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MATHEMATICAL MODEL OF DAM BREAK

(Applied Study of Derna Dam)

نموذج رياضي للانهيار ســـد (دراسة تطبيقية على ســـد درنة)

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ملخص :-

إن انهيار سد مشكلة معقدة جداً ولا نستطيع توقعها بالضبط و هناك العديد من الباحثين قاموا بدراسة المشكلة بتوقع السلوك والتقـدم لموجة الفيضان مستخدمين افتراضات مختلفة وفي هذا البحث تم دراسة انهيار السد مستخدماً نموذجين رياضيين كأداة لتوقع العوامل المسببة للانهيار و محاكاة مسار السريان بعد حدوث انهيار السد. تم اختيار سد درنة (الساحل الشمال الشرقي للجماهيرية الليبية) كسد ترابي للدراسة التطبيقية كما تم جمع المعلومات اللازمة عن مدينة و سد درنة و اعتبر انهيار التفريغ الأنبوبي هو المتوقع لهذا السد. ومن الدراسة الحالية, وضح أن انهيار السد يسبب خطر كامن على مدينة درنة عند حدوث فيضان حسبما تعطي التوقعات المفترضة. و بناءا عليه يوصى البحث بدراسة مسار السريان بعد الانهيار في حالة انخفاض منسوب المياه المرتدة في الخزان وكذلك عند زيادة ارتفاع الحوائط الحامية للمدينة.

ABSTRACT

The dam break is a very complicated problem and it can't 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 the present research, the study of dam break was performed using a mathematical modeling as a tool to predict and simulate the dam break problem. Two major mathematical models were employed namely BREACH and FLDWAV for predicting such failure parameters as well as simulating the flow routing after dam break.

Derna dam (northern east coast of Libya) was taken as a case study of an earthen dam. So, collected information about Derna city and Derna dam were included in the current work. Piping failure was considered the most predictable one for Derna dam. For the current study it was shown that dam break cause a potential danger towards Derna City, since flooding occurs in the city for all hypothetical scenarios. It was recommended to predict the routing of the flow after failure due to lowering the retained water surface elevation in the reservoir and due to the use of levees and increasing walls heights in the city area.

INTRODUTION

A dam is a special kind of structure which is constructed to achieve several goals like: flood protection, human supply, irrigation, livestock water supply, energy generation, recreation, or pollution control. One of the most important is the study of Dam-break, that because dam failures are severe threats to life and property. The dam failure's degree of danger depends on many factors includes; the quantity of retained water, dam height, dam position relative to the settlement (city, farm, industry…etc.), how dam fails (partially or totally), how quick the dam fails (suddenly or take a specified time), whether the dam-break comes at the same time as a flood, and the downstream channel's characteristic (shape, roughness and geometry).

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In general, dams may fail, because one or more of the following causes takes place:

* Piping and seepage of water in the dam body as a result of bad construction and compacting of fill material, or animal holes.
* Settlement of dam's foundation, or the constructed material.
* Cracks or liquidized earthen dams, due to earthquake or any sudden shock.
* Instability of dam resulting from sliding, or overturning.

The objective of present study:-

The main aim of the current dam-break study is to predict the dam failure's behavior, and to estimate the flow along the downstream channel (flow routing) by calculating flows and water depths (stages) at different predefined locations along the water course. Derna dam was chosen as a case study of an earthen dam and the data regarding the study area was collected. The present work includes:

- Predicting the dam failure (piping type), and breach's parameters.
- Simulate different scenarios of dam break by applying FLDWAV Mathematical Model, Fread et al [2,3,4] on Derna Dam, and by using different parameters.
- Observe the results and produce conclusions and recommendations.

DAM-BREAK SIMULATION

The primary tasks are essential to simulate the dam break, the first is prediction of the reservoir outflow hydrograph through the breach, and the second is the flow routing of that hydrograph along the downstream.

- **The Breach** :-

The breach is defined as the opening formed in the structure as it fails. Many reasons may contribute in the development of the breach, but the main breach formation can be classified as due to overtopping or due to piping and that will affect the outflow hydrograph from the breached dam.

Different models were developed to predict the formation of breach for earthen dams, Ponce and Tsivoglou [10] presented a computationally complex breach erosion model for the sediment transport equation which introduced by Meyer and Muller [7]. A review of literature performed by Wahl [11], revealed numerous breach parameter prediction equations developed since 1984 (about 108 dam failures). The number of models available that allow an estimation of piping failure are limited and include the NWS BREACH model Fread, 1998 [4] and models under development by Mohamed, 1999 [9].

