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HYDRAULIC PERFORMANCE OF A STORMWATER FACILITY OUTFALL STRUCTURE

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The Florida Department of Transportation

by

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HYDRAULIC PERFORMANCE OF A STORMWATER FACILITY OUTFALL STRUCTURE

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16. Abstract

This report details a performance analysis of a new outlet control structure proposed for use by The Florida Department of Transportation. This attenuator comprises a skimmer and weir, enclosed in standard precast elements, to protect against mowing accidents and vandalism. An analysis of the hydraulic performance of the attenuator was formulated and compared to a one-quarter scale physical model study. These results were then used to predict full scale performance.

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NOMENCLATURE

In this report, dimensions are given in English units. Where appropriate the SI equivalent units are denoted (SI). If no units are given then the quantity is nondimensional.

A = area, ft^2 (m^2)
b = channel width, ft (m)
B_s = thickness of skimmer, ft (m)
B_w = thickness of weir (characteristic dimension), ft (m)
C_D = discharge coefficient (based on pond elevation)
C_d = discharge coefficient (conventional)
C_{cw} = weir contraction coefficient
C_{cs} = skimmer contraction coefficent
Fr = Froude Number
g = acceleration of gravity, ft/s^2 (m/s^2)
h = depth of flow from channel bottom, ft (m)
h_w = height of weir, ft (m)
h_s = height to skimmer lip, ft (m)
H = reservoir elevation above weir crest, ft (m)
K = loss factor
l_p,l_m = model and prototype dimensions, ft (m)
L = width of weir opening, ft (m)
L_r = characteristic length ratio
n = exponent in Villamonte equation
P_w = height of weir crest, ft (m)
P_s = dimension of skimmer lip below weir crest, ft (m)
q = flow per unit width
Q = flow rate, ft^3/s (m^3/s,)
s =submergence ratio
V = velocity, ft/s (m/s)

CONVERSION FACTORS

To convert	British	SI	multiply by
Acceleration	ft/s^2	m/s^2	3.048E-1
Area	ft^2	m^2	9.290E-2
Density	$slugs/ft^3$	kg/m^3	5.154E+2
Length	ft	m	3.048E-1
Pressure	lb/ft^2	N/m^2	4.788E+1
Velocity	ft/s	m/s	3.048E-1
Volume flowrate	ft^3/s	m^3/s	2.832E-2
Volume flowrate	gal/min	l/s	6.310E-2

Constants

Acceleration of gravity	32.19 ft/s^2	9.81m/s^2
Density of water	1.94 $slugs/ft^3$	1000 kg/m^3
Manning's constant	1.485	1.0

EXECUTIVE SUMMARY

The Florida Department of Transportation has proposed a novel outlet control structure for detention facilities. While this device is fundamentally the same as a weir/drop box configuration in function, the new design incorporates several improvements. The underlying concept of this structure is first that it be constructed from standard components and secondly that it be integrated into the berm wall surrounding the detention pond. The purpose of this latter feature is to reduce damage incurred during mowing operations while providing a reasonably aesthetic appearance.

This report presents a study of the hydraulic performance of the control structure. To this end, a one-quarter scale model was constructed and tested under simulated field conditions, including both free discharge and flooded regimes. Head-discharge relationships were measured for various configurations. Observations included the sensitivity of performance to the weir configuration. As the device incorporates an entrance grating, skimmer and drop box, the influence of each of these components on overall performance was investigated and compared to an analytical prediction technique also developed during this investigation. Influences of an additional bleed orifice and trash accumulation were also considered. Sizing for application is discussed.

INTRODUCTION

Management of stormwater runoff is an important issue impacting satisfactory pavement drainage. The typical stormwater pond outfall structure is a box inlet with an opening in the side to release water at a controlled rate. These structures are installed out from the edge of the pond, near the toe of the berm, but are difficult to mow around and are hard to access due to elevation from the berm slope. Often the structures are so tall that a ladder is required. As a result of location, shape, and maintenance difficulties the structures are not usually aesthetically pleasing. An oil skimming device is almost always attached to the exterior of the structure which further reduces aesthetics and is subject to theft.

The FDOT Roadway Design Section plans to develop a standard outfall structure which would address the concerns mentioned above and could be used in most situations. This device was originally suggested by Frank Chupka of the Department and was presented in Reference 1. Throughout this report the device will be referred to as the "attenuator". The proposed configuration resembles a U-endwall specified as in Index 261 [2] conforming to the pond berm slope and incorporating a traversable grate over the opening. The skimmer and the control opening (the weir and/or orifice) are located behind the U-endwall and within the pond berm. The discharge pipe (if employed) is located beyond that point. In some situations requiring high capacity, tandem units could be employed. Maintenance personnel will be able to mow over the structure since it conforms to the berm slope and has a traversable grate. The skimmer is internal to the structure and therefore hidden. The only component clearly visible is the grate itself. Theft and vandalism usually associated with externally mounted skimmers should be eliminated.

The principal goal of the research effort reported here is to examine the hydraulic performance characteristics of the proposed outfall structure prior to adoption and present design parameters. In addition to measurement of discharge under various conditions, several specific questions must be considered. For example, the influence of the endwall grate and debris blockage on the control opening is not known. The hydraulic characteristics are essential in order to properly size the stormwater facility and minimize Department liability due to flooding.

DESCRIPTION OF THE ATTENUATOR

The attenuator discussed in this report is shown in Figure 1 as envisioned, and as originally installed in Figure 2. A schematic diagram is presented in Figure 3. The device consists of a conventional culvert endsection modified by the addition of a weir set in the end of the channel, and a skimmer at the end of the mitered entrance. A receiving box has been added to form a transition for the flow into a drainline for eventual disposal. As designed, the control (discharge limiting) point along the flow path is intended to be the weir, and it is the elevation of the detention pond that is to be regulated.

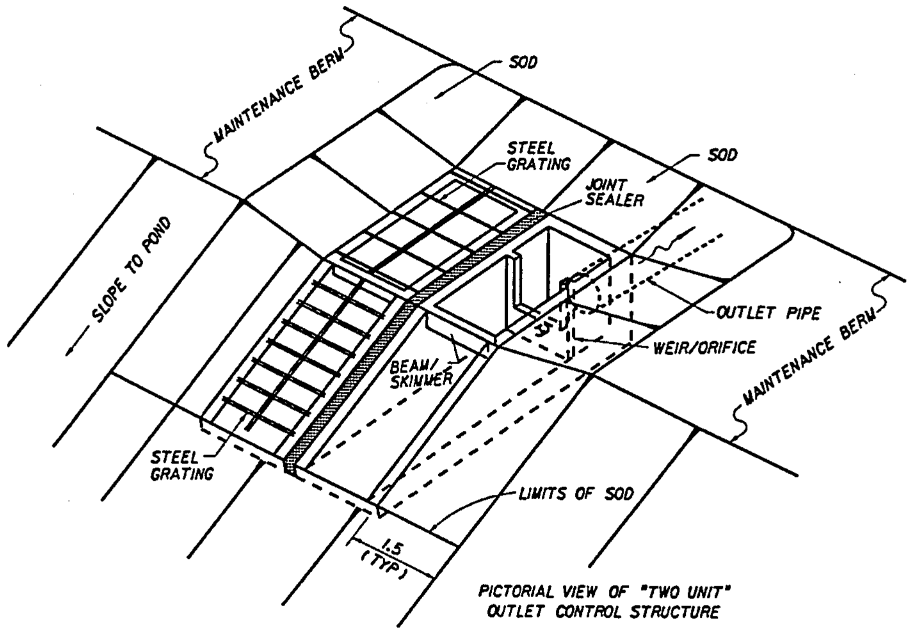

Figure 1: Conceptual diagram of proposed attenuator (provided by FDOT)

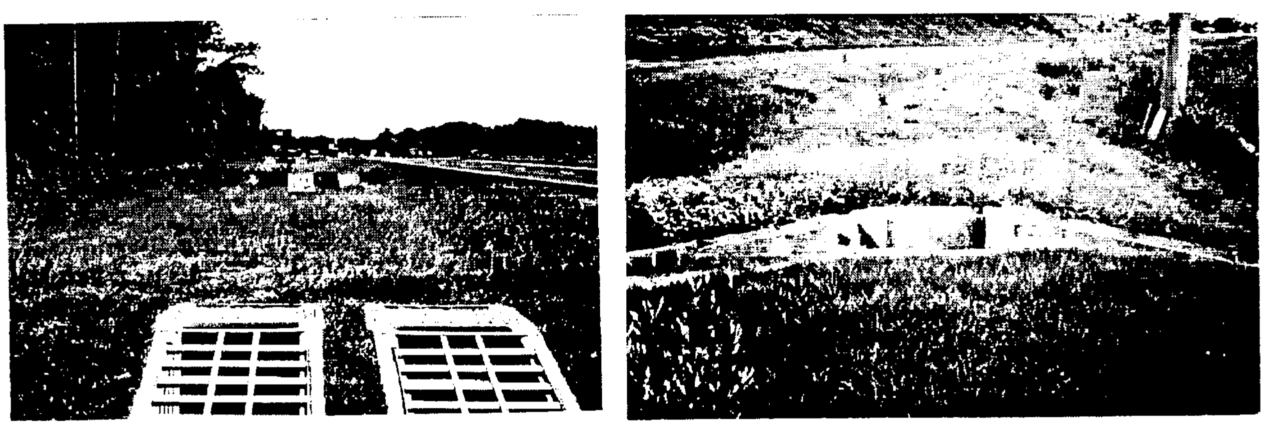

Figure 2: Field installation of attenuator as originally conceived, in a tandem installation with grate in place (left) and with grating removed to show skimmer and weir (right). Photographs courtesy of Frank Chupka, FDOT.

