Peer-reviewed version available at *Water* **2017**, *9*, , 324; doi:10.3390/w9050324

Article

Simplified Simulation Method for Flood-Induced Bend Scour—A Case Study Near the Shuideliaw Embankment on the Cho-Shui River

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Abstract: The modeling of flood-induced bend scour near embankment toes can provide important information for river engineering, embankment safety warnings, and emergency action management. During the rainy seasons, short-term general scour and bend scour are the most common causes for the failure of reinforced concrete embankments in Taiwan. To gain a deeper understanding of the scouring process near levee foundations, this study proposed a straightforward and practical method for bend scour simulation. The proposed simulation method is subdivided into three stages: two-dimensional flow simulation, general scour estimation, and bend scour estimation. A new bend-scour computation equation is proposed and incorporated into a two-dimensional hydraulic finite-volume model for simulating the evolution of bend scour depth around embankment toes. The proposed method is applied to simulate the temporal evolution of bend scouring near the Shuideliaw Embankment on the Cho-Shui River in Taiwan, where serious failure occurred during the June 2012 monsoon. Field data were gathered using the numbered-brick technique at the Shuideliaw Embankment to demonstrate the accuracy of the proposed method. The results of the bend scour simulations compared reasonably well with field measurements, indicating close agreement in terms of water levels and bend scour depths near the Shuideliaw Embankment. The proposed method was found to quickly estimate the maximum short-term general scour and bend scour depths for further enhancement of the safety of the embankment toe.

Keywords: bend scour; embankment toe; finite-volume model; numbered bricks

1. Introduction

Embankments are natural or man-made hydraulic structures used to prevent inundation disasters in adjacent flood-prone areas. In general, embankment failure occurs during a flood as the result of several possible factors: (1) overtopping of the embankment crest, (2) scouring at the toe, (3) piping or internal erosion, and (4) toe or slope sliding of the embankment [1-4]. These potential mechanisms can result in the breaching or failure of embankments, which can occur as a single mechanism or a combination of different mechanisms. The failure of embankments due to multiple mechanisms may cause widespread flooding and result in a significant loss of life. In Taiwan, the Water Resource Agency (WRA) [5] reported that approximately 43% of embankment failures are associated with scouring at the toe. On August 8, 2009, embankment failures due to Typhoon Morakot caused catastrophic inundation and serious property damage in Southern Taiwan. The Shuideliaw Embankment on the Cho-Shui River was damaged during the June 2012 monsoon and Typhoon Saola (August 2012), with peak discharges of 6,120 m^3/s and 10,600 m^3/s, respectively. Moreover, the adjacent irrigation channels were damaged during these flood events.

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To mitigate potential embankment hazards, the prediction of flood-induced embankment failure is a basic necessity. A number of numerical models have been developed over the past 50 years to predict the embankment erosion and failure process [6-13]. These models can be divided into two major categories: (1) parametric models and (2) physically based models. The parametric model uses regression analysis to establish statistically based equations that can be used to estimate embankment breaching width and discharge. The physically based model uses sediment transport formulas to estimate the rate of embankment erosion according to a reasonable approximation of the evolution of embankment breaching. According to a review of the literature, these models can be used to simulate embankment breach formation, growth, and closure. Although many published studies have investigated earthen embankment erosion and failure processes, there has been little focus on reinforced concrete embankments. In Taiwan, an extreme flood can erode the riverbed near embankment toes and result in their failure. Hence, scour has been the major safety concern for reinforced concrete embankment structures.

The more commonly used approach adopts numerical methods to simulate the scouring around cross-river structures. Several studies have adopted numerical methods to investigate the scour process around bridge piers [14–18] and super dikes [19-21]. Time-consuming three-dimensional (3D) numerical models produce sharp resolutions of 3D flow fields and local scour around hydraulic structures under laboratory conditions. Moreover, these 3D models have not been as successful in practical applications, where more field data have been required. By contrast, two-dimensional (2D) numerical models can provide accurate hydraulic characteristics at the locations far from the local structures. Additionally, from a practical perspective, 2D models can provide fundamental approach flow information to estimate both general and local scour depths. This enables the combination of 2D models with reliable scour-depth equations for the simulation of total scour processes [22].

According to the literature review, most previous research has focused on embankment erosion and earthen embankment failure. As the majority of embankment failure in Taiwan is due to toe scouring, a new practical method is required that explicitly relates to embankment toe scour. The scouring process around the embankment toe in a river bend leads to bend scouring at the toe of the outer embankment, in which riverbed variation during typhoon periods results in the undermining of embankment structures [23]. Thus, a reliable method is needed for estimating the maximum scoured depth in a river bend in order to design appropriate embankment protection or establish an embankment warning system.

This paper presents a new modeling methodology for simulating the temporal evolution of scour depth at embankment toes in river bend reaches. Its practical applicability is demonstrated by a case study of the Cho-Shui River, in which the Shuideliaw Embankment is located in a gravel-bed river bend. Due to river bend effects, bend scour occurs at the outer embankment; in other words, the riverside of the Shuideliaw Embankment. Hence, this paper integrates the approach flow of the hydraulic model [24-26], which is a 2D shallow water flow model based on the finite-volume model, with a general-scour computation equation [22], three existing bend scour equations [27-29], and a new bend-scour computation equation. In addition, this paper investigates flood-induced embankment toe scour by using the field data collected with the numbered-brick method. The proposed model is then verified through simulations of bend scour depth at the Shuideliaw Embankment under two flood events. Comparisons are made between the simulated and field-measured maximum scour depths [30, 31]. The practicability of the method is further confirmed through its application to the embankment failure event that occurred during Typhoon Saola in 2012. Based on nine scenario simulations, a new practical embankment safety curve representing the scoured depth-discharge relationship is finally proposed for use in river management.

2. Field Data Collection and Formula Derivation

Field measurement of embankment erosion or scour is very difficult, especially during high flow conditions. Flow conditions during floods are extremely dangerous, complicating the measurement of riverbed variation such as riverbed scour. In this study, 12 sets of bend scour data were collected to

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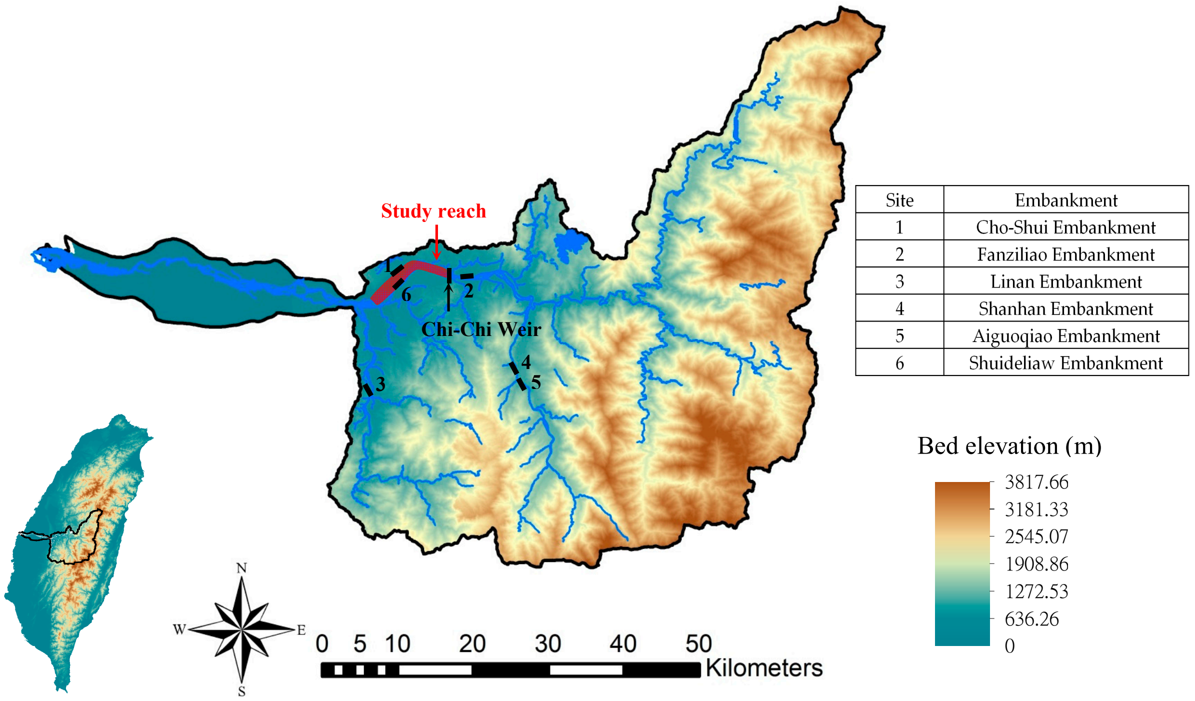
develop a new bend scour computation equation for rivers in Taiwan. Specifically, 10 and 2 sets of scour data were collected from the basins of the Cho-Shui and Da-Chia Rivers, respectively, for the database. Two sets of the data collected in the Cho-Shui River basin were near the Shuideliaw Embankment and an irrigation channel directly behind the embankment. Therefore, the scour data collected near the Shuideliaw Embankment can be used to accurately evaluate the performance of the proposed method.

