DEVELOPMENT OF CCHE2D EMBANKMENT BREAK

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Abstract: Flooding due to breaching of earthen embankments often results in detrimental impact on the people and their properties downstream in the flooded zone. The embankment breaching process is often caused by overtopping of excessive water in a reservoir or a river. The purpose of this study is to develop a practical numerical model for simulating overtopping and embankment breaching process. To achieve the goal, the key physical-empirical dam breaching mechanism of earthen embankment is adopted and implemented into CCHE2D surface flow model. A special function describing the shape of the breaching channel profile is introduced which significantly simplifies breach modeling. The developed model is validated using experiment data. The simulated flooding hydrograph, headcut migration, and breaching embankment profiles agree well with the observed data. Because this is a 2D model, the development makes it possible to simulate breaching in more complex conditions and study multiple embankment breaches at the same time. This broadens the applicability of the embankment break model significantly.

Keywords: Dam break, flooding, numerical simulation, two dimensional modeling, WinDAM, cohesive soil, overtopping

INTRODUCTION

Earthen embankments are effective infrastructures constructed across waterways or along rivers as dams or levees for flood protection. Due to aging or extreme hydrological events, embankments may fail and result in detrimental flooding. Embankment breaches have been studied by using physical experiments, numerical model simulations, and field observations. Most publications focus on dam breaching, but studies on the embankment erosion processes are relatively rare. Without considering the gradual breaching process of embankments, one may overestimate the dam break flood discharges. In this study, a numerical model to simulate embankment breaching is developed. Considering the complexity of the breaching process and the resulting flooding, a combination of a two dimensional flow model and a reliable dam break process model would have the potential for the best outcome. One can fully take the advantage of the capabilities of 2D models to accurately and reliably simulate embankment breaches in rivers and reservoirs.

Embankment breaching is often simplified as an instantaneous problem which can be simulated using only numerical flow models. There have been many developments in this area in recent years. Frazão and Zech (2002) measured flood waves in a channel bend and developed a finite volume model to simulate the flood. Aureli et al. (2008) simulated dam breach flooding using a finite volume code, the data were obtained by an imaging technique in a small scale experimental flume. Ying and Wang (2008) have developed a 1D finite volume model for flood wave simulation, where the HLL shock capturing scheme was used to solve the hydrodynamics implicitly. Zhou et al. (2004) validated a 2D model of dam breach flow using physical experiment data, with the complex channel geometry handled using cut-cell technique. Jia et al. (2010)

simulated the 2008 Midwest flood in Mississippi River using CCHE2D, a 2D flood model based on the finite element method. Multiple levee breaches and gradual breach processes have been considered, the simulation results are validated using satellite imagery.

Macchione and Sirangelo (1988) simulated the dam break process by solving 1D Saint Venant and sediment continuity equations. The breaching process was treated similar to that of non-cohesive sediment transport. This model was later applied to simulate multiple dam failures and had reasonable results (Macchione and Sirangelo, 1990). Wu and Wang (2006) simulated dam break processes with a 1D model, small-scaled experimental that considered dams constructed of loose materials. The density of sediment was accounted for in the flow modeling, and the non-equilibrium bed load sediment transport approach was applied. Ying and Wang (2010) have simulated the dam breaching process using a 2D model, simulating the flow using the 2D shallow water model based on the control volume method. The breaching process of an earthen dam composed of cohesive materials is simulated based on the linear model for cohesive soil erosion, which assumes that the erosion rate is proportional to the excessive shear stress acting on the soil surface. Wu (2013) proposed a method to simulate the erosional process causing dam breaching. The dam break slope, connecting the dam crest to the front toe, is assumed to be a straight line for non-cohesive sediment and a vertical drop for cohesive earth dams.

Embankment breaching processes have also been studied using physical experiments. Because embankments are constructed using cohesive materials, it takes extensive efforts to study and quantify the process using large scale physical experiments. Research engineers at the USDA-ARS Hydraulic Engineering Research Unit (HERU) constructed scale models of earthen embankments, and then breached them under controlled conditions. Hanson et al. (2005) conducted several outdoor experiments on large size models constructed of cohesive materials providing a unique set of data for dam breaching analyses and model validation. Their experiments have made significant contributions in collecting detailed observation data. Several other large scale dam break experiments have been carried out in Europe (Hassan, 2008; Vaskinn et al. 2001); in addition to homogeneous cohesive materials, breach processes with composite dam material were also tested.

