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Seung Oh Lee

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Jun Seon Lee

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Estimates of Discharge Coefficient in Levee Breach Under Two Different Approach Flow Types

Seung Oh Lee ¹, Kwang Seok Yoon ², Jun Seon Lee ³ and Seung Ho Hong ^{3,*}

¹ School of Urban and Civil Engineering, Hongik University, 94 Wausan-ro, Mapo-gu, Seoul 04066, Korea; seungoh.lee@hongik.ac.kr

² Korea Institute of Civil Engineering and Building Technology, Goyang 10223, Korea; ksyoon@kict.re.kr

³ Department of Civil and Environmental Engineering, West Virginia University, 1306 Evansdale Drive, Morgantown, WV 26506, USA; jl0124@mix.wvu.edu

* Correspondence: sehong@mail.wvu.edu; Tel.: +01-304-293-9926

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Abstract: The amount of released water (discharge) in a levee breach is a primary input variable to establish an emergency action plan for the area next to the levee. However, although several studies have been conducted, there is still no widely applicable discharge coefficient formula; this needs to be known to estimate discharge amount through an opening caused by a levee breach. Sometimes, the discharge coefficient developed for a sharp crested side weir is used to rate the discharge, but, in case of a levee breach, the resulting geometry and flow types are similar to that over a broad crested weir. Thus, in this study, two different openings—rectangular and trapezoidal shape—are constructed in the center of a levee at a height of 0.6m to replicate levee breach scenarios, and the effect of two different approach flow types—the river type approach and reservoir type approach—are explored to suggest a discharge coefficient formula applicable for discharge rating for a levee breach. The results show that the ratio of head above the bottom of an opening and the opening width is a key variable for calculating the discharge coefficient of a reservoir type, but the approach Froude number should also be considered for a river type approach. The measured data are used to improve rating equations and will be useful in the future to validate computational fluid dynamics simulations of wave propagation during levee failure into the inundation area.

Keywords: levee break; flood management; discharge coefficient; large scale hydraulic experiment

1. Introduction

Since the late 1970's, levees have been constructed adjacent to rivers to protect people and industries from flooding and to provide residential areas or cultivated land behind the levees. While levees accompanied by ongoing levee reinforcement are a viable method to reduce the risk of flooding, any flooding that does occur will have an even greater impact on the low-lying land behind the levees. For example, if a breach in the levees occurs during flooding season, the damage will be catastrophic, as occurred for the levee failures in New Orleans in the U.S. (2005). Fourteen billion dollars have been spent to shore up New Orleans' levee system and a total of \$135 billion dollars have been spent to restore and repair the damages. More recently, in 2017, Tropical Storm Harvey overwhelmed the entire levee system near Houston, TX and resulted in at least 18 deaths and a total of \$125 billion in damages in the southeast of Texas.

The term 'failure' is defined as inability to achieve a defined performance threshold. Thus, in the case of levees, failure includes (1) deterioration-process over time, such as an overtopping over the levee during large flooding and internal/external erosion by hydraulic loading, and (2) a resulting breach in the end. The main cause of levee deterioration process leading to a breach is geotechnical stability.

Geotechnical stability of a levee is outside the scope of this paper, but the example failure shown in the previous paragraph is a sobering reminder that the levee structure can fail with catastrophic consequences. For the case of New Orleans, a number of contributing factors have been identified by several external reviews in the aftermath of the failures. These include inadequate geotechnical design of the concrete I-wall foundations intended to strengthen the levees, elevations of the tops of the levees that were below storm design levels with resultant overtopping and erosion of the outer toe of the levees, lack of soil protection in the event of overtopping, and failure to account for seepage under the levees. These various mechanisms are characterized and may be combined in a failure scenario, but the ultimate breaching of the levees occurred really quickly [1]. Even though reasons for failure are still a matter of some controversy with many other contributing factors identified, perhaps one of the main lessons of Katrina is that when levees are the last line of defense against the floods, catastrophe is bound to occur at some time. Thus, for reducing the potential risk from a levee failure, an accurate estimation of flood inundation zone based on the correct amount of discharge by a levee breach is required to prepare an evacuation plan and to establish a Flood Alarming System.

For dam break analysis, initial discharge amount through the opening can be estimated with the conventional format of weir equation with corresponding discharge coefficient because the volume of the water constrained by a dam can be considered as static. However, in case of a levee breach, flow characteristics from the opening are more complex than in a regular dam break because flow velocity and flow depth as well as geometric variables in the approach section play an important role in deciding the discharge amount over the breached section and, therefore, approach variables should be considered when a discharge coefficient is formulated. Based on several recent studies, similar flow characteristics in a levee breach can be found to that over a side weir (also called to lateral weir). A side weir is usually installed along the side of a channel for irrigation and flood protection when water depth within the channel is too high. Researchers including Subramanya and Awasthy [2], Ranga Raju et al. [3], Hager [4], Cheong [5], Singh et al. [6], Borghei et al. [7], and Ji et al. [8] have extensively studied the subject of a side weir. They applied energy and momentum conservation to flow over a sharp-crested rectangular side weir and derived a theoretical discharge rating equation. Then, corresponding discharge coefficient was given by laboratory experiment. Several proposed formulas for discharge coefficient (C_D) are as follows,