Earthen dams, which exceedingly outnumber all other types of dams, do not tend to completely fail, nor do they fail instantaneously. The fully formed breach in earthen dams tends to have an average width ($\bar{b}$) in the range of ($\mathbf{0.5\ H_d \leq \bar{b} \leq 8\ H_d}$) where $\mathbf{H_d}$ is the height of the dam, Singh and Snorrason [12]. The breach requires a finite interval of time (τ)

for its formation through erosion of the dam materials by the escaping water, this interval of time is known as the time of failure. The breach bottom width is assumed to start at a point, Fig. (1) then enlarges at a linear or nonlinear rate over the failure time (τ) until the terminal bottom width (**b**) is attained and the breach bottom has eroded to the terminal elevation ($\mathbf{h_{bm}}$). Collapse failure can be defined by assuming (τ) too small, and let the width of the breach bottom starts at a value rather than zero.

The terminal width (**b**) is related to the average width of the breach ($\bar{\mathbf{b}}$) by the following:-

$$\mathbf{b} = \bar{\mathbf{b}} - \mathbf{z}\,\mathbf{h_d} \tag{1}$$

in which $\mathbf{h_d}$ is the height of water over the breach bottom which is usually about the height of the dam and z is the user-specified side slope of the breach. The bottom elevation of the breach ($\mathbf{h_b}$) is simulated as a function of time (τ) according to the following:-

$$\mathbf{h_b} = \mathbf{h_d} - (\mathbf{h_d} - \mathbf{h_{bm}})\left(\frac{\mathbf{t_b}}{\tau}\right)^{\rho_o} \qquad \text{if } 0 \le t_b \le \tau \tag{2}$$

in which $\mathbf{h_{bm}}$ is user-specified final elevation of the breach bottom, $\mathbf{t_b}$ is the time since beginning of breach formation and ρ_o is the parameter specifying the degree of nonlinearity, e.g. $\rho_o = 1$ is a linear formation rate, while $\rho_o = 2$ is a nonlinear quadratic rate; the range for ρ_o is $1 \le \rho_o \le 4$; however, the linear rate is usually assumed.

The instantaneous bottom width ($\mathbf{b_i}$) of the breach was given by the following:-

$$\mathbf{b_i} = \mathbf{b}\left[\frac{\mathbf{t_b}}{\tau}\right]^{\rho_o} \qquad \text{if } 0 \le t_b \le \tau \tag{3}$$

Also, the following predictive equations were obtained from Froelich's work [5]:

$$\bar{\mathbf{b}} = 9.5\,\mathbf{K_o}\left(\mathbf{V_r}\,\mathbf{h_d}\right)^{0.25} \tag{4}$$

$$\tau = 0.59\left(\frac{\mathbf{V_r^{0.47}}}{\mathbf{h_d^{0.9}}}\right) \tag{5}$$

in which $\mathbf{V_r}$ is the reservoir volume (arce-ft) and $\mathbf{K_o} = 0.7$ for piping and 1.0 for overtopping. The average breach width $\bar{\mathbf{b}}$ in (ft) and time of failure (τ) in (hrs).

Formation of the breach produces additional outflow discharge through the dam, then total outflow was defined by:-

$$\mathbf{Q_T} = \mathbf{Q_s} + \mathbf{Q_b} \tag{6}$$

in which $\mathbf{Q_T}$ is the total outflow, $\mathbf{Q_s}$ is the summation of outflows through all the existing outflow structures and $\mathbf{Q_b}$ is the outflow discharge through the breach.

When the breach outflow ($\mathbf{Q_b}$) was computed as broad-crested weir flow, Fread [3], i.e.

$$\mathbf{Q_b = C_v\, K_s \left[3.1\, b_i \left(h - h_b\right)^{1.5} + 2.45\, Z \left(h - h_b\right)^{2.5}\right]} \tag{7}$$

in which **h** is the computed elevation of the breach bottom which was assumed to be a function of the breach formation time (τ) and ($\mathbf{K_s}$) was the computed submergence correction due to the downstream tail-water elevation ($\mathbf{h_t}$), i.e.,

$$\mathbf{K_s} = 1.0 - 27.8\left[\frac{\mathbf{h_t - h_b}}{\mathbf{h - h_b}} - 0.67\right]^{3.0} \tag{8}$$

that if $\dfrac{\mathbf{h_t - h_b}}{\mathbf{h - h_b}} > 0.67$ otherwise, $\mathbf{K_s = 1.0}$

Also, the correction factor of the approach velocity ($\mathbf{C_v}$) was computed from the following, Brater [1]:

$$\mathbf{C_v} = 1.0 + 0.023\frac{\mathbf{Q_b^2}}{\mathbf{B_d^2\left(h - h_{bm}\right)^2\left(h - h_b\right)}} \tag{9}$$

in which $\mathbf{B_d}$ is the reservoir width at the dam.

- **<u>Flow Routing</u> :-**

Flow routing is a mathematical procedure for predicting the changing magnitude speed and shape of floods wave as a function of time at one or more points along a water course. Because of the importance of the flow routing in simulation of any dam break, United Stated Army Engineer Waterways Experiments Station (WES) [13] collected one of the most complete sets of laboratory data on dam break flood waves.

