LARGE SCALE PHYSICAL MODEL TO INVESTIGATE THE MECHANICS OF EMBANKMENT EROSION DURING OVERTOPPING FLOW

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ABSTRACT:

This paper presents the results of large scale physical model embankment failure experiments. The model simulates the failure by overtopping of three embankments that are varying in dimensions and soil type. The soil is either homogeneous pure sand or mixture of sand, silt and clay. A new measuring technique consists of the labeled rod bridge; grids lined and captured instantaneous pictures are utilized to monitor the failure. It was used to analyze the morphological evolution during embankment failure. The flow pattern and progressive failure of embankment overtopping were investigated. The results showed four types of phenomenon during embankment failure: erosion; erosion with sliding; erosion with mass failure and head cut migration. The results revealed that the recorded time to reach the full breach failure in case of mixture materials (sand, silt and clay) is seven times that with pure sand. Also, reducing the embankment height to 50% (0.9m), led to increase the time to peak discharge 8.4 times that of large one (1.8m). Adding 15% clay to pure sand changes soil characteristics to (c and ϕ) soil which makes soil to have a critical vertical cut height. Set of dimensionless regression equations are developed by using the experimental data. In conclusion, types of embankment failure are defined. Adding a percentage of clay (15%) to the pure sand led to the maximum remain vertical cut height of about 50% of the calculated critical vertical cut height (Z_c). Bearing in mind the limited number of large scale testes for embankment breach in literatures, the set of data could be used for calibration of mathematical breach models and provides a reference for the flood risk management of embankments.

تعرض هذه الورقة البحثية دراسة تأثير نوعية المواد المستخدمة في بناء الجسور أو السدود الصغيرة وكذلك ارتفاع هذه الجسور على عملية إنهارها نتيجة اجتياز المياه قممها وزمن الإنهيار وأقصى تصرف له وعمقه وعرضه. ولتحقيق هذا الهدف، تم إنشاء نموذج طبيعي ذات أبعاد تقارب الأبعاد الطبيعية لهذه الجسور والسدود من خلال ثلاث إختبارات باستخدام مواد متجانسة مثل الرمل أو خليط بنسبة ١٥% طين و ٤١% رمل و ٤٤% طمي. وقد تم إختبار ارتفاعين لهذه الجسور هما ٨،١م و ٩،٠ م وبمعدل أمامية وخلفية متساوية وقيمتها ١ رأسى: ٢،٥ أفقى. ولقد تم استخدام طريقة جديدة فى قياس ومتابعة وتحليل عملية إنهار الجسور أثناء الإختبارات مكونة من كوبرى ذو أعمدة متحركة ومركمه وشبكة خطوط على الميول الخلفية للجسر أو السد بالإضافة إلى التصوير الحظي. وقد أوضحت النتائج أنواع الظواهر التى يمكن أن تحدث أثناء عملية الإنهيار وهى النحر أو الانزلاق أو النحر والإنهيار الكتلى أو القطع الرأسى ذو الحركة تجاه جسم الجسر أو السد. كما أظهرت النتائج تأثير إضافة الطين والطينى للرمل على زمن الوصول إلى أقصى تصرف والمصاحب للإنهيار ومقارنته بحالة الرمل الخالص. وقد أثبتت ذلك أن إضافة الطين والطينى إلى الرمل، يحول الرمل إلى مادة لها خصائص الطين ولها القدرة على الوقوف رأسياً بدون دعائم. وقد استخدمت أيضاً النتائج فى إستنباط معادلتين أحصائيتين لحساب تصرف الإنهيار. وأهم ما يمكن إستخلاصه من هذه الدراسة أنه هو تعريف أنواع الإنهيارات و أنه بترتبه رملية تحتوي على نسبة تساوى ١٥% فأكثر منها طين، يكسب هذه التربة خصائص الطين من تآسك وهذا يزيد من معامل الأمان للجسور والسدود ويغير من شكل عملية الإنهيار ويزيد من زمن التحذير من قبل حدوث الإنهيار ونظراً لندرة مثل هذه الإختبارات بالأبعاد التى تشابه الأبعاد الطبيعية فإنهم يمكن استخدام نتائج هذه الإختبارات فى معايرة النماذج الرياضية.

Key words: Flow pattern, Embankment failure, Overtopping, Large scale physical model

1 INTRODUCTION

Dams or embankments are constructed for the retention of water for irrigation, hydropower generation, water supply, and the protection of people, land, and property from flooding. Tools currently available for simulating dams or embankment failure are not very accurate and can only be used as indicative assessments. Consequently, the prediction of flood risk from dam or embankment breach may be similarly inaccurate. This applies particularly in the critical zone downstream the embankment where the risk to life is greatest.