$$C_D = 0.864 \left(\frac{1 - Fr_1^2}{2 + Fr_1^2} \right)^{0.5} \quad (\text{Subramanya and Awasthy 1972}) \quad (1)$$

$$C_D = 0.81 - 0.6Fr_1 \quad (\text{Ranga Raju et al. 1979}) \quad (2)$$

$$C_D = 0.485 \left(\frac{2 + Fr_1^2}{2 + 3Fr_1^2} \right)^{0.5} \quad (\text{Hager 1987}) \quad (3)$$

$$C_D = 0.45 - 0.22Fr_1^2 \quad (\text{Cheong 1991}) \quad (4)$$

$$C_D = 0.33 - 0.18Fr_1 + 0.49 \frac{P}{y_1} \quad (\text{Singh et al. 1994}) \quad (5)$$

$$C_D = 0.7 - 0.48Fr_1 - 0.49 \frac{P}{y_1} + 0.06 \frac{L}{B_1} \quad (\text{Borghei et al. 1999}) \quad (6)$$

where, Fr_1 is the Froude number based on the approach channel velocity (V_1) and approach channel water depth (y_1), P is the side weir height measured from the bottom of the channel to the crest of the weir, L is the bottom width of the weir, and B_1 is the approach channel width. The detailed schematics for the variables are shown in Figures 1 and 2.

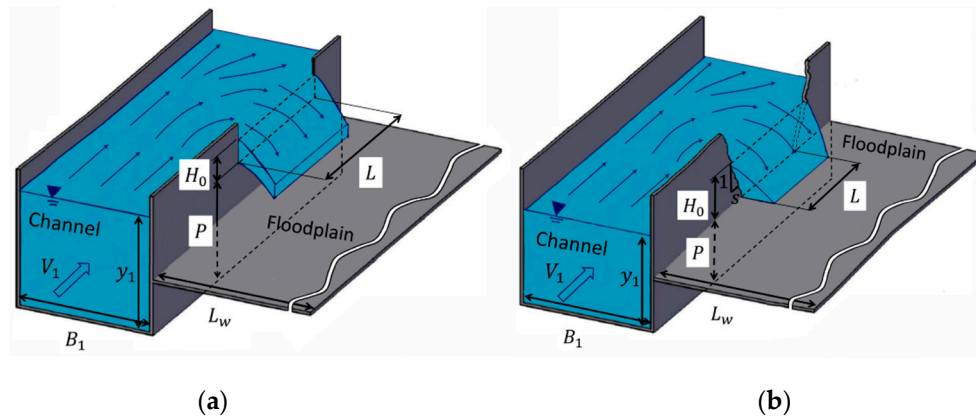


Figure 1. Comparison of flow over side weir in (a) and over levee breach in (b).

The discharge per unit width (q) of a sharp crested rectangular side weir [9,10] is as follows and total discharge can be estimated by integration of Equation (7) with respect to the width of weir.

$$q = \frac{2}{3} C_D \sqrt{2g} H_0^{3/2} \quad (7)$$

In which H_0 is the upstream head relative to the crest of weir. As shown in the equations above, most of investigators selected approach Froude number (Fr_1) as an important parameter to account for hydraulic characteristics in the approach channel on the side weir discharge coefficient. In addition to the approach Froude number, Singh et al. [6] included the effect of the relative weir height (P/y_1), and Borghei et al. [7] included effect of the ratio between bottom width of a weir and a channel width (L/B_1) as well on their discharge coefficient. Recently, Cosar and Agaccioglu [11] found that discharge coefficient of a triangular side-weir located on a curved channel is a function of approach Froude number, and the discharge coefficients are greater than a coefficient in a straight channel.

Many side weirs are constructed within levee systems to maintain water level in a channel for irrigation and flood protection. As shown in Figure 1a, the shape of a side weir is mostly a rectangular sharp crested weir for easier operation and construction. When water overtops over a side weir, shallower and faster moving flow over the side weir adjacent to the slower moving flow in the channel results in a complex interaction that includes strong transverse transfer of longitudinal momentum from the levee to the channel flow. This phenomenon is more pronounced in the immediate interface region between the channel and the corner of the levee, where there exists a strong transverse gradient of the longitudinal velocity. A similar flow field can be found in the very beginning of levee failure during the deterioration process as explained in the previous paragraph. However, as explained by Heerden and Li [1], time required to reach total breach of a levee is short. Thus, after a short period of time when a levee starts to fail, the resulting geometry and flow types are similar to that over a broad crested weir as shown in Figure 1b because the floodplain (inundation) area is long enough that parallel flow and critical depth occur at some point in the floodplain as in a broad crested weir [12]. Furthermore, the shape of a levee failure zone is site-specific and closer to a trapezoidal shape, which has a wide top and narrow bottom, not a rectangular shape. Thus, the application of empirical discharge coefficient shown in Equations (1) to (6) to rate discharge over an opening caused by a levee breach is questionable.