Mohamed Alam et al [8] used the one-dimensional hydrodynamic equations based on shallow water theory to compute the flood wave resulting from the total instantaneous collapse of a dam. Wang, et al [15] used a second order hybrid type of total variation diminishing (TVD) finite-difference scheme to investigate the solution of the dam break problems.

In Fig. (2) a dam is represented by a vertical gate, with an upstream water depth of y_o, while the other side is dry. The gate was suddenly removed allowing the water to the downstream side. If the vertical accelerations were neglected, then the instantaneous water depth y varied from $\mathbf{y_o}$ to zero, and the velocity of flow at any section (**V**) was given by;

$$\mathbf{V = -2.0\sqrt{g}\left(\sqrt{y_o} - \sqrt{y}\right)} \tag{10}$$

Water surface was shaped as a parabola which concaved upward where the vertex was at the leading edge, and the shape of water surface was given by;

$$\mathbf{X = \left(3\sqrt{g\,y} - 2\sqrt{g\,y_o}\right)t} \tag{11}$$

in which **X** is the distance away from gate location ($X = 0$) and t is the time elapsed from the removal of gate. At any time, both the water depth (**y**) and the velocity (**V**) at dam's location ($X = 0$) were kept constants, as follows:

$$y = \frac{4\,y_o}{g} \qquad \text{and} \qquad V = -\frac{2}{3}\sqrt{g\,y_o}$$

The leading edge of the wave feathers out to zero height and moved downstream at velocity:

$$V = C = -2\sqrt{g y_o} \tag{12}$$

in which **C** is the local dynamic wave velocity.

Governing equations of the current mathematical model are the two Saint-Venant equations [11], the mass conservation one;

$$\frac{\partial (A\,V)}{\partial t} + \frac{\partial A}{\partial t} = 0 \tag{13}$$

and the momentum conservation equation:

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial X} + g\frac{\partial y}{\partial X} + g\left(S_f - S_o\right) = 0 \tag{14}$$

in which (S_f) is the boundary friction slope, (S_o) is the channel bottom slope < 15 %. and (**A**) is cross-sectional area of flow. The general equation to find the water flow is:

$$Q = Q_N\left[1 - \frac{1}{S_x}\,\frac{\partial y}{\partial x} - \frac{V}{S_x\,g}\,\frac{\partial V}{\partial x} - \frac{1}{S_x\,g}\,\frac{\partial V}{\partial t}\right]^{1/2} \tag{15}$$

in which **Q** is the general flow, Q_N is the normal flow, S_x is the frictional slope (constant for steady flow), **y** is the flow depth, **x** is the distance in the flow direction and **t** is the time elapsed from initiation of failure.

In FLDWAV Model, Saint Venant equations were extended and other variables were included, these equations in the extended from are as follows:

Conservation of mass:

$$\frac{\partial Q}{\partial x} + \frac{\partial\left[S\left(A + A_o\right)\right]}{\partial t} - q = 0 \qquad \text{, and} \tag{16}$$

Conservation of momentum:

$$\frac{\partial (S_m\,Q)}{\partial t} + \frac{\partial\left(\frac{\beta Q^2}{A}\right)}{\partial x} + g\,A\left[\frac{\partial h}{\partial x} + S_f + S_e + S_i\right] + L + W_f B = 0 \tag{17}$$

in which **A** is the active cross-sectional area of flow, A_o is the inactive cross-sectional area, **q** is the lateral inflow or outflow, S_m is the sinuosity factor which represented the ratio of the flow-path distance along a meandering channel to the mean flow-path distance along the flood plain, **B** is top width of active cross section, **L** is the momentum effect of lateral inflow or outflow, $\mathbf{W_f}$ is the resistance wind effect coefficient on the water surface, $\mathbf{S_e}$ is the slope due to expansion or contraction effects, $\mathbf{S_i}$ is the internal friction slope due to non-Newtonian fluid properties and β is the momentum coefficient for non-uniform velocity distribution within the cross section.

Based on the upwind explicit numerical scheme, Jin and Fread [6], the general equation could be written as follows:

$$\int_{t_j}^{t_{j+1}} \int_{x_i}^{x_{i+1}} \left[\frac{\partial \mathbf{Q}}{\partial \mathbf{x}} + \frac{\partial (\mathbf{A} + \mathbf{A_o})}{\partial \mathbf{t}} \mathbf{q} \right] \mathbf{dx\,dt} = 0 \tag{18}$$