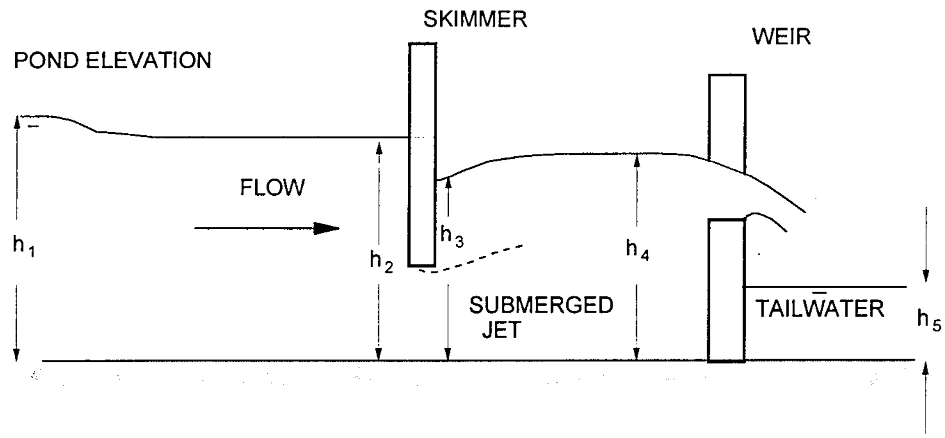

Figure 3: Definition diagram showing the water elevations along the flow path in relation to various components.

The operation of the attenuator may be described as follows, beginning at the detention pond (cf. Figure 3). No significant velocity develops in the pond, except near the entrance to the control structure. Water flows along the rectangular channel from the entrance (1) to the receiving box (5), with the elevation reduced in accord with the specific energy relation accounting for losses along the flow path. Flow under the skimmer resembles the submerged discharge from a sluice gate and the flow through the weir is similar to that of a conventional weir. The average velocity in the channel may be estimated from the continuity relation in terms of the flow rate from the pond. Ignoring channel friction because the channel is short, only losses at the grating and the skimmer need to be considered. Thus, the discharge can ultimately be related to the water elevation in the pond.

It is important to note however, that several features are present which require special consideration in evaluating the performance of the attenuator. The first of these issues is the fact that as proposed, both the skimmer and the weir will be fabricated in concrete necessitating an extended wall thickness in the direction of flow. This feature is typical of control weirs and has been examined in Reference 3.

Another type of complication occurs as a result of the relative size of the weir aperture and the approach channel dimension. When weirs are utilized for measurement of flow rate (rather than a control function), specific requirements are made for the height of the weir crest as well as the side clearance in the case of a contracted weir. While these specifications can be violated, special consideration must be given to the modification of the discharge relationship. For example, a weir formed with a very low crest would not be expected to obey the conventional empirical correlations. For the control structure considered here, consideration must be given to the spacing of the weir sides away from the side wall of the channel. For very wide weirs, data may reflect a modification of the flow at the edge. As shown in Figure 4, this region has been already determined to be influential due to the variability of attachment as the flow turns around the corner at the edge [3]. Furthermore it is noted that earliest designs for the control structure proposed suppressed weirs (a crest extending from wall to wall in the channel with no side lip). If employed, additional concerns for nappe ventilation and reproducible performance will be important.

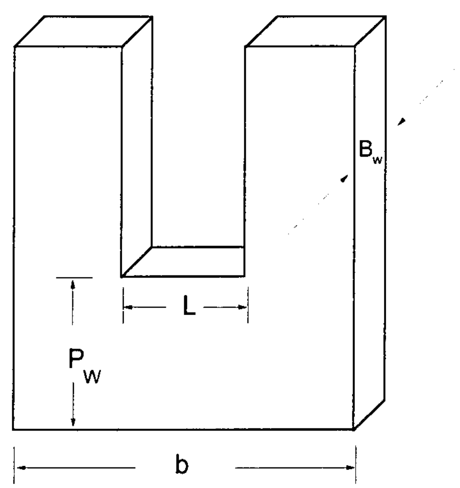

Figure 4: Definition diagram for weir notation [3]

In recent years, regulatory agencies have increasingly required the addition of skimmers to block the flow of oily waste and floating debris from passing through the weir. The skimmer must be positioned somewhat below the crest of the weir to preclude passage of this material as the pond elevation drops due to discharge or water losses. The effect of skimmer placement on discharge has not been thoroughly investigated however. In this investigation P_s=0.5 ft (measured between the weir crest and the lip of the skimmer) was assumed to be typical of this dimension. Other related issues include the possible addition of bleed orifices and trash blockage of the flow apertures.

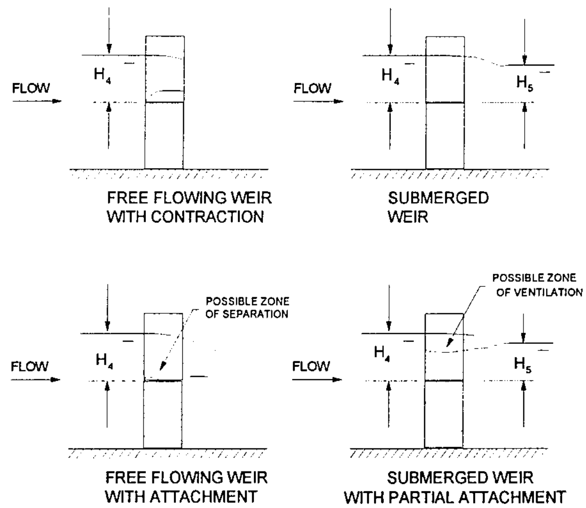

Figure 5: Separation in flows associated with weirs [3]

HYDRAULIC ANALYSIS OF ATTENUATOR PERFORMANCE

Viewing the attenuator as an integrated control structure, performance information can be represented as discharge as a function of pond elevation. Geometrical parameters can then be factored into an overall discharge coefficient and if submergence is a factor, the downstream tailwater would then become a secondary parameter. To develop this overall relationship analytically, it is necessary to link hydraulically each of the components of the structure along the flow path, as shown in Figure 3. Although the weir crest is the preferred datum (denoted with a capital "H"), in the discussion below it is more convenient to reference some elevations to the bottom of the flow channel (denoted with a small "h"). Dimensional notation for the flow channel is shown in Figure 3 and for weirs, Figure 4.

The control weir

The conventional treatment for a simple sharp-crested rectangular notch weir, relates actual performance to ideal via a discharge coefficient (denoted C_d).

$$Q_f = C_d L \sqrt{2g}\,(h_4 - h_w)^{\frac{3}{2}} \qquad (1)$$

Here the elevation h_4-h_w represents an elevation in the flow channel upstream of the weir [4], assuming that the weir is freely discharging at flow rate Q (denoted with a subscript f).

The discharge coefficient is in turn formulated to include a contraction coefficient (accounting for the influence of the edges of the weir) and a term accounting for the velocity of approach)

$$C_d = \frac{2}{3}C_{cw}[(1+\frac{V_0^2}{2gH_4})^{3/2} - (\frac{V_0^2}{2gH_4})^{3/2}] \tag{2}$$

where $H_4=h_4-h_w$.

Sharp edged notch weirs are typically very efficient in that the actual hydraulic loss is negligible but substantial flow contraction occurs through the notch. The coefficient of contraction is nearly constant (C_{cw}=.39) as long as the weir dimensions are not extreme [4] (cf. also Reference 5). A weir constructed in some other configuration may well include losses which could easily be incorporated in this formulation. With respect to the typical control structure configuration of interest here, a notch weir cut into a concrete wall of finite thickness may be treated in a similar fashion as shown in Reference 3, except that the coefficient of contraction, C_{cw}, was found to be a complex function of weir geometry. In that investigation there was no velocity of approach so $C_{cw}= C_d$. Weirs of finite crest width exhibit a reduced discharge coefficient as the water elevation above the crest is decreased and the flow approaches that of a broad crested weir. Furthermore, the discharge is very strongly influenced by the leading edge condition of the crest and sides which influences separation, especially at higher discharge. Results from [3] are reproduced in Table 1. The testing reported in [3] was performed with no velocity of approach. To represent the performance of the control weir in the attenuator, the coefficient of contraction was taken as that reported as the coefficient of discharge in [3]. It is noted also that in that investigation the factor of 2/3 was absorbed into the discharge coefficient.

Table 1: Recommendations for unconfined rectangular notch weirs (excerpted from [3]). In all cases, it may be necessary to interpolate to obtain a smooth relationship for the discharge coefficient.