In this study, to suggest new or modified discharge coefficient formula applicable to levee breach (similar to the broad crested weir) with respect to different approach flow conditions (reservoir type and river type), laboratory experiments are conducted with different opening shapes including rectangular and trapezoidal shape under various flow conditions. To overcome possible flaws stemming from scale effect, all of the tests are conducted in a large test basin, instead of using a small scale flume for the experiment. The experimental results show that the dimensional parameter (head above bottom of the opening and the opening width) has a dominant effect on the formula determining discharge

coefficient of a levee breach in a reservoir type approach, but approach Froude number should also be considered for a river type approach as well. Comparisons between measured and predicted discharge values using suggested discharge coefficient formula show good agreements.

2. Methodology

2.1. Theoretical Background

The main difficulty in obtaining an accurate method to predict discharge amount during levee failure is the large number of variables affecting the discharge coefficient. The following variables are introduced.

$$C_D = f_1(V_1, y_1, H_0, B_1, S_1, P, L, L_w, s, g, \rho, \gamma, t) \quad (8)$$

where parameters related to flow properties: V_1 is the approach velocity, y_1 is the approach water depth, and H_0 is the water head above the crest of the opening; parameters related to geometry: B_1 is width of the approach channel, S_1 is the approach channel bottom slope, P is the height measured from channel bottom to the crest of the opening, L is the opening width measured at the bottom, L_w is the length of the floodplain (inundation area) parallel to flow, and s is the side slope of the opening; parameters related to fluid properties: g is the gravitational acceleration, ρ is the density of fluid, and γ is the kinematic viscosity of fluid; t is time. In order to elucidate and identify the effect of each variable, dimensional analysis should be carried out using Buckingham pi-theorem before an experimental study. Also, experimental data can be unified through a normalizing process (i.e., scaling) and presented in terms of dimensionless parameters. With reference to the definition sketch for each variable in Figure 2, the results of dimensional analysis of the problem of discharge coefficient over the opening caused by the levee failure can be given as

$$C_D = f_2\left(Fr_1, Re_1, \frac{H_0}{L_w}, \frac{P}{H_0}, \frac{H_0}{L}, \frac{L}{B_1}, \frac{P}{y_1}, S_1, s, \frac{V_1 t}{y_1}\right) \quad (9)$$

Reynolds number based on the head (H_0) is of the order of 10^5 , which ensures a fully turbulent flow regime. Therefore, the effect of Reynolds number (Re_1) can be negligible [13]. The relative ratio of head and floodplain length can also be negligible (H_0/L_w) because the value of the inundation area length (L_w) is large compared to the small increment of the head (H_0). Based on the studies for side weirs [2,14], the effect of the relative weir height (P/H_0) on a discharge coefficient is not important. Furthermore, the effect of channel slope (S_1) and the time term ($V_1 t/y_1$) in Equation (9) can be ignored in this study because channel slope is constant and the discharge coefficient in case of total levee breach is only considered in the experiment. Thus, the result of dimensional analysis can be given as

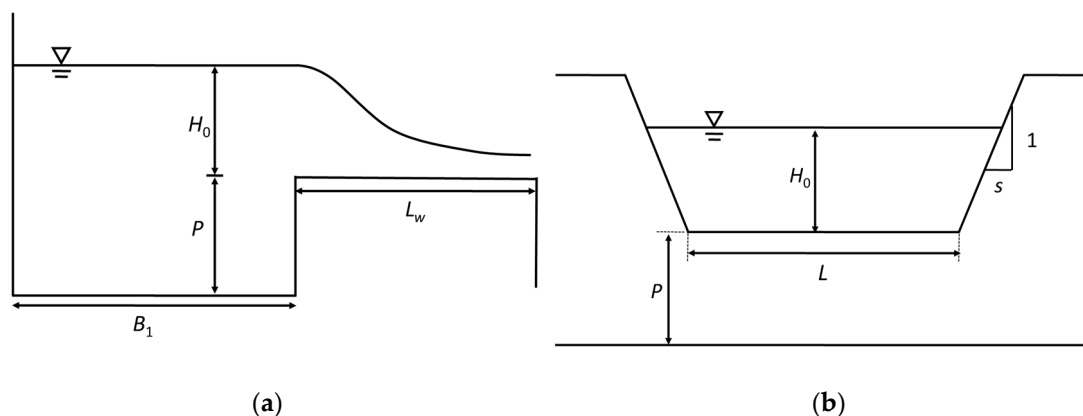


Figure 2. Variables influencing discharge coefficient in levee breach.

$$C_D = f_3\left(Fr_1, \frac{H_0}{L}, \frac{L}{B_1}, \frac{P}{y_1}, s\right) \quad (10)$$

Each dimensional variable in Equation (10) is testified with extensive laboratory measurements together with head–discharge relationship as shown in Equation (7).

2.2. Experimental Setup

As shown in Figure 3, all of the experiments are conducted in a 30 m long by 30 m wide large outdoor basin at Korea Institute of Civil Engineering and Building Technology (KICT), Goyang, Korea [15]. Within the basin, lowlands 30 m long by 25 m wide are constructed for the inundation area. Along each side of the inundation area except one end, under-ground laboratory sumps are located to drain the water after passing the inundation area from the opening. Along one end of the basin, instead of constructing an under-ground sump, a rectangular-shaped straight channel is constructed to align with the lowland. The channel is 30 m long and 5 m wide and the width of channel is wide enough to avoid any influences of flow characteristics around the levee failure zone on the approach uniform flow section. The slope of the channel bed is horizontal, and the channel bed elevation is 0.4 m lower than the invert of the inundation area. The maximum flowrate is 1.0 m³/s supplied by pumps and the actual flowrate is measured by magnetic flow meters installed in supply pipe systems during the experiment. Water supply to the channel is recirculated such that water flows into the laboratory under-ground sump at the end of the channel from which the water is continuously pumped into the entrance of channel.