The computer model Windows Dam Analysis Modules (WinDAM, Temple et al. 2006; Visser et al. 2012) has been developed for evaluation of dam breaching processes subjected to overtopping. It includes a reservoir routing model for the auxiliary spillway and the embankment. The model is capable of evaluating the ability of vegetation or riprap on the downstream face of the dam to withstand the overtopping flow. A homogeneous embankment constructed from cohesive soil materials of simple cross section is assumed.

The objective of this study is to develop an embankment breach model based on a two dimensional model of surface flow and the breach mechanism of the WinDAM model. Because the two–dimensional model is capable of simulating flows in both rivers and reservoirs, it will be applicable to both dam and levee breach cases with general flow conditions. In addition, two dimensional models can provide local flow solutions directly for the overtopping flow and the flow in the downstream flooding zone, the simulation results can be more accurate than one dimensional models. Two-dimensional models make it possible to simulate multiple breaches along a channel at multiple locations and under differing local flow conditions. The finite element method based model CCHE2D (Jia et al. 1999, 2002) has been used for this study.

NUMERICAL MODEL

CCHE2D is a depth-integrated model based on the finite element method for simulating free surface flows, sediment transport, bank erosion, vegetation effect, and water quality, and it is also applicable to coastal processes (Jia and Wang 1999, Jia et al. 2002, 2006, Ding et al. 2003, and Zhu et al. 2008). The model uses a collocation method and quadrilateral mesh system. Velocity-pressure coupling is achieved using a partially staggered grid arrangement. The velocity correction method is used to achieve momentum and mass conservation at each time steps. Temporal integration is of a second order Range-Kutta method. In order to increase computational efficiency, wet-dry capability was developed for handling moving boundary problems in areas with complex topography. It has been applied to a wide range of flow conditions from dam breaches of real scale reservoirs, large rivers such as the Mississippi, to small-scale experimental laminar flows.

EMBANKMENT BREACH MODEL

For cohesive earth embankments, the overtopping flow profile has two segments: a segment at the crest and a segment and the headcut. The conceptual evolution of the geometry of dam and levee breaches is depicted in figure 1.

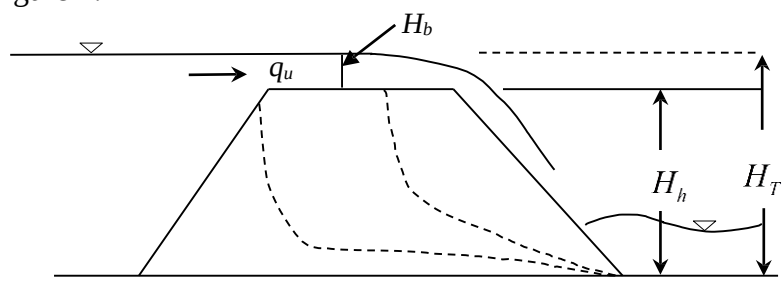

Figure 1 Dam break or levee breaching process sketch

Overtopping induced embankment breach process (figure 1) is normally recognized to have three stages:

1. Overtopping stage: the flow erodes the dam crest surface and front slope surface, these two surfaces lower their elevation almost uniformly. The discharge is relatively small, depending on how much higher the water surface in the reservoir is than that of the crest.
2. Headcut stage: The crest is continuously eroded. Headcut(s) are formed by soil erosion over the front slope and migrate upstream while incising deeper. In this stage the embankment is eroded significantly by the incised channel, but the flow discharge is relatively small because the crest elevation is still quite high. The headcut in the embankment widens due to both lateral erosion and soil mass failure.
3. Embankment breach stage: The headcut migrates to and intercepts with the back of the embankment (upstream slope), the flat dam crest disappeared. Because the remaining embankment is thin and has a sharp crest, its strength to resist flow erosion and water pressure is weakened. The headcut migration rate therefore increases significantly and the discharge increases quickly to the peak flood. The increasing discharge further erodes the remaining embankment crest, deepening and widening this incised channel. The breach

channel widens in this stage with a slower rate. Because the vertical erosion and lateral widening occur at the same time in numerical simulation, the third stage includes the III and IV stages suggested by Hanson (2005).