UNCONFINED WEIRS

SHARP LEADING EDGE (LOWER LIMIT)	
($.25<H/B_w<2.0$)	$C_D=0.053(H/B_w) + 0.278$
($H/B_w>2.0$)	$C_D=0.39$
ROUNDED OR BEVELED EDGE	
($H/B_w<2.0$)	ELEVATE SHARP EDGED VALUE ABOVE BY 10%
($H/B_w>2.0$)	C_D=0.42-0.5

Discharge conditions for the weir

Normally, control structures are intended to operate with a free discharge from the weir. However, special consideration should be given to performance when substantial water elevation is present in the receiving box due to tailwater conditions or flow limitation in the drainline from the box. If the weir flow is partially submerged then the tailwater at Station 5 affects the discharge. This issue has been previously examined [6] and it was determined that the reduction in discharge can be adequately represented by the Villamonte relationship even for weirs with extended crests. Thus

$$\frac{Q_s}{Q_f} = [1 - s^n]^{0.385} \tag{3}$$

where n=3/2 for a rectangular weir and s is the submergence ratio, based on elevations above the weir crest.

$$s = \frac{H_5}{H_4} \tag{4}$$

The relationship between the elevation H_5 and the ultimate tailwater is essential to predicting the performance of the control operating in a submerged regime, but can only be calculated on a case by case basis since the configuration of the drain pipe is dependent on the specific installation. The following method is suggested. Starting at the entrance to the drainline (in the drop box downstream of the weir) a performance rating for the drainline should be established (elevation in box as a function of capacity). This computation is similar to culvert analysis since at least three possible modes of operation are possible (weir conditions at the inlet, inlet or outlet control). Inlet control is not likely since the drainline capacity is large. Outlet control most likely will occur in situations where the ultimate tailwater submerges the drainline termination, in which case the pipe losses will determine the elevational difference. Weir conditions at the inlet and a free surface flow in the pipe perhaps represent the most difficult situation to analyze. For the present, it will be assumed that either h_5 represents the ultimate tailwater or a relationship can be made between this point and the ultimate tailwater via the drainline connection to the receiving box.

Flow in the approach channel with skimmer

The flow rate through the weir may be related to the flow per unit width in the rectangular channel (width b in notation sketch Figure 4)

$$Q = qb \tag{5}$$

where

$$q = Vh \tag{6}$$

There are several approaches to modeling the flow between the skimmer and the weir and two will be examined here. The first of these is to follow a suggestion by Benson [7] to treat the skimmer as a submerged under flow gate, the discharge under the skimmer can be related to the water elevation in the entrance channel h_2. The conventional approach to this problem (as discussed in Henderson [4]), involves splitting the flow into a submerged jet emerging from beneath the gate at elevation $h_s C_{cs}$ where C_{cs} is a contraction ratio for the skimmer. A relatively stagnant water mass overlays this jet so that the hydrostatic pressure is elevated. Assuming no losses the specific energy equation for flow under the skimmer yields

$$h_2 + \frac{q^2}{2gh_2^2} = h_3 + \frac{q^2}{2g(h_s C_{cs})^2} \tag{7}$$

Note that the hydrostatic pressure acting on the flow emerging from under the gate is given by the water elevation at this point, even though the flow area is assumed to be computed from the opening height modified by a contraction coefficient. Further assuming that momentum is conserved between Stations 3 and 4 (no force acting)

$$\frac{h_3^2}{2} + \frac{q^2}{gh_s C_{cs}} = \frac{h_4^2}{2} + \frac{q^2}{gh_4} \tag{8}$$

Thus two equations in two unknowns, q and h_3, result. It should be noted that the actual contraction coefficient for flow under the skimmer is not known. For the case of a simple sluice gate with a sharp edge, a value of about 0.6 is frequently suggested. However, the proposed design of the skimmer comprises a substantially extended edge along the flow path. By comparison to the situation for a weir cut in to a thick concrete headwall, some degree of reattachment might occur, but this question can only be resolved by experiment.

The second method of treating the flow between the skimmer and the weir is to assume a simple turbulent loss mechanism, proportional to the velocity of flow under the skimmer. The apparent velocity (assuming no contraction) is given by

$$V_s = \frac{q}{h_s} \tag{9}$$

and

$$h_2 + \frac{q^2}{2gh_2} = h_3 + \frac{q^2}{2gh_s^2}K_s \tag{10}$$

For computation, the kinetic energy upstream at station 2 is negligible in comparison with the loss term. The flow through the weir is governed by the elevation h_3 and even though some velocity components may remain in this region, all of the kinetic energy is assumed to be dissipated. The solution to the governing equations is easily accomplished by assuming a flow rate through the weir, solving for h_3 and q, then computing h_2.

One of the most important points emerging from this discussion is that the final state of the flow issuing from under the skimmer determines the magnitude of the discharge over the weir. Ideally, a sufficient length of flow path downstream of the skimmer will be allowed so that the flow will be reequilibrated (uniform velocity, hydrostatic pressure distribution) at h_4 before passing over the weir. In that situation, the first method for computing the flow between the skimmer and the weir would probably be successful. In the proposed design however, the spacing between the skimmer and the weir is kept relatively short for practical reasons and it is unlikely that well developed flow is actually achieved (ie some artifacts of the submerged jet are likely to remain at the upstream side of the weir). In this case, the second (loss) model is more likely to be appropriate. Both approaches will be compared to experiment in the discussions following.

Flow at the entrance to the channel

To develop an overall performance characteristic or rating for the control, the elevation of the pond is related (as before) to the elevation of the weir crest

$$H_1 = h_1 - h_w \tag{11}$$

and this result is correlated with the flow through the weir. It is easiest to pick a range of discharge then solve for the respective elevations at each station, working back to the pond elevation. The weir equation (1) can be solved easily by an iterative procedure since the contraction coefficient is a function of the elevation as is the velocity of approach in the channel. The skimmer flow can be solved in a similar fashion. While the discharge can be written in terms of the upstream elevation h_2 and a discharge coefficient (which depends on the coefficient of contraction associated with the bottom of the gate, the opening under the gate and the downstream water elevation), it is more direct to eliminate h_3 then set the discharge over the weir equal to the discharge under the skimmer, and solve for elevation h_2, in terms of consistent values of h_4 and q, as obtained from the flow over the weir.

Once the flow state in the channel is known, the elevation of water in the pond can be related to the elevation in the inlet channel h_2 by the specific energy relationship (assuming the possibility of loss as flow enters the endsection. This loss can be written in terms of a kinetic energy loss factor K_{ent} based on the velocity in the entrance section, and a quiescent pond as

$$\text{losses} = K_{ent}\frac{q^2}{2gh_2^2} \tag{12}$$

so that

$$h_1 = h_2 + \frac{q^2}{2gh_1^2}(1+K_{ent}) \tag{13}$$

The entrance loss (based on the velocity just inside the channel) will account for both the geometry of the entrance section and also the presence of an entrance grate (specified as in Index 261[2]).

Solution to the governing equations

Ultimately, the goal of this analysis is to develop an overall coefficient of discharge C_D, based on pond elevation as defined by

$$Q = C_D L\sqrt{2g}\,H_1^{\frac{3}{2}} \tag{14}$$

(the capital D denotes the difference between this formulation and the conventional C_d). The solution to the governing equations for either modeling approach can be accomplished by spreadsheet or other means.

SCALING

The experimental models used in this investigation were constructed at one-quarter scale and no attempt was made to simulate concrete roughness. The generally accepted modeling relationships were assumed to apply, maintaining equivalent Froude numbers between model and prototype.

$$\frac{V_m^2}{gy_m} = \frac{V_p^2}{gy_p} \tag{15}$$

The relationship between the velocities and discharge derive from this assumption and the length ratio $L_r = l_p/l_m$.

$$\frac{V_p}{V_m} = L_r^{1/2} \tag{16}$$

continuity yields

$$\frac{Q_p}{Q_m} = L_r^{5/2} \tag{17}$$

Transfer relationships for a scale factor of one-quarter are summarized in the table below.

Table 2: Scaling relations for a quarter scale model

LINEAR RATIO	L_r	4
FLOW RATIO	Q_r	32
VELOCITY RATIO	V_r	2

It is possible to develop an overall empirical discharge coefficient for the entire control structure as defined in Equation 14. Rearranging

$$C_D = \frac{Q}{L\sqrt{2g}\,H^{3/2}} \tag{18}$$

where

$$C_D = C_D\left(\frac{h}{B_w},\frac{h_w}{B_w},\frac{B_s}{B_w},\frac{b}{B_w},\frac{h_s}{B_w},....\right) \tag{19}$$

Here the discharge coefficient is assumed to be a function of several geometrical ratios accounting for the configuration of the weir, and possibly other factors. It is emphasized that written in this form the discharge coefficient is a parameter including all significant losses and contractions in the attenuator and not simply related to the conventional discharge parameter associated with a weir. It is also emphasized that C_D is a nondimensional empirical parameter, not to be confused with the dimensional weir coefficient, which is the product of C_D and $(2g)^{1/2}$. The extent of the weir crest in the flow direction, B_w, (thickness) has been chosen as the nondimensionalizing factor. This discharge coefficient can be developed by experimental measurement or modeling. Both free and submerged discharge can be represented. Once developed, it is assumed that C_D is invariant with scale (following the scaling relations discussed previously), but may depend on many other factors. The philosophy of the present investigation is to approach the question of performance prediction by a) measuring an overall discharge coefficient for the attenuator at model scale, b) develop a predictive, analytical model, c) to compare these two results, with the goal of eventually improvement of the analytical approach to be a useful design method. With the tacit assumption that these results scale to the prototype, design performance predictions will then be possible.

EXPERIMENTAL INVESTIGATION

An experimental study of the performance of the attenuator was initiated using a 1/4 scale model. As shown in Figure 6, a simulated detention pond was constructed from resined plywood with the attenuator located at one end, discharging through a short length of pipe to a second ponding area (for tailwater control). Water pumped from a reservoir sump was introduced into the upstream pond through a large tee fitting behind a multiple V-notch baffle intended to minimize the motion in the pond. Water elevation in both the upper and lower basin could be independently controlled.