The values of $\mathbf{(A+A_o)}$ and $\mathbf{Q}$ are known at all computational nodes at time $\mathbf{t_j}$, and the equation in finite difference form can be written as follows:

$$\begin{aligned} &\Delta \mathbf{t_j} \left[\mathbf{Q}_{i+1}^{j+1} + \mathbf{Q}_{i+1}^{j} - \mathbf{Q}_{i}^{j+i} - \mathbf{Q}_{i}^{j} \right] - 2\bar{\mathbf{q}} \Delta \mathbf{x_i} \, \Delta \mathbf{t_j} \\ &+ \Delta \mathbf{x_i} \left[(\mathbf{A} + \mathbf{A_o})_{i+1}^{j+1} + (\mathbf{A} + \mathbf{A_o})_{i}^{j+1} - (\mathbf{A} + \mathbf{A_o})_{i+1}^{j} - (\mathbf{A} + \mathbf{A_o})_{i}^{j} \right] = 0 \end{aligned} \tag{19}$$

in which $\mathbf{i} = 1$ for the upstream boundary and $\mathbf{i} = n - 1$ for the downstream boundary ($\mathbf{n}$ is the total number of computational cross-sections).

THE APLLIED CASE STUDY

Derna City

Derna City is one of the cities in the northern east of Libya, Fig. (3) shows its location. It lies in a small plateau about 4 km long and 800 m width in the northern coast of El-gebal El-akhdar Mountain. The plateau is separated from the hilly area by rocky clefts. The area is crossed by number of valleys and the most important one is Derna Wadi. The weather is moderate and the highest rain fails recorded at Derna station was 300 mm/year. Due to the activity of earthquake, the city exist in the second region according to the geological classification. In Sept., 1998 the population in the city reached 90,000 and it was estimated to reach 145,000 people in year 2015.

Derna Dam

Derna Dam was designed by Hidroprojekat, 1979. The main purpose of constructing the dam was the protection of Derna City from flood and to supply some local farms with irrigation water, so in the past, no considerations were taken to reduce the infiltration. The dam was located in the vicinity of the Derna city, presented a certain danger on the city itself and its population, therefore, determination of the hydraulic consequences of the dam failure was of primary importance. Dam function was intended not only as a protection from flood but also to store water to cover some of the increased demands. So, for the purpose of maintenance, removal of sedimentation, improvements of the bed and sides ability to impound water, and stability studies are required all.

Fig. (4) shows a vertical cross section of Derna Dam, which has a height of 26 m, top width of 6m, top length of 100m, front slope of 1 : 1.5 and rear slope of 1: 1.4. There is a shaft spillway with a crest at (41.00) m and 16.50 m diameter, passes a maximum flow of 350 m^3/sec at a head of 2.50 m above the crest. The spillway's tunnel has a diameter of 6.0 m and length of 190 m.

ESTABLISHMENT OF THE MATHEMATICAL MODEL

The main objective in establishing a mathematical model is to represent the prototype physical properties by suitable and equivalent mathematical parameters.

Dam failure is defined by a set of values defining the breach and time of failure. Other parameters are required to perform the routing (water course geometry, roughness, boundary condition and initial conditions). Solutions were obtained using numerical techniques each required essential parameters like; distance steps, time steps, depth tolerance, distance tolerance and weighting factors.

For the earthen dam, FLDWAV manual was suggested to be used according to the following ranges in breach parameter predictions, $0.1 < \tau < 0.5$, $0.5 < \mathbf{b} < 8\,h$, $\mathbf{Z} \geq 1$ In which h is the dam's height equals 26 m and **b**, τ could be calculated from equations (4) and (5) in **SI** units.

In the current work BREACH mathematical model was used to perform many runs, each with certain initial piping elevation, main output data were demonstrated by three curves and produced in Figs. (5,6 and 7). From these figures, the following values were adopted:

$\tau_{min} = 0.10$ hr, $\tau_{max} = 0.37$ hr, $\tau_{ave} = 0.24$ hr

$\mathbf{b}_{min} = 10$ m, $\mathbf{b}_{max} = 27$ m, $\mathbf{b}_{ave} = 18$m

and breach side slope $\mathbf{Z} = 0$, 1.0 or 1.3

Final bottom elevation of breach is 20 m, this elevation was near the dam's bottom and that produce breach with maximum height.

Water course is formed from many vertical perpendicular cross sections at different locations along the route. The downstream point results from the intersection between the Wadi course and sea was considered as (0+00) m distance. Derna dam is located at distance of (2+53) m so, all sections of distances more than (2+53) m exist in the reservoir. The last upstream cross section location has a distance of (4+80) m.

Due to the uncertainty of the roughness, different values were used for the simulation, with detail sensitivity analysis.

FLDWAV default tolerance for discharge is 2.831m^3/s, for stage is 0.003 m. For weighting factors in finite difference technique a value of 0.6 was recommended by FLDWAV manual. Minimum discharge of 10 m^3/s was assumed as the base flow before failure. Also, the distance steps were kept almost equal to 10 m, where time steps used were noticed very small (range from 0.36 sec to 5.0 sec).

SIMULATION SCENARIOS:-

Simulation runs were categorized into four groups each was concerning in certain findings, all groups were assumed to occur with the maximum inflow of constant value (350 m^3/s), initial water surface elevation in the reservoir was at the crest elevation of dam's spillway (41.00 m), failure was defined to occur as elevation of water reach (41.50 m).

