950.406	1.53	2782.667	1.64	2653.244
74	22.5552	1.43	2623.310	1.35	2541.578	1.37	2454.013
78	23.7744	2.00	2350.985	2.20	2377.209	1.95	2380.296
82	24.9936	2.80	2228.812	2.66	2279.392	2.56	2312.923
86	26.2128	1.96	2755.706	1.82	1511.722	2.24	1689.146
90	27.432	1.93	2820.644	1.91	2832.566	1.91	2832.510
94	28.6512	1.85	2845.367	1.85	2844.376	1.79	2841.289
98	29.8704	1.89	2873.574	1.85	2868.901	1.87	2880.314
102	31.0896	1.85	2863.039	1.90	2869.014	1.91	2867.315
106	32.3088	1.93	2848.284	1.92	2867.91o	1.93	2850.012
110	33.528	2.03	2850.578	2.02	2857.092	1.98	2852.249
114	34.750	2.02	2865.389	2.02	2861.481	2.02	2864.823

To declare the effect of changing initial piping elevation on the peak breach outflows and also the time failure, two histograms are drawn, the first histogram is to show the initial piping elevation versus the resulted outflow peaks, where the second one shows time failure due to the initial piping different elevations, Figures (10, and 10). It can be seen that using of different roughness coefficients for the same initial piping elevation had made some effects on peak breach outflows and time of failure. Also, one important conclusion can be reached in which that the piping failure at location near the mid height of the dam (at elevation 27m) is to be the most critical failure case due to the large peak outflow (2880 m3/s), and the minimum time of failure occurred at elevation 23m.

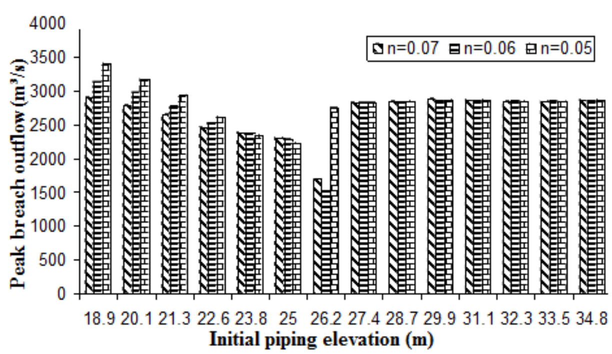


Fig. 9 Initial piping elevation with peak breach outflows.

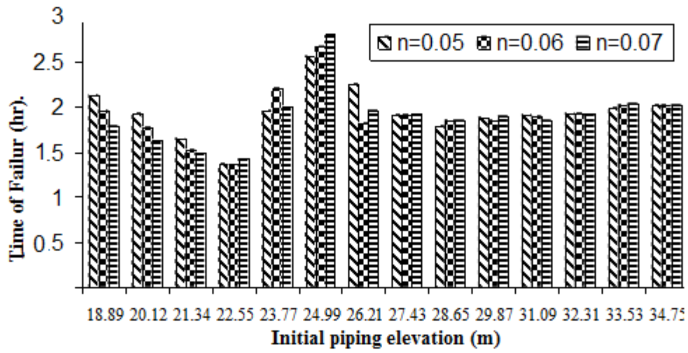


Fig. 10 Initial piping elevation with time failure.

CONCLUSIONS

In this work Wadi Libda, Libya was taken as case of study. BREACH and FLDWAV (flood wave) were applied for the simulating failure of the dam. BREACH model was used to simulate earth dam failure duo to either piping or overtopping. FLDWAV model was used in the study for dam break simulation. The predominant mechanism of the breaching of the studied dam was selected duo to piping phenomenal .Mathematical parameters such as breach characteristics and the time of failure were obtained as output values from BREACH model, and were used as inputs data for the FLDWAV. Topographic contour map (1/10000) was used to determine the cross-section channel of the Wadi fixed roughness coefficient values were assumed in the simulation. In all scenarios the following constants are used, the maximum probable floods for the dam and spillways as 698m³/s and 594m³/s respectively, the base flow at the dam and downstream channel is assumed 10m³/s, and initial water surface elevation of the reservoir was taken as 36.0 m.
Two simulation running cases were performed, steady and unsteady state flow conditions. Steady state case represents the conditions due to the maximum discharge flow at the most upstream location without the occurrence of dam failure. The flow in the downstream section (the reach from location of the dam to its intersection with the sea) is characterized as non-uniform flow. The stage increases as the value of roughness coefficient decreases along the downstream channel.
Flow at the most downstream locations considered as unsteady state. Piping failure occurred at 19.0m (a.m.s.l).

The most critical failure case is duo to the large peak outflow nearly 3250m³/s and the time of failure of 1.25hr (shortest) that occurred at piping elevation of 22.5m (a.m.s.l). Nearly at the mid height of the dam, the change of initial piping elevation has a little effect on the breach outflow and the time of failure.

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