2.1. Site Description

The field study site was chosen at the middle reach of the longest river in Taiwan, Cho-Shui River, from Chi-Chi Weir to Formosa Freeway No. 3 Bridge on the Cho-Shui River. A map of the Cho-Shui River basin and an aerial photograph near the Shuideliaw Embankment are shown in Figures 1 and 2, respectively. The Cho-Shui River upstream of the Chi-Chi Weir, which was constructed to provide water for irrigation and industrial consumption, has a watershed area of 2,034 km^2. The aerial photograph in Figure 2(a) shows two concave embankments located near the Mingchu Bridge: one on the right-hand side upstream (but facing downstream) of the Mingchu Bridge, and the other on the left-hand side downstream of the bridge, where the Shuideliaw Embankment is located.



Site	Embankment
1	Cho-Shui Embankment
2	Fanziliao Embankment
3	Linan Embankment
4	Shanhan Embankment
5	Aiguoqiao Embankment
6	Shuideliaw Embankment

Figure 1. Topographic map of study site in the Cho-Shui River basin.

To prevent embankment erosion, the Fourth River Management Office constructed the Cho-Shui and Shuideliaw Embankments on the right- and left-hand sides (facing downstream) near the Mingchu Bridge, respectively. The Mingchu Bridge and Formosa Freeway No. 3 Bridge are respectively 6.5 km and 8.5 km downstream of the Chi-Chi Weir. According to a field survey by the Water Resources Agency, the mean channel slope in the middle reach of the Cho-Shui River is 0.006, and there is no tributary between the Chi-Chi Weir and the Formosa Freeway No. 3 Bridge; therefore, the flow discharge at Mingchu Bridge and Shuideliaw Embankment was assumed to be the flow released from the Chi-Chi Weir. Because the distance between the field site and the river mouth is more than 45 km, the tidal effect does not influence the flows at the Mingchu Bridge and Shuideliaw Embankment.

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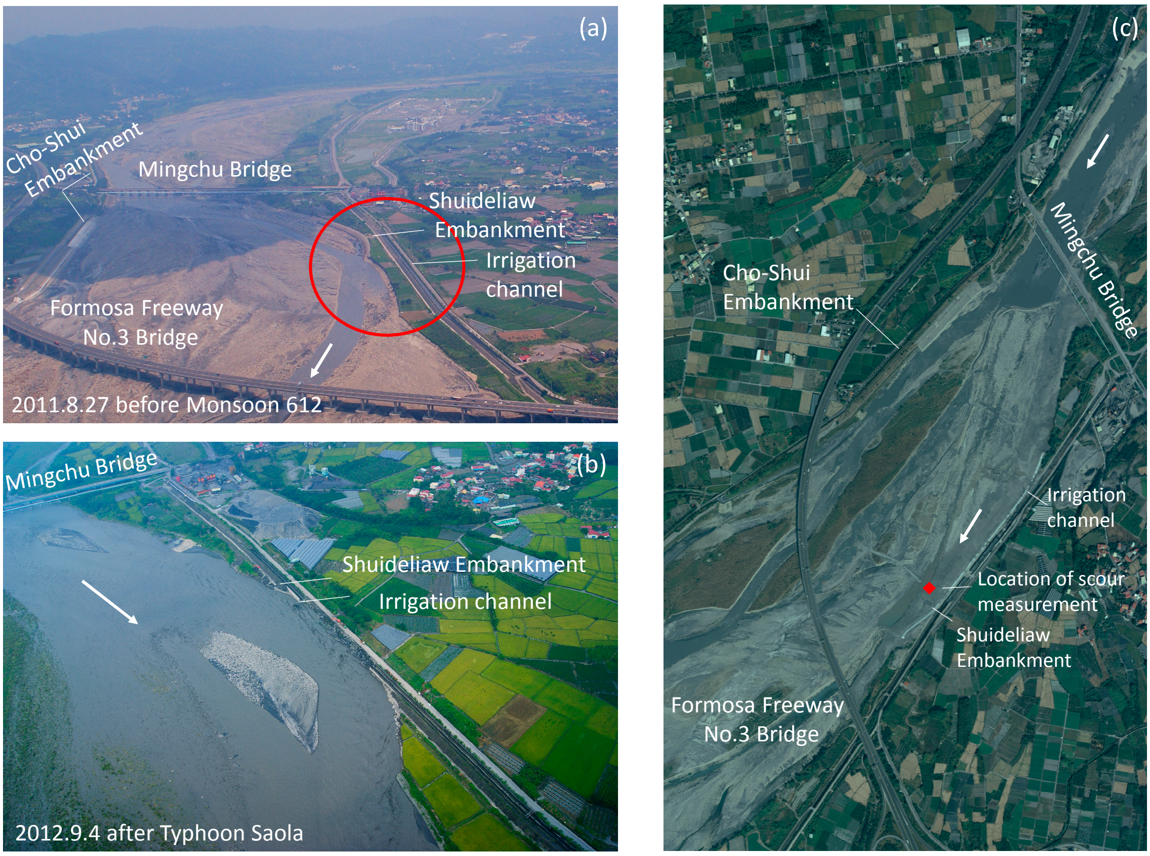

Figure 2. Scour measurement system for study site of Shuideliaw Embankment showing (a) an aerial view before Typhoon Saola; (b) looking downstream after Typhoon Saoloa and (c) location of scour measurement.

Bed samples obtained in 2013 in the vicinity of the Shuideliaw Embankment show that the sediment was composed of noncohesive alluvial gravels with various sands. Figure 3 shows the size distribution of sediment particles near the Shuideliaw Embankment, clearly demonstrating that the size distribution of sediment particles is bimodal. A survey of the bed materials revealed that the returned sediment sizes of approximately d_{16} = 1.7 mm, median size d_{50} = 108 mm, and d_{84} = 329 mm.