Warning and evacuation time can dramatically influence the loss of life from embankment failure. When establishing hazard classifications, preparing emergency action plans, or designing early warning systems, good estimates of warning time are crucial. Warning time is the sum of the breach initiation time, breach formation time, and flood wave travel time from the dam to a population centre. Case history based on procedures developed by the Bureau of Reclamation indicate that the loss of life can vary from 0.02 percentage of the population at risk when the warning time is 90 minutes to 50 percentage of the population at risk when the warning time is less than 15 minutes [1]. More work shows similar extreme sensitivity to warning time [2]. The average number of fatalities per dam failure is 19 times greater when there is inadequate or no warning [3].

The prediction of potential breaches and the consequent flooding are thus important steps in managing the risk from potential dam or embankment failure. However, breach simulation and breach parameter prediction are considered to have the greatest uncertainty of all aspects of dam break flood forecasting [4, 5, and 6]. The breach formation and the eventual dam failure may be linked to hydraulics, hydrodynamics, hydrology, sediment transport mechanics, and other geotechnical factors. Earthquake, landslide, extreme storm, piping, equipment malfunction, structural damage, foundation failure and sabotage are prominent causes of dam failures [7].

Breach Parameter Definitions

The term breach parameters will include the parameters needed to physically describe the breach (breach depth, breach width, and side slope angles) as well as parameters that define the time required for

breach initiation and formation. These parameters can be summarized as follows [8]:

Breach depth: is the vertical extend of the breach measured from the crest down to the invert of the breach.

Breach width: is ultimate breach width which can dramatically affect the peak flow and resulting inundation levels downstream the dam.

Breach initiation time: is the breach initiation time begins with the first flow over or through a dam that will initiate warning, evacuation, or heightened awareness of the potential for dam failure. The breach initiation time ends at the start of the breach formation phase

The time of failure: is the duration of time between the first breaching of the upstream face of the dam until the breach is fully formed. For overtopping failures the beginning of breach formation is after the downstream face of the dam has eroded away and the resulting crevasse has progressed back across the width of the dam crest to reach the upstream face.

Researchers have found that the nature and magnitude of compaction used to construct the embankment significantly affects the rate of erosion during embankment overtopping and breach [9, 10 and 11]. A common method for specifying compaction of earth embankments is to require a given degree of compaction (the compacted soil must be at a dry density equal to or higher than the stated percentage of the control compaction test), and within a range of acceptable water contents. Often, small embankments (less than about 20 m high) have acceptable engineering properties if they are compacted to about 95% of Standard Proctor dry density, with the specified water content dependent on the soil type and other site specific conditions. Higher embankments or those built with more highly plastic soils are often specified to be compacted to a higher degree, perhaps 100% of Standard Proctor (ASTM D698A) [12] dry density, to achieve acceptable shear strength [13].

Three main failure modes for levees and embankment dams are reported in literature: overtopping, piping failure or internal erosion and structural failure (e.g., slope stability and foundations). About 34 % of embankments and levees fail by overtopping, 28 % by piping and 30 % by foundations defects [14].

The experimental studies under controlled laboratory flow conditions, and further prototype observations, showed that the embankment breach starts with an initial phase, followed by rapid

development of the breach, and then an enlargement of breach width once the breach invert reaches the channel bed rock [15 and 16].

Physical Model Tests

Breach formation and development in embankment is a very complicated process with still little understanding. Therefore physical model tests on embankment breaching are at least for the moment, very valuable and irreplaceable for gaining more insight into the process, and for calibration and validation of the embankment breach mathematical models.

Physical modeling should be performed at large scales to overcome the problems of simultaneous scaling of hydraulic conditions and material properties. Although hydraulic modeling at small scales is a well-developed science, material properties do not scale uniformly and can be difficult to reproduce at small scales. Using large-scale models allows the use of near-prototype size materials, making results more reliable and easier to interpret. It will, of course, be impossible to conduct true full-scale testing for larger dams, but scales should be sufficiently large to allow the use of prototype embankment materials. The tests should produce erosive mechanisms and conditions comparable to those encountered in a prototype condition.

The objectives of this study are to examine the effects of cohesive and non-cohesive fine sediment materials used in construction of embankment in the development of the large embankment breach and to study the effects of soil properties on the breach hydrograph. The current work adds to the understanding of the effect of the soil properties on embankment breach by overtopping.