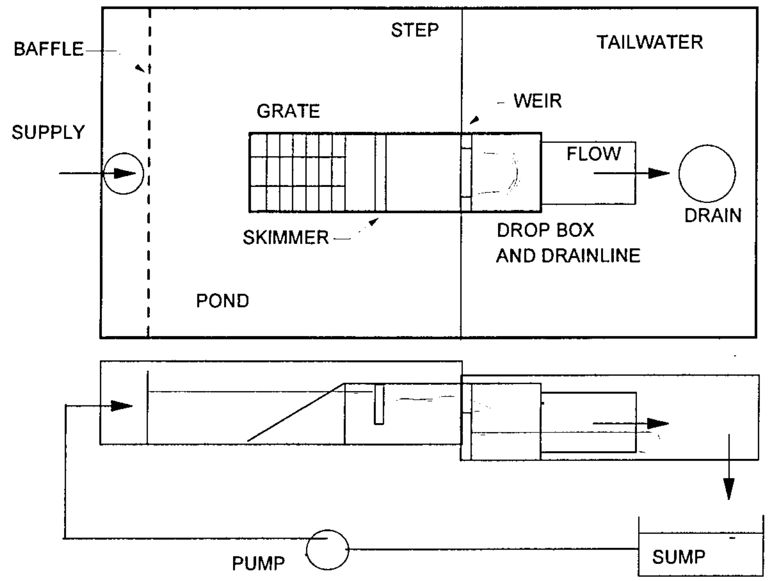

Figure 6: Schematic of experimental apparatus used in this investigation.

The model attenuator was also fabricated from resin coated plywood. In accordance with current FDOT designs, the entrance consisted of a nominal 4:1 sloped culvert end section resembling FDOT Index 261[2]. The receiving box bottom was dropped by a small amount (0.23 ft) representing an optional configuration. The model included a removable grating and provision for an adjustable skimmer. Although both the grating and skimmer would be installed in field applications, some testing was conducted without these components to investigate modifications to hydraulic performance.

Experiments were conducted by first establishing a datum at the weir crest. A secondary reference elevation was taken at the channel bottom. Upstream and downstream piezometers were fitted with stilling tubes and measured directly with a sharp pointed scale to an accuracy of .003 ft to observe pond elevations. Measurements of the water depth at stations 2, 3 and 5 were made directly with a scale for additional comparisons. The water surface at stations 3 and 5 was considerably disturbed and difficult to measure accurately. Flow into the pond was measured by a paddle wheel type flowmeter measuring pipeline velocity. Although the manufacturer provides a calibration for velocity, it was decided to independently calibrate the paddle wheel flowmeter directly for discharge while in place. First a sharp crested, retangular metal weir 0.417 ft wide was installed directly in position in the attenuator (without the skimmer and the grate in place. Pond elevation, channel elevation and flow meter output were recorded. Standard weir relations [4,5] were used to reduce the data and develop a calibration factor (assuming that a contraction factor =.39). As a second calibration, a weigh tank capture was used to measure discharge and correlate with flowmeter output. Even with these steps, an examination of the coefficient of discharge with the metal weir in place exhibited a slight trend, strongest at the lowest flow. The most likely cause of this discrepancy is that the weir in channel configuration does not conform to all requirements for boundary spacing usually recommended for weir testing. Some interference in the development of the flow through the weir may be expected from the proximity of the sidewalls and channel bottom to the weir opening.

The experimental procedure was relatively straightforward. Prior to the initiation of a test, the configuration and appropriate dimensions were recorded. Various points in the model were leveled prior to testing as part of the test. The flow rate was set to a particular value and the elevation in the pond was allowed to stabilize, allowing several minutes for the system to equilibrate. The head and discharge readings were recorded, including elevations along the flow path. The head was then varied through an appropriate range to form a data set. In some cases the experiments were repeated with the skimmer or grating removed to assess the influence of that component. Finally, experiments were further modified by measuring the head discharge relationship under flooded conditions downstream.

Several different weir configurations were tested and analyzed as discussed below. Except as noted, in these tests the crest height, P_W, was 0.417 ft, the crest breadth, B_W, was 0.125 ft and the skimmer thickness B_S, was 0.205 ft positioned at P_S = 0.125 ft below the weir crest. The width of the approach channel b, was 0.863 ft. Table 3 summarizes the geometry of weirs tested.

Table 3: Configuration of weirs tested (dimensions in ft).

WEIR	HEIGHT	WIDTH	THICKNESS
A	0.42	0.90	0.13
B	0.67	0.61	0.13
C	0.42	0.42	0.13
D	0.42	0.86	0.13
E	0.42	0.17	0.13
CAL	0.42	0.42	THIN

RESULTS AND DISCUSSION

The results of this investigation are presented in Figures 7 to15. Maintaining a constant weir crest elevation of 0.417 ft from the channel bottom, the dependence of discharge coefficient on pond elevation was measured for several weir widths, as seen in Figure 7. The empirical correlation for an unconfined weir (Table 1) has been added to the graph for comparison. The narrowest weir tested follows the unconfined correlation well, presumably because the flow rate is low and the effect of channel, grate and skimmer are negligible. It can also be seen that the discharge coefficient for weirs A and C decreases as the elevation increases. Two trend lines for higher elevations (above $H/B_w > 1.75$) were developed for these weirs (Table 4). The data for the weir spanning the full channel has been omitted from the graph for clarity, but similar results were obtained and the correlation is included in Table 4. Some problems with fluctuations in the nappe were noted as is often the case for full channel weirs.

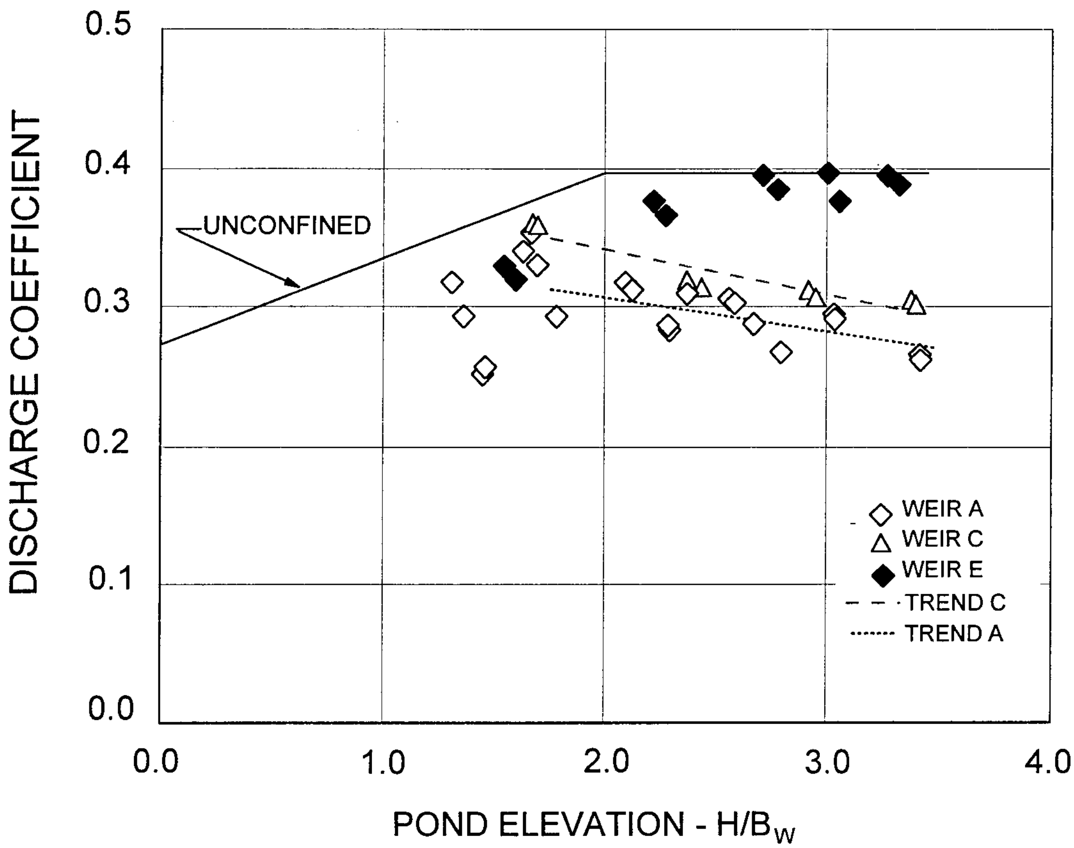

Figure 7: Correlation of data for constant weir height (0.417 ft) and various weir widths.

Figure 8 shows the effect of several skimmer and weir configurations. Again the same trend away from the unconfined correlation is obvious. The discharge coefficient rises for lower heads then falls instead of maintaining the nearly constant value reported in [3], presumably due to the influence of the skimmer. When the thick skimmer was replaced by a thin sheet skimmer, little change was noted. If the leading edge of the weir was rounded, the discharge coefficient was elevated over the value observed for a square edged weir but still tended to drop off. The effect of rounding the leading edge of the skimmer was negligible. In all tests except for two, the skimmer was maintained at $P_s/B_w=1$ (corresponding to 0.5 ft in a full size model). Lowering the skimmer caused a substantial reduction in the discharge coefficient. Correlations developed for the various configurations shown in Figure 8 are included in Table 4.

