Notch Weirs for Use in Stormwater Detention Basin Control Structures

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By

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FOREWORD

The impetus for this research was a contact from an employee, Michael McCarthy of Kitsap County, Washington whose responsibilities include checking plans for stormwater detention basins. Outlet structures for some of these facilities include what is referred to as notch weirs, which are simply rectangular slots cut in a section of pipe used as a riser through which outflow from the basin exits the pond. Various design references provide equations that can be used to design the size of the notch weir to provide a design outflow at a given pond elevation. It had been observed that the equations yield physically unrealistic results in some situations. The original contact was for guidance on an alternative approach that could be implemented in the design process. In response to this request, two different efforts were initiated. The first was a review of existing data that may be relevant to this application. Specific recommendations were generated as a result of this data review; a report was prepared and is included as Appendix 2 to this report. However, this review also identified several unanswered questions. A research project sponsored by the University of Michigan under the Undergraduate Research Opportunities Program was initiated to investigate answers to these questions. Braden Baldwin conducted the research included in this report for his project under this program.

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INTRODUCTION

Stormwater detention facilities typically require flow control structures to limit offsite discharges for prescribed hydrological events. A typical design utilizes flow constrictors in a riser connected to the outlet pipe as indicated in Figure 1. The riser and outlet pipe are typically the same diameter and a combination of orifices or weirs installed in the riser are used to control the flow. An orifice and elbow assembly mounted near the bottom of the riser is generally used to regulate the lower regulatory discharge (e.g. a 2 year, 24 hour storm event). Higher regulatory discharges such as the 10 and/or 100-year events may require a second and third orifice/elbow assembly at higher elevations. In the case that there is insufficient vertical extent to install these in the riser, a notch weir is commonly used to provide for the discharge control at the higher flow rates. The notch weir is simply a slot cut in the perimeter of the riser pipe intended to operate in the weir mode with a free surface at the top; the horizontal width of the slot is designed to pass the required flow. The King and Kitsap County (State of Washington) design manuals specify that the slot length cannot exceed more than one-half of the pipe circumference. In addition, an oil-water separator baffle is required. This baffle is simply a section of larger diameter pipe bolted to the riser on the side opposite the weir and extending at least one foot below the weir crest. The only requirement in the design manuals is that the area between the riser and the baffle be sufficient to pass the design flow; it is understood that a common configuration may involve a twelve-inch diameter riser and an eighteen-inch diameter baffle, for example.

Standard weir equations are recommended for use in determining the relationship between the weir discharge and the water surface elevation in the detention pond. These equations were developed for weirs installed in flat plates installed perpendicular to an approach flow. The actual riser/weir configuration deviates from this in several fundamental aspects, calling into question the validity of the discharge equation. Often, the designed notch weir crest length (slot width) is fairly small and less than the weir head (detention pond water surface elevation relative to the weir crest). Standard weir discharge equations call for a correction in the crest length that is intended to account for the flow contraction when the weir projects into the approach flow. However, this correction was developed from experimental data in which the weir head was considerably less than the crest width. In extreme situations of high weir heads, the discharge equation will result in the prediction of a negative discharge, a situation that is physically unrealistic.

The literature was searched for experimental data in which the weir head to crest length ratio was greater than one in order to test the ability of the standard weir equations to reproduce the data. One such data set was located: *A Comprehensive Discharge Equation for Rectangular-Notch Weirs, R.W. Carter, M.S. Thesis, Georgia Institute of Technology, June 1956.* Analysis of the data in this thesis was performed and the results of the analysis are included as Appendix 2 to this report. However, the data in the thesis was not adequate to answer all questions and an experimental investigation was conducted to obtain additional information. Tests were performed on various sizes of notches cut in the sides of PVC pipe. Two different pipe diameters were tested in order to assess the role of pipe diameter on the head discharge relation. In addition, tests were performed both with and without baffles surrounding the notch weir in order to provide guidance as to the required baffle size in order to not materially impact the weir discharge.