2 MODEL DESCRIPTIONS AND SETUP

Three large scale laboratory experiments were constructed one by one in an area of 600 m². The

experimental model consists of the three major parts: (1) a storage reservoir (400m²), (2) modeled embankment (80m²) and (3) exit of the model (120m²) (settling basin, weir, sump). Figures (1) and (2) show the general layout of the model.

The height (H) and the crest width (T) of the embankment were varied from 0.90 m to 1.80m and from 0.75m to 1.5m respectively. The width of the embankment (W) was 5.0m, Figure (3). The upstream slopes (S₁) and downstream slopes (S₂) were the same, 1V: 2.5H. Two different types of materials were tested as embankment materials (Table1). Figures (4) and (5) show grain size distribution of pure sand and mixture soil, respectively. The storage volumes of water at the crest level were varied from 360m³ to 720m³ according to the height of the embankment.

Four pumps, with a maximum combined flow capacity of 1400 l/s, were used during the experiments. Two pumps with a maximum combined flow capacity of 700 l/s were used to deliver the water to the reservoir from Nile River and the other two pumps were used to pump the water from the end of the model to the Nile River. The valves were opened at different times to maintain a nearly constant head during the first stage of the experiment. The valves were calibrated prior to the experiments to obtain the desired discharge at different opening.

The embankment constructed in layers of approximately 0.15m. each layer is compacted using a previously calibrated roller to obtain a dry unit weight of about 95% of the standard proctor maximum dry unit weight, (ASTM – D698) [12]. A 0.03 m deep and 0.5m wide pilot channel was created at the center of the embankment crest prior to the beginning of each experiment. The pilot channel conveyed the flow through the center of the embankment, leading a controlled failure.

Figure (1) General Layout of the Model

Figure (2) Photo of the General Layout of the Model

Not: Location 1- Measurement location of breach at crest

Location 2- Measurement location of breach at downstream face

Figure (3) Dimensions of the Embankment and Measurements Location of Breach

Table1 Description and Properties of the experimental Runs

Run s	Dimensions					Type of soil
	W (m)	H (m)	T (m)	S ₁	S ₂	
1	5	1.8	1.5	2.5	2.5	100% Sand
2		1.8	1.5	2.5	2.5	41% Sand 44% Silt 15% Clay
3		0.9	0.75	2.5	2.5

Figure (4) Grain Size Distribution of Sand

Figure (5) Grain Size Distribution of Soil Mixture (sand, silt and clay)

Instrumentation

To measure the breach width and height evolution, a labeled road bridge as shown in figure (6) and three high definition digital cameras with high resolution and speed were used. These cameras were located at suitable positions of the embankment. The breach width evolution was directly measured by taking snapshots of the experiment recorded at regular intervals. These images were rectified and analysed to obtain the breach width temporal evolution during the experiments.

One distance measuring sensor was placed upstream the embankment to monitor water level during the experiments. This sensor has a measuring range from 0.4 m to 2.5 m with a resolution of less than 0.001 m. Another three distance measuring sensors were placed over the rectangular weir to monitor the water level, used to calculate the discharge through the breach. These sensors had a measurement range from 0.1 m to 1 m with a resolution of less than 0.001 m as shown in figure (7).

Figure (6) Labeled Road Bridge Used for Measuring the Breach Width and Height

Distance Measuring Sensor at the Reservoir
Distance Measuring Sensors over the Weir

Data Logger to Collect the Data
Acoustic Doppler Current Profiler
(ADCP),
(Sontek)

Figure (7) Measuring Water Levels Instruments

3 MODEL RUNS AND DISCUSSION

Three embankments were tested, with different filling material and height, Table (1).

Overtopping Flow Patterns of an Embankment

Generally, after overtopping of embankment occurs, three hydraulic flow regimes and erosion zones can be observed, figure (8). In the subcritical flow region on the dam crest, energy slopes, velocities, and tractive stresses are relatively low and erosion will occur only if the crest materials are highly erodible. A transition to supercritical flow occurs on the downstream portion of the crest. Energy slopes and tractive stresses are higher in this region, and erosion is sometimes observed at the knick point at the downstream edge of the crest. The third zone of erosion is the downstream face of the dam. On which the flow accelerates at supercritical conditions until reaching uniform flow after

leaving the toe of the embankment. Tractive stresses (shear stresses) are very high, and changes in slope can concentrate stresses and initiate

Figure (8) Flow Regimes over an Embankment

Initiation Time of the Breach

The initiation time is defined as the instant at which the flow from the reservoir entered the pilot

channel. During the initial phase of the test, the flow acted like sheet-flow, as it approached the downstream toe of the embankment. Following this period, considerable erosion took place as the flow velocity increased, gullies and reels developed. For embankment with weaker soils (Run_1), the pilot channel eroded faster in the vertical direction. For embankment with stronger soils (Run_2 and Run_3), the major erosion on the pilot channel is observed to be in the lateral direction.