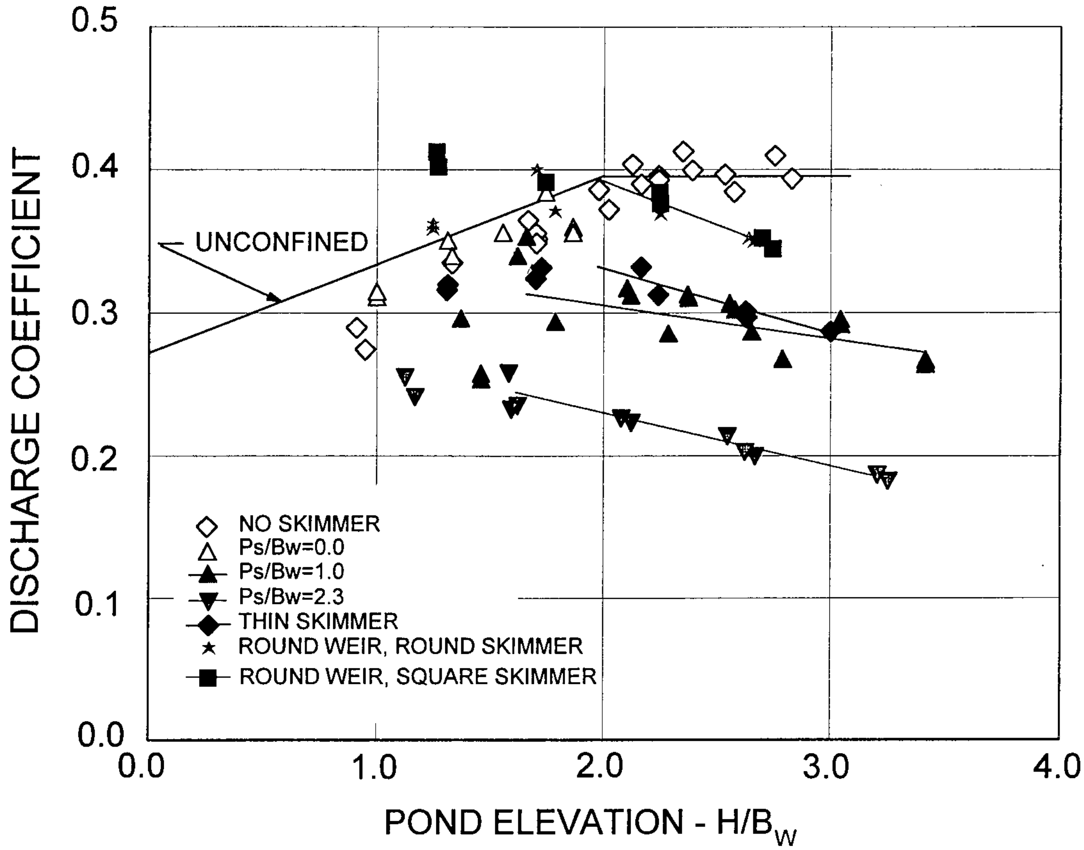

Figure 8: Effect of skimmer and weir variations for thick weirs.

Table 4: Empirical correlations for the attenuator developed in this investigation. The discharge coefficient should be merged with the unconfined value at lower heads. In all cases, it may be necessary to interpolate to obtain a smooth relationship for the discharge coefficient.

ATTENUATOR $P_s/B_w=1$ (for use above $H/B_W=1.7$)

WEIR A	L/b=0.70	$C_D= 0.355-0.0240(H/B_w)$
WEIR C	L/b=0.48	$C_D= 0.406-0.0325(H/B_w)$
WEIR D	L/b=1.0	$C_D= 0.364-0.0290(H/B_w)$
WEIR E	L/b=0.19	USE UNCONFINED RELATION
WEIR A and C combined		$C_D=.364-.0229(H/B_w)$

WEIR A MODIFICATIONS	L/b=0.70	$P_s/B_w=1$, unless noted
SKIMMER POSITION ($P_s/B_w=2.3$)		$C_D=.303-.0365(H/B_w)$
THIN PLATE SKIMMER		$C_D=.422-.0452(H/B_w)$
ROUNDED EDGE WEIR		$C_D=.512-.0597(H/B_w)$
(rounded and square edged skimmer data combined)		

Another set of experiments intended to clarify the effect of configurations was conducted using the metal calibration weir instead of the thick plate (Figure 9). Even though the metal weir is thin, the thickness of the extended weir was used to nondimensionalize the elevation above the crest, to assist in comparisons. For a thin rectangular notch weir at the side of an unconfined reservoir, a discharge coefficient of 0.39 is expected and this value may be used as a point of reference. An examination of the results obtained show that the effect of the approach channel and grate is quite small. Order-of-magnitude calculations confirm this result and it is probably realistic to neglect the influence of these configurational variations in design. Adding a skimmer causes a reduction in the discharge coefficient but rounding the leading edge of the skimmer does not alter this reduction.

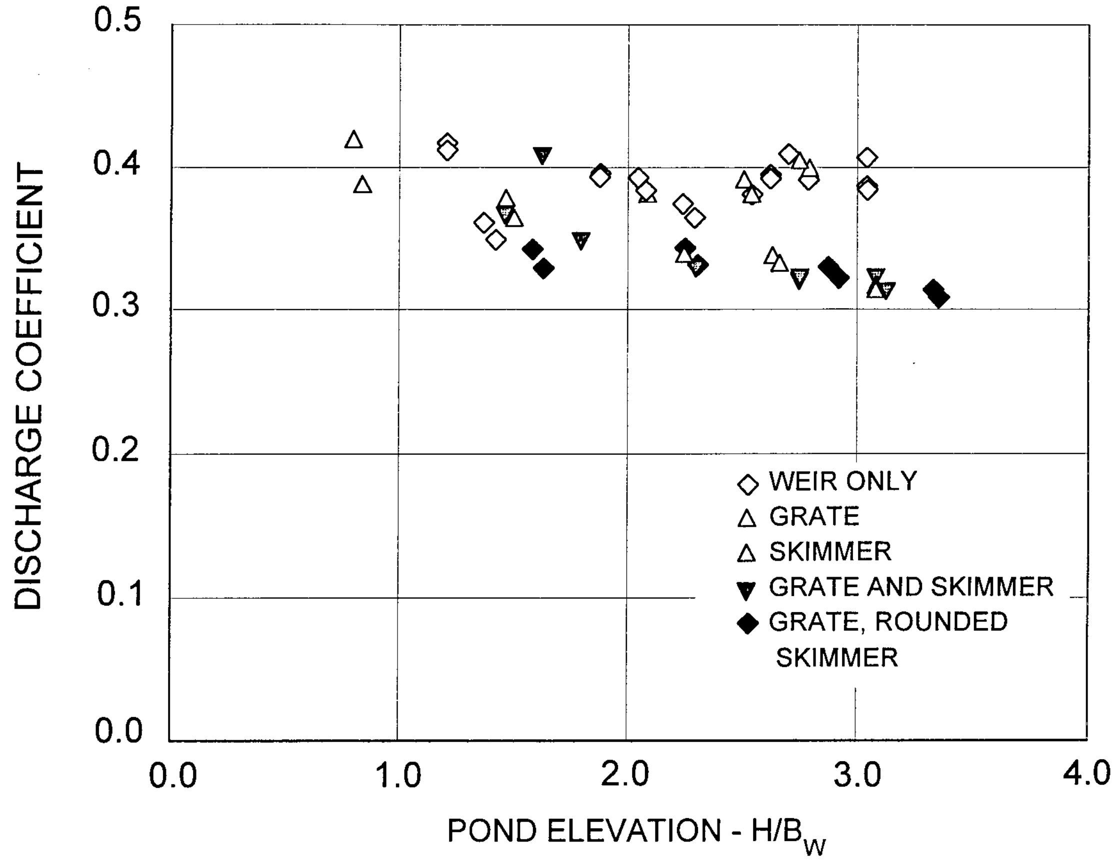

Figure 9: Effect of grate, skimmer and weir thickness on discharge coefficient (all tests made with thin plate, sharp edged weir).

An effort was made to correlate the analytical predictions discussed above with the data obtained during the investigation. Figure 10 shows a comparison between the data and modeling based on the first approach discussed (sluice gate analogy) for the two intermediate width weirs set at 0.417 ft above the channel bottom. While the analytical prediction does reproduce the trend of the data, the magnitude of the discharge coefficient predicted is somewhat larger than the data indicate (similar comparisons for weir E yielded better results, probably because the flow rate is low and the effect of the additional elements is small).

It was observed during experiments that the condition of the flow just upstream of the weir did not resemble the smooth surface associated with an unconfined weir freely discharging but rather a disturbed surface with an obvious upwelling along the upstream side of the weir plate. This characteristic is assumed be associated with a persistent submerged jet from under the skimmer and is consistent with the fact that lowering the skimmer further reduces the discharge coefficient. It can be inferred from these observations that the first approach to modeling results in poor predictions because the assumption of energy recovery from the submerged jet is not met. Instead, the small confined region between the skimmer and the weir results in mixing and dissipation of energy.