BACKGROUND

Figure 2 is a sketch of a standard sharp-crested weir. In this sketch, H is the weir head, i.e. the elevation difference between the weir crest and the upstream water surface elevation. Other geometrical variables include L, the crest width (normally referred to as the crest length) and P, the weir height, which is the elevation difference between the weir crest and the upstream floor level. The weir discharge Q is generally assumed to be related to these geometric variables. The weir equation prescribed in the King and Kitsap County design manuals (see Figure 4.4.7 on page 4.4.7-2 of the King County Surface Water Design Manual or Figure 5-22 on page 5-44 of the Kitsap County manual) is

$$Q = (3.27 + 0.4\ H/P)(L - 0.2\ H)\ H^{3/2} \qquad (1)$$

This equation is valid when all of the geometric dimensions are given in ft and for Q in ft^3/s. The H/P term in the first set of parentheses is to account for the approach flow conditions and the exact form varies among various recommended weir equations. The second term involves a correction of 0.1H for each side of the weir, which is supposed to account for the contraction of the flow at the side of the weir. A suppressed weir is one in which the approach channel and the weir have the same width L and the 0.2H correction would not be applied in this situation. This term for the end contraction was developed in 1883 by Francis *(Lowell Hydraulic Experiments, D. van Nostrand, New York, 1883)* and has been stated to be restricted to L > 3H and there are additional restrictions on the width of the approach channel, B, to the notch width L.

Inspection of the notch width correction term indicates that the effective width L – 0.2H is zero if H/L = 5. The predicted discharge would be zero in this case and the discharge is predicted to be negative for H/L > 5. Neither of these results is physically realistic and is a consequence of applying the correction outside the range of the experimental data from which it was derived (presumably H/L < 0.33 from the above mentioned restriction on applicability). The computer software, *Waterworks*, utilizes a similar length correction but restricts all discharges to be zero or positive by specifying that whenever a discharge Q is predicted to be negative, it is set to zero. However, Equation (1) can be shown to predict a maximum discharge at H/L = 3 if the H/P term is neglected and therefore that the discharge will decrease with increasing head above H/L = 3. This is also not a physically realistic result and indicates a more severe restriction on the applicability of the weir equation than simply that it predicts a non-negative discharge. Since H/L = 3 is considerably above the range for recommended applicability of the equation, there may well be even more restrictive limitations to the use of this length correction in the weir equation.

A literature search was conducted to determine the availability of data sets on weir flows with large H/L ratios. One source was discovered: *A Comprehensive Discharge Equation for Rectangular-Notch Weirs, R.W. Carter, M.S. Thesis, Georgia Institute of Technology, June 1956* and a copy of the thesis was obtained from the Georgia Tech Library system. A total of 346 individual sets of head and discharge measurements were presented for both suppressed (B/L = 1) and contracted weirs (B/L > 1). A wide variety of H/L ratios were investigated, but in particular, a number of individual experiments were performed with L = 0.1, 0.2 and 0.4 ft with weir heads H that yield H/L values up to a maximum value of 7.1 and include data for both suppressed and contracted weirs. Although the largest H/L values are for L=0.1 ft which may be out of the range for detention pond applications, the other values are apparently within a typical range for many applications. In addition to the three L values mentioned above, there are more limited data for L values of 0.118, 0.121, 0.281, 0.292, 0.577 and 0.60 ft. These data were analyzed to determine the degree of correspondence with Equation 1. This equation was found to grossly under-estimate the discharge under the conditions of high weir head to notch width. At lower heads, the tendency for the discharge equations recommended in the King County and Kitsap County (State of Washington) Surface Water Design Manuals was to slightly over-estimate the discharge. Removing the weir crest length contraction effect from Equation 1 resulted in more consistent discharge predictions that were generally in the range of five to twenty five percent greater than observed in the experimental data. See Appendix 2 for a more detailed discussion of this analysis.

One issue with this data set is that most of the experiments were performed for fairly small P values. Since the original experiments were performed with the weir plates set across the channel, it is not clear whether the effect of the weir height P upstream or downstream of the weir controls the flow behavior. In the Georgia Tech experiments, the dimension P is the same both upstream and downstream of the weir plate. In an outlet structure where the flow drops down the riser to a horizontal outflow pipe and with a very large approach flow area, the effect of P should be small in any case. It is not clear that the data analyzed with relatively large values of H/P can be directly applied to the current application. New experimental data was collected in order to address this issue as well as the fact that the notch weirs in a riser pipe are actually on a curved surface as opposed to a flat plate. In addition, the effect of a baffle structure on the weir discharge equation was investigated.