Figure 3. A large test basin in Korea Institute of Civil Engineering and Building Technology (KICT).

The inundation area and the channel are separated by a 0.6 m high and 30 m long vertical levee. The height of levee in the experiment is decided based on the field measurements in South Korea which show an average levee height is about 10% of the channel width [16]. In the middle section of the levee system, sliding opening gates moving into the opposite direction are installed to simulate the levee breach. Based on the research by Lee and Han [16], opening widths during levee breach in South Korea vary around a width of one to three times the height of the levee or 1/8 to one times the channel width. Several other studies [17,18] for dam break show that an average width of failure zone varies from two to five times larger than height of the dam. Thus, in this study, the maximum

bottom opening width is chosen to be 3 m and the width can be adjusted to the desired value by using an attached variable motor. Shapes of levee breach section vary depending on the approaching flow characteristics and geotechnical properties of the levee. Thus, to account for a different shape of breach section, two different shapes of opening, a rectangular shape and a trapezoid shape, are replicated in the middle of levee. As shown in Figure 4, four different slopes of gates (1V:0H, 1V:0.1H, 1V:0.3H and 1V:0.5H) are used in the experiments and these arrangements allow simulation of the effect of the shape on the discharge coefficient. For the measurements of velocity and water depth through the entire channel section, an Acoustic Doppler Velocimetry (ADV) and capacitive wave height meters are used, respectively.

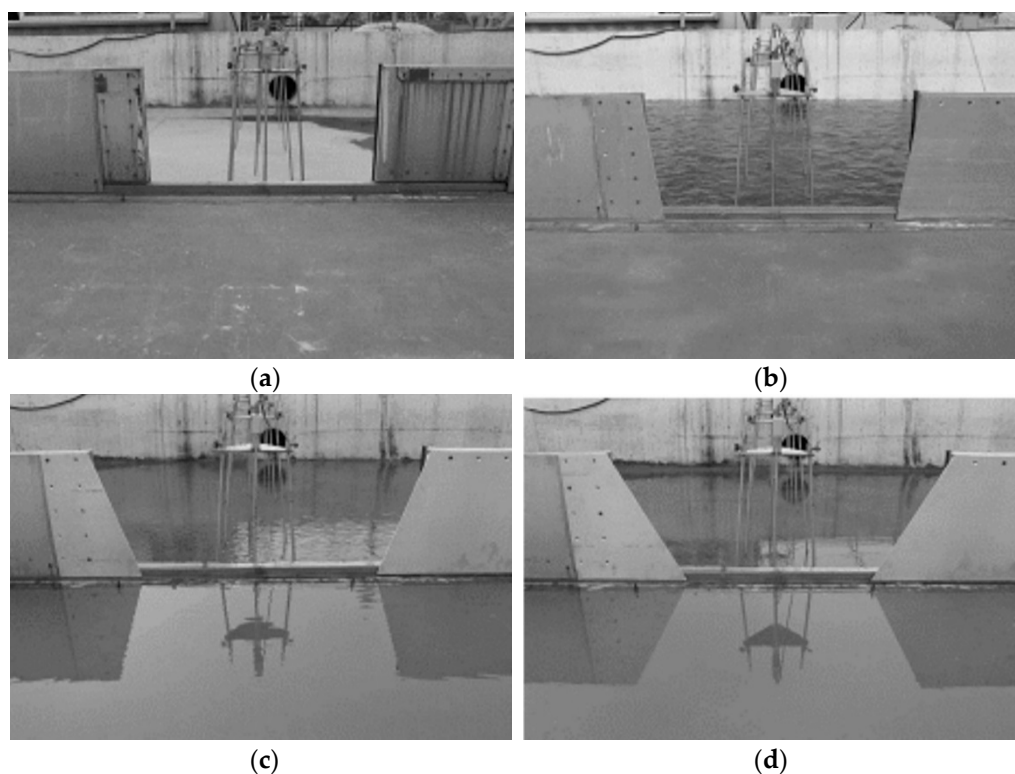


Figure 4. Shape and slope of gate for levee failure; (a) rectangular shape (1V:0H) and trapezoidal shape (b) (1V:0.1H), (c) (1V:0.3H) and (d) (1V:0.5H).