• **Group A :-** This group represents the steady flow conditions, for different roughness, occurred due to the maximum discharge flow at the most upstream location without the occurrence of dam failure. From Fig. (8), it was clear that the variations in the recorded stages at the same location using different roughness coefficients were small. Water surface elevation of the reservoir upstream the dam reached nearly a constant value of (43.50 m).

• **Group B :-** In this group, the effect of the initial piping elevation was changing, while keeping other parameters constants. Two main simulation cases were considered, each with constant manning coefficient (n =0.050 and n =0.075) and with breach parameters of the following values (τ = 0.24 hr, b = 18 m, and Z = 1.3), Figs. (9 and 10). From the two histograms, it was noticed that, the peak outflow stared high at low initial piping elevation, then reduced as the elevation was increased up to a certain value above mid height of the dam, then peaks were increased as the initial piping elevation went higher toward the top of the dam. The time of occurrence started with low values, then reduced by small values and as initial piping elevation went higher, the time of occurrence were clearly increased.

• **Group C:-** In this group, the effect of changing breach parameters on the peak outflow from breach were studied. These parameters were to be assumed as following; Time of failure τ (0.10 hr, 0.20 hr or 0.50 hr), breach bottom width **b** (10 m, 20 m or 27 m), breach shape factor **Z** (0, 1, 1.3) and final breach bottom elevation for all runs is 20.0 m. Figs. (11, 12 and 13) are representing three samples (from nine cases) of histograms, keeping two parameters constant, while the third was changing against the peak outflows.

In general, by keeping both shape factor and final breach bottom width constants, peak outflows were noticed to be decreased as time of failure was increased. The peaks are direct proportional to the final breach bottom width. Also, when time of failure and final breach width were considered constants, peak outflows were increased as shape factor was increased. It is also noticed that, the current established model was more sensitive to the shape factor than other breach parameters.

• **Group D :-** This group represented the effect of changing Manning roughness on the flow routing after dam's failure. Two graphs were prepared in Figs. (14 and 15) for seven values of **n** (0.045, 0.050, 0.055, 0.60, 0.065, 0.070 and 0.075) and constant breach's parameters

(τ = 0.24 hr, **b** = 18 m and **Z** = 1). Generally, it was shown that, the increasing of Manning coefficient, caused the stages to be higher and flows be reduced.

Stages and flow hydrographs resulted from eight different locations, where maximum flows, depths of water and water surface elevations were calculated and analyzed. Figs.(16 and 17) represented stage and flow hydrograph at dam' location (2+530) m, and it was noticed that, maximum water surface elevation was (41.93) m which was more than the predefined required elevation (41.50) m to initiate the failure (breach formation).

CONCLUSIONS AND RECOMMENDATIONS

The current situation of the Derna Dam, reservoir and downstream channel, which were used to make the simulations in the present work, showed that there was certain potential danger on Derna City due to predicating piping failure of the dam.

From Derna Dam established model, and the mathematical simulation using different scenarios computer outputs, the following points were concluded:

1- Flooding occurs in the city for all assumed scenarios, depths of water and peak flows differ as simulation parameters differ.

2- The relationships between changing Manning coefficient on both stages and flows were non-linear.

3- At location (0+260) m (near the end of the system) the times of arrival was too short (from 13 min to 35min). That means there was no enough time for warning and evacuations.

4- The worst piping failure cases were due to initial piping elevation near the bottom of the dam. These elevations produced larger peak outflows in shorter time.

5- The current established model was more sensitive to the shape factor than the other breach parameters.

6- For upstream dam locations the established model was more sensitive to breach parameters compared to the water course roughness. But for the most downstream locations, the effects of water course roughness were more than that due to cases of failure (breach's parameter combinations).

7- For the study of flow simulations in terms of modeling technique without occurrence of dam failure, the following were deduced:
   - Water surface didn't rise more than walls' height.
   - Stages differ from location to another, which means that the flow was non uniform in the system.
   - At the same locations, stages were increased as the flows were decreased, by a small values, while the value of Manning roughness coefficient was increased.

It is recommended to study the effect of lowering final water surface elevation on breach parameters and on the flow routing after failure. Also, it is advised to study the effect of increasing the walls' heights in the city area on the flow routing.

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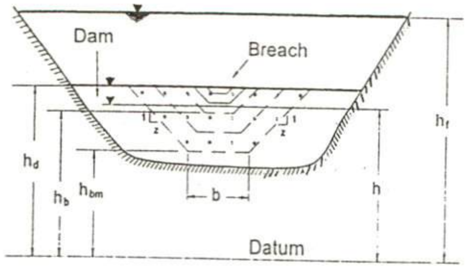

Fig. (1) Front View of Dam Showing Formation of Breach.