EXPERIMENTAL SETUP

An experimental configuration was developed that more closely mimics the conditions of a notch weir in a vertical riser pipe. Figure 3 is a schematic of the setup. The weir itself was simply a uniform width notch cut in a section of PVC pipe (with the exception of one experiment that was conducted for a weir in a flat plate). The pipe was mounted over a hole cut in the center of the bottom of a plywood box 2 ft high by 4 ft square. Flow was introduced into the box outside of the pipe from constant head source and allowed to pass through the weir and vertically down through the outlet into a weigh tank used to determine the discharge. The weir head was measured with a point gauge mounted in a corner of the plywood box. In order to minimize waves on the water surface where the point gauge was located, a transparent acrylic pipe open at the bottom was mounted on the box wall and the point gauge operated within it. Experiments consisted of measuring the discharge with the weigh tank and the water surface elevation in the box outside the pipe. The flow was regulated by means of a control valve to typically provide eight to ten different flow rates for each weir configuration.

The size of the box was chosen to be sufficiently large to eliminate high approach velocities in the box. The inflow into the box was through two different relatively small diameter inlet pipes and these created sufficiently high velocities so that the flow over the weir was affected at high discharges. This problem was minimized by installing the inflow pipes on the size of the box opposite to the direction the weir was facing. In addition, the inflow was deflected to break up the inflow jet and a set of fiber baffles further straightened the flow before passing into the portion of the box where the weir was located. Even with all this care, there was probably some approach velocity effect on the results at the highest discharges but these are felt to be minimal in most cases.

The point gauges read to a basic precision of 0.001 ft. The uncertainty in the measurements is probably somewhat greater than that. In order to determine the weir head, the water level in the box needs to be determined relative to the elevation of the weir crest. In order to accomplish this, two point gauges were installed in the box, one directly above the weir crest. The outlet to the box was blocked and the box was filled with stagnant water to an arbitrary level above the weir crest. Readings from both point gauges were obtained and additional reading was obtained by lowering the one point gauges until it contacted the weir crest. Repetitions indicated that the weir crest elevation could be established to within 0.001-0.002 ft. Therefore weir heads are considered to be accurate to within about 0.0025 ft.

As mentioned, flow rates were determined with the weigh tank by measuring the time required to accumulate a known weight of water in the tank. This determination is considered to be accurate to within 1-2 percent except possibly for the very highest discharges in which the accuracy may be less due to the limited volume of the weigh tank.

The basic variables in the testing were the crest width, L, and the pipe diameter D. Experiments were performed for two different pipes, eight- and twelve-inch Schedule 40 PVC pipe. The inside diameters of these pipe are the nominal diameter while the wall thickness for the eight-inch pipe is 3/8 inches and for the twelve-inch pipe is 7/16 inches. The notches were cut perpendicular to the pipe circumference. The twelve-inch pipe had four different notch widths with nominal dimensions of 1.5, 3, 6 and 9 inches while the notch widths for the eight-inch pipe had nominal dimensions of 2, 4, and 6 inches. The actual notch width for each case was determined by measuring the linear (straight-line) width from the outside circumference of one side of the notch to the other side. Trigonometric relations were used to convert this dimension into a circumferential length; actual values for both lengths are recorded in Table 1. Width variations along the height of the weir typically were less than 0.005 inches. The bottom of the weir was located approximately 1.2 ft above the bottom of the box. The outflow opening in the box was sufficient to pass all flows studied without backing the flow up into the pipe. Figures 4 and 5 provide images of a typical weir setup. The flow was visually observed to contract off the outside edge of the pipe wall and to not touch the inside of the weir notch. One single additional set of experiments was performed with a six-inch weir cut in a piece of 1/4-inch acrylic sheet. This sheet was affixed to the side of a twelve-inch square box such that the configuration resembled the round pipe installations; Figure 6 is an image of that setup.

Table 1. Lengths of weir crests using various definitions.

Pipe Diameter	Nominal Length	Straight Line Length	Circumferential Length
12	1.5	variable (appr. 1.483, actual measured, used in analysis)	
12	3	2.963	2.990
12	6	5.585	5.778
12	9	7.795	8.387
8	2	1.990	2.008
8	4	3.860	3.998
8	6	5.550	6.012
Flat Weir	6	6.03	6.030

Note: All dimensions in inches

For certain experiments, the influence of a baffle on the flow over the weir was investigated by varying the diameter of a baffle structure placed around the pipe. Figure 7 is an image of the baffle installation. In order to allow for a variable baffle diameter, the baffle was created from a length of expanded metal that was rolled into a cylinder of the desired diameter. This cylinder was then covered with polyethylene sheeting to make it watertight and bolted to the side of the PVC pipe opposite the weir. The bottom of the baffle extended 0.5 ft below the weir crest, leaving approximately 0.7 ft of open flow area beneath it for the inflow to approach the weir crest. In some cases, the outside of the baffle approached to within a few inches of the side of the box. It was also visually observed that in some high discharge configurations and small baffle diameters, a persistent coherent vortex formed between the baffle wall and the weir opening. There did not seem to be a strong effect on the flow behavior associated with this occurrence. Baffle diameters were generally varied in two-inch increments and a complete set of data was compared to the no-baffle configuration in order to assess the impact of the baffle on the weir discharge.