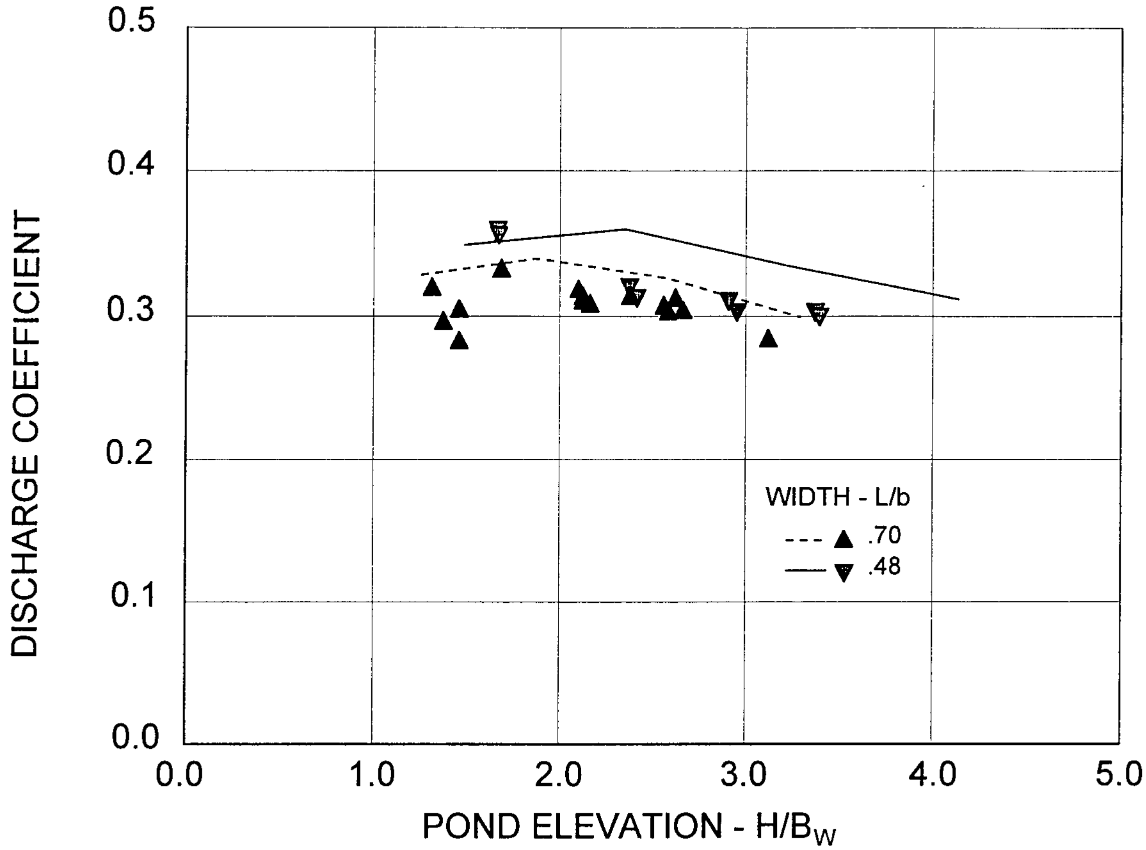

Figure 10: Correlation of data for weirs A and C with an analytical model based on a sluice gate analogy.

A comparison of the loss model (second method) with data were more successful, as can be seen in Figure11. A loss value K_s=3 was identified by combining a large number of data sets (weirs A, C, E and the calibration weir, all with skimmer and grate in place) and minimizing the residual square error by trial (neglecting the kinetic energy at station 2, as discussed previously). A value of 3 is quite large and confirms the substantial effect of skimmer clearance.

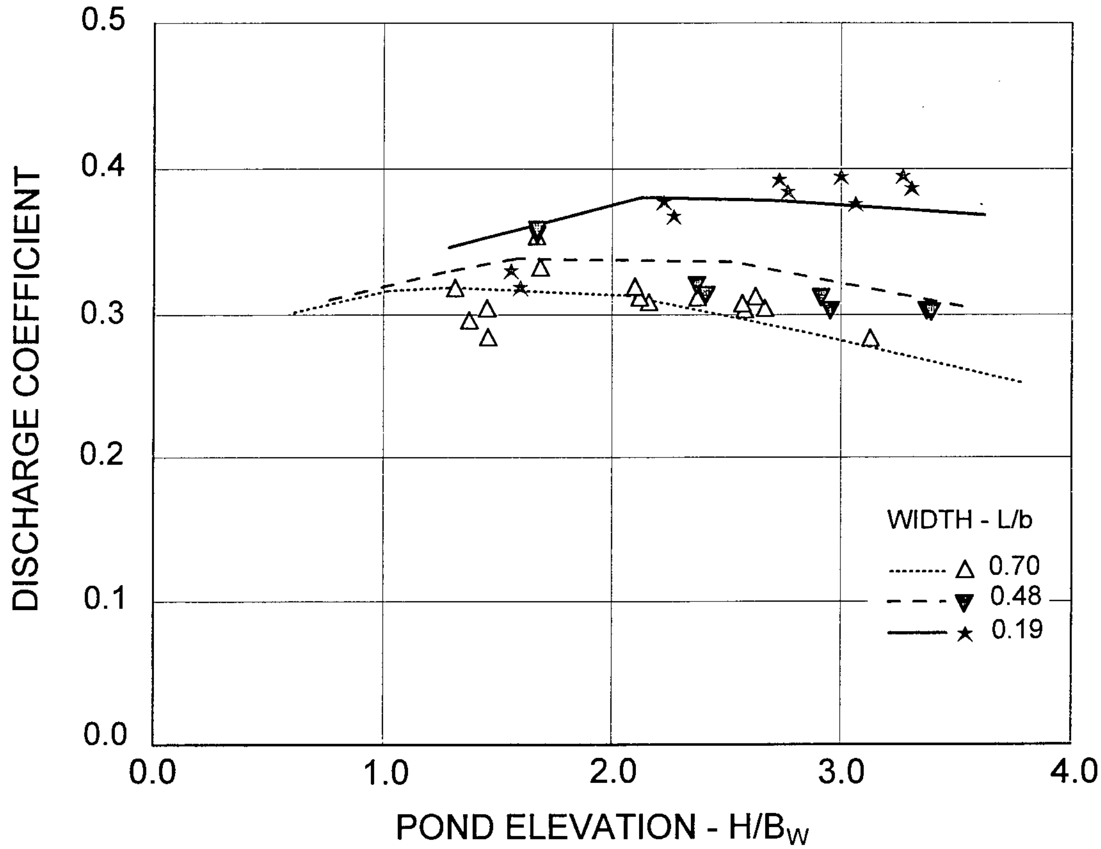

Figure 11: Comparison of several data sets of an analytical model assuming a turbulent loss at the skimmer (K_s=3).

As a further test of this approach, the method was used to predict the performance of both the rounded weir (assuming a weir contraction coefficient of 0.5 [3]), the suppressed, full channel width weir (assuming a weir contraction coefficient of 0.42), the case of the lowered skimmer (discussed earlier) and the sharp edged weir configured with skimmer and grate. The comparison with data obtained is shown in

Figures 12 and 13. For the rounded weir both square skimmer data and rounded skimmer data are presented and the comparison is quite reasonable, considering that the weir contraction coefficient is only an estimate. Again it is noted that the rounding of the skimmer lip seems to have little effect. Comparison for the channel spanning weir was less successful but not unrealistic. The sharp edged data were included in the original correlation, so good agreement would be expected (Figure 13), but most surprising was the result obtained for the case of the lower skimmer position. Agreement here indicates that the model may be used with confidence when the skimmer position is changed by small amounts (obviously removing the skimmer or lowering the lip by an extreme amount would not result in good agreement).

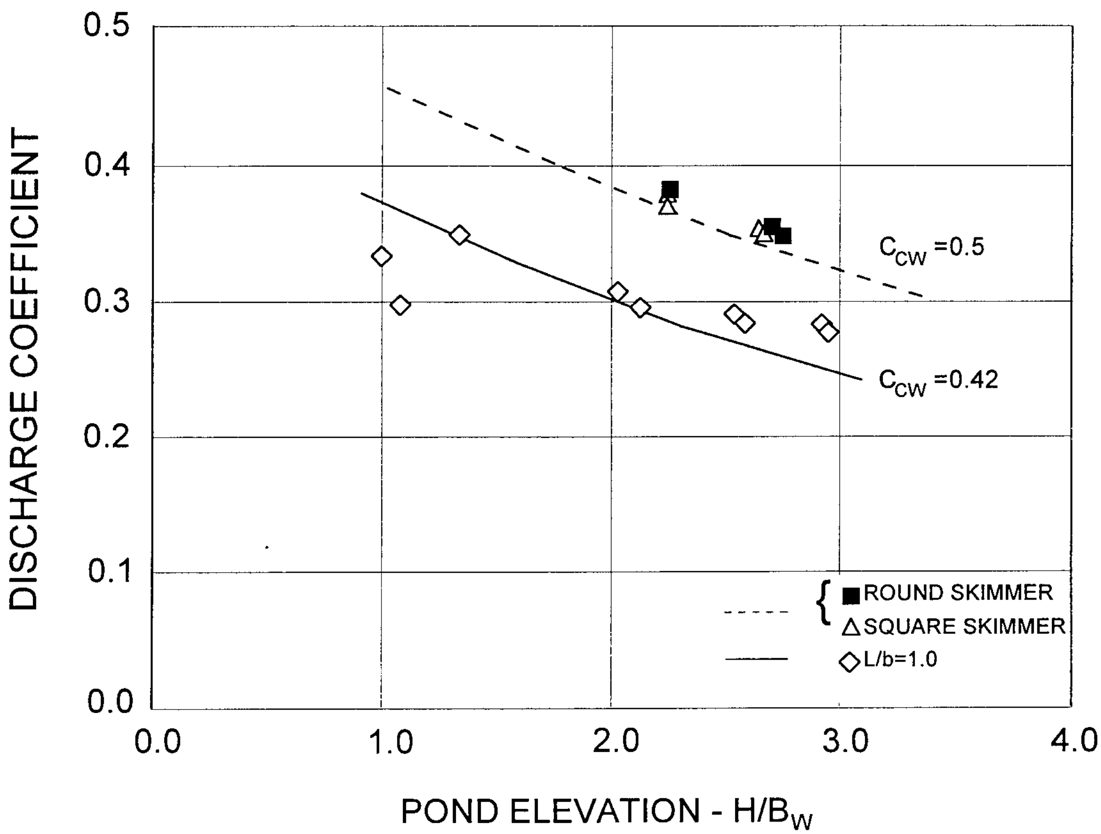

Figure 12: Extension of the model to the case of a rounded notch weir (L/b=0.48, Ccw=0.5 assumed) and a full channel width weir (Ccw=0.42 assumed).