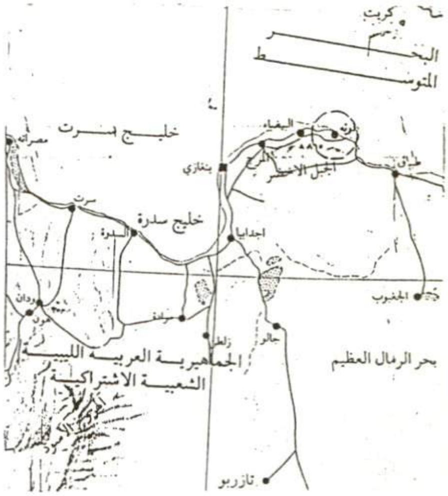

Fig. (3) General Location of Derna City.

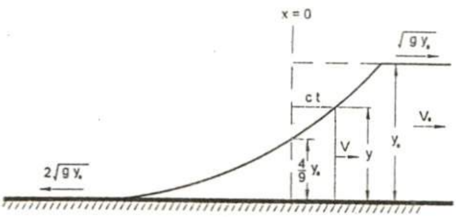

Fig. (2) Dam-Break Profile

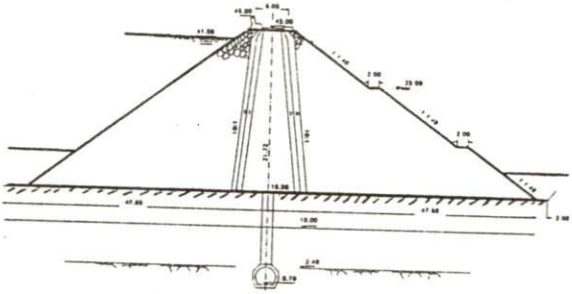

Fig. (4) Vertical Cross Section of Derna Dam.

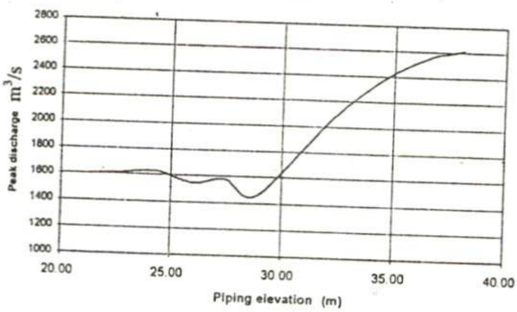

Fig. (5) Initial Piping Elevation and Peak Outflow Through the Breach.

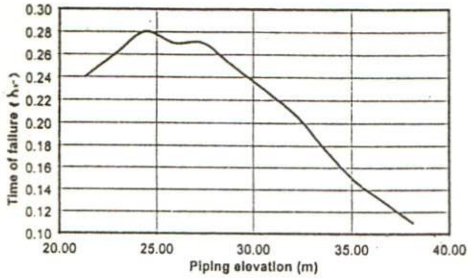

Fig. (6) Initial Piping Elevation and Time of Failure.

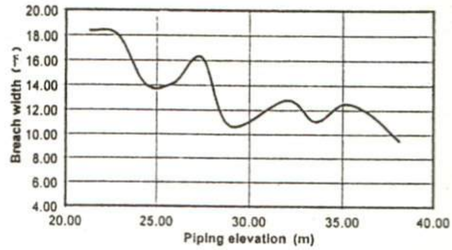

Fig. (7) Initial Piping Elevation and Final Breach Bottom Width.

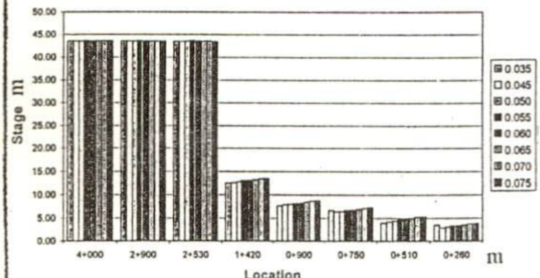

Fig. (8) The Variations of Stages for Different Roughness.

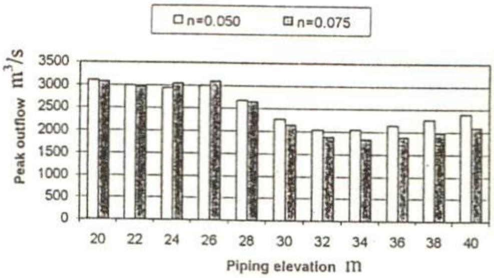

Fig. (9) Piping Elevation With Peak Breach Outflows

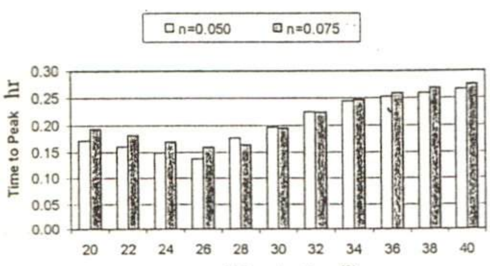

Fig. (10) Time of Occurrence of Peak Reach Outflow for Different Initial Piping Elevation

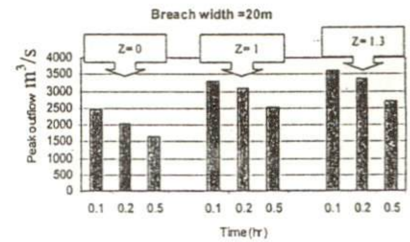

Fig. (11) Peak Outflows With Respect to the Change in Time of Failure.