2.2. *Field Data*

Su and Lu [32] proposed a highly efficient numbered-brick method to measure the general scour in natural channels. Lu [31] adopted this direct observation method for measuring the maximum bend scour depth near the Shuideliaw Embankment. In the present study, bend scour field data were collected as listed in Table 1. All of the study sites were associated with the scour measurements near the embankment toe and all of the scour depths were measured using the same numbered-brick method. The peak flow discharge $\boldsymbol{Q_P}$ ranged from 164 m^3/s to 5309 m^3/s and the flow intensity (unit peak flow discharge $\boldsymbol{q_P}$) ranged from 1.31 m^2/s to 25.37 m^2/s. The channel slope $\boldsymbol{S_0}$ was also an important factor, with a range from 0.0063 to 0.0153, and the median particle size $\boldsymbol{d_{50}}$ of the bed materials was from 3.43 mm to 168 mm. The characteristic parameter for bend scour is the outer radius of the bend R_0, which was recorded to range from 169 m to 800 m. Finally, the measured maximum scour depth in a river bend was from 0.22 m to 5.23 m. With these measured data, a reliable bend-scour computation equation was developed, as described in Section 2.3. Furthermore, the measured data for two flood events, Typhoon Trami and Typhoon Usagi, were adopted in the present study to verify the accuracy of the proposed method, as described in Section 4.2. Figure 4 shows the flow hydrographs released from the Chi-Chi Weir for Typhoon Trami (Aug 2013) and Typhoon Usagi (Sep 2013).

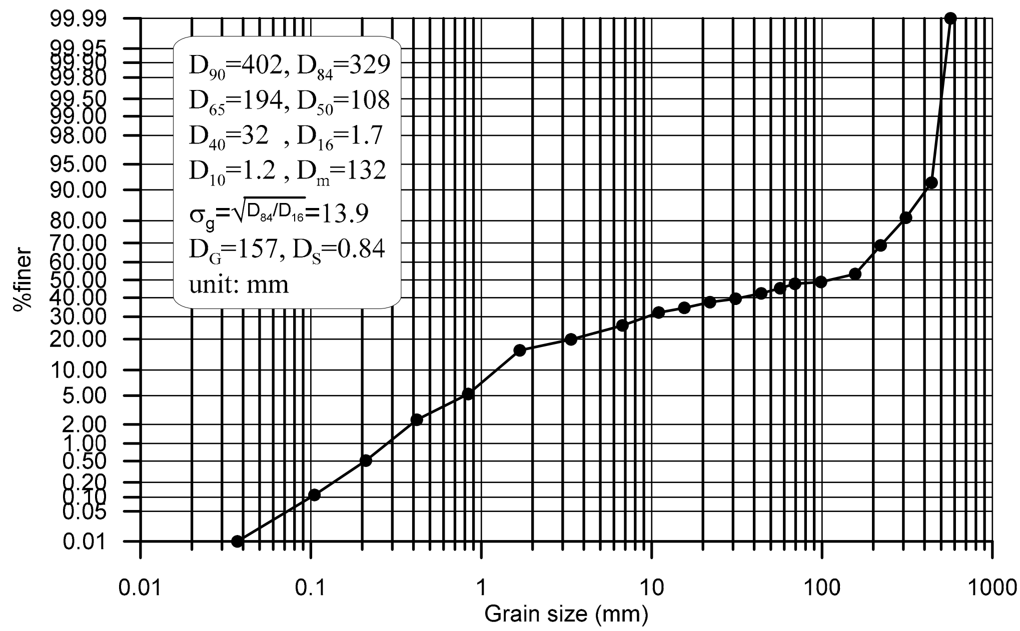

Figure 3. Size distribution of sediment particles in the study site, where the survey conducted in 2013.

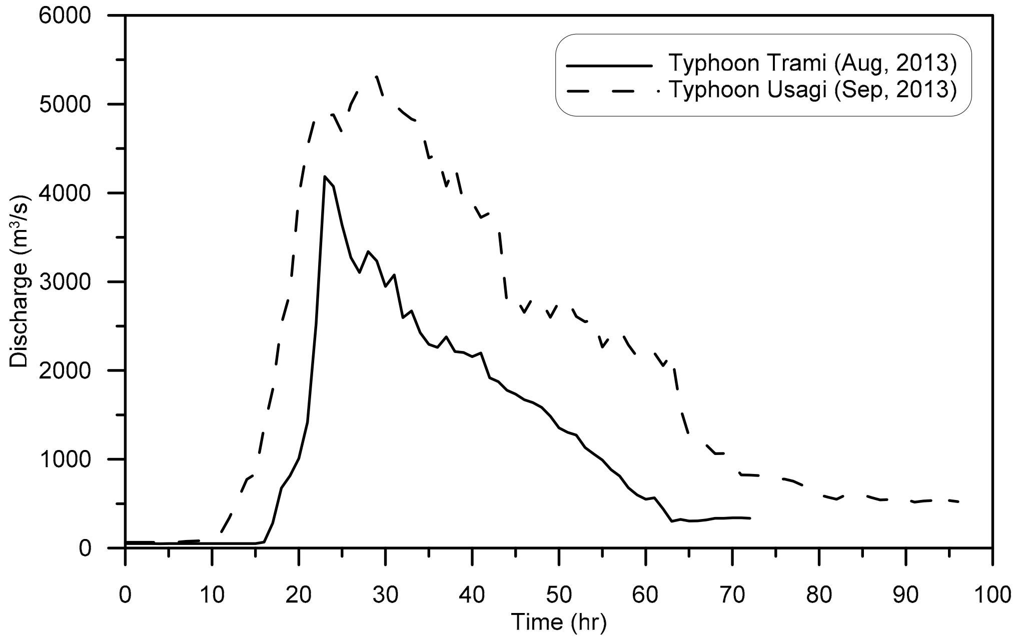

Figure 4. Flow hydrographs for Typhoons Trami (Aug, 2013) and Usagi (Sep, 2013).

Table 1. Field data collection for short-term bend scour induced by floods.

River	Site	Flood Event	Q_p (m³/s)	q_p (m²/s)	s_0	d_{50} (mm)	R_0 (m)	d_{bs} (m)	Remark
Cho-Shui River	Cho-Shui Embankment	1. Typhoon Nanmadol (August, 2011)	164	1.48	0.0063	3.43	605	0.67	Lin and Lin [30]
	Fanziliao Embankment	2. Typhoon Talim (June, 2012)	346	1.86	0.0071	23.5	418	0.22	
		3. Typhoon Saola (August, 2012)	475	2.97	0.0057	7.5	418	0.8	
	Linan Embankment	4. Typhoon Nanmadol (August, 2011)	186	1.31	0.0066	85	558	0.36	
		5. Typhoon Saola (August, 2012)	2279	13.21	0.0066	34.5	558	2.36	
	Shanhan Embankment	6. Typhoon Talim (June, 2012)	562	4.44	0.0153	19.5	471	1.9	
	Aiguoqiao Embankment	7. Typhoon Talim (June, 2012)	1182	11.87	0.0112	48	169	0.47	
		8. Typhoon Saola (August, 2012)	1837	14.94	0.0148	25.5	169	2.45	
	Shuideliaw Embankment	9. Typhoon Trami (August, 2013)	4186	25.37	0.00527	108	800	1.71	Lu [31]
		10. Typhoon Usagi (September, 2013)	5309	18.31	0.00527	108	800	3.30	
Da-Chia River	Fengzhou Embankment	11. Typhoon Soulik (July 2013)	6692	22.01	0.00755	168	500	5.23	
		12. Typhoon Trami (August, 2013)	2393	10.64	0.00755	168	500	0.55	

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2.3. Proposed Bend Scour Computation Equation

Several empirical equations for estimating bend scour have been developed in the literature [23]. Three previous empirical equations are used in this study, and the corresponding schematic definition terms are shown in Figure 5.