Run₁

Run_1 represents the embankment whose dimension as show Table (1) and filling material is pure sand with $D_{50}=0.0005m$, Figure (4).

Mechanics of Embankment Erosion during Overtopping Flow for Run₁

Flow through the pilot channel initially erodes a small breach channel on the embankment downstream face from the crest to the toe of the embankment. After a breach channel was formed in the embankment, the breach was wider at the downstream end, figure (9). From the experiment, it was observed that, breach failure changes from primarily vertical erosion to predominantly lateral erosion, the latter occurring as the breach channel invert approaches the embankment foundation level. Embankment material is eroded by the processes of tractive shear and turbulence, with lateral erosion

occurring at all points along the channel side slopes [17]. Undermining of the breach channel side slopes causes large volumes of unsupported material to collapse into the center of the channel and be transported downstream, figure (9).

For the embankment profile tested, erosion of the breach channel initially proceeds with the invert slope parallel to the slope of the downstream face of the embankment, figure (9b,c). After the downstream toe of the channel invert has eroded upstream to a pivot point along the base of the embankment, the channel invert slope then starts to flatten while rotating about this fixed (pivot) point, figure (9e). As erosion continues, the channel invert slope reduces to a terminal value of minimum slope, figure (9f), the channel subsequently widening with a central flatbed region of S_{min} .

In this run, a combination of embankment surface erosion and sliding failure were observed. The embankment crest was severely eroded within a few minutes due to the loss of supporting embankment material, which resulted from head-cut in the upper part of the downstream embankment surface, figure (9f).

Under the action of upstream static water force and the dynamic flow drag force, some part of soil near the upstream entrance of the breach channel may lose its stability and suddenly collapse, figure (10).

(a): Time =60 second

(b): Time =160 second

(c): Time =260 second

(d): Time =358 second

(e): Time =460 second

(f): Time =562 second

Figure (9) Breach Development for Pure Sand Embankment (Run₁)

Figure (10) Instable Part of Pure Sand Embankment

Temporal Evolution of the Breach in Pure Sand Embankment (Run₁)

Figures (11) and (12) present the hydrograph, water level, reservoir inflow, breach width and maximum breach depth at the crest and at the middle of downstream face of the embankment respectively. Flow entering the model was adjusted to ensure a constant reservoir level as the breach developed as shown figures(11) and (12), the test thereby simulating the failure of an embankment dam impounding a large upstream reservoir. A decrease in reservoir water level was observed, regardless of an increase in inflow to the reservoir. Figures (11) and (12) show the results of run₁, the peak outflow is 2557.868 l/s and the time to peak is 10.25minutes (615 seconds). The duration of the embankment failure process due to flow overtopping was less than 13 minutes (778 seconds). The results show that, the growth rate of the breach increased until it reached a constant breach width and height. From the results of the run₁, it has concluded that, the embankment crest was severely eroded within few minutes due to the loss of supporting material, which resulted from head-cut in the upper part of the downstream embankment face.

Run₂

Run₂ represents the embankment whose dimensions and the upstream and the downstream slopes are the same of the embankment in Run₁, but

with different in filling material, Table (1).

Mechanics of Embankment Erosion during Overtopping Flow for Run₂

Based on test observation and data recorded, a good description of the mechanics of embankment erosion is provided, figure (13). Breaching takes place by head-cutting rather than tractive stress erosion as in Run₁. A small head-cut is typically formed near the downstream toe of the embankment and then advances upstream until the crest of the dam is breached. Small head-cuts started forming at the construction layer interfaces. The small head-cuts continued to develop into small over-falls that dominated the erosion process in the downstream slope of the embankment. A series of stair-step head-cuts forms on the embankment downstream face after the erosion is initiated. This action is similar to that described by Ralston, Dodge, Powledge et al. and Zhang et al, [18, 19, 20 and 21].