A summary of all experimental data collected in this study is included in Appendix 1.

RESULTS

For reasons of convention, results are presented in terms of a weir coefficient C defined by

$$C = \frac{Q}{LH^{3/2}} \qquad (2)$$

Note that C is not dimensionless in this representation and values therefore depend on the system of units employed. The values presented in this report utilize the U.S. Customary system of units because of the convention in engineering practice in the U.S. In order to convert to S.I. or other units, one only need recognize that C has units of the square root of gravity. To convert to S.I. units for example, the C values reported herein must be multiplied by $0.3048^{1/2}$ where 0.3048 represents the conversion from feet to meters.

With the notch weir in the side of a circular pipe, there is a choice on how to represent L. It could be given by the circumferential distance from one side of the weir to the other or it could equally well be given by the straight-line distance. Logically, the first definition may be more valid at very low weir heads in which the flow passing over the pipe circumference determines the nature of the flow. At high weir heads, the straight-line distance is perhaps more appropriate as it defines the area that the approach flow "sees" as it approaches the weir. The King and Kitsap County Surface Water Design Manuals suggest that the circumferential length is the appropriate one to the used in the discharge equation. In the data analysis, both definitions of L were employed and the second definition proved to be more capable of collapsing the data in a non-dimensional sense; i.e. to provide more similar values of C for each weir width. The data for the 12-inch pipe and the various weir widths with each definition of L are presented in Figures 8 and 9. When the weir width is very short such as 1.5 inches, the differences between the two definitions of L are minimal but become more significant when the weir width is a larger proportion of the pipe circumference. From these two figures, it appears that the use of the straight-line distance provides a more consistent presentation of experimental results. Therefore, this definition of L is utilized in all further results presented herein.

Weirs Without Baffles

The majority of the experiments were performed without the baffle installed since it is intended that the baffle not affect the head-discharge relation for the weir. A typical result for a weir width of 1.5 inches is given in Figure 10. Also included on this figure is data from Carter at a similar weir width (1.42 inches). The two data sets are reasonably consistent in terms of the general magnitude of the weir coefficient. All sets of experiments performed in the current study clearly demonstrated the trend that the weir coefficient decreases with increasing head. This may be due to the issue discussed above where the crest length at low heads may be more appropriately correlated with the circumferential length in the sense that a longer length in Equation (2) would reduce the C value; hence using the straight line distance for L yields too large a weir coefficient. This effect would not occur for a weir in a flat plate and is consistent with the observations for the one experiment performed with a flat plate (and discussed below). Carter's Georgia Tech data were inconsistent in the trends of C vs. H for the various weir configurations, in some cases decreasing and in others, increasing with H. This may be due to other effects not investigated in the current study such as small values of P.

Also included in Figure 10 is the prediction of Equation (1) in the limit with H/P = 0; this is considered to be the most appropriate description for the experimental configuration studied. It can clearly be seen that Equation (1) does not capture the trend in the data, supporting the previous conclusion (see Appendix 2) that it is an inappropriate equation for this particular application with relatively large H/L values.