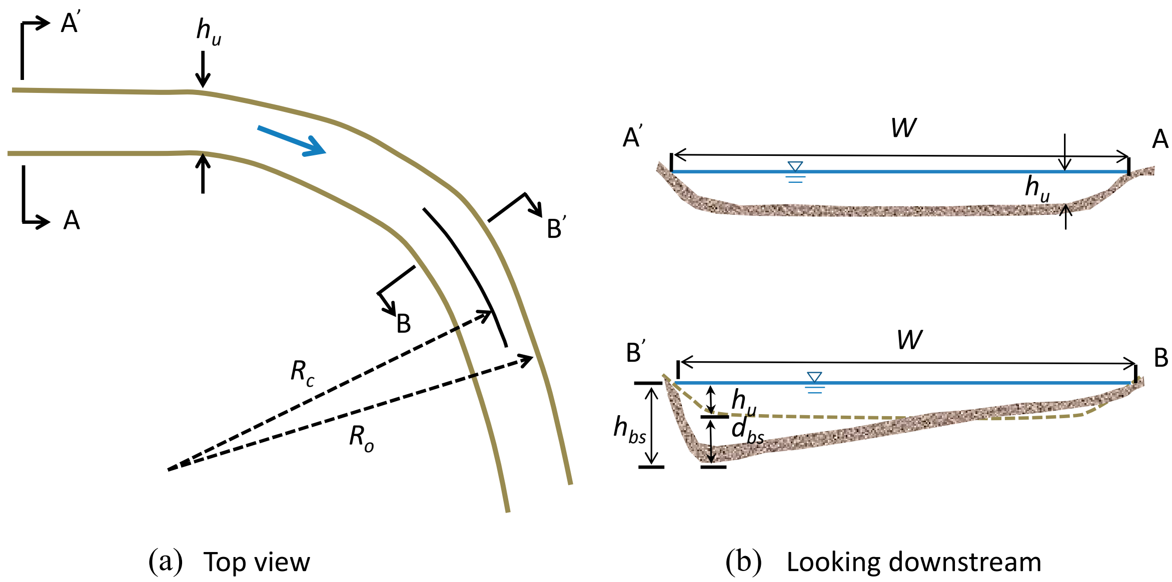

Figure 5. Definition sketch showing the bend scour: (a) top view; (b) cross-sections looking downstream.

First, Galay et al. [27] developed an empirical equation for rigid (noneroding outer bank) bend of 60° in a gravel-bed river reach:

$$\frac{h_{bs}}{h_u} = 1.2 + \frac{W}{R_c} \tag{1}$$

where h_{bs} denotes the maximum flow depth in the bend cross-section $B'B$, h_u is the water depth in the upstream straight reach $A'A$, W denotes the water surface width and R_c is the centerline radius of the bend. Next, using the scour data from 70 bends in a reach of the Red River between Arkansas and Louisiana in the United States, Thorne [28] developed the following regression equation:

$$\frac{h_{bs}}{h_u} = 2.07 - 0.19 \ln\left(\frac{R_c}{W} - 2\right) \tag{2}$$

Notably, Equation (2) is limited to R_c / W greater than 2.0 based on the scour data set. However, a best-fit regression equation is found in U. S. Army Corps of Engineers [29] and presented as:

$$\frac{h_{bs}}{h_u} = 2.57 - 0.36 \ln\left(\frac{R_c}{W}\right) \tag{3}$$

These three equations were developed using the bend factors R_c and W as the primary parameters. However, on the basis of the bend scour data collected in this study, the sediment size as well as the standard deviation of particle size distribution are found to significantly affect the bend scour depth. Therefore, this study proposes a new computation equation. The characteristic parameters of bend scour estimation can be defined as:

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$$\phi\left(\underbrace{d_{bs},q,\rho,s_0,g}_{\text{flow}},\underbrace{d_{50},\sigma_g,\rho_s}_{\text{sediment}},\underbrace{W,R_0}_{\text{bend}}\right) \tag{4}$$

where d_{bs} is the bend scour depth; q is the unit discharge; ρ is the density of fluid; g is the gravitational acceleration; s_0 is the channel slope; d_{50} is the median particle diameter; σ_g is the geometric standard deviation of the particle size distribution; ρ_s is the density of sediment particles; and R_0 (= R_c + 0.5W) is the outer radius of the bend. Based on Buckingham's π theorem, bend scour depth can be written as the following dimensionless form:

$$\frac{d_{bs}}{d_{50}}=\phi'\left(\frac{q}{\sqrt{(\rho_s/\rho-1)gd_{50}^3}},s_0,\sigma_g,\frac{W}{R_0}\right) \tag{5}$$

In the present study, multiple regression analysis is then conducted with the measured 12 data listed in Table 1, and Equation (5) is obtained as:

$$\frac{d_{bs}}{d_{50}}=0.106\left(\frac{q}{\sqrt{(\rho_s/\rho-1)gd_{50}^3}}\right)^{0.932} s_0^{\ 0.122}\sigma_g^{\ 0.459}\left(\frac{W}{R_c+0.5W}\right)^{0.116} \tag{6}$$

which has a coefficient (R-squared value) of 0.915, indicating that the short-term bend scour depths computed by Equation (6) fit the measured data well as shown in Figure 6. Bend scour depth was found to be directly proportional to unit discharge, channel slope, and water surface width, and inversely proportional to the centerline radius of the bend.

3. Method for Embankment Toe Scour Simulation

For an embankment located on a river bend, bend scour can lead to increased scour at the embankment toe. According to Melville and Coleman [23], bend scour may occur in a bend in a river reach and is categorized as short-term general scour. This paper proposes a simple and practical method to estimate the evolution of scour depth at an embankment toe on a river bend. The evolution of general scour depth is estimated by combining a 2D finite-volume model with a general-scour computation equation. The approach flow depth and velocity upstream of the embankment toe are then modified and the revised approach flow conditions are used to determine bend scour depth evolution.

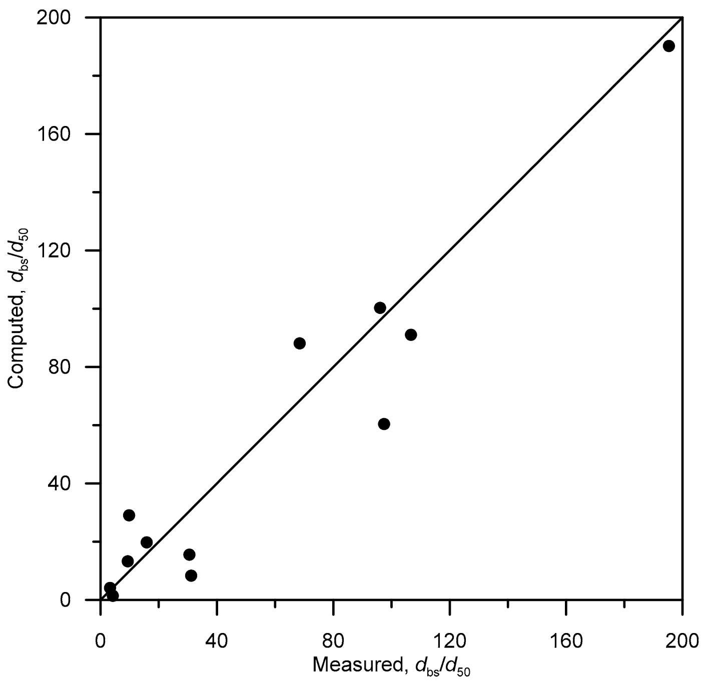

Figure 6. Measured versus computed bend scour depth using the proposed Equation (6).

