Flow Measurements in Tidal Spillways

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Abstract

Fresh water flows in coastal and estuarine environments may create ecological problems due to reduction in salinity levels. Therefore, accurate measures of regulated freshwater discharges are important for an effective management of coastal waters. Traditionally flow rates through spillways are estimated, depending on the gate opening, either as orifice-type flow or flow over a weir. However, for fully open gates under low-head and tidally-effected conditions estimates based on the traditional formulas are inaccurate. This paper summarizes the state-of-the-art of flow through spillways. A new formula for estimation of flow rate in tidally-affected spillways is presented. The formula was developed by applying dimensional analysis and π-Theorem and was calibrated and verified by using field data from different coastal spillways in Southeast Florida.

Keywords

Coastal waters, Dimensional analysis, Flow measurements, Modeling, Spillways

1. Introduction

The exponential anthropogenic growth in southern Florida necessitated the development of water controls in order to maintain adequate fresh water supplies for the rapidly growing population along the lower east coast, to sustain agricultural activities and to restore and protect the Everglades National Park and other delicate wetland ecosystems. The agency in charge of the regional water controls is the South Florida Water Management District (SFWMD) which works in cooperation with the U.S. Army Corps of Engineers and other federal, state and local agencies. The SFWMD operates and maintains over four hundred hydraulic structures including spillways, weirs, culverts and pump stations. Approximately one hundred and twenty of the hydraulic structures are gated spillways and about one-fifth of them are coastal spillways that discharge excess runoff water directly into the environmentally sensitive area of Biscayne Bay, south of the Miami-Dade metropolitan area. Over the past years serious concerns have been raised regarding the large fluctuations of salinity levels in Biscayne Bay caused by the freshwater releases. Salinity variations can have a negative impact on the quality and biodiversity of the native environment. Thus, for an effective and efficient management of the regional water resources system there is a substantial need for accurate estimates of the freshwater discharges.

Water managers at the SFWMD estimate flows at gated spillways using an in-house developed program known as FLOW that is based on instantaneous upstream and downstream water stages obtained from operational control information. The equations used in the FLOW model are basic orifice and weir type

discharge formulas that were developed by the U.S. Army Corp of Engineers in the early sixties by using data from small-scale physical models, as well as field observations from inland spillways (U.S. Army Corps of Engineers, 1963). Flow calculations, based on existing formulas for orifice-flow (free and/or submerged) and for free flow over a weir, provide adequately accurate predictions of the discharges. However, for the case where the gate is fully open and discharge occurs under submerged flow conditions the estimates are inaccurate. The inaccuracy in flow estimations worsens during extreme events (e.g., hurricanes) and/or at structures directly affected by tides. The reason for this inaccuracy is that the flow is controlled in a very complex way by a variety of parameters involving water properties, flow regime, structure and canal geometry, as well as other occurring factors such as floating debris, gate leaking, etc. A detailed description of the flow regime in the near-the-gate zone is extremely complicated. Vortices of all scales are continuously being generated, cascaded and dissipated in a very dynamic manner. However, for practical purposes the simulation of those localized turbulent effects is unnecessary since the net flow can be calculated by utilizing a number of bulk and easily measurable, in real-time, parameters. Notably, these parameters can be engaged in an empirical rather than a physically-based model. For that purpose the Buckingham Theorem (π-Theorem) can be employed so that a regression type formula is established (Ansar and Raymond, 2005).

This study summarizes the state-of-the-art of methods related to flow estimation over spillways, and extends the work of Ansar and Raymond (2005) by applying a wider range of physical and geometric parameters to the predictive formula. The approach utilizes dimensional analysis and π-Theorem for the derivation of a general formula. Then the formula is calibrated through regression analysis by using field measurements and other data obtained from six coastal spillways in southeast Florida.

2. Formulas for Flow Estimation Through Gated Spillways

Depending on the gate operating conditions and the water elevation on either side of the spillway flow at coastal spillways is classified into five different flow categories (Ansar and Raymond, 2005), (Tillis and Swain, 1998). More specifically those categories are (Figure 1):