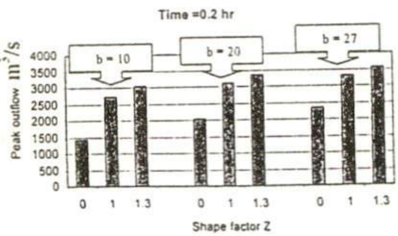

Fig. (12) Peak Outflows With Respect to the Change in Shape Factor.

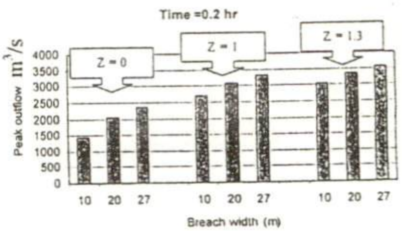

Fig. (13) Peak Outflows With Respect to the Change in Final Breach Width.

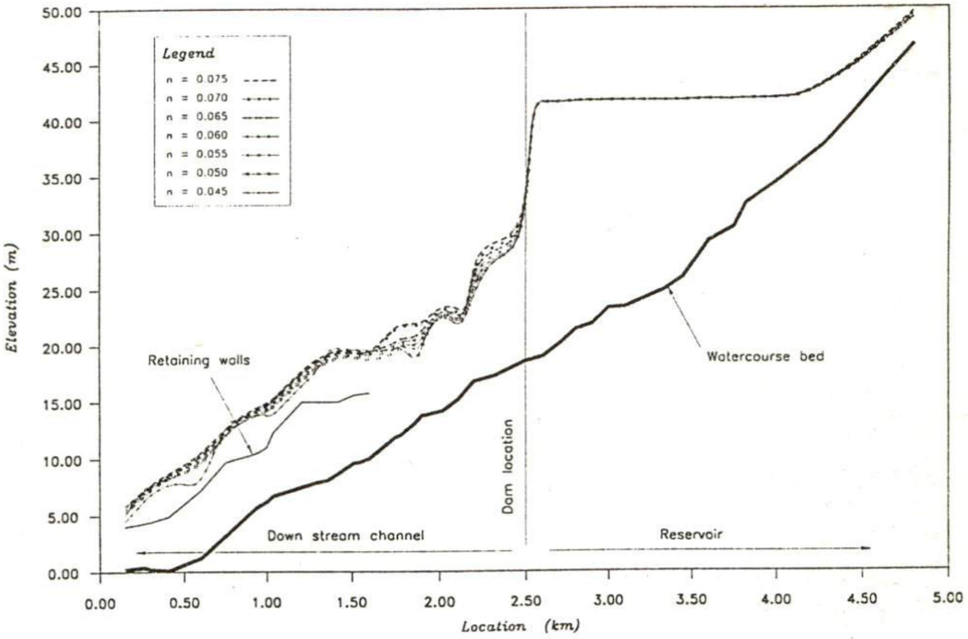

Fig. (14) Peak Water Surface Elevation Along Watercourse for Group D.