3. Results and Discussion

Opening bottom width (L), head above the opening (H_o), approach velocity (V_1), approach water depth (y_1), and total discharge into the channel (Q_T) are measured for two different types of approach conditions in the channel that are expected to cause a levee failure: (a) reservoir type approach and (b) river type approach. The chosen values of H_o satisfied the recommendation with a minimum value of head of 0.07 m recommended for spillway models [19,20] to eliminate surface tension as well as viscous effects. Flow depths in the model are generally greater than 0.07 m, which is another criterion for avoiding surface tension effects manifested by capillary waves in free-surface flow models [13]. The reservoir type approach stands for where velocity in a channel is lower, such as a channel in which water resources are controlled by series of spillways. A good example can be found along Kissimmee River in Florida [21]. The South Florida Water Management District operates a series of five ogee spillways through Kissimmee River to maintain a certain water depth for irrigation and transportation. Thus, the average velocity is lower than in a natural river except for emergency conditions when the gates of all five structures should be open at the same time. To establish reservoir type in the laboratory channel similar as in the Kissimmee River, the tail gate is raised up to its maximum and then the flowrate is gradually increased until the target head over the opening is achieved. During

the procedure, increment of flow rate should be small enough not to make any overflow through the downstream tailgate. When the H_o has been stabilized with the target value, approach velocity and approach water depth are measured 7.5 m upstream from the opening. In the reservoir type approach experiments, discharge amount over the levee breach opening (Q) is the same as the total inflow discharge (Q_T) through the channel inlet. Contrary to the reservoir type approach, the river type approach stands for where a decent value of velocity can be found in the approach flow. Thus, to set up the river type approach in the channel, flow rate is gradually increased up to 0.8 m³/s with the tailgate raised until the head over the opening is higher than the target value. Then, the tailgate is adjusted to produce a required target value of H_o over the levee opening. After setting up each experimental condition, all of the required data are measured in a similar way as in the reservoir type. In addition to the approach water depths and velocities, velocities and water depths are measured 10 m downstream from the opening by using an ADV and capacitive wave height meters to predict the discharge amount at the downstream section in the river type approach experiments. Then, the actual amount of the discharge over the opening can be calculated by using continuity between inflow from the inlet and outflow at the exit section. The range of experimental parameters has been summarized in Table 1.

Table 1. Range of experimental parameters.

Approach Type	Bottom Width, L (m)	Side Slope, s	Weir Height, P (m)	Channel width, B_1 (m)	Inflow Discharge, Q_T (m ³ /s)	Head, H_o (m)	Fr_1
Reservoir	0.5, 1.0, 1.5, 2.0	1V:0.0H	0.4	5	0.05~0.8	0.15, 0.2, 0.25, 0.3, 0.35, 0.4, 0.45	0.005~0.06
	0.5, 1.0, 1.5, 2.0	1V:0.1H				0.15, 0.2, 0.25, 0.3, 0.35, 0.4, 0.45	
	0.5, 1.0, 1.5, 2.0	1V:0.3H				0.15, 0.2, 0.25, 0.3, 0.35, 0.4, 0.45	
	0.5, 1.0, 1.5, 2.0	1V:0.5H				0.15, 0.2, 0.25, 0.3, 0.35, 0.4, 0.45	
River	0.5, 1.0	1V:0.0H			0.8	0.15, 0.2, 0.25, 0.3, 0.35, 0.4, 0.45	0.06~0.12

In case of a rectangular shape of an opening, flow rate per unit width is essentially constant across the opening. Thus, the discharge coefficient can be directly decided by using Equation (7). However, for a case of trapezoidal shape, flow distribution across the cross section is non-uniform and dependent on geometries, particularly on the side slope. Thus, to account for the effect of shape on discharge coefficient, side slope, s , in Equation (10) is absorbed in the flow area calculation, and the corresponding discharge coefficient for the trapezoidal shape of a breach opening is given below [22,23].

$$C_D = \frac{Q}{(LH_o + sH_o^2) \sqrt{2gH_o}} \quad (11)$$

A theoretical discharge coefficient over a typical broad crested weir is 1.0, but friction losses reduce the value of the discharge coefficient, C_D , to 0.848. In addition to the friction losses, in the case of levee breach, flow over the opening is curvilinear to the entire crest of the opening, as shown in Figure 1b, leading to additional energy losses, therefore, the value of discharge coefficient in case of the levee breach is even lower than 0.848. Degree of energy losses associated with the flow curvature around the opening is related to the approach flow characteristics. Thus, in this study, the behavior of discharge coefficient over the broad crested side weir as in a levee breach are explored with respect to the different approach flow conditions and the results are summarized below.

The significant non-dimensional parameters affecting value of discharge coefficient are obtained by dimensional analysis as shown in Equation (10). Figures 5 and 6 shows the relationship between discharge coefficients and dimensionless parameters (P/y_1 and L/B_1) for the reservoir type approach and the river type approach, respectively. As shown in Figures 5 and 6, values of P/y_1 and L/B_1 for

discharge coefficient (C_D) under different approach flow types do not show any functional relationships based on the value of coefficient of determination (R^2). Even if any comprehensive relationship cannot be determined from Figures 5 and 6, results show that the discharge coefficient slightly decreases as the value of P/y_1 and L/B_1 increased. Since the value of relative weir height (P/y_1) and relative opening width (L/B_1) determines ratio of the surface flow to the bed flow that is deflected through the weir, it influences the discharge coefficient. In this experiment, height of weir (P) and width of channel (B_1) is constant. Therefore, the value of C_D for both approach flow types decreases as the value of opening width (L) increases, but increases when the water depth (y_1) increases. However, to develop any functional relationships between discharge coefficients (C_D) and the values of P/y_1 and L/B_1 , additional experiments should be conducted with various geometries; this is a point that deserves further studies in detailed hydrodynamic simulations.