- Submerged orifice-flow (partially opened gate)
- Free orifice-flow (partially opened gate – i.e. gate is in the water)
- Submerged weir-flow (fully opened gate)
- Free weir-flow (fully opened gate – i.e. gate is out of the water)
- Submerged tidally-affected weir-flow (fully opened gate)
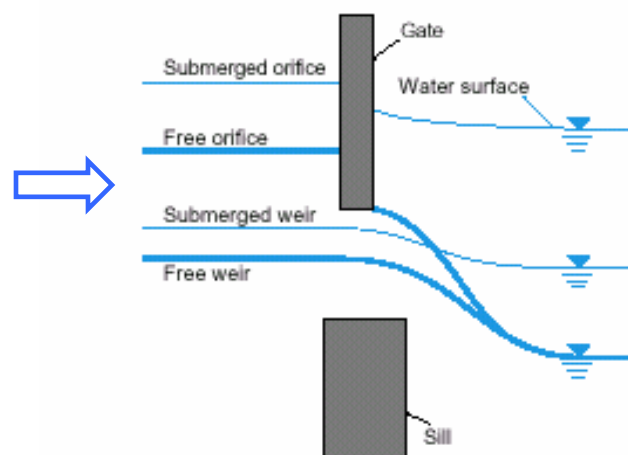
Figure 1: Classification of Flow Through Gated Spillways (Tillis and Swain, 1998)

2.1 Broad-crested weirs – Free flow conditions

Flow over a fully open spillway structure is hydraulically similar to flow over a broad-crested weir. A general classification of broad crested weirs as related to the upstream hydraulic head and their length in the direction of flow was provided by Jain (2001). For subcritical upstream flow conditions, the discharge over a broad-crested weir is estimated by the well-known general formula

$$Q = C_w BH^n$$

where B is the top width, H is the upstream hydraulic head, C_w is a correction coefficient, and n = 3/2. The coefficient C_w depends on a number of variables including the displacement thickness of the boundary layer (Rouse, 1950), the weir height (Hwang and Houghtalen, 1996), and the weir length and configuration of the upstream crest corners (Khosrojerdi and Kavianpour, 2002). The correction coefficient requires calibration from field measurements. The coefficient C_w can also be approximated as:

$$C_w = C_d C_v$$

where C_d is a correction coefficient that accounts for local friction losses, flow curvature and nonuniform flow distribution, and C_v is a correction coefficient that accounts for the velocity head (U.S. Bureau of Reclamation, 2001). The U.S. Army Corps of Engineers provided extensive design criteria for different types of spillways (i.e., sharp-crest, ogee, spillway chutes, and morning glory spillways). Those criteria include design characteristics of the spillway as well as charts for selection of discharge coefficients (U.S. Army Corps of Engineers, 1987).

2.2 Broad-crested weirs - Submerged flow conditions.

Under submerged flow conditions the governing equation for weir discharge is, (U.S. Army Corps of Engineers, 1963):

$$Q = C_s Bh\sqrt{2g(H - h)}$$

where h is the tailwater head. The coefficient C_s is commonly expressed as a function of the ratio H_t/H as, (Tillis and Swain, 1998):

$$C_s = \alpha\left(\frac{h}{H}\right)^\beta$$

where α and β were experimental coefficients. Another approach used by the USGS for submerged flow is given by the equation (Collins, 1977),

$$Q = C_{so} C_w BH\sqrt{H}$$

where C_w is the discharge coefficient under free flow conditions, and C_{so} is an additional flow correction factor. Leliavsky (1965) expressed the discharge for submerged flow conditions as:

$$Q = \frac{2}{3} CB\sqrt{2g}(H - h)^{\frac{3}{2}} + CBh\sqrt{2g(H - h)}$$

The recommended value for the empirical coefficient is $0.57 < C < 0.63$. Experimentally has been shown that the shape of the upstream corner of a weir can also have an effect on the discharge for both free and submerged flow conditions. Using a combination of momentum theory and dimensional analysis, Skogerboe and Hyatt (1967) investigated the effects of submergence on flow measuring flumes. The expression that they proposed is:

$$Q = \frac{C_1(\Delta H)^{n_1}}{[-(\log S + C_2)]^{n_2}}$$

where $\Delta H = H - h$ is the head difference, $S = h/H$ is the submergence, and n_1, n_2 are experimental exponents. The values for the exponents were given as $n_1 = 1.5$, and $1.0 < n_2 < 1.5$. The same authors suggested a similar formula for submerged subcritical flows over embankments (Skogerboe and Hyatt, 1967a). For long crested weirs, it is proven that submerged flow, Q_s, causes a reduction in discharge as compared to free flow conditions, Q, by the following ratio, (Irrigation Research and Training Center, 2002),

$$\frac{Q_s}{Q} = \left[1 - \left(\frac{h}{H}\right)^{1.5}\right]^{0.385}$$

The above equation is known as the Villemonte relationship (Florida Department of Transportation, 1999). Generally, Mossa et al. (2003) suggested that for a general study of flow over a spillway the Froude, Reynolds, and Weber numbers, as well as the relative roughness may be necessary to be considered for discharge estimations.