Because of the complexity of the breaching process, it is difficult to simulate by simply applying a numerical model for hydrodynamics, and sediment erosion and transport. A conceptual model similar to WinDAM was adopted in this study that considered with the following three major processes:

- Frictional erosion on embankment top segment
- Headcut erosion and migration
- Channel widening

Breaching Channel Profile: A simple power function is proposed to represent the headcut profile as indicated in figure 2. The elevation and length of the flat crest is lowered and shortened by top friction erosion and headcut migration. The friction erosion is controlled by the embankment soil property and the flow shear stress. Higher flow rate and more erodible soil would result in faster crest erosion and decrease in elevation. The headcut erosion is physically dominated by frictional erosion and flow plunging impact on soil surface, it is more intensive than crest erosion. One major headcut is considered. In figure 2, (x_T, z_T) is the location of the brink of the dam breach flow or headcut, it connects the two segments and moves in time. (x_b, z_b) is the location of the front toe of the embankment. (x_{tr}, Z_{tr}) and (x_{br}, Z_{br}) is back brink of the embankment crest and embankment back toe. The base line is considered non-erodible in this study.

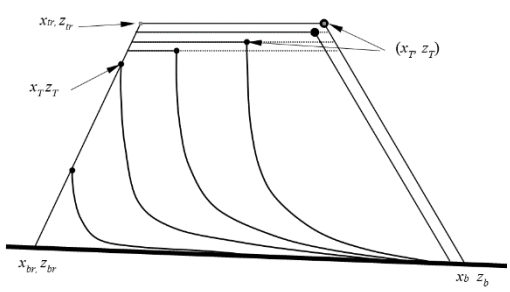

Figure 2 Side view of an overtopping induced dam-break process

It can be seen that the profile across the embankment is mainly represented by three segments:

$$\text{Crest segment}: \quad z = z_T \qquad x \le x_T \tag{1}$$

$$\text{Headcut segment}: \quad \left(\frac{z - z_b}{z_T - z_b}\right)^s = \frac{x - x_b}{x_T - x_b} \qquad x_T \le x \le x_b \tag{2}$$

$$\text{Back slope segment}: \quad \frac{z_T - z_{br}}{z_{tr} - z_{br}} = \frac{x_T - x_{br}}{x_{tr} - x_{br}} \qquad x_{br} < x_T < x_{tr} \tag{3}$$

The crest segment is the horizontal and flat top of the embankment. Its elevation is determined by surface friction erosion; its length can be shortened due to front slope erosion and head cut

migration. The headcut segment is approximated by a function combining the initial straight front slope and the head cut profiles. The shape parameter, *s*, defines a continuous transition from the first stage to second stage of headcuting:

s=1.0 : downstream slope at initial overtopping stage
s>1.0 : headcut stage

Figure 3 illustrates several headcut curves modeled using equation 2, in which *s* varies from 1 to 10.

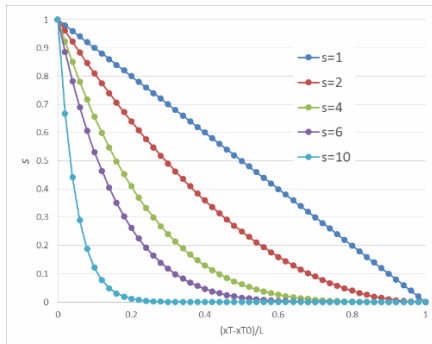

Figure 3 Non-dimensional headcut profile function

The exponent parameter *s* is formulated as a function of x_T and the width of the crest, L_T . It is assumed *s*=1 at the beginning, it increases to its maximum value (*s*=10) when the head cut advances to half of the crest width:

$$\begin{aligned} s &= 1 + 20\frac{(x_{T_0} - x_T)}{L_T}, \qquad (x_T >= x_{T_0} - \tfrac{1}{2}L_T) \\ s &= 10 \qquad\qquad\qquad , \qquad (x_T < x_{T_0} - \tfrac{1}{2}L_T) \end{aligned} \tag{4}$$

Equation 4 is a model established to approximate the process of the changing headcut profile. Figure 4 shows this headcut profile parameter changing from the overtopping stage (*s*=1) to the fully advanced headcut stage (*s*=10). This equation assumes that after the headcut has reached a half of the dam crest (0.5L_T), *s* reaches its maximum value, and the headcut steepness remains unchanged in further process. The assumption would not affect the accuracy because the breach is dominated by the location of headcut $[x_T, z_T]$ and less by the shape of the profile.