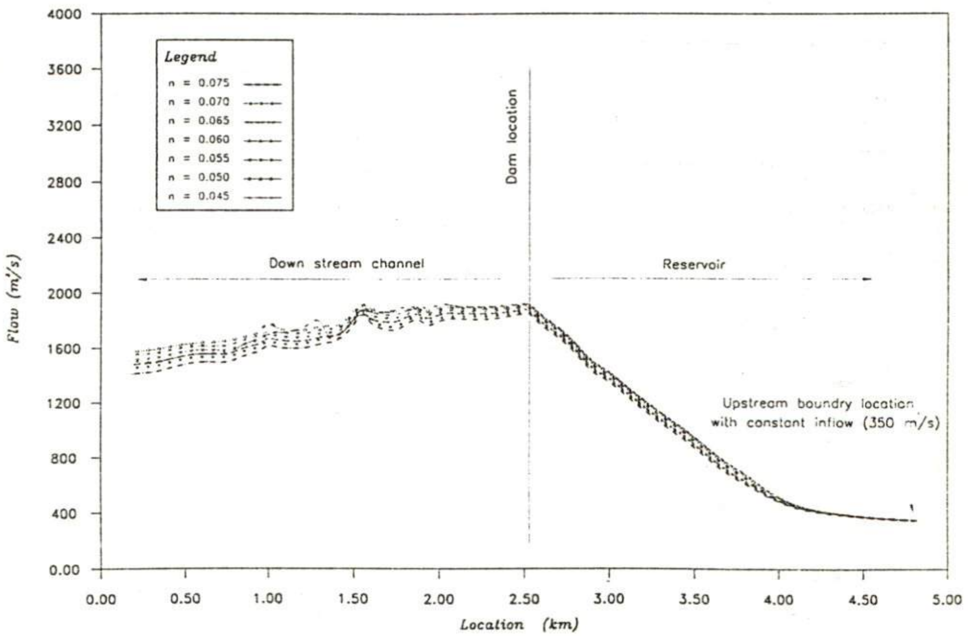

Fig. (15) Peak Water Flows Along Watercourse for Group D.

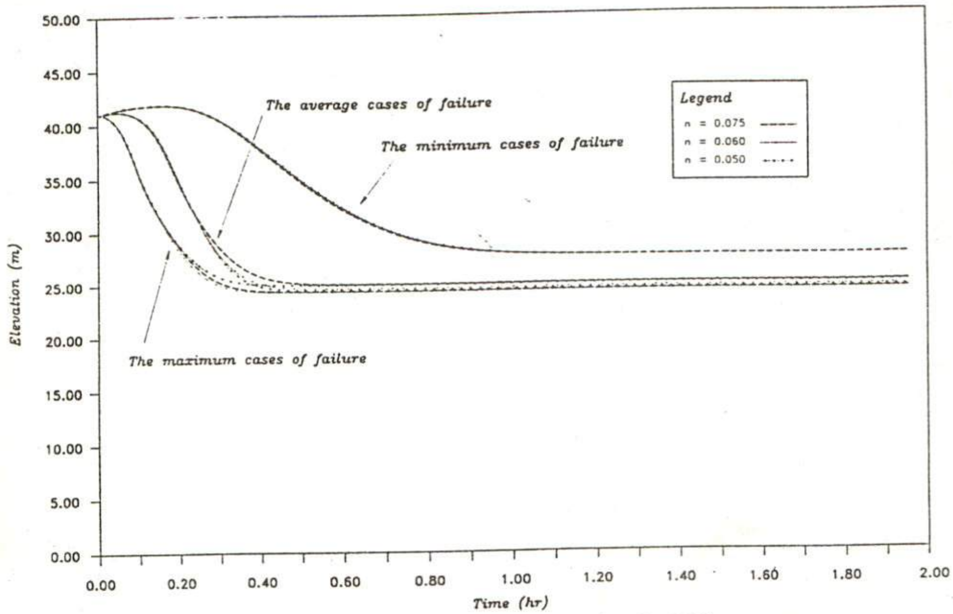

Fig. (16) Stage Hydrograph at Location (2+539) m.

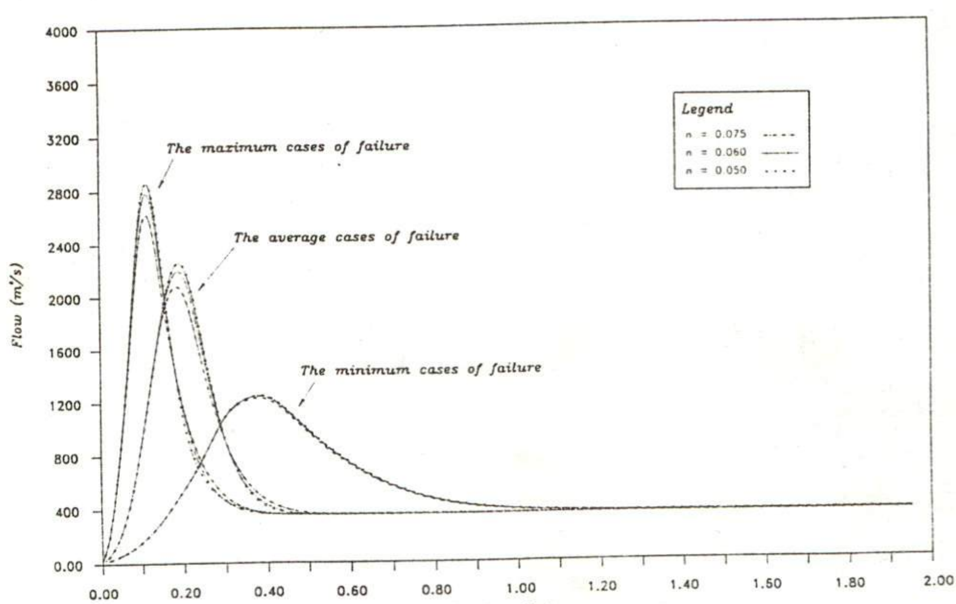

Fig. (17) Flow Hydrograph at Location (2+539) m.