2.3 Spillway gate flows in tidal environments

Flow over a partially constructed barrage under tidal conditions was investigated by Halliwell (1967). In his study, flows were discretized into four categories: flood tide free flow (critical flow depth over the crest), flood tide submerged flow, ebb tide free flow and ebb tide submerged flow. The velocity, U, over the barrage was assumed to depend on seven parameters, i.e.

$$U = f(H, B, g, A, T, A_i, t)$$

where g is the gravitational acceleration, A is the tidal amplitude, T is the tidal period, A_t is the area of the confined water body, and t is the time in the cycle. If only the maximum velocity within the cycle is considered, then the parameter, t, is dropped from consideration. On another study, Ansar and Raymond (2005) assumed that the discharge at a tidal gate is a function of eight parameters, i.e.,

$$Q = f(H, P_w, B, g, \mu, \rho, A, T)$$

Where P_w is the weir height, μ is the dynamic viscosity, ρ is the water density, A is the tidal amplitude and T is the tidal period. By using π-theorem and dimensional analysis they derived

$$\frac{Q^{\frac{2}{3}}}{B^{\frac{2}{3}}g^{\frac{1}{3}}P_w} = \lambda\left[\frac{A}{T\sqrt{gP_w}}10^6\right]^{\xi}\left[\frac{H}{P_W}\right]^{\zeta}$$

This regression type model improved the flow predictions as compared to those obtained by the U.S. Army Corps of Engineers formulas (1963). However, still there was a discrepancy in the prediction of maximum and minimum flows (Ansar and Raymond, 2005).

2.4 Special conditions affecting flow estimations over spillways

The flow over a spillway gate can be also be affected by a number of localized conditions such as: (a) channel constriction and skewness (Fiuzat and Skogerboe, 1984), (b) density stratification due to salinity or suspended sediments (Denton, 1987), and (c) air entrainment during gate operations (Emiroglu and Baylar, 2003).

An extensive literature review regarding flow over spillways, weirs and embankments indicated that although there is a plethora of information regarding free flow conditions, there is only a miniscule amount of information regarding submerged flows under tidal effects (Scarlatos, 2005).

3. Methodology for Flow Estimation Through Coastal Spillways

3.1 Application of the π-Theorem

Flow through coastal spillways depends on a number of natural and artificial quantities of varying degree of importance. These quantities can be generally related to the properties of the transported water, the characteristics of the flow field, and the geometric design of the spillway structure. An extensive list of dimensional variables that can be potentially involved in the description of flow through coastal gates is discussed in the following sections. Those variables were particularly selected based on literature review, field observations, and as-built blueprints for structures S-123, S-20F, S-20G, S-21, S-21A, and S-22.

3.1.1   Water Properties Variables.
The predominant variables related to fluid properties that can affect the discharge are the following: density of fresh water (ρ_f), density of saline water (ρ_s), density of transported solids (ρ_t), concentration of transported solids (C), dynamic viscosity of water (μ), surface tension (σ) and gravitational acceleration (g). Regarding those fluid-related variables, the densities of fresh water, saline water and transported solids are important to characterize any stratification effects and identify the possible formation of a halocline or lutocline interface. Salt water intrusion (saline wedge) through the gates and sedimentation effects (shoaling or scouring) if ignored, can cause inaccuracies to discharge calculations. The amount of transported solids is also significant because in high concentrations it can even change the Newtonian behavior of the water to that of a pseudo-plastic or Bingham fluid (Mehta et al., 1991). Surface tension is of minor importance, while gravitation acceleration is the driving force of the entire phenomenon and thus the most important of all the parameters involved.

3.1.2   Flow Characteristics Variables.
The most important variables that control the flow regime are the following: upstream hydraulic head (H), downstream (tail-water) hydraulic head (h), flow discharge (Q), tidal amplitude (A), tidal period (T), time within the tidal period (t) and location of saline interface (H_i). Regarding those flow-related variables, under submerged flow conditions discharge is controlled by both the upstream and tail-water hydraulic heads. These heads are measured in reference to the crest of the weir. Tidal effects are characterized by the tidal amplitude and tidal period. The tidal amplitude is measured in reference to the MSL (Mean Sea Level). Consideration of the time within the tidal period is necessary for flow estimations and the period can be divided into three time segments representing the ebb, flood and slack tides. Discharge through a spillway is enhanced during ebb retarded during flood and unaffected during slack tide since there is not any tidal current affecting the discharge. Tidal effects during slack tide are implicitly accounted for through the tail-water elevation (high or low water). In case of saline wedge the elevation of its interface

is important since it can cause reverse flow and salt water intrusion, resulting not only in inaccurate flow estimations but also undesirable environmental conditions.