Equation 3 represents the back slope segment of the embankment which intercepts with the headcut brink (x_T, z_T) when the headcut advance exceeds the back brink of the crest. At this time the remaining embankment becomes a sharp crest and the migration rate will be increased. By inserting x_T into equation 3, one can compute the crest elevation z_T. Inserting the new (x_T, z_T) into equation 2, a headcut profile for the breaching stage can be computed.

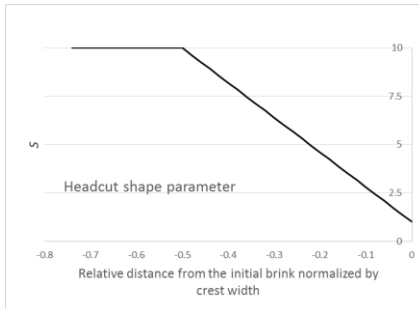

Figure 4 Headcut shape parameter *s*

<u>Embankment Erosion Process:</u> The headcut brink, (x_T, z_T), is the hydraulic control of the entire process. The downward movement of z_T is controlled by friction erosion of the crest. In this study, the crest erosion is governed (Hanson and Cook, 2004 and Hanson and Hunt, 2007) by

$$\frac{dz_T}{dt} = \varepsilon_r = k_d(\tau_e - \tau_c) \tag{5}$$

ε_r is the soil detachment rate in volume per unit time, k_d is the detachment rate coefficient which is an embankment material property parameter, τ_c soil critical shear stress, and τ_e the effective shear stress. Observed data indicate that k_d varies between 0.02~1000 depending on water content, compaction energy and texture of the soil (Henson et al. 2011).

The length of the crest segment will decrease because of the headcut migration. Computing headcut migration is to track the upstream movement of the brink point (x_T, z_T). Temple and Hanson's headcut advance rate model (Temple et al. 2005) is adopted for simulating this process:

$$\frac{dx_T}{dt} = C_h(q_u H_h)^{1/3} \tag{6}$$

where H_h is the headcut height (figure 1), C_h is a calibration parameter for headcut migration rate, q_u is the unit discharge of the overtopping flow. In this study q_u is directly computed by the 2D flow simulation model. The unit discharge would be a constant if the crest is resistant to erosion. In this case, the headcut will advance at a constant rate. If the headcut advances into the reservoir and intercepts the upstream slope of the dam, the brink point will lower its elevation while moving upstream, allowing for more flow to accelerate the breaching process. Experiments indicated that the migration rate would significantly increase when the upstream crest is reached (Hanson et al. 2005). As mentioned earlier, the speed-up is attributed to the weaker embankment condition, the coefficient C_h may have a larger value at the breaching stage. The C_h value is doubled in this study for the headcut migration during the breaching stage.

Widening of the Headcut Channel: In the process of breaching, the breaching channel incises into the embankment, the incision profiles can be computed as indicated in the previous section. However, the incised channel will also be widened due to lateral erosion and mass failure as its bed gets deeper and deeper. The lateral shear force of the flow will erode the materials from the embankment side walls and the steep side slope of the embankment may collapse due to gravity. The widening rate is correlated to the rate of deepening in WinDAM model (Temple et al. 2006): the rate of widening of the breaching channel is of 1.4 times of the rate of deepening. A side wall will therefore retreats 0.7 times of the mean depth erosion in a cross-section. This is equivalent to about 55 degrees along the wet toe of the side slope. In the case of two dimensional modeling, the incised breaching channel is represented by a number of mesh lines and the widening is dominated by the vertical toe erosion of the sidewalls. An effective angle of repose is used to model the channel widening: if the slope near a toe mesh point of the sidewall exceeds this angle due to toe erosion, the sidewall elevation at the higher node is "collapsed" to enforce the angle (figure 5). In the numerical tests, the effective angle of repose is adjusted slightly to fit the observed widening process (Tab. 1).