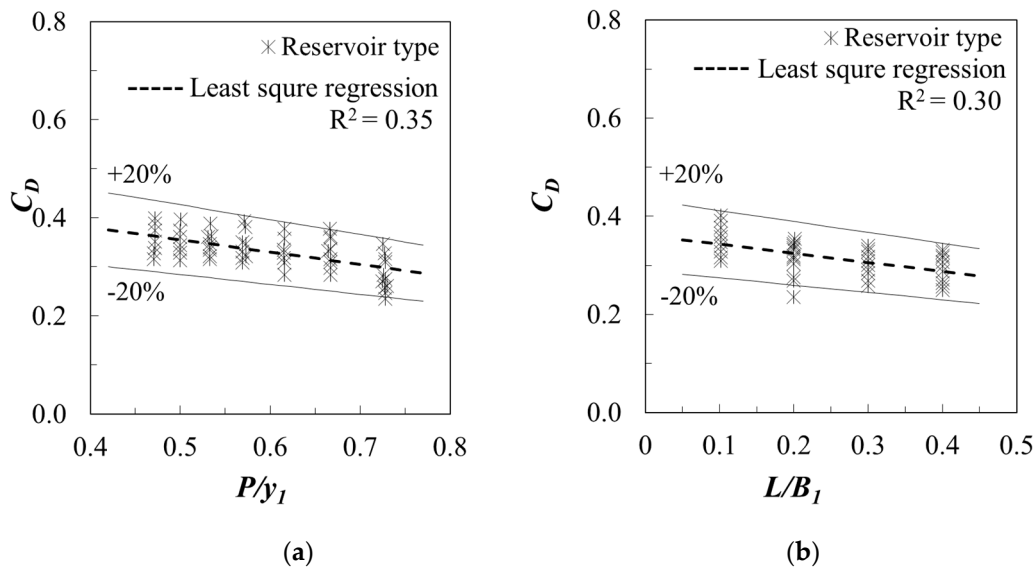


Figure 5. Discharge coefficient for reservoir type approach with non-dimensional variable P/y_1 in (a) and L/B_1 in (b).

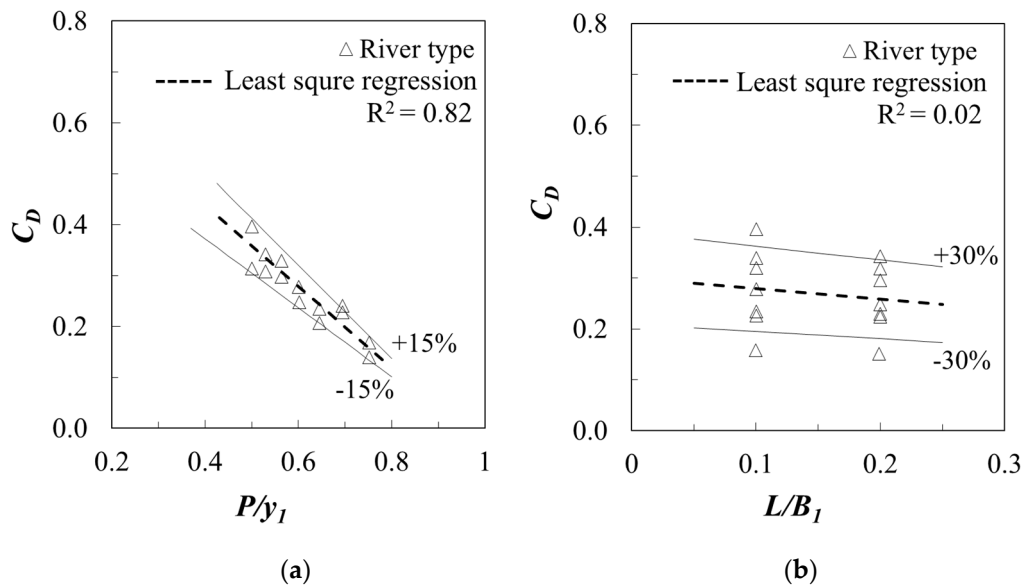


Figure 6. Discharge coefficient for river type approach with non-dimensional variable P/y_1 in (a) and L/B_1 in (b).

As shown in Equation (10), another influential parameter is H_0/L . Thus, the measured discharge coefficients for both approach types are plotted in Figure 7 according to the values of H_0/L . Based on the classical researches including Kindsvater and Carter [24] and Bos [19], there is a negligible influence of H_0/L on discharge coefficient. However, as shown in the Figure 7, as the dimensionless variable, H_0/L , in the x -axis increases, the value of discharge coefficient gradually increases. Thus, power function is used in the regression analysis and the results clearly reveal that the discharge coefficient has strong relationship with the value of H_0/L for the reservoir type approach. For the river type approach, the value of C_D increases as H_0/L increases, but they are a slightly lower range than for the reservoir approach flow type. However, it seems that higher value of scatter can be found based on the value of coefficient of determination with respect to the regression line. Hence, it can be assumed that this preliminary results for the river type approach is over-simplified and additional variables should be considered for the analysis. This point will be discussed in next paragraph. The best-fit equation on the data given in Figure 7 is given by

$$C_D = 0.397\left(\frac{H_0}{L}\right)^{0.141} \quad \text{for reservoir type} \quad (12)$$

$$C_D = 0.338\left(\frac{H_0}{L}\right)^{0.303} \quad \text{for river type} \quad (13)$$

with coefficients of determination of 0.92 and 0.65 for the reservoir type approach and the river type approach, respectively.