3.1.3   Structural Design Variables.

Before discussing the structural design variables, it is important to outline the particular features of the coastal spillways under consideration. The structures can be generally grouped into two main groups (a) those with an elevated rectangular spillway structure (Figure 2), and (b) those with a low-elevation gradually-rising spillway design (Figure 3). Spillways S-123 and S-21-A belong to the former group while spillways S-20F, S-20G, S-21 and S-22 belong to the latter design group. In addition, it should be noted that on the top of the spillway there is a weir located directly under the gate. It is anticipated that those two structural designs to create different flow patterns. Furthermore, it is expected that for the elevated rectangular spillway design flow characteristics would be different under high and low upstream head. Under high head conditions the massive spillway structure would control the flow while for low head conditions flow may be controlled only by the top weir. In the proceedings for clarification purposes the entire structure will be called "*spillway*" and that includes both the base structure and the "small" weir on the top, while the top weir under the gate will be called as "*weir*".

Based on the above observations the structural design parameters are defined as follows: spillway height (P_w), spillway crest length (L_w), top weir height (P), top weir length (L), spillway gate opening width (b), maximum gate opening (D), spillway skin roughness (k_s), angle of upstream spillway wing-wall (φ), width of upstream channel (B) and eccentricity of operating gate (ε). The spillway height is measured from the top of the canal bottom to the weir crest, while the spillway length is measured longitudinally over the entire spillway structure excluding the length of any protective upstream and downstream riprap. The weir height is measured from the top of the massive spillway structure to its crest, while the weir length is measured longitudinally over its crest. The spillway opening is the total width when all gates are opened. The skin roughness accounts both for the spillway surface (concrete) as well as the protective riprap. The angle of the upstream wing-walls is affecting the flow by streamlining the water towards the gate. Flow calculations can also be affected by the eccentricity of the structure and the location of the operating gate(s) in reference to the axis of the upstream canal. This is important since some structures have two or three gates that may not operate all at the same time. Finally, the width of the upstream canal is included since it is indicative of the flow convergence from the wide canal into the gates.

3.1.4   Formation of Dimensionless Groups.

Considering the specifics of the 21 variables listed in the preceding paragraphs, and accounting for the specifics of the local conditions the parameters can be combined through the Buckingham Theorem into the following theoretical expression:

$$\frac{Q}{\sqrt{g(H-h)}Hb} = \alpha\left(\frac{QL}{\nu b(H-h)}\right)^{\beta}\left(\frac{A}{T\sqrt{gP_w}}\right)^{\gamma}\left(\frac{Q^2}{k_s^3 gL_T^2}\right)^{\delta}\left(\frac{A}{h}\right)^{\varepsilon}\left(\frac{h}{P_w}\right)^{\zeta}\left(\frac{H-h}{P}\right)^{\eta}\left(\frac{b}{B}\right)^{\vartheta}$$