Figure (13) show that, first rill and micro-rill erosion on the downstream slopes of the embankment were initiated by the sheet flow, figure (13a). Following this, considerable erosion took place as the flow velocity increase. Gullies and rills developed, and small head-cuts started forming at the construction layer interface. The small head-cuts continued to develop into small over-falls that dominated the erosion process in the downstream slope of the embankment, figure (13b). After that the head-cuts remain with constant height and simultaneously lateral erosion takes place, figure (13c). The head-cut eventually migrate to the upstream end of the embankment dam crest, from then on any further upstream migration of the head-cut resulted in a crest lowering and increased discharge from the reservoir. This progression finally led to a full breach, figure (13d).

The gully widening and the head-cut upstream migration were believed to be due to the turbulence and hydraulic stresses within the jet impingement area just downstream of the head-cut. The hydraulic stresses erode the base of the head-cut, cause it to

become unstable and ultimately produce mass failure. The mass failure induced the breach widening and head-cut upstream migration.

Temporal Evolution of the Breach in Soil Mixture Embankment

Figures (14) and (15) present the hydrograph, water level, reservoir inflow, breach width and maximum breach depth at the crest and at the middle of the embankment respectively. Flow entering the model was adjusted to ensure a constant reservoir level as the breach developed as shown figures (14) and (15), the test thereby simulating the failure of an embankment dam impounding a large upstream reservoir same as in case (run₁). A decrease in

reservoir water level was observed, regardless of an increase in inflow to the reservoir. Figures (14) and (15) show the results of run₂, the peak outflow is 2064.121 l/s and the time to peak is 82 minutes (4920 seconds). The duration of the embankment failure process due to flow overtopping was less than 85 minutes (5100 seconds). From Figures (14) and (15), the breach is indicated to widen (breach width is increasing) at a rate that increases with time, the breach crest lowering at a low rate that increases with time. Initially, the rate of breach widening is greater than the rate of breach deepening. From the results, it can be conclude that, the major erosion is in lateral direction.

Figure (11): Results for Pure Sand Embankment (Run₁) at Crest

Figure (12): Results for Pure Sand Embankment (Run₁) at Middle of Downstream Face

(a) Time = 68 second

(b) Time = 548 second

(c) Time = 1928 second

(d) Time = 4868 second

Figure (13) Breach Development for Soil Mixture Embankment (Run ₂)

Figure (14) Results for Soil Mixture Embankment (Run ₂) at Crest

Figure (15) Results for Soil Mixture Embankment (Run₂) at Middle of Downstream Face

Run₃

Run₃ represents the embankment whose dimensions and its filling material as show, Table (1).

Mechanics of Embankment Erosion during Overtopping Flow for Run₃

An initial rectangular notch is created on the center of the crest. Water flows through the initial notch. Flowing water erodes downstream surface, (initial erosion), figure (16a). This initial erosion progresses into stair-stepped multiple overfalls, figure (16b). Over-falls then merge into a single upstream migrating head-cut, figure (16c). The head-cut then migrates upstream, lowering the crest by advancing into the upstream embankment face, figure (16d). This progression finally led to a partial breach failure, figure (16 d). Therefore, the breach reaches its maximum dimensions.

Temporal Evolution of the Breach in Soil Mixture Embankment (Run3)

Figures (17) and (18) present the hydrograph, water level, reservoir inflow, breach width and maximum breach depth at the crest and at the middle of the embankment respectively. Flow entering the model was adjusted to ensure a constant reservoir level as the breach developed as shown figures (17) and (18), the test thereby simulating the failure of an embankment dam impounding a very large upstream reservoir. A reservoir water level stays constant for a longer time during the breach process regardless of an increase in inflow to the reservoir. As a result, the peak outflow is 685.2318 l/s and the time to peak is 11.76 hour (42350 seconds). The duration of the embankment failure process due to flow overtopping was almost 12hour (43200 seconds). From figures (17) and (18), the breach is indicated to widen (breach length is increasing) at a rate that increases with time, with the breach crest lowering at a low

rate, that increases with time. Initially, the rate of breach widening is greater than the rate of breach deepening. From the results, it can be conclude that, the major erosion is in lateral direction as the result of run₂ but it takes longer time.

Effect of Non-Cohesive Fines and Cohesive Soil on Breach (Run₁ and Run₂)

The data from run₂, show that the presence of fine non-cohesive soil (44% silt) and cohesive soil (15% clay) in the soil mixture have profound effect on the breach process. These effects are demonstrated in figure (19), by showing a comparison of the breach evolution in Run 1 and 2 at $t=562$ s. For Run₁, the breach is fully developed (Figure 19a), while in Run₂, the breach is still a stair-stepped multiple overfalls on downstream slope (Figure 19b). After 480s, it can be observed that the breach width rate of change is 2.34mm/s in Run₁, figure (20) and its maximum breach depth reach to the full breach, while in run₂ the breach width rate of change is 0.59mm/s and its maximum breach depth has no significant change.