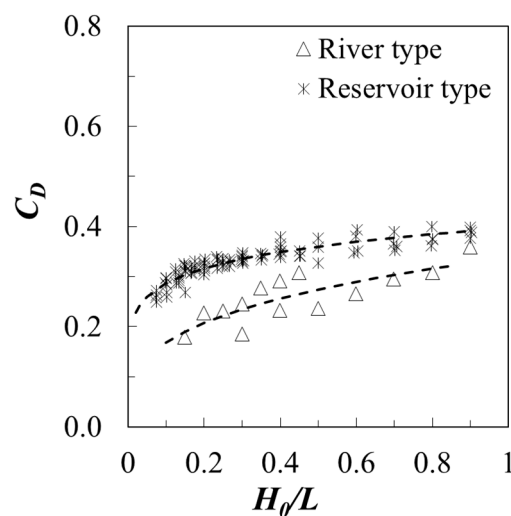


Figure 7. Discharge coefficient with non-dimensional variable H_0/L .

The other important parameter for the problem of discharge coefficient is approach Froude number. Usually, reservoir can be characterized as deep water depth and very low flow velocity. Thus, the corresponding value of Fr_1 for a reservoir type approach is “close to zero” and the effect of approach Froude number can be negligible. Therefore, as shown in Equation (12), discharge coefficient can be correctly estimated without considering the effect of Fr_1 . However, for the river type approach, approach Froude number is important variable for the problem of rating discharge in levee breach. Thus, the effect of approach Froude number (Fr_1) on the discharge coefficient is plotted in Figure 8. The flow curvature around the sharp edged side wall resulted in higher pressure at the control section leading to lower values of the discharge coefficient. Furthermore, due to the strong adverse pressure gradient imposed by the opening along the curved flow, the boundary layer separates at the edge of the side wall and the separation resulted in the formation of large turbulent structure. The higher turbulence also lowered the value of the discharge coefficient. Those symptoms become higher when the value of Fr_1 increase. As shown in Figure 8, as the value of Fr_1 increases, the coefficient of discharge

gradually decreases for the river type approach. However, there is no direct relationship between discharge coefficients for the reservoir type and the values of Fr_1 .

As shown in Figures 7 and 8, the discharge coefficient for river type approach shows a relationship with both of the values of H_0/L and Fr_1 . Thus, based on the multi variable regression analysis, the best-fit regression equation on the data given in Figures 7 and 8 for the river type approach is

$$C_D = 0.338(Fr_1)^{-0.832}\left(\frac{H_0}{L}\right)^{0.065} \quad \text{for river type} \quad (14)$$

The value of coefficient of determination ($R^2 = 0.95$) for Equation (14) shows higher than the coefficient of determination ($R^2 = 0.65$) on Equation (13). Furthermore, Equation (14) shows that the function of H_0/L has a relatively small dependence in the regression equation based on the value of the exponent.

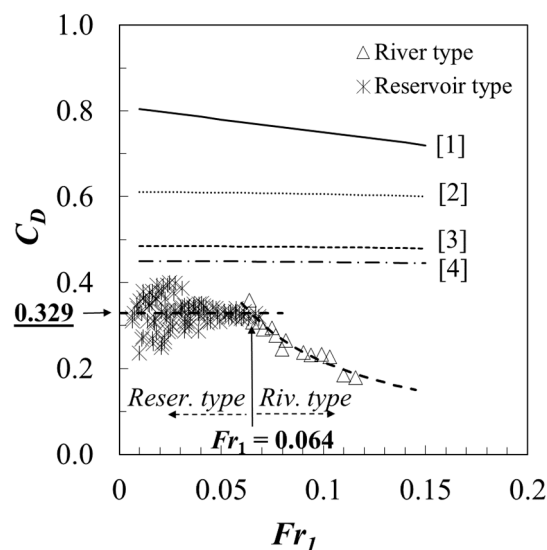


Figure 8. Discharge coefficient with approach Froude number (Fr_1).

Discharge coefficient can be calculated by using Equations (12) and (14) applicable to a reservoir type approach and a river type approach, respectively. However, approach flow type for given conditions must be identified first to ensure that appropriate discharge coefficient formulas is used. Thus, to suggest the transition criteria, data in Figure 8 are examined further. As explained in the previous paragraph, the value of C_D is independent of Fr_1 for a reservoir type approach, and thus, varies from 0.23 to 0.4 in the lower bound of the x -axis in Figure 8. However, values of C_D are aggregated as Fr_1 increases and approached to arithmetic mean of discharge coefficient (=0.329); hence, it can be assumed that the value of C_D at the transition from the reservoir type approach to the river type occurs at the value of $C_D =$ their arithmetic mean (0.329). Then, the regression line for the discharge coefficient of the river type approach is plotted in the figure. Figure 8 shows when the $Fr_1 < 0.064$, the approach section can be considered as a reservoir type and the corresponding discharge coefficient formula is Equation (12); when the Froude number is larger than 0.064, the approach flow type is a river type, then Equation (14) should be used.