The results for the four different crest widths in the twelve-inch PVC pipe are presented in Figure 8. The concept implied by Equation (1) is that an "effective crest length" is computed by reducing the length of the crest by a function that depends on the weir head. The underlying philosophy is cast into doubt by the results in Figure 11 since the weir coefficient should then decrease with crest length since end effects would be greater for smaller crest lengths and the opposite trend is actually indicated. It should be noted that the Georgia Tech data also supports this conclusion for weirs in flat plates although a bit more ambiguously. In the *Handbook of Hydraulics, 7th edition, (E.F. Brater, H.W. King, J.E. Lindell and C.Y. Wei, McGraw Hill)* page 5.11, a correction to the weir coefficient to account for end contractions is discussed based in part on the Georgia Tech data. Figure 5.3 on page 5.14 indicates a positive correction to be made to the actual length over nearly the entire range of flow conditions, consistent with the results presented in Figure 8. Such results call into question the structure of Equation (1), even though it currently is the most commonly used equation for contracted weirs.The experiments with the 8-inch pipe with crest lengths of two, four and six inches were intended to correspond to geometrically similar conditions for the 12-inch pipe (i.e. the 2-inch crest length corresponding to the 3-inch, the 4-inch to the 6, etc.) However if the results for the two different pipes are plotted non-dimensionally in terms of H/D where D is the pipe diameter as in Figure 11, there is no correspondence between the results for the two pipes for equal values of H/D. If, however, the results are plotted simply as a function of head H and crest length L as in Figure 12, then the two sets of results appear to be consistent with each other. This finding led to the additional experiment with the 6-inch weir width on a flat plate as described earlier. Figures 13 and 14 show the results for all three six-inch weirs with the two different definitions of L and again support the conclusions based on the data in Figures 8 and 9. The fact that the data in Figure 13 are reasonably consistent with each other implies that the curvature of the pipe that the weir is installed on is basically irrelevant so long as the crest length is interpreted as the linear distances between the two sides of the weirs. Additional experiments may be in order to verify this conclusion over a wider range of geometries.

Attempts were made to collapse the data in some sort of dimensionless presentation without success. Instead, following the development by Kindsvater and Carter as described in Equation 5-35 of the 7th edition of the Handbook of Hydraulics, the only successful means of collapsing the data was by adding a fixed correction to the length as

$$L_{eff} = L + k_L \tag{4}$$

Kindsvater and Carter recommend k_L values on the order of 0.008 ft for their studies while a fixed value of 0.0038 ft appears to be adequate for collapsing the data for the present study. The value of k_L can, however, be doubled without a significant change in the standard deviation of the fitted equation (Equation 6 below) and the exact value is subject to some uncertainty. Figure 15 shows the result of this correction to the crest length and the ability to present experimental results for several crest lengths as a unique function of the weir head. The differences between the current study and that by Carter (from which the Kindsvater and Carter's correction was derived) appears to be quite substantial in regard to this correction so there is likely to be a difference mechanism associated with the correction in the two studies. It should be noted that many of Carter's experiments were conducted with fairly small P values (H/P less than one) and this may well have an important and significant difference on the flow downstream of the weir crest.

Kindsvater and Carter also recommend using a weir equation in which the head is corrected in a fashion similar to Equation (4):

$$H_{eff} = H + k_H \qquad (5)$$

After attempts to nondimensionalize the results were not successful, this approach was ultimately selected for the current study as well. Again, a much larger value of k_H, on the order of 0.018 ft, was required to collapse the data as indicated in Figure 16 than the value of 0.003 ft recommended by Kindsvater and Carter. A portion of the difference between the two studies is believed to be due to the curved nature of the notch weir constructed in the side of a pipe, but the limited data with the flat weir do not indicate that this is the case. It also appears that the conditions downstream from the weir are likely to play a significant role on the pressure distribution at the weir crest and therefore to affect the values of the weir coefficients. A future study to investigate this issue is planned. In any case, a flow equation that is adequate to predict the entire range of experimental data to within an accuracy of about three percent (to cover all the experimental data; the standard deviation is about 1.7 %) is:

$$Q = 3.06\,(L + 0.0038)\,(H + .018)^{3/2} \qquad (6)$$

This equation is dependent on the system of units employed and should be applied for Q in ft^3/s and L and H in feet. In the SI system of units, a similar equation would be