Figure (20) shows a comparison between Run₁ and Run₂. This figure indicates that while the peak discharge decreased by 20%, the time to peak increased 7 times. Also, the breach width rate of change is decreased by 75% and the maximum breach depth at crest after completed failure decreased by 33%. These happened due to the presence of silt (44%) and clay (15%) in soil mixture (Run₂), when compared with an embankment built of pure compacted sand (Run₁).

Effect of Non-Cohesive Fines and Cohesive Soil on Vertical Cut Height

Figure (21a, b) shows the final stage of the failure of two clayey embankments with different height, 1.80m and 0.90m, (run₂ and run₃). The examined soil in both cases (run₂ and run₃) according to its soil contents (15%clay, 41%sand and 40%silt)

classified as clayey sand and silt, [22]. The clay percent even with its low percent dominate, this due to the geotechnical behavior of the mix, which can be explained by the fact that the employed clay is an active clay with activity ranged between (2.25 and 2.5) where the activity define as :

$$\text{Activity} = \text{P.I.} / \text{clay percentage}$$

Where P.I. = Plasticity Index, [23]. From this Figure we observed that, the maximum remain vertical cut height of the two embankments after complete failure is almost the same = 0.60 m which is about 50% of the calculated critical vertical cut height ($Z_0 = 1.22\text{m}$) for such soil. The critical vertical cut height for the clayey soils mixture is constant disregarding the height of the embankment without exposit to external load or overtopping condition on it. The Z_0 is a function of the shear-parameters for the clayey soils (c & ϕ) and independent of the height. This phenomenon matches the observed final-shape of the two tested case where the embankments end with equal height (50% of Z_0) independent of the embankment height. This Z_0 can be calculated from the following equation, [24]:

$$Z_0 = 2c / (6 * K_s^{0.5}) \quad (1)$$

$$\text{Where } K_s = (1 - \sin\phi) / (1 + \sin\phi)$$

Effect of Embankment Height on Breach (Run₂ and Run₃)

The data from Run₃, show that the effect of embankment height has profound effects on the

breach process. These effects are demonstrated in figure (22), by showing a comparison of the breach evolution in Run 2 and 3. From this Figure, after 2700s, it can be observed that the breach width rate of change is 0.55mm/s and its maximum breach depth has no significant change (Run₂), while the breach width rate of change is 0.1mm/s and its maximum breach depth has no significant change (Run₃). Figure (23) shows how the peak discharge decreased by 66% and breach width rate of change at crest decreased by 82% due to decrease the embankment height by 50% (Run₃, $H=0.9\text{m}$), when compared with Run₂ ($H=1.8\text{m}$). Also, the time to peak of Run₃ increased 8.4 times of Run₂. Figure (23) shows a comparison of the breach evolution in Run₂ and Run₃ at $t=5100\text{ s}$. For Run₂, at $t=5100\text{ s}$, the breach is fully developed (Figure 23a), while in Run₃, the breach is still a stair-stepped multiple overfalls on downstream slope (Figure 23b).

Concerning the time of failure both embankments, the one with high 1.8 m has a failure time of 1.4 hr. and the other one with high of 0.9 m has a failure time of 11.7 hr., that is because the static and dynamic pressure on the 1.8 m embankment is much higher than the other with 0.9 m high. Also, taking into consideration that dynamic and static pressure has a nonlinear relation with high. That is why the time of smaller dam takes longer time to reach the same critical high of the 1.8 m embankment as shown in figures (22).

(a) Time = 358 second

(b) Time = 1768 second

(c) Time = 36026 second

(d) Time = 45884 second

Figure (16) Observed Breach Process during Embankment Test for Soil Mixture (Run₃)

Figure (17) Results for Soil Mixture Embankment (Run₃) at Crest

Figure (18) Results for Soil Mixture Embankment (Run₃) at Middle of Downstream Face

(a) $t=562$ (s), Run₁ (pure sand)

(b) $t=562$ (s), Run₂ (soil mixture)

Figure (19): Effect of Non-Cohesive Fines and Cohesive on Breach Evolution of the Embankment (Run₁ and Run₂)

Figure (20): Effect of Non-Cohesive Fines and Cohesive Soil on Breach Discharge and Evolution of the Embankment (Run₁ and Run₂)