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3.1. 2D Finite-Volume Hydraulic Model

The present 2D model [24-26] is based on a finite-volume model in the unstructured arbitrarily shaped mesh system. The governing equations for 2D shallow water flow, including the continuity and momentum equations, can be expressed as:

$$\frac{\partial \mathbf{Q}}{\partial t}+\frac{\partial \mathbf{F}_I}{\partial x}+\frac{\partial \mathbf{G}_I}{\partial y}=\frac{\partial \mathbf{F}_V}{\partial x}+\frac{\partial \mathbf{G}_V}{\partial y}+\mathbf{S} \quad (7)$$

in which

$$\mathbf{Q}=\left[h, hu, hv\right]^{\mathrm{T}} \quad (8)$$

$$\mathbf{F}_I=\left[hu, hu^2+1/2\,gh^2, huv\right]^{\mathrm{T}};\ \mathbf{G}_I=\left[hv, huv, hv^2+1/2\,gh^2\right]^{\mathrm{T}} \quad (9)$$

$$\mathbf{F}_V=\left[0, hT_{xx}/\rho, hT_{xy}/\rho\right]^{\mathrm{T}};\ \mathbf{G}_V=\left[0, hT_{xy}/\rho, hT_{yy}/\rho\right]^{\mathrm{T}} \quad (10)$$

$$\mathbf{S}=\left[0, gh\left(s_{0x}-s_{fx}\right), gh\left(s_{0y}-s_{fy}\right)\right]^{\mathrm{T}} \quad (11)$$

where $\mathbf{Q}$ is the vector of conserved variables; $\mathbf{F}_I$ and $\mathbf{G}_I$ are the inviscid flux vectors in the x- and y-directions, respectively; $\mathbf{F}_V$ and $\mathbf{G}_V$ are the viscous flux vectors in the x- and y-directions, respectively; $\mathbf{S}$ is the source term; h is the water depth; u and v are the depth-averaged velocity components in the x- and y-directions, respectively; ρ is the density of flow; T_{xx}, T_{xy}, and T_{yy} are the depth-averaged turbulent stresses; g is the gravitational acceleration; s_{0x} and s_{0y} are the bed slopes in the x- and y-directions, respectively; and s_{fx} and s_{fy} are the friction slopes in the x- and y-directions, respectively.

The finite-volume model was adopted to solve the 2D governing equations, leading to the following basic vector equation:

$$A\frac{\mathrm{d}\mathbf{Q}}{\mathrm{d}t}+\sum_{m=1}^{M}\mathbf{T}(\theta)^{-1}\mathbf{F}(\bar{\mathbf{Q}})L^m=\tilde{\mathbf{S}} \quad (12)$$

in which A is the area of the cell; m is the index that represents the side of the cell; M is the total number of the sides for the cell; $\mathbf{T}(\theta)^{-1}$ is the inverse of the rotation matrix corresponding to the m side; θ is the angle between the outward unit vector $\mathbf{n}$ and the x-axis; $\mathbf{n}$ is the outward unit vector normal to the boundary of the control volume; L^m is the length of the m side for the cell; $\mathbf{F}(\bar{\mathbf{Q}})=\mathbf{F}_I(\bar{\mathbf{Q}})-\mathbf{F}_V(\bar{\mathbf{Q}})$ is the numerical flux; $\mathbf{F}_I(\bar{\mathbf{Q}})$ represents the inviscid numerical flux; $\mathbf{F}_V(\bar{\mathbf{Q}})$ denotes the viscous numerical flux; and $\tilde{\mathbf{S}}$ is the integral form of the source terms. To resolve discontinuous shock waves or hydraulic jumps, many different types of approximate Riemann solvers have been developed for estimating the inviscid numerical flux $\mathbf{F}_I(\bar{\mathbf{Q}})$. This paper employs an upstream flux-splitting finite-volume scheme [24] to obtain the inviscid numerical flux. In addition, the Jacobian transformation method [25] is used to estimate the viscous numerical flux.

To avoid the generation of numerical errors due to unphysical high velocities at wet/dry fronts, the wet/dry boundary treatment of the model is improved herein. This paper introduces a positive tolerance depth h_{tol} to deal with wet/dry boundary tracking:

1. The cell interface is the dry edge, when $h_L \le h_{tol}$ and $h_R \le h_{tol}$, in which h_L and h_R are the left and right water depths at the center of the cell interface, respectively. There is no flux estimation.
2. The cell interface is the wet edge, when $h_L > h_{tol}$ and $h_R > h_{tol}$. The upstream flux-splitting finite-volume scheme coupled with the hydrostatic reconstruction method [24] is adopted to estimate the well-balanced numerical fluxes.
3. The cell interface is the partially wet edge with flux, when $h_L > h_{tol}$, $h_R \le h_{tol}$, and $h_L+(z_b)_L > h_R+(z_b)_R$, where z_b is the bed elevation. The momentum flux is set at zero and the mass flux $(hu)_{LR}$ at cell interface LR is estimated as:

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$$\delta\eta = h_L + (z_b)_L - h_R - (z_b)_R \tag{13}$$

$$(hu)_{LR} = 1.42\delta\eta\sqrt{|\delta\eta|} \tag{14}$$

4. The cell interface is the partially wet edge without flux, when $h_L > h_{tol}$, $h_R \le h_{tol}$, and $h_L + (z_b)_L \le h_R + (z_b)_R$. According to the bed elevation condition, there is no flux across the cell interface. Thus, no flux estimation is required.

A detailed description of the model can be found in the literature [24-26]. In practice, the presented 2D model adopts a well-balanced upwind numerical scheme coupled with a robust wet/dry algorithm. Hence, the model is a useful hydraulic modeling tool to simulate 2D hydraulic flows involving irregular bed topography.

3.2. General-Scour Computation Equation

Many of the equations for scour estimation (general, contraction, or local scour) were developed according to the analysis of data from laboratory research. However, these equations derived from laboratory studies may not be suitable for field application of scouring. To overcome this problem, Hong et al. [22] proposed a general-scour computation equation. This reliable equation is based on the results of general scour depth measurements through the numbered-brick method. The dimensionless equation can be expressed as:

$$\frac{d_{gs}}{d_{50}} = 7.271\left(\frac{q}{\sqrt{(\rho_s/\rho - 1)gd_{50}^3}}\right)^{0.514} s_o^{\;0.071}\left(\sigma_g\right)^{-0.014} \tag{15}$$

in which d_{gs} denotes the short-term general scour depth. Compared with several commonly used equations, the proposed equation [22] utilized more parameters that significantly affect the general scour depth. Hence, this equation [22] is the optimal choice to date for estimating the short-term general scour depth in a gravel-bed river, especially for the Cho-Shui River in Taiwan.

3.3. Method for Simulating Bend Scour-Depth Evolution

As shown in Figure 7, the algorithm for estimating bend scour depth evolution is summarized as follows:

1. Simulating the 2D flow field in the bend reach through the proposed 2D finite-volume hydraulic model, as described in Section 3.1.
2. Obtaining approach flow conditions, including water depth, velocities, water surface width, and the centerline radius of the bend.
3. Adopting a general-scour computation equation (i.e., Equation (15)) to estimate general scour depth evolution.
4. Re-evaluating hydraulics properties based on the unit flow discharge.
5. Obtaining the revised flow conditions: $\tilde{h} = h + d_{gs}$ and $\tilde{v} = q/\tilde{h}$.
6. Using bend-scour computation equations (Section 2.4) to estimate the bend scour evolution of the embankment toe (i.e., $h_u = \tilde{h}$).

The proposed bend scour equation is used to directly estimate the scoured depth d_{bs} in a river bend. However, the other existing equations used to estimate the maximum flow depth is the bend cross-section h_{bs} . To compare the computed results with these existing equations, the computed maximum flow depth can be transferred to the bend scour depth as $d_{bs} = h_{bs} - \tilde{h}$.