$$Q = 1.69\,(L + 0.0012)\,(H + 0.0055)^{3/2} \qquad (7)$$

Weirs with Baffles

A limited number of experiments were performed with baffles of varying diameters surrounding the 12-inch diameter pipe. The baffles were constructed as described previously and were varied in 2-inch increments until a noticeable reduction in the discharge relative to the unbaffled weir was observed. The procedure involved varying the baffle diameter and observing the influence on the computed weir coefficient. An example is presented in Figure 17 for the 3-inch weir. It can be seen that as the baffle diameter decreases, the weir coefficient decreases further below the no baffle condition, indicating that the presence of the baffle restricts the approach flow in such a way as to reduce the discharge at a given weir head. It is not possible in general to specify a unique diameter for which the baffle has a given influence in discharge reduction since the effect is related in large part to the flow rate; a baffle of a given diameter will have more impact at higher weir heads. Considering a somewhat arbitrary definition that the discharge be reduced by no more than about five percent or more at heads of 0.5 ft or less, Table 2 identifies the approximate baffle diameter that would satisfy this constraint. Because of the relatively coarse adjustment on baffle diameters (two inches), the results are not distinct, for example the data for the six inch weir indicate that the 20 inch diameter baffle comes close to meeting the specified criterion for no flow impact. No similar testing was performed for the 8-inch diameter pipe, but it is likely that the minimum baffle diameters would be only slightly smaller. One method to estimate the required baffle diameter would be to compute the annular area between the baffle and the pipe (since this dictates the flow area approaching the weir crest as the flow passes beneath the baffle and up to the crest) and keep it the same for a given weir length. Although this is a plausible approach, it should be noted that testing to verify the concept was not conducted. However, given the limited number of standard diameters of pipe in this range, it should be possible to use the results in Table 2 to estimate the baffle diameter required for an application to a pipe risers in the range of 8-16 inches.

Table 2. Required baffle diameter to yield less than five percent reduction in discharge at a head of 0.5 ft.

Weir Crest Length (inches)	Baffle Diameter (inches)
3	20
6	22
9	22

CONCLUSIONS

Testing was performed for several different configurations with notch weirs cut in the side of a standard PVC pipe. The general configuration is with the pipe oriented vertically and the notch cut along the pipe length as would be the case with an application which uses the pipe as a riser in the outlet control structure in a stormwater detention pond. Measurements were made of the discharge through the weir as a function of the reservoir head upstream of the weir crest. Experiments were performed both with and without a baffle surrounding the riser. The purpose of the baffle is to limit the discharge of floating trash and oil through the outlet weir but an important objective is that it not limits the discharge through the weir. The following conclusions are drawn from an analysis of the experimental results:

1.) A rational analysis of Equation (1) indicates that it provides illogical predictions at high values of H/L. Although these are beyond the range for which the equation was originally developed, the correction associated with H/L is small at low H/L values calling into question the particular structure of the equation at any value of H/L. Since Equation (1) is recommended for use by the King County drainage manual as well as many other sources without any stated restrictions on its usage, the possibility for misapplication in many situations is considerable.
2.) The length of the weir crest for notch weirs installed in a circular pipe wall should be defined as the straight-line distance between the outside edges of the weir. Using this definition instead of the circumferential length provides good correspondence between results obtained from different pipe diameters

and also for similar weirs installed in a flat plate. For small weir heads, the use of this length instead of the circumferential length may raise the value of the weir coefficient relative to the large head results. However, this effect was accounted for in the recommended weir equation by a correction to the weir head. The more appropriate definition of the weir crest length is different that that suggested in the King and Kitsap County Surface Water Design manuals and this difference needs to be recognized in applying the recommended equations.

3.) Analysis of the experimental results obtained in this study also indicates that the formulation in Equation (1) is inappropriate. Whereas the correction based on H/L in Equation (1) serves to reduce the effective length of the weir crest, data collected for very narrow weirs indicate higher weir coefficients than wider weir lengths. In order to reduce the data to a form given by a standard weir formula, it is necessary to add a fixed length to the weir crest in order to account for an effective crest length. This correction appears to be independent of the weir head although it may also be possible to reduce the data in such a way that the weir coefficient depends directly on the value of H/L.

4.) Given the above two equations, it is not recommended to use Equation (1) to design notch weirs or any sharp-crested weir with a significant H/L value. Since the correction to the crest length based on H/L is negligible at small H/L values, there is no justification for using a weir equation with the length correction suggested by Equation (1).

5.) Weir coefficients for notch weirs were found to decrease with increasing weir head, again in a manner inconsistent with Equation (1). Therefore, the entire structure of Equation (1) is not suitable for describing the flow behavior of notch weirs. Alternatives were sought to replace this equation. Although non-dimensional representations of the data were investigated, it was not possible to find one that was rationally based that captured the trends in the data. Following the approach by Kindsvater and Carter, an equation was developed that will be sufficiently accurate for the purpose of designing notch weirs as outlet control structures for a stormwater detention basin:

$$Q = 3.06 (L + 0.0038) (H + .018)^{3/2}$$

for English units or

$$Q = 1.69 (L + 0.014) (H + 0.0055)^{3/2}$$

for SI units.