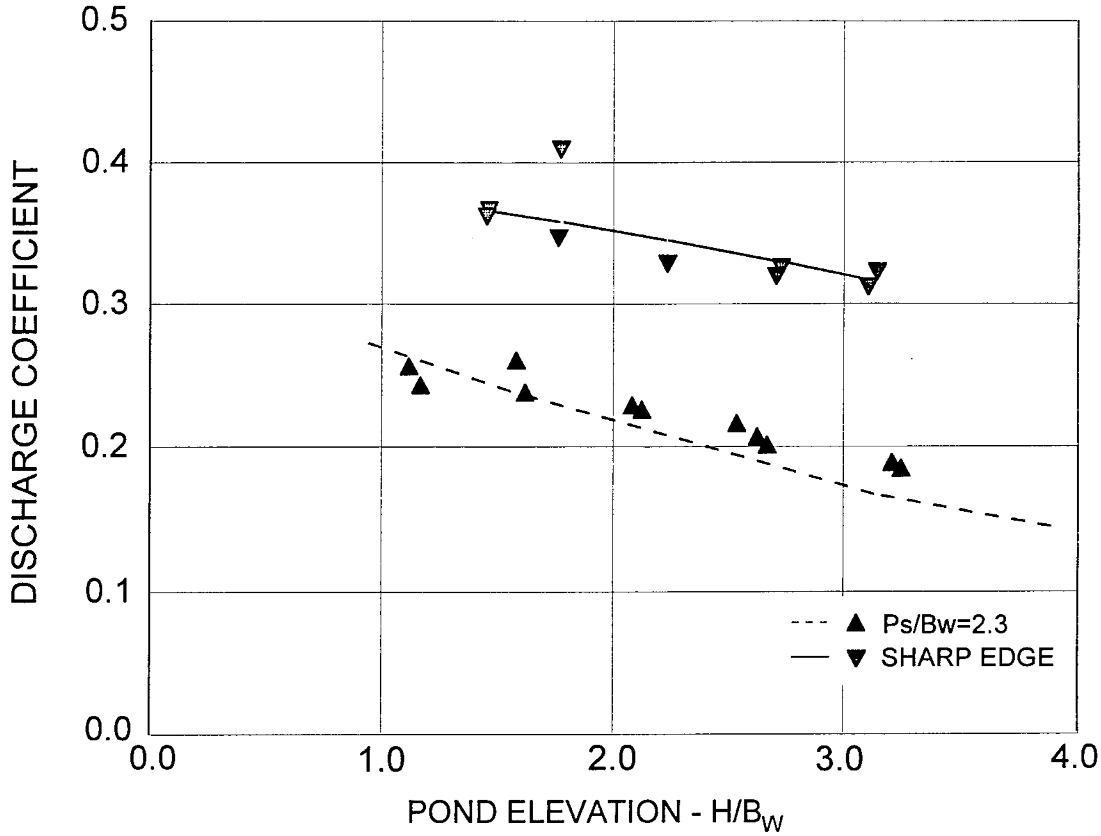

Figure 13: Extension of the model to the case of a sharp edged weir (L/b=0.48, Ccw=0.5 assumed) and weir A with a lowered skimmer (P_s/B_w=2.3).

Most of the data obtained in this investigation were for a constant weir height of 0.417 ft. Dropping the weir crest would limit the installation of a bleed orifice and possibly alter the weir performance adversely. Raising the crest is possible, although this change would tend to limit the control range. As shown in Figure 14, some experiments were conducted at a crest height of 0.67 ft. Over the range tested the data are comparable to the data taken at lower crest, but seem to be displaced slightly to the left along the abscissa. One possible cause for this displacement may be the reduction in the disturbance caused by the submerged jet from beneath the skimmer

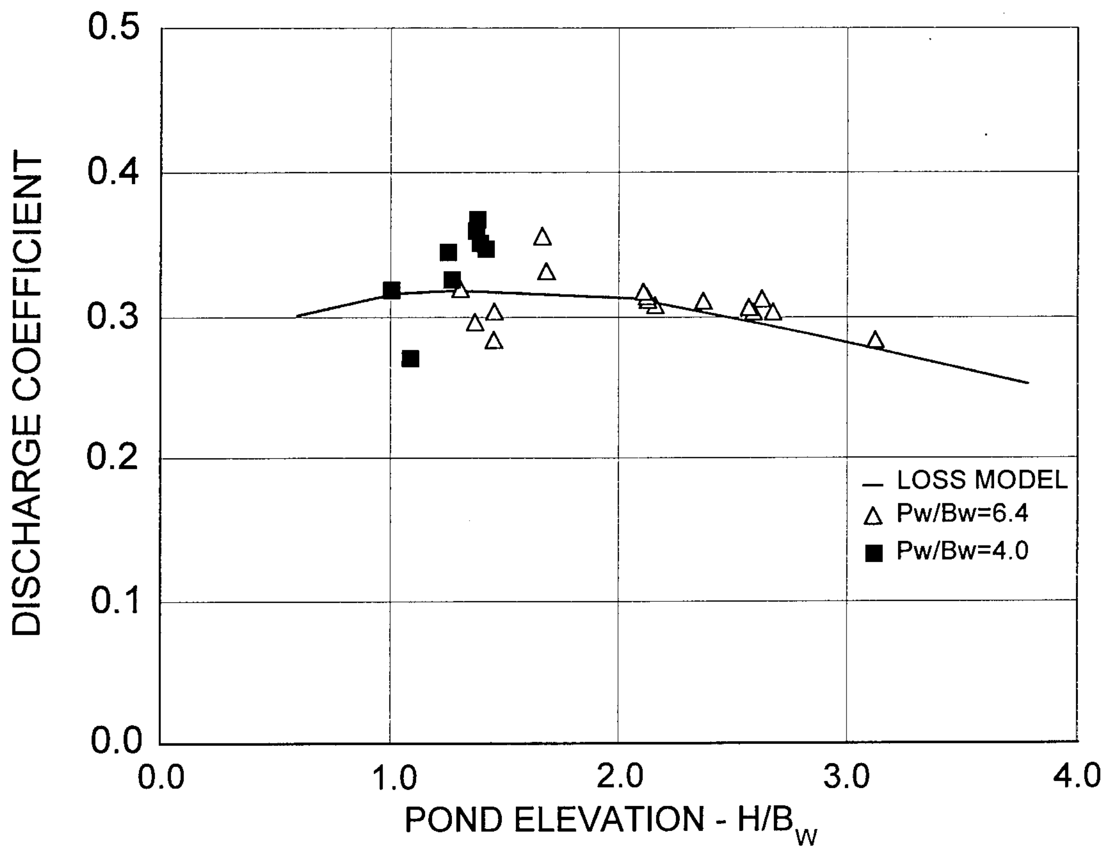

Figure 14: Comparison of data obtained at two crest heights for weirs A and B (loss model for A added).

Finally,a limited study of the effect of submergence was made. Discharge under several submergence conditions was measured using both the square edged weir C and the rounded edge weir A, for various submergences (Equation 3). For comparison the submerged flow rate has been calculated using the Villamonte relationship (Equation 4), and the appropriate empirical correlation for the discharge coefficient from

Table 4. The measured value for submerged discharge was cross plotted with the predicted value and compared to a 45° line (perfect agreement) as shown in Figure 15. Since reasonable agreement was obtained, use of the Villamonte relationship can be recommended at present.

