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CONCRETE BLOCK WEIR AND ORIFICE
DISCHARGE CHARACTERISTICS
for
STORMWATER DETENTION OUTLET CONTROL

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee, Knoxville

Anthony N. Wylie

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DEDICATION

The author wishes to dedicate this thesis to his wife, Christine F. Wylie for her understanding and encouragement during his graduate study.

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ABSTRACT

A series of twelve (12) full-scale laboratory tests was conducted to evaluate the performance and hydraulic characteristics of various orifice and weir configurations commonly used in single- and multiple-stage stormwater detention outlet control structures constructed of concrete block. The use of such concrete block structures in conjunction with detention storage appears to be growing because of apparent practical, economic, and esthetic advantages over other types of structures. However, lack of published orifice and weir discharge coefficient data for openings formed from concrete blocks leaves the hydraulic engineer with little choice but to design the detention storage and outlet control opening(s) using "guesstimate" coefficients. Inaccurate coefficients could result in either expensive over-designed or ineffective under-designed ponds.

Several combinations of weir and orifice opening geometries were tested and evaluated to account for pond bottom effect, "stacked-weir" arrangement, multiple orifices, and combined weir and orifice configurations.

This thesis presents the results of a series of twelve (12) tests, which relate discharge coefficient to orifice and weir configurations and heads. It also demonstrates that required (design) detention storage will vary significantly within a relatively small range of assumed outlet weir and orifice discharge coefficients. The sensitivity analysis between required detention storage and assumed weir/orifice discharge coefficients was performed using common HEC-1 and TR-55 routing procedures. Recommendations are presented for hydraulic design, construction, and maintenance of concrete block stormwater control structures.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

TABLE PAGE

SECTION I

INTRODUCTION

GENERAL

With the upsurge in urbanization occurring throughout the country the use of Stormwater Detention Facilities as a part of an overall flood control strategy is becoming increasingly important. Many municipalities have developed regulations that require newly developed areas to be designed such that, for specific storm events, downstream flooding will not be increased by the new development (17,19)*. One of the best ways minimize flooding is through the use of Stormwater Detention Facilities. Although detention is being required it is suspected that detention is often required by local governments without an adequate understanding of the underlaying principles or the ramifications of the effects of detention on a basin-wide or regional stormwater system (21). The growing use of Stormwater Detention Facilities and the proliferation of various design approaches used to configure the discharge structures led to the establishment of a Task Committee within The American Society of Civil Engineers (ASCE) to review and report on progress in the area of outlet control structures for Stormwater Detention Facilities (26). In their report published in 1985, the Task Committee has identified compound or stacked-weirs, costs and benefits of outlet control configurations, structural design of discharge structures, and weir/orifice interaction as areas the Committee feels need special attention in order to improve the state of the art in the area of Stormwater Detention Facilities discharge structure

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* Numbers in parentheses refer to similarly numbered items in the Bibliography.

design and construction. The study presented in this thesis will provide laboratory data and rationale to support the use of concrete block construction of outlet control structures for Stormwater Detention Facilities, and discuss the use of regular weirs, stacked-weirs, orifice, and combined weir/orifice type configurations for outlet control structures. The use of concrete block structures in conjunction with detention storage appears to be growing because of apparent practical, economic, and aesthetic advantages over other types of structures. Several reasons concrete block makes an excellent alternative material for the construction of discharge structures are as follows:

1. For many applications, concrete block discharge structures can have a cost advantage over the large sections of corrugated metal pipe most often used for constructing discharge structure risers. This cost advantage is more likely for small projects where the sophistication of the contractor is low.
2. Properly constructed concrete block structures can be more easily cleaned with standard construction equipment (shovels, buckets, etc.) thus reducing maintenance problems and cost.
3. Configuration flexibility is enhanced by using concrete block discharge structures since each structure is custom constructed to fit the particular site conditions, and if the need arises, concrete block discharge structures are easily modified in the field to meet changing watershed conditions.
4. Concrete block discharge structures are more aesthetically pleasing than those constructed with corrugated metal pipe risers,

and they blend easily into the urban neighborhood setting. The material used for the construction of concrete block discharge structures can be the same material used for the primary structure consequently increasing the overall aesthetic impact of the completed facility.

Figure 1 illustrates how a concrete block discharge structure could be configured. The components of a concrete block discharge structure are as follows:

Orifice Opening - This type of opening controls the discharge for one storm event, e.g., 10 year storm. In the configuration shown, an orifice is used to regulate the first-stage storm flow. A handy way to produce the orifice openings is to build in concrete blocks laid on their side. The required area can be achieved by providing the proper number of block openings around the perimeter of the discharge structures.

Regular Weir - This type of weir opening has a constant width throughout its height. Using regular weirs in conjunction with lower orifice opening(s) is a way to control the discharge for a second storm event, e.g., 25 year storm, thus producing a multiple-stage discharge structure. It is important that the block voids on all weir surface openings be filled with concrete and trawled to a smooth surface to ensure proper flow performance.

Stacked-Weir - Figure 1 shows a stacked-weir that could be used to control the discharge for two storm events. In cases when a stacked-weir is used for multiple-stage stormwater control, the orifice opening may be omitted. State-of-the-art practice currently favors control of two (rather than three or more) storm events (26).

Emergency Spillway - The emergency spillway protects the integrity of the earth dam that is most often used to produce a detention pond in the event of