The calculated values of discharge coefficient in this study with respect to the Froude numbers are compared with the calculated discharge coefficient using formulas from four other investigators [2–5]. As shown in Figure 8, values in this study are lower than the data set published in the previous researches. One of the possible reasons is the broad-crested shape of the weir used in this study resulting in higher amount of friction over the crest compared to a sharp-crested rectangular weir in their study. Another possible reason can be found from scale effect. Lopardo [25] made an interesting model-prototype comparison that showed the limits of Froude scale models. The extreme values in a scale model were overestimated by a 1/50, due to the absence of aeration. When flow passes through

the opening, the local flow motion around the vertical wall include a separated shear layer adjacent to the wall, formation of bubbles within the shear layer, and a wake region immediately downstream of the wall. The unique flow characteristics cannot be reproduced in small scale model as explained by Lopardo. Finally, Figure 9 shows correlation between the measured and the calculated discharge over the opening. The value of stand error was 0.0012 and 0.0031 and the value of $R^2 = 0.98$ and 0.96 for the reservoir type approach and the river type approach, respectively, and the comparison shows good agreement between the measured and the predicted discharge.

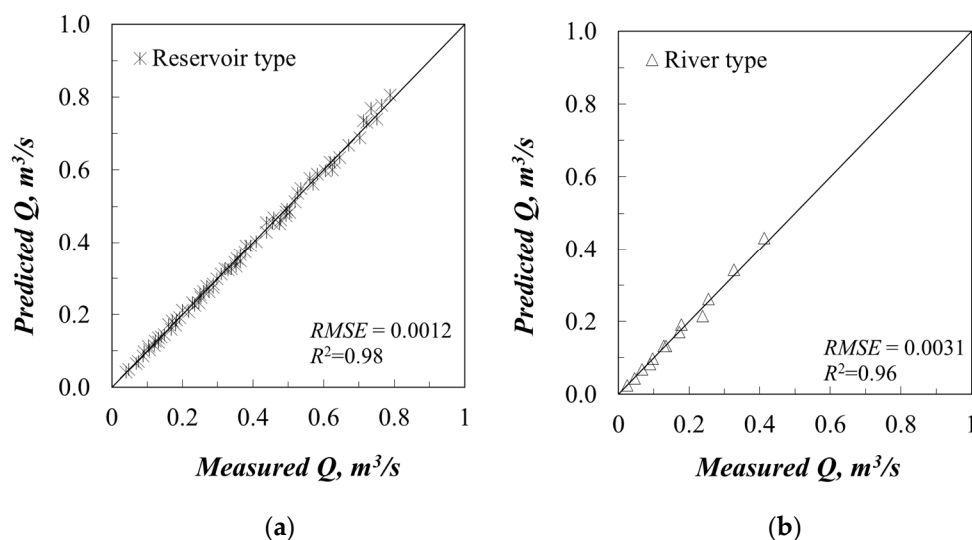


Figure 9. Correlation of discharge data for (a) reservoir type and (b) river type.

4. Conclusions

One of the important purposes of building a levee is to confine the flow during large floods thereby protecting the land use and population located on the other side of the levee. Recently, issues of levee system failure have attracted attention of government agencies as well as engineers because of climate change that may cause historically unprecedented flood stages in the rivers and their tributaries. Thus, state planners are facing the challenge of preparing for and responding to future flooding by a levee failure in the populated land use area for more sustainable environments. Sometimes political parties need to decide “Levee Upgrades” to protect their failure or “No upgrades” based on the economic consideration. On the other hand, engineering communities can emphasize the importance of a central control center with remote operation of all control structures based on real-time meteorological forecasts, past historical system behavior, and risk-based decision-making techniques. However, “all of the above”, primary information that needs to be known first is the discharge amount though an opening caused by a levee failure. Thus, in this research, discharge coefficient in levee breach under two different approach flows including a reservoir type approach and river type approach are analyzed by using a large scale laboratory experiment. A trapezoidal shape as well as a rectangular shape of levee breach are considered in the experiment. The results show that the discharge coefficient is a function of the ratio between head over the bottom of the breach section (H_0) and the width of the breach opening (L), but effect of approach Froude number (Fr_1) is negligible for a reservoir type approach. However, for a river type approach, the approach Froude number is tightly related to the behavior of discharge coefficient as well as the value of H_0/L . Based on the findings, discharge coefficient formulas are suggested that can be readily useable for discharge rating with respect to each approach flow type. The side slope (s) that account for the shape of a levee breach is used to calculate flow area over the breached section, and the results show that different shape of levee breach on each discharge coefficient can be ignored if the shape of geometry is correctly taken into account for the flow area calculation. The experimental results also explore transition between the reservoir

type approach and the river type approach with respect to the value of approach Froude number which is an important finding because an approach flow type must be identified first to ensure that the appropriate discharge coefficient formula is used to compute discharge. Based on other researcher's findings, the discharge coefficient can be a function of other geometry variables. However, in this research, geometric variables including weir height, width of approach channel, and shape of approach channel are constant. Thus, determining a comprehensive relationship with those geometric variables is impossible. Additional experiments should be conducted to establish more general relationship for discharge coefficient including several other geometric variables as well as hydraulic variables.

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