where the coefficient α and the exponents β, γ, δ, ε, ζ, η and θ are to be estimated numerically. Those constants must be evaluated under three tidal flow conditions i.e. ebb, flood and slack tide. For slack tide the exponent γ should be set equal to zero. Also, for smooth flow regime $\delta = 0$, while for rough flow regime $\beta = 0$. The dependent variables that need to be provided as input for the theoretical expression are the upstream head, tail-water head, tidal amplitude, and the gate opening width if all gates are not operational. All of the other quantities are constant. Any other potentially contributing factors (i.e., salinity, suspended solids, surface tension, eccentricity, etc,) are lumped within the coefficient, α, unless existing data indicate the need to express them explicitly.
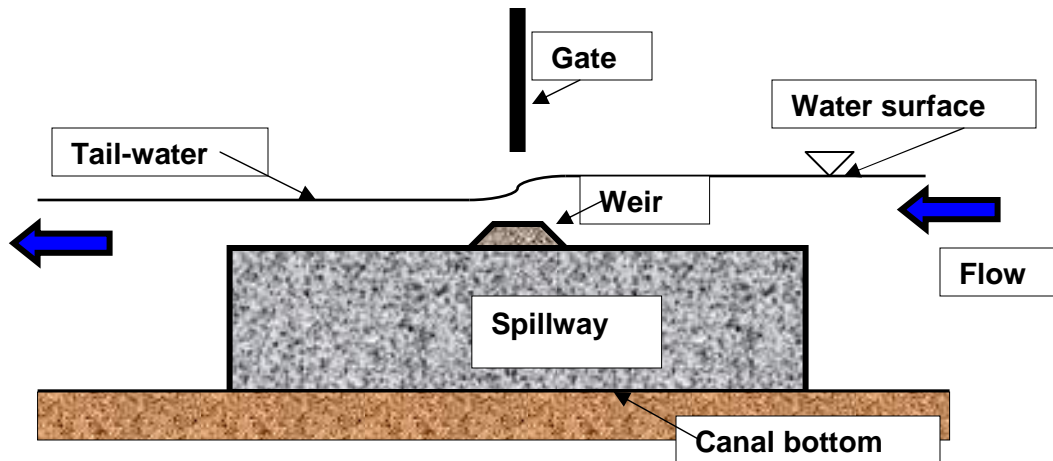
Figure 2: Schematic configuration of Structures S-123 and S-21A
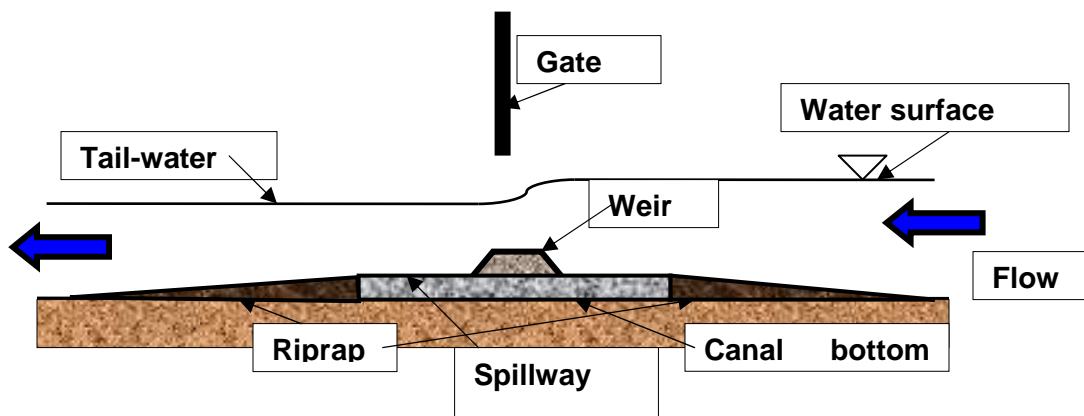

Figure 3: Schematic configuration of Structures S-20F, S-20G, S-21 and S-22

The physical significance of the individual terms in the above equation is identified as: $\dfrac{Q}{\sqrt{g(H-h)}Hb} = F_r$ is the Froude number and shows the relative significance between inertia and gravitational forces; $\dfrac{QL}{\nu b(H-h)} = R_e$ is the Reynolds number and shows the relative significance between inertia and viscous forces; although the bulk flow is turbulent this term represents the effects of

the laminar sub-layer in case of smooth or transitional flow conditions; $\dfrac{A}{T\sqrt{gP_w}}$ shows the relative significance of the vertical tidal fluctuating motion (rising/falling of the water surface) versus the horizontal tidal propagation speed; $\dfrac{Q^2}{k_s^3 gL_T^2}$ shows the relative significance between inertia and frictional forces; $\dfrac{A}{h}$ shows the relative significance of the tidal amplitude versus the tail-water head; $\dfrac{h}{P_w}$ shows the relative significance between the tail-water and the height of the entire spillway structure; $\dfrac{H-h}{P}$ shows the relative significance between the head difference and the height of the weir; $\dfrac{b}{B}$ shows the relative significance between the gate opening width and the average width of the canal and can become important in case that not all of the gates are operational.