(a) Embankment 2, Run₂, 1.8m (height)

(b) Embankment 3, Run₃, 0.9m (Height)

Figure (21): Effect of Non-Cohesive Fines and Cohesive Soil on Vertical Head-cut Height (Run₂ and Run₃)

Figure (22): Effect of Embankment Height on Breach Discharge and Evolution (Run₂ and Run₃)

(a)Embankment 2, 1.8m (height), $t=5100$ s

(b) Embankment 3, 0.9m (Height), $t = 5100$ s

Figure (23): Effect of Embankment Height on Breach Evolution of the Embankment (Run₂ and Run₃)

Regression Equation Development

Within the comprehensive analysis performed for collected data from the previously mentioned experimental program, regression analysis was performed to develop closed form dimensionless empirical equation expressing the relationship between the breach discharge (Q_b) divided by the square root of acceleration of gravity (g) and height of the embankment to the power five (H^5) presented as (Y) in one side as a function of a dimensionless term (x) that includes dimensionless breach width (W/H) multiplied by dimensionless breach depth (D/H), on the other side.

The equations that express the dimensionless, Q_b for the pure sand embankment and the mixture (41% sand, 44% silt and 15% clay) embankment and Figures 24 and 25 represent the fitting of the proposed equations to the measured data are presented as follows:

Sand embankment (Run₁) Regression Equation

$$Y = a + bx^3 \quad (2)$$

Where:

Y=Dimensionless term represents $\left(\frac{Q_b}{\sqrt{g} \cdot H^5}\right)$

x = Dimensionless term represent $\left(\frac{W \cdot D}{H^2}\right)$

$$a = 0.001$$

$$b = 0.021$$

Figure (24) Regression Equation Fitting Sand Embankment Data

Mixture (Run₂&Run₃) Regression Equation

$$Y = a + be^{-x} \quad (3)$$

Where:

Y = Dimensionless term represent $\left(\frac{Q_b}{\sqrt{g} \cdot H^5}\right)$

x = Dimensionless term represent $\left(\frac{W \cdot D}{H^2}\right)$

$$a = 0.137$$

$$b = 0.136$$

Figure 25 Regression Equation Fitting Mixture Embankments Data

4 CONCLUSIONS

An experimental investigation to study the effects of soil composition and embankment height on the embankment breach process has been presented. A new methodology to measure the subaqueous breach bathymetry without stopping the experiment has been developed. The main conclusions of this experiment are as follows:

- Physical model tests, especially full-scale and large- scale test are vital for a better understanding of the erosion mechanism of embankment dam breaching and for calibration and verification of the mathematical models.

- The erosion process and soil type have significant effect on the timing and rate of discharge during overtopping.
- The soil erodibility affects the breaching process significantly. Soil with high erodibility has large final breach geometry, shorter failure time and larger peak outflow rate.
- Adding a percentage of clay to filling material of pure sand embankment increases the breach time which will increase the warning time.
- Head-cuts tend to initiate in the construction layer interfaces and in the downstream toe of the embankment in case of clayey (cohesive) soils is observed.
- As the soil strength increases, the peak discharge decreases and the time to peak increases,
- The active clay percent even with its low percent will domain the geotechnical behavior of the mix.
- Breach process has been shown to be highly dependent on soil properties.
- The peak discharge decreased by 20%, the time to peak increased 7 times, the breach width rate of change is decreased by 75% and the maximum breach depth at crest after completed failure decreased by 33% due to the presence of silt (44%) and clay (15%) in the soil mixture, when compared with an embankment built of pure compacted sand.
- The peak discharge decreased by 66%, the time to peak increased 8.4 times, and breach width rate of change at crest decreased by 82% due to decrease the embankment height by 50%.
- The maximum remain vertical cut height for the clayey soils mixture embankments after complete failure is constant disregarding the height of the embankment and show some connection with the critical vertical cut height.
- The short breach initiation time in case of sand (non-cohesive) soils will limit opportunities for operational response, warning, or evacuation.
- Regression equations are developed for the sand embankment (Run_1) and the mixture embankment (Run_2 & Run_3).

5 REFERENCES

[1] Brow, C.A., and W.J. Graham, (1988), "Assessing the Threat to Life from Dam Failure," Water Resources Bulletin, vol.24, no. 6, 1303-1304.

[2] Deky, M.L., and G.H. McClelland, (1991), "Setting Decision Thresholds for Dam Failure Warnings: a Practical Theory-Based Approach," CRJP Technical Report no. 328, Center for Research on Judgment and Policy, Colorado University, Boulder, Colorado.