These equations describe all experimental results to within about three percent.

6.) An investigation was conducted to determine the required diameters for a baffle that would surround the riser pipe and prevent floating trash and oil from passing over the weir. The objective was to determine the minimum diameter that only reduced the weir discharge minimally. For weir crest lengths on the order of three to nine inches, the baffle diameter is required to be in the range of 20 to 22 inches with the larger diameter required for the longer crest length.

APPENDIX 1

EXPERIMENTAL DATA

APPENDIX 2

REVIEW OF EXISTING DATA

AIIM SCANNER TEST CHART # 2

Spectra

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Times Roman

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Century Schoolbook Bold

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

News Gothic Bold Reversed

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Bodoni Italic

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Greek and Math Symbols

4 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋫⋪⋗⋜≡
6 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋫⋪⋗⋜≡
8 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋫⋪⋗⋜≡
10 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋫⋪⋗⋜≡

White

Black

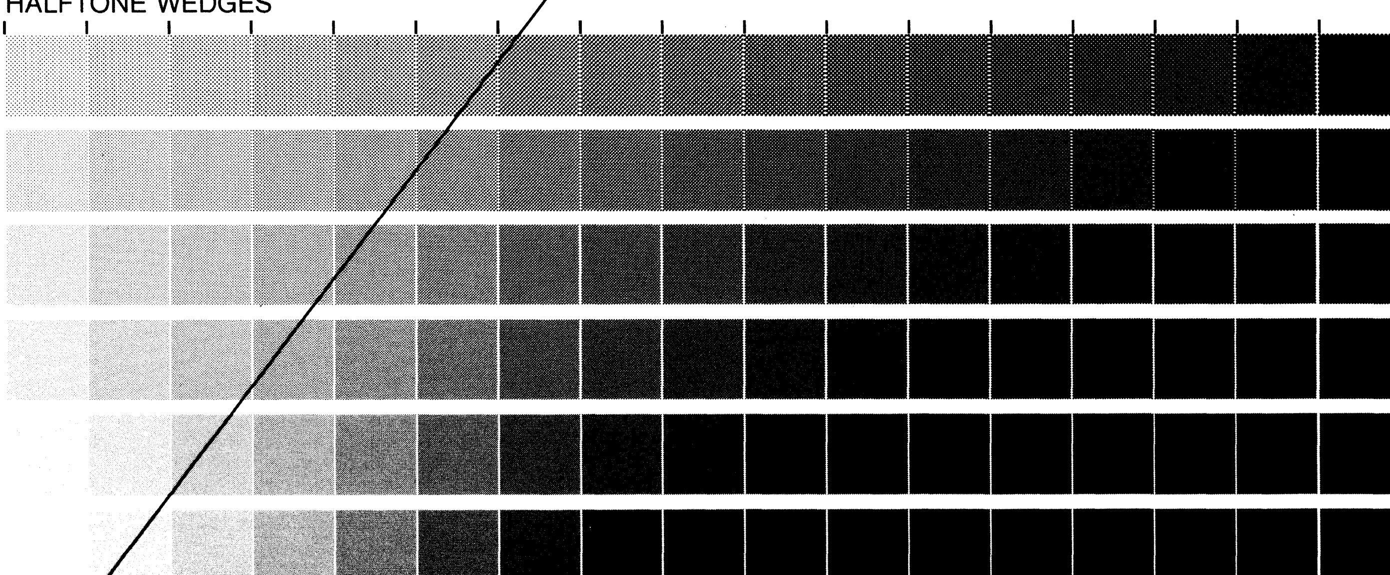
Isolated Characters

e	m	1	2	3	a
4	5	6	7	o	•
8	9	0	h	I	B

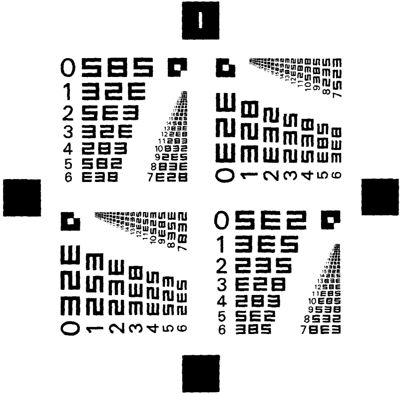
0123456 A4 Page 6543210

MESH HALFTONE WEDGES

65
85
100
110
133
150



RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-1-71



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