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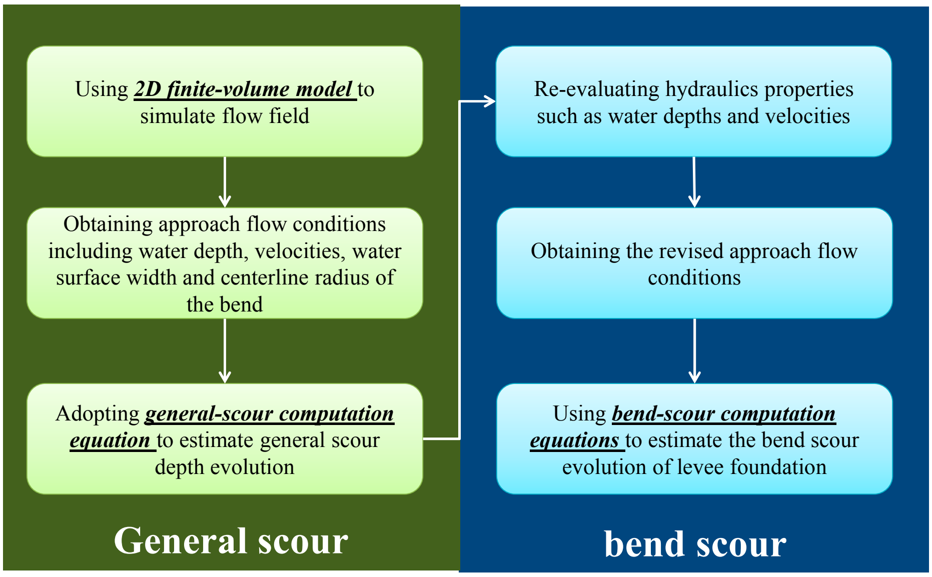

Figure 7. The algorithm of the proposed method for estimating the evolution of bend scour depth.

4. Flow Field and Embankment Toe Scour Simulations

4.1. Verifications of 2D Finite-Volume Model

This section verifies the accuracy of the finite-volume model for 2D flow field simulations. Figure 8 shows the simulation domain of the study channel bend, namely a cross-section and LiDAR (0.5 m × 0.5 m) bed elevation data. The study reach was approximately 12 km long between cross-section 117 (the upstream inflow boundary) and cross-section 90 (the downstream outflow boundary). The field-measured water level data at the Mingchu gauging station was used for Manning roughness coefficient calibration and model verification. Toe scouring at the Shuideliaw Embankment usually results from heavy rainfall during typhoon events, and two such flood events are analyzed herein: Typhoon Trami (August 2013) and Typhoon Usagi (September 2013). The Manning roughness coefficient was calibrated by comparing the simulated and measured water level data at the Mingchu gauging station.

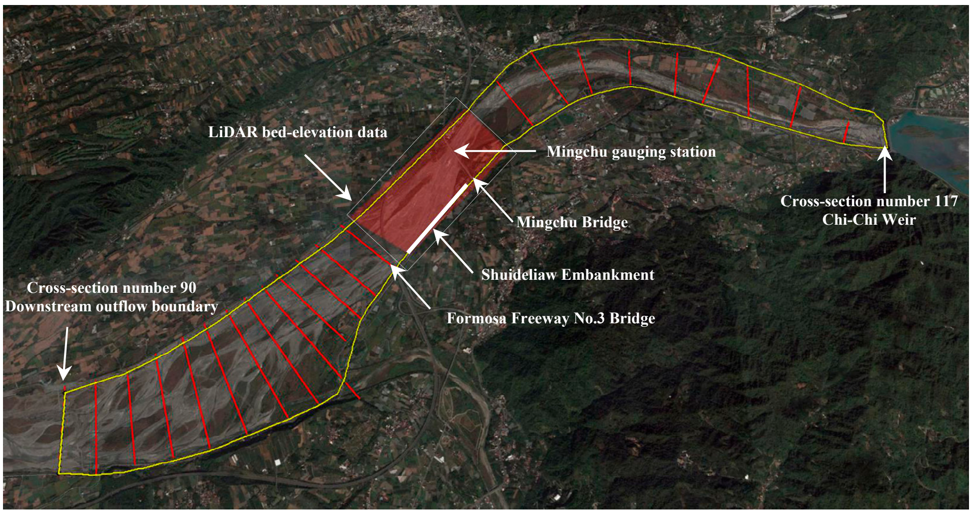

Figure 8. The simulation domain in a gravel-bed bend river reach of the Cho-Shui River

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The flow discharge hydrographs released from Chi-Chi Weir, as shown in Figure 4, were used for the subcritical inflow boundary conditions. The left and right sides along the simulation domain were considered the land boundaries and the time step was 0.5 s. The numerical accuracy of the 2D model was evaluated by comparing the simulated results with the measured water levels using the following two criteria:

$$E\eta_p(\%) = \frac{\left|\eta_p^{sim} - \eta_p^{mea}\right|}{\eta_p^{mea}} \times 100, \quad ET_p = T_p^{sim} - T_p^{mea} \tag{16}$$

in which $E\eta_p$ is the peak water level error; ET_p represents the error of time to peak water level; η_p^{sim} and η_p^{mea} denote the simulated and measured peak water levels, respectively; and T_p^{sim} and T_p^{mea} are the simulated and measured time to peak water level, respectively. The results for the two examined flood events, for which the $E\eta_p$ results were 0.13% and 0.46%, respectively, are provided in Table 2. Because the overall performance of $E\eta_p$ is less than 0.5%, the results demonstrate that the model can provide high numerical accuracy for the peak water level in modelling 2D flow with variable bed topography.

Figure 9 depicts the simulated and the measured water levels at the Mingchu gauging station, which are compared by calibrating the Manning roughness coefficients (0.025). The simulated results reveal close agreement with the measured data for the periods before the peak flow. However, for the periods after the peak flow, the model overestimates the water level. The error between the simulated and measured data may be due to the adopted assumption of a “fixed” bed. Nevertheless, previous studies have indicated that this does not affect the scour simulation results. Figure 10 displays the simulated results for velocity contours under peak flood conditions. Reasonable results are obtained using the 2D model, indicating that the velocities adjacent to the outer Shuideliaw Embankment increase significantly with peak discharge. Due to the river bend effects, high-velocity floods can attack and erode the riverbank, meaning that the outer embankment is the most vulnerable area. The results also show that the proposed 2D model is accurate for modeling flood flow with the irregular bed topography and complex geometry in a channel bend.

Table 2. Simulated results for two flood events using two criteria.

Events	**Two Criteria**	
	$E\eta_p$ (%)	**ET_p (h)**
Typhoon Trami (August, 2013)	0.1335	2
Typhoon Usagi (September, 2013)	0.4600	2

4.2. Verifications of Proposed Method for Bend Scour Depth

The results presented in Section 4.1 demonstrate that the proposed finite-volume hydraulic model determines accurate approach flow conditions. In this section, the evolution simulations of the bend scour depth obtained through the proposed approach are presented.

The approach flow conditions for the estimation of bend scour depth include the water depth, velocity, water surface width, and centerline radius of the bend. The proposed finite-volume hydraulic model was adopted to compute the required approach flow conditions for two typhoon events. The evolution of bend scour depth was then computed using the proposed method as well as the three existing equations identified earlier. Comparisons between the measured data and the computed results are displayed in Figure 11. Globally, all simulated scour-depth variations yield a similar trend compared with the flow hydrograph. To further discuss the differences among the four methods, the relative error is used and defined as:

$$Ed_{bs}(\%) = \frac{d_{bs}^{sim} - d_{bs}^{mea}}{d_{bs}^{mea}} \times 100 \tag{17}$$

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where d_{bs}^{sim} and d_{bs}^{mea} represent the simulated and measured maximum bend scour depth, respectively. Table 3 compares the simulated results with the measured data, including the maximum bend scour depth and the relative error. The results indicate that the existing empirical equations by Thorne and the U. S. Army Corps of Engineers tend to overestimate scour depth. However, the proposed method herein provides a good fit compared with the measured maximum bend scour depth, and is thus the most accurate of all the tested methods.