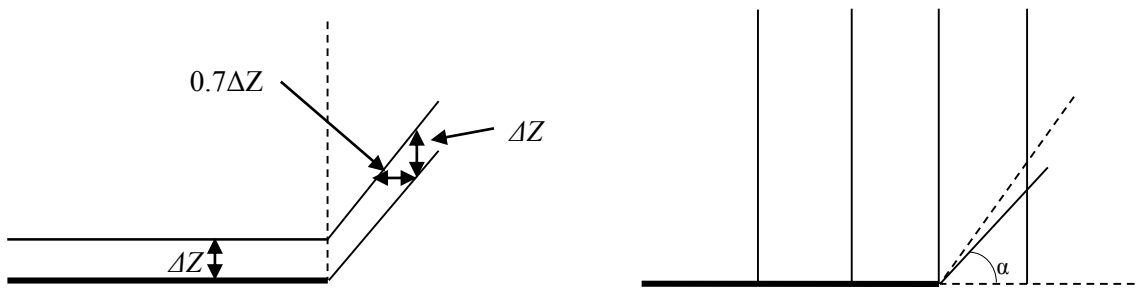

Figure 5 The width increases in time with the lower end being wider.

VALIDATION OF THE IMPLEMENTED CCHE2D DAMBREAK MODEL

Data from the experiment are required to validate the developed model. Large-scale overtopping breach tests on homogenous cohesive embankments conducted by Hanson et al. (2001, 2005) were selected for validation. Three sets of data from the experiments (E1S1, E1S2, and E1S3) were selected for testing. These three tests have the same shape and size of the embankment but built with three different soils. The embankment is 2.3 m high, 22 m long; a 0.46m deep by 1.83m wide notch was opened for the overtopping flow to develop. During the experiments, a flow of 1.0 m3/s is discharged into the reservoir.

Table 1 Parameters used for simulations

	Critical shear stress (Pa)	**Soil erodibility (cm^3/N-S)**	**Headcut coeff C_h**	**Effective angle of repose α(°)**	**Inflow discharge (m^3/s)**	**Soil**
E1S1	0.14	10.3	0.0035	62	1.0	1
E1S2	0.14	2.0	0.000288	50	1.0	2
E1S3	14	0.39	0.00004	55	1.0	3

It is generally recognized that the erosion to the embankment made of cohesive soil is related linearly to an erodibility parameter and the excessive shear stress (Hanson et al. 2001; Hanson, 1989a.) Parameters related to this approach have been calibrated using experimental data to obtain good agreements on comparing the simulated and observed flood process. Table 1 lists the

parameters for the experiments. C_h and α have been calibrated to obtain correct values for headcut advance and breach channel widening.

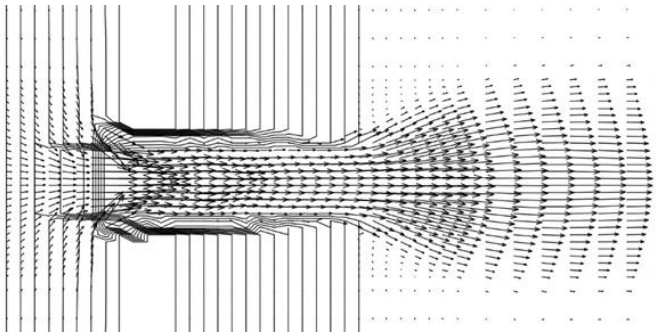

Figure 6 Simulated velocity distribution for E1S1 at t=38 min. Contours indicate the embankment