[3] Costa, John E., (1985), "Floods from Dam Failures," U.S. Geological Survey Open- File Report 85-560, Denver, Colorado, 54p.

[4] Wurbs, Ralph A., (1987), "Dam-Breach Flood Wave Models," Journal of Hydraulic Engineering, Vol. 113, no. 1, p. 29-46.

[5] Singh, V.P., (1996), "dam Breach Modeling Technology," Kluwer Academic Publishers, Boston, Massachusetts.

[6] Morris, M.W., (2000), "A European Concerted Action Project on Dam Break Modeling," Biennial Conference Proceedings, British Dam Society. Thomas Telford.

[7] U.S. ARMY CORPS OF ENGINEERS, (1997), "Engineering and design: Hydrologic engineering requirements for reservoirs, Engineer manual 1110-2-1420, Washington D C.

[8] Wahl, Tony L., (1998), "Predicting Embankment Dam Breach Parameters-A Literature Review and Needs Assessment." DSO-98-004, Dam Safety Research Report, U.S. Bureau of Reclamation.

[9] Hahn, W., Hanson, G.J. and K. R. Cook, (2000), "Breach Morphology Observations of Embankment overtopping Tests." In Proc. Joint Conference of Water Resources Engineering, Planning, and Management. Reston Va.: ASCE.

[10] Fell, R., C.F. Wan, J. Cyganiewicz, and M. Foster, (2003), "Time for development of internal erosion and piping in embankment dams." J. of Geotechnical and Geoenvironmental Engineering, ASCE Vol. 129(4): 307-314.

[11] Hassan, M., M. Morris, G. Hanson, and K. Lakhal, (2004), "Breach formation: Laboratory and numerical modeling of breach formation." In Proc. Dam Safety 2004, CD-ROM. Lexington, ky.: Association of State Dam Safety Officials (ASDSO).

[12] ASTM, (2003). Annual Book of ASTM Standards, Section 4: Construction, Vol. 04.08. Philadelphia, Pa.: ASTM.

[13] Hanson, G. J., and Hunt, S.L., (2007), "Lessons learned using laboratory jet method to measure soil erodibility of compacted soils." Applied Engineering in Agriculture, 23(3):305-312.

[14] ASCE/EWRI, T., (2011). "Earthen embankment breaching." Journal of Hydraulic Engineering, 137(12), 1549-1564.

[15] Coleman, S., Andrews, D., and Webby G. (2002), "Overtopping breaching of noncohesive homogeneous embankments." *Journal of Hydraulic Engineering*, 128(9), 829-838.

[16] Rozov, A. (2003), "Modeling of washout dams." *Journal of Hydraulic Research*, 41(6), 565-577.

[17] Coleman, S.E., Jack, R. C., and Melville, B.W. (1997), "Overtopping breaching of noncohesive embankment dams." *Proc., 27th Congress of the Int. Association for Hydraulic Research*, San Francisco, D42-D47.

[18] Ralston, David C., (1987), "Mechanics of Embankment Erosion During overflow," *Hydraulic Engineering, Proceedings of 1989 ASCE National Conference on Hydraulic Engineering*, Williamsburg, Virginia, August 3-7, 1987, p. 733-738.

[19] Dodge, Russell A. (1988), "Overtopping flow on low embankment Dams, Summary Report of Model Tests", REC-ERC-88-3, U.S. Bureau of Reclamation, Denver, Colorado, August 1988, 28p.

[20] Powledge, George R., D.C. Ralston, P. Miller, Y.H. Chen, P.E. Clopper, and D.M. Temple, (1989a and b), "Mechanics of Overflow Erosion on Embankments. I and II: Hydraulic and Design Considerations," *Journal of Hydraulic Engineering*, vol. 115, no. 8, p. 1040-1075.

[21] Zhang, J., Xuan, G., Wang, X., and Li, J. (2009), "Overtopping breaching of cohesive homogeneous earth dam with different cohesive strength." *Science in China Series E: Technological Sciences*, 52 (10), 3024-3029.

[22] Egyptian code for soil mechanics, no, 202/2-2007, ministry of housing and new communities, national research center for housing and construction, p.4.

[23] A.W. Skempton, (1948), "A study of the geotechnical properties of some post-glacial clays", *Géotechnique*, vol 1, pp 7-22

[24] Egyptian code for soil mechanics, no, 202/7-2007, ministry of housing and new communities, national research center for housing and construction, p.11.