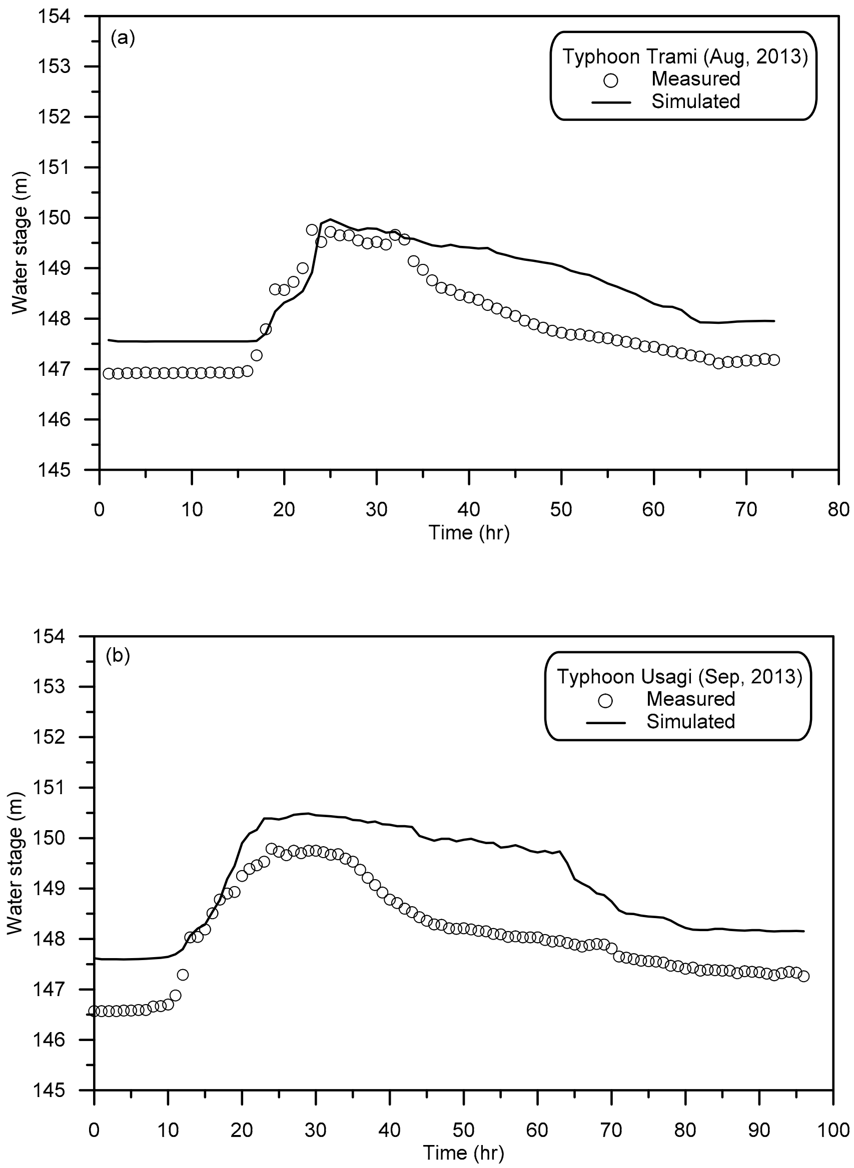

Figure 9. Comparisons of simulated and measured water level hydrographs for (a) Typhoon Trami and (b) typhoon Usagi.

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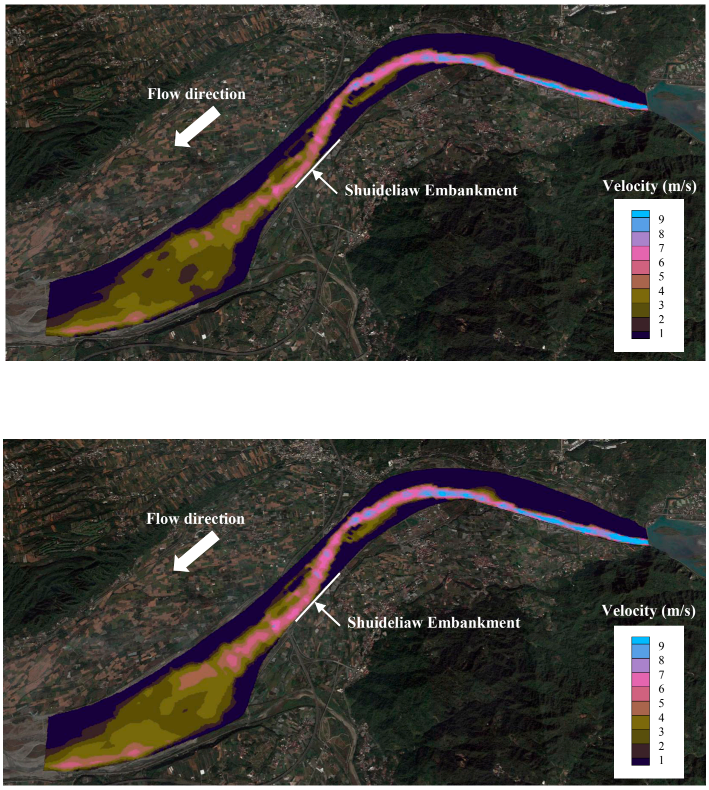

Figure 10. 2D contours showing simulated velocities under peak-flood condition for (a) Typhoon Trami (Q_p = 4186 m^3/s) and (b) Typhoon Usagi (Q_p =5309 m^3/s).

Table 3. The simulated maximum bend scour depths and the relative errors by four approaches for Typhoon Trami and Typhoon Usagi.

Methods	**Typhoon Trami** (Measured = 1.71 m)		**Typhoon Usagi** (Measured = 3.3 m)	
	Simulated	Ed_{bs} (%)	Simulated	Ed_{bs} (%)
Galay et al. (1987)	1.52	-10.98	2.22	-32.66
Thorne (1988)	3.04	77.62	4.85	46.95
U. S. Army Corps of Engineers (1994)	3.25	90.35	4.37	32.34
Proposed method	**1.81**	**5.65**	**2.67**	**-19.15**

4.3. Practical Embankment Safety Curve

A rational and practical tool to determine embankment safety in real time is of great importance for decisions made during typhoon periods. Based on the proposed method, this study attempts to provide a quantitative curve to evaluate the influence of discharge on bend scour depth at the embankment toe. The numerical simulations are performed with nine different discharges of 1.11-, 2-, 5-, 10-, 20-, 25-, 50-, 100-, and 200-year return period floods. According to results described in the

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previous section, the proposed bend-scour computation equation was verified to be suitable for simulating embankment toe scour in this case study. Thus, the bend scour depth at the peak flow condition obtained from the proposed equation is utilized to develop the embankment safety curve. Figure 12 shows the results of numerical experiments, indicating that the bend scour depth increases significantly with an increase in the discharge. Using statistical regression analysis results in the following regression equation: $d_{bs} = 0.0005Q$. On the basis of this equation, the bend scour depth at the embankment toe downstream of the Chi-Chi Weir in Cho-Shui River can be easily estimated in real time for a given flow discharge.