4. Calibration and testing

The proposed curve was calibrated using data from four coastal spillway structures in southeastern Florida, U.S.A. The actual flow rates were estimated by using a combination of continuously recording Ultrasonic Velocity Meter (UVM) and data collected from an Acoustic Doppler Current Profiler (ADCP). All geometric data were obtained from structural design blueprints while the water elevations and other related data were obtained from SFWMD records.
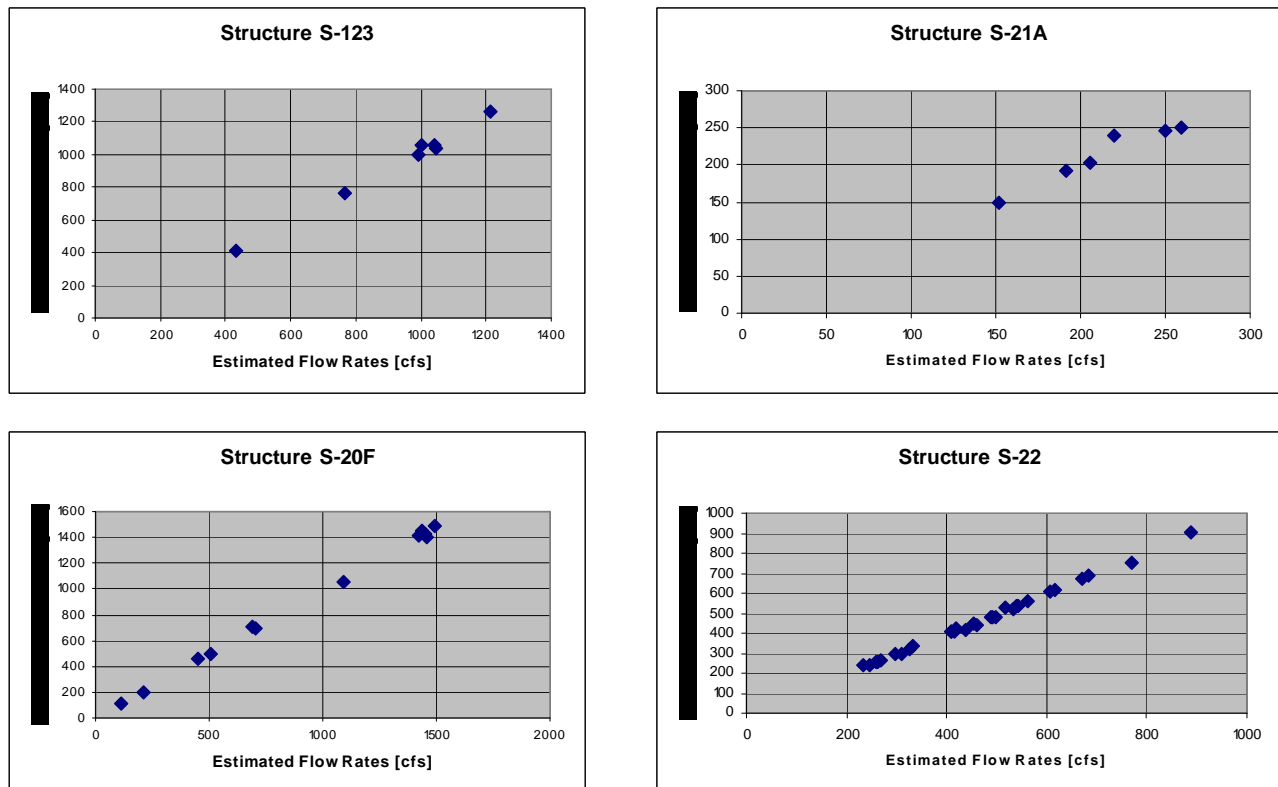

Figure 4: Flow Rate Simulation Results

A single calibration curve applicable to all of the spillways was still not feasible since there were some significant differences among the various structures under consideration. Those differences included design features of the structure (particularly the magnitude of the ratio of P/P_w) and the number of the spillway gates (ranging from one to three). The equations were fitted in terms of the unknown coefficient and the exponents by using data series of the discharge, Q, the headwater, H, and the tailwater, h. For that purpose, the governing equation was written in terms of natural logarithms so that a linear regression was applied. The simulation results as compared to actual field measurements are shown in Figure 4. As it can be seen the results are very promising, However, calibration and verification of the proposed formulas is still an undergoing process and the accuracy is anticipated to be improved by using additional data as they become available.

5. Conclusions

Based on the research and analysis conducted in this study, the following conclusions can be drawn:
1. There is a need for more accurate formulas for the estimation of flow rates over coastal spillways operating under submerged and tidally-affected systems.
2. A regression-type model based on dimensional analysis and π-Theorem is very effective in producing accurate flow predictions.
3. A wide variety of parameters involving water properties, flow characteristics and structural design can have a major effect on the flow rates simulation accuracy.

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