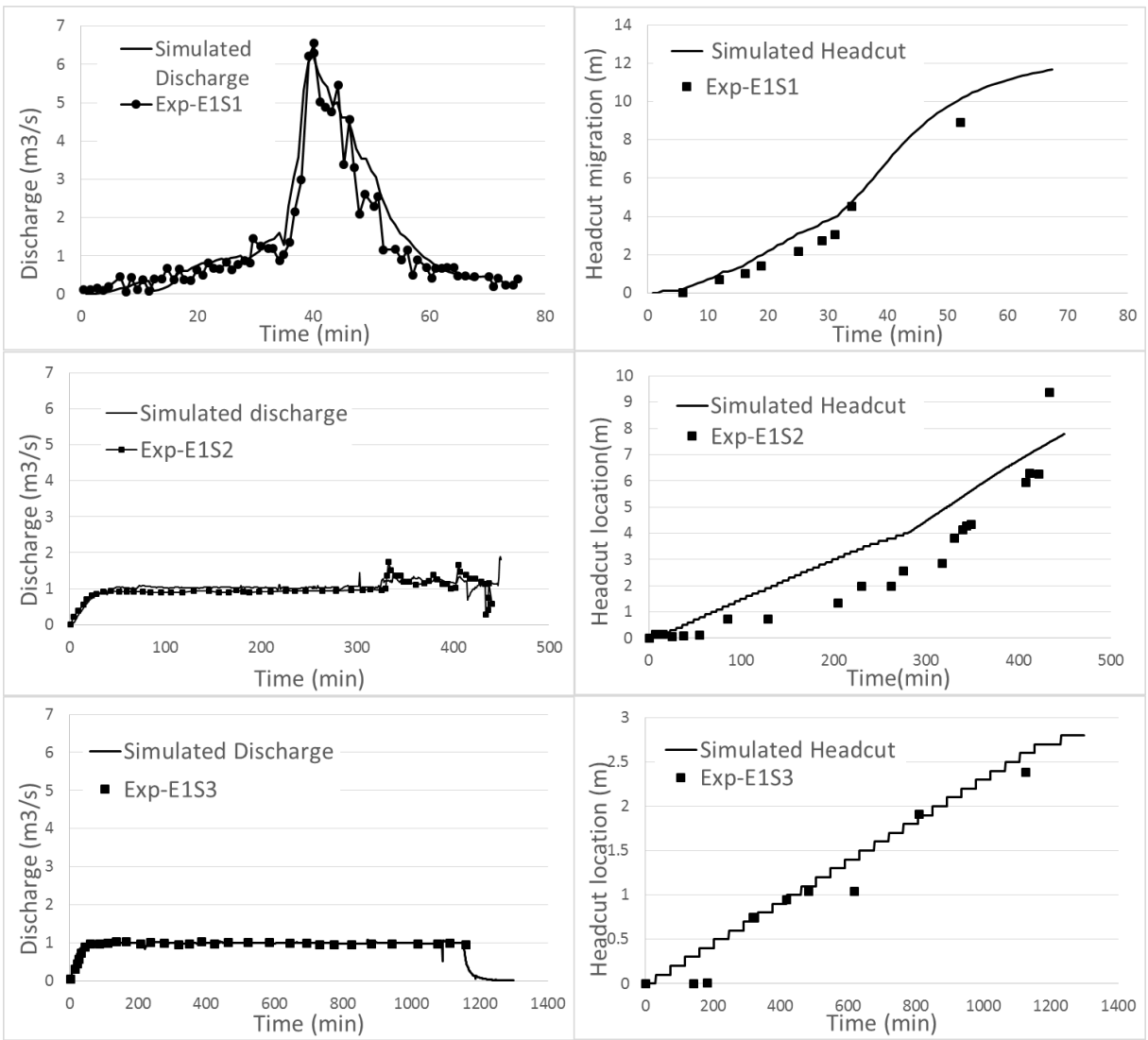

Figure 7 Comparisons of the computed and measured breaching flow discharge and headcut advancement

Figure 6 is a snap shot of the simulated flow field for the case E1S1. The headcut intercepts the back slope of the embankment at t=38 min approximately, the breaching stage starts and flow discharge is increasing rapidly. The vector field indicates the converging flow on the reservoir side and the diverging flow on the outside of the embankment. The variables in equation 5 and 6, τ_c, τ_e, H_h and q_u, are averaged from the simulated two dimensional flow in the friction erosion zone. It is seen the 2D model can represent the hydrodynamics of the breaching flow, the model predicted breaching flood can directly be used for inundation and mitigation analysis.

Figure 7 compares the computed and measured breaching flood discharge and headcut advance of all three test cases. The discharge hydrographs agreed very well including the peak of the flood and the shape of the hydrographs. The simulated headcut migrations also match those observed. Because the advance rate of the headcut is set to be doubled, the simulated migration speed up when the upstream brink of the embankment is reached by the headcut, as seen for E1S1 and E2S2. For the case E1S3, the experiment was terminated before breaching stage, both computed and observed migration are of a constant rate. One notes that the computed headcut location of the case E1S2 is a little ahead of the measured one, about 1 to 2 meters, and the two curves intercept at later time when the observed headcut migration suddenly accelerated.

Figure 8 shows the computed and measured breaching channel profiles of the case E1S2. The general agreement of the two sets of data is very good. The shape of the function-modeled headcut profile at different times are very similar to the measured ones. The 1 minute profile of the simulation has little movement from the initial profile; the measured counterpart has advanced about 1meter. The profiles of 19 minute and 85 minute agreed very well. The simulated profile of 203 minute and 314 minute advanced faster than the measured for about 1-1.5 meters. The agreements for the profiles of 342 minute and 407 minute become good again. These are consistent with headcut advance showing in Fig. 8, that the predicted and measured headcut locations are close at the beginning and near the end, but they have larger differences in the middle of the process.