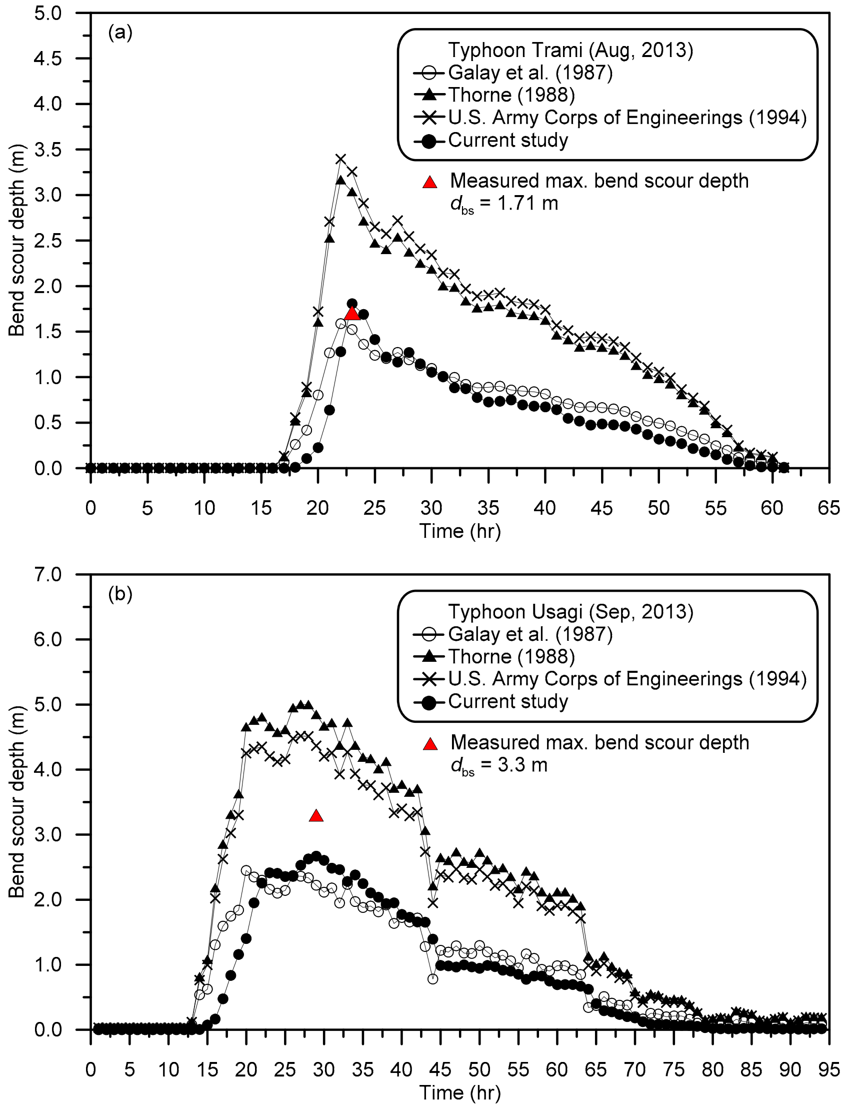

Figure 11. The simulated time variations of bend scour depths using four approaches for (a) Typhoon Trami and (b) Typhoon Usagi.

After the embankment safety curve is established, its accuracy must be verified. From a process standpoint, the safety of an embankment is controlled by the relationship between the resisting and driving forces of the embankment. For reinforced concrete embankments, the driving forces can represent the hydraulic scouring around the embankment toe, and the resisting force can stand for the depth of the embankment foundation. During the flood induced by a monsoon in June 2012, the

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bend-scour depth near the Shuideliaw Embankment was larger than the depth of the embankment foundation. Hence, the embankment failed due to the embankment toe erosion caused by flooding with a peak discharge of 6,122 m³/s. By using the proposed equation $d_{bs} = 0.0005Q$, when discharge is equal to 6,122 m³/s, the total bend scour depth is estimated at 3.06 m, which is larger than the depth of embankment foundation (3.0 m). Therefore, the results demonstrate that the proposed simplified equation could be used to correctly assess embankment toe safety. To further provide advantageous assessment information, the discharge warning value can be achieved from this equation fairly simply. Moreover, the depth of the embankment foundation can be substituted into this equation, resulting in 6,000 m³/s waring discharge. The results herein suggest that the study embankment will be unstable and may further fail if discharge released from the Chi-Chi Weir is greater than 6,000 m³/s. By contrast, the numerical experiments demonstrated that the maximum bend scour depth near the Shuideliaw Embankment is greater than the designed foundation depth, because the flow discharge is greater than 6,000 m³/s (which is between the flood discharges for the 2- and 5-year return periods, similar to the bankfull discharge). This similarity between the flow and bankfull discharges denotes a significant change in the river bed, and indicate why usually embankment failure occurs during bankfull discharge.

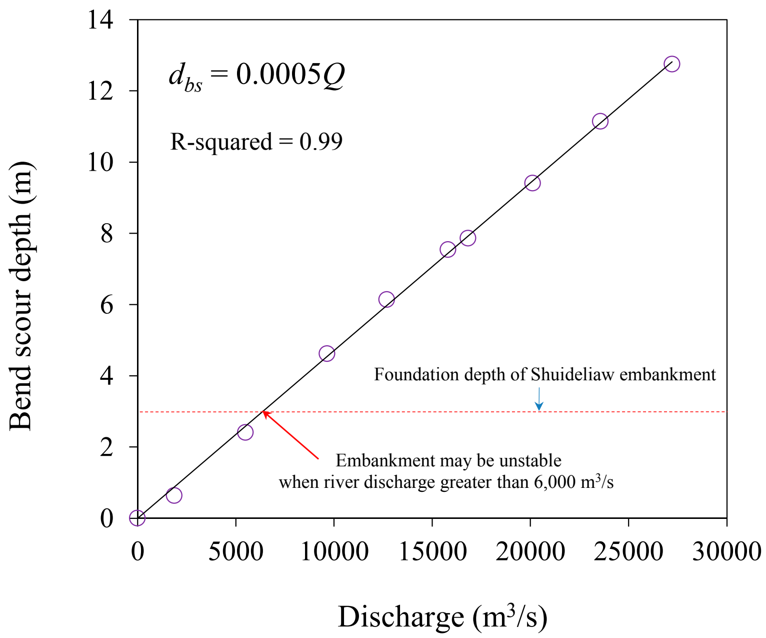

Figure 12. The relationship between the bend scoured depth and the flow discharge at Chi-Chi Weir.

In summary, the proposed embankment safety curve provides a quick and advantageous assessment for river managers to use when making management decisions associated with embankment stability or waring.

5. Conclusions

Scour at the embankment toe is a complex fluvial process, especially in bend reaches. A new composite method for simulating general and bend scour depths was therefore proposed in this paper. This composite method consists of three components: a 2D finite-volume model, a general-scour computation equation, and a new bend scour computation equation. An assumption linking the flow, general, and bend scour evolution was presented, and it allows the proposed method to simulate both general and bend scour evolutions in a river bend with complex bed topography under typhoon-induced flood conditions. The 2D flow fields and the temporal evolutions of bend scour depths in a bend reach of the Cho-Shui River were simulated in detail for two typhoon flood events, and the simulated results were compared to field data measured using the numbered-brick method.

The primary conclusions of this study are as follows: (1) the new practical method is capable of simulating the temporal evolution process caused by the development of bend scour at embankment toes; (2) the simulated results were generally in agreement with the field-measured data, including water level data at the Mingchu gauging station and maximum bend scour depths at the Shuideliaw Embankment; (3) the proposed embankment safety curve enables river managers to make rational decisions when responding to embankment emergencies during the typhoon season.

The new method has been demonstrated to accurately simulate bend scour evolution based on the separating assumption of scour components. However, further testing must be conducted in comparison with the field-measured evolution of embankment toes, specifically with consideration for the sediment deposition process. In addition, our method can be integrated into a flood warning system that can be used for automated real-time forecasts of water levels and embankment toe scouring. The system provides information for effective and timely decision-making on hazard mitigation actions prior to expected critical situations.

Acknowledgments: The authors would like to thank the Fourth River Management Office, Water Resources Agency, and Ministry of Economic Affairs for providing the flow discharge hydrographs at Chi-Chi Weir on the Cho-Shui River.

Author Contributions: Wen-Dar Guo and Jian-Hao Hong performed the scour modeling and wrote the paper. Cheng-Hsin Chen and Chih-Chiang Su collected the relevant data and reviewed the manuscript critically. Jihn-Sung Lai provided suggestions for improvements to the manuscript.

Conflicts of Interest: The authors declare no conflict of interest.

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