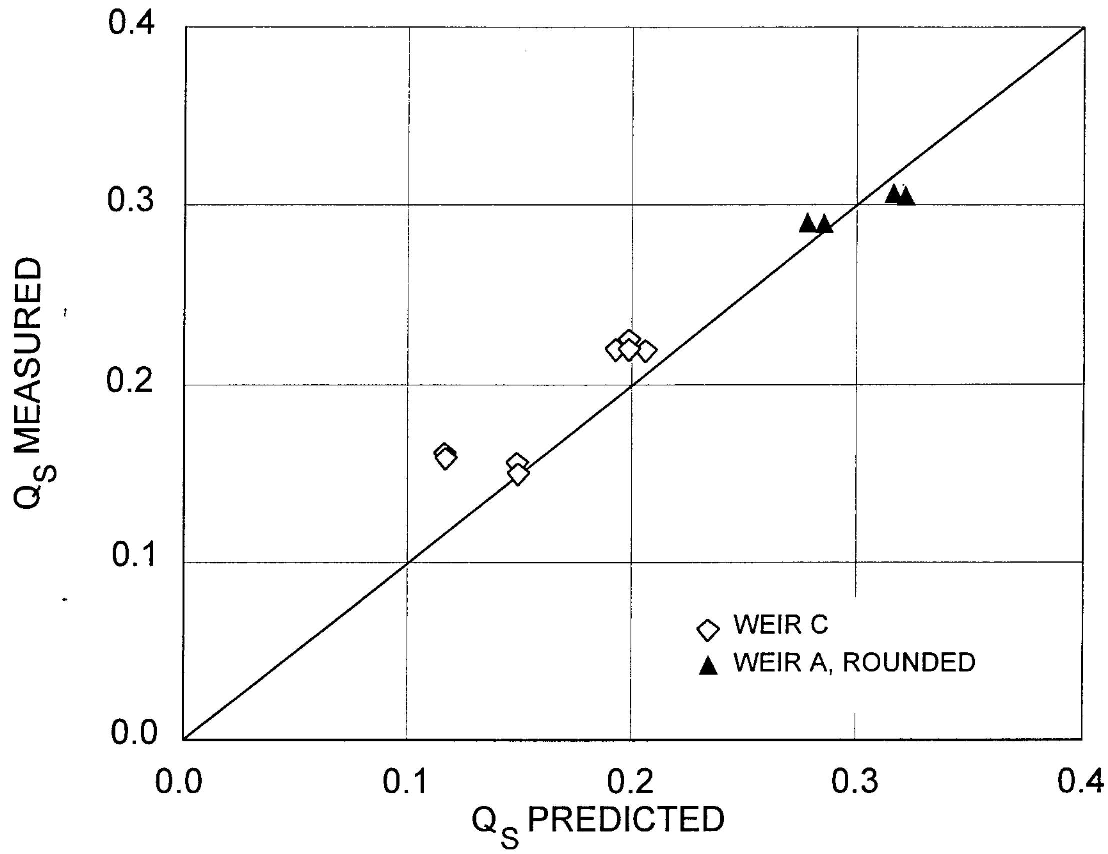

Figure 15: Comparison of submerged discharge as measured vs. that predicted for various submergence conditions.

PERFORMANCE OF A FULL SIZE ATTENUATOR

As an example of the use of the data obtained in the experiments reported here, the hydraulic capacity of a full size structure has been predicted. The proposed dimensions are as follows.

Table 5: Full size prototype dimensions for sample calculation.

Crest height	1.67 ft
Channel width	3.44 ft
Lower lip of skimmer	0.50 ft below weir crest
Weir width	1.67 ft
Weir crest breadth	0.50 ft
Skimmer thickness	0.82 ft
Top of weir plate	1.60 ft

Using the empirical fit to model data, a head-discharge diagram has been developed as seen in Figure 16. To this graph have been added two curves representing reduced discharge at constant submergence (generated from the Villamonte relationship) and a prediction based on the loss model outlined above. Finally, an alternative case was generated by assuming that the weir plate did not extend to the full height of the structure, allowing for some overtopping inside the endsection for emergency discharge. In this case the weir was treated as a compound weir with the upper section extended to the walls of the channel. The computation is divided into two parts in the upper region, with the narrower weir computation continued and the remaining area treated as a full width weir. It is emphasized that this calculation is at best an estimate, since many assumptions are required.

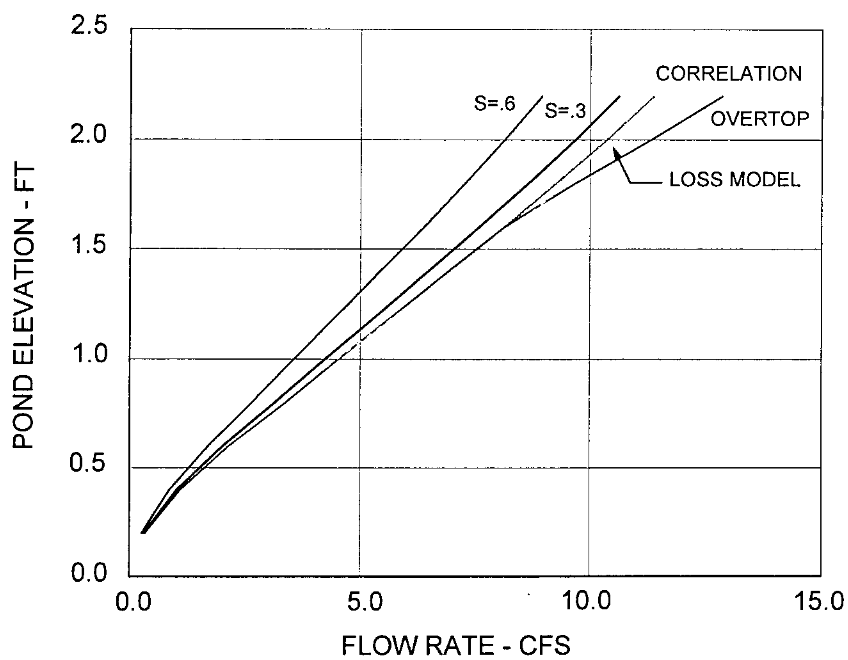

Figure 16: Predicted hydraulic performance of attenuator; discharge vs. pond head above weir crest.

CONCLUSIONS AND RECOMMENDATIONS

The principal conclusions and recommendations resulting from the research reported here are as follows:

1. Experiments have been conducted for a one-quarter scale model of a proposed stormwater attenuator and the results have been reduced to a discharge coefficient utilizing the conventional weir equation. Width of the weir aperture is an important parameter affecting the discharge coefficient. Overall the performance of the attenuator was acceptable.

2. It was found that the skimmer contributes substantially to a reduction in capacity of the attenuator from that which might be expected from a simple weir arrangement. This fact should not limit application, however, if suitable discharge relationships are employed. It does not appear that the thickness or the condition of the leading edge of the skimmer exert substantial influence on capacity.

3. The experimental measurement of discharge as a function of head was compared to the predictions of two analytical model developed during this investigation. It was found that modeling based on a significant loss developed downstream of the skimmer gave realistic predictions for the discharge coefficient.

4. As observed in previous investigations, the rounding of the leading edge of the weir contributes to a substantial improvement in discharge and should be accounted for in design.

5. The use of suppressed (full width weirs) is not recommended due to potential reproducibility problems.

6. Under normal conditions it is most likely that the control of discharge occurs at the weir as long as the discharge is free. If the tailwater elevation rises above the crest of the weir, discharge will be diminished. Design of the attenuator to operate in a submerged mode is not anticipated but for estimation of capacity, the Villamonte relationship is recommended. To calculate submergence, the correct upstream elevation is at the weir rather than the pond elevation.

7. Trash accumulation was only considered in a limited manner during this investigation. While low velocities at the face of the grate limit the amount of trash accumulated at this point, it is still possible for heavy objects (metal signs, etc) to lodge at the entrance and limit flow. Routine inspection and cleaning will be necessary.

8. Design capacities for an example of a full size attenuator as currently proposed have been calculated using the results of this investigation. Predictions can be made using either the empirical values for the discharge coefficient or the loss model.

Based on the experimental observations, a simplified picture of the flow path in the attenuator is as follows. All along the channel, the kinetic energy of the flow is small enough to be neglected except under the skimmer. A portion of the energy developed at this point is lost, reducing the head at the weir. A residual flow jet from under the skimmer rises along the weir plate and may interfere with the flow through the opening. Contraction through the weir is much like that observed in the static case [3], in that some reattachment may be observed and the condition of the leading edge is important.

Any skimmer configuration plays an important role in determining the discharge coefficient of the attenuator. A simple loss model is reasonably successful in explaining this fact. It is emphasized that nothing in these observations limits the application of the attenuator since as long as an experimental rating is available, an attenuator could be designed. It does appear however that some configurations may be less desirable. Locating the weir crest lower than that currently tested (0.417 ft for the model, 1.67 ft for the full size prototype) would mean that the skimmer would also be lowered and a higher velocity for the submerged jet would result. A reduced skimmer clearance may also increase debris accumulation. Locating the weir crest higher would reduce the control range. Wide weir openings are undesirable if higher velocities develop at the entrance and grate. Not only will losses increase at this location, but trash capture on the grate is likely to increase. While no specific recommendations regarding maximum capacity will be made here, it would be advisable to examine alternatives such as tandem installations when high capacity is required.

Caution must be exerted in the installation of bleed orifices in conjunction with a weir. To meet regulatory concerns, it may be necessary to cover the orifice by the skimmer or to install an elbow type cover on the orifice. Covering the orifice by a skimmer may require a very small clearance with the channel bottom and increased losses. Compound weir designs would require the same consideration.

Overtopping of the weir can occur if the weir plate does not extend to top of the attenuator structure. This type of compounding can be utilized to provide some emergency capacity. In this case, the results for the full width weir can be used to calculate the discharge. As conceived, the attenuator includes an open grating on the top and is set into a berm, possibly with some local depression rather than flush. Thus if the entire structure is overtopped, discharge from the pond will be limited by the width of the depression along the top of the berm. Standard correlations may be used to estimate the flow over the crest, which must be added to the flow through the attenuator.

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