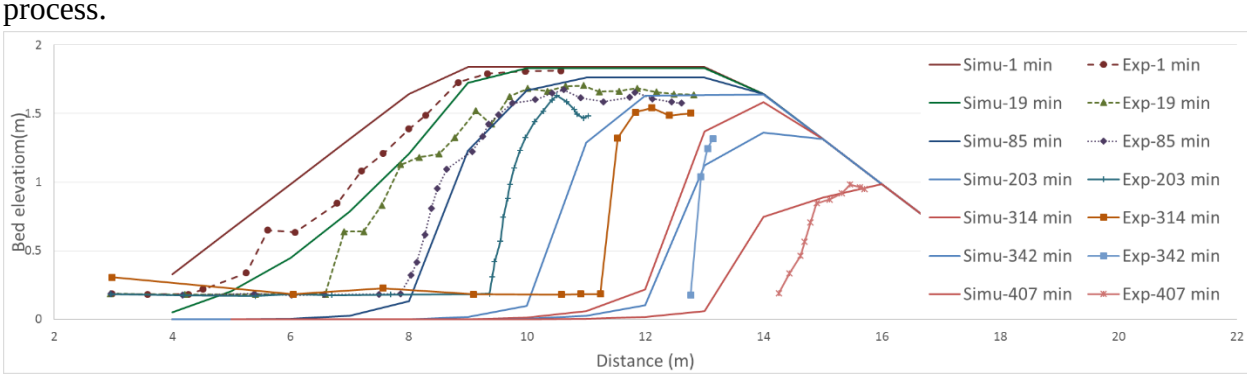

Figure 8. Comparisons of simulated and observed headcut profiles for case E1S2.

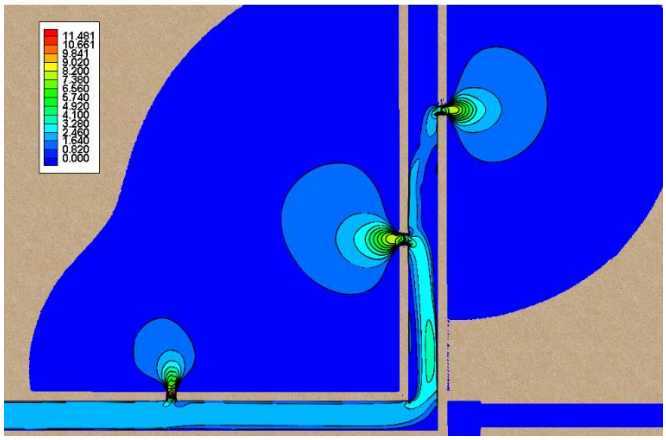

Figure 9 Simulated large scale test case with multiple levee breaching. The horizontal length of the figure is 2 km, the channel width is 100 m.

Figure 9 shows a hypothetical levee embankment breaching test. The flow velocity distribution of the simulated flooding results at about 14 hours is displayed. The maximum velocity exceeds 11m/s. The spatial scale of the simulation is quite large, the width of the domain is 2000 m, the width of the channel is about 100 m, the flow discharge is 1200m^3/s, and the relative levee height is about 7 m. A steady flow is established first, and three notches are made at the breach locations. A continuing run of the model resulted in three breaches along the channel. Because the parameters of the embankment at the three locations are set differently, the flow conditions along the channel and the simulated breach developments are different. This example demonstrates that a two dimensional model can significantly extend the application of the embankment breach model such as WinDAM.

CONCLUSIONS

A two-dimensional embankment breach model is developed by integrating the physical-empirical breach mechanism of the WinDAM model and the general surface flow model, CCHE2D. The new model is capable of simulating one or multiple embankment breaches at the same time in more complicated general surface water conditions. It therefore has a wider potential to be applied for flood prediction and disaster mitigation. A function is proposed to model the breach channel of the headcut which transitions the breach channel profile from the straight line at the beginning to the steep headcut at the end of the process. The function simplifies the computation because no erosion simulation is needed in the area from the headcut brink to the toe of the embankment. Validation tests using three sets of large scale experimental data have had good results indicating the developed model can really predict the physical process of dam breach or levee breaching. In particular, the modeled headcut profiles are very close to those measured. The developed model is embedded in the graphic user interface of the CCHE2D model, making applications of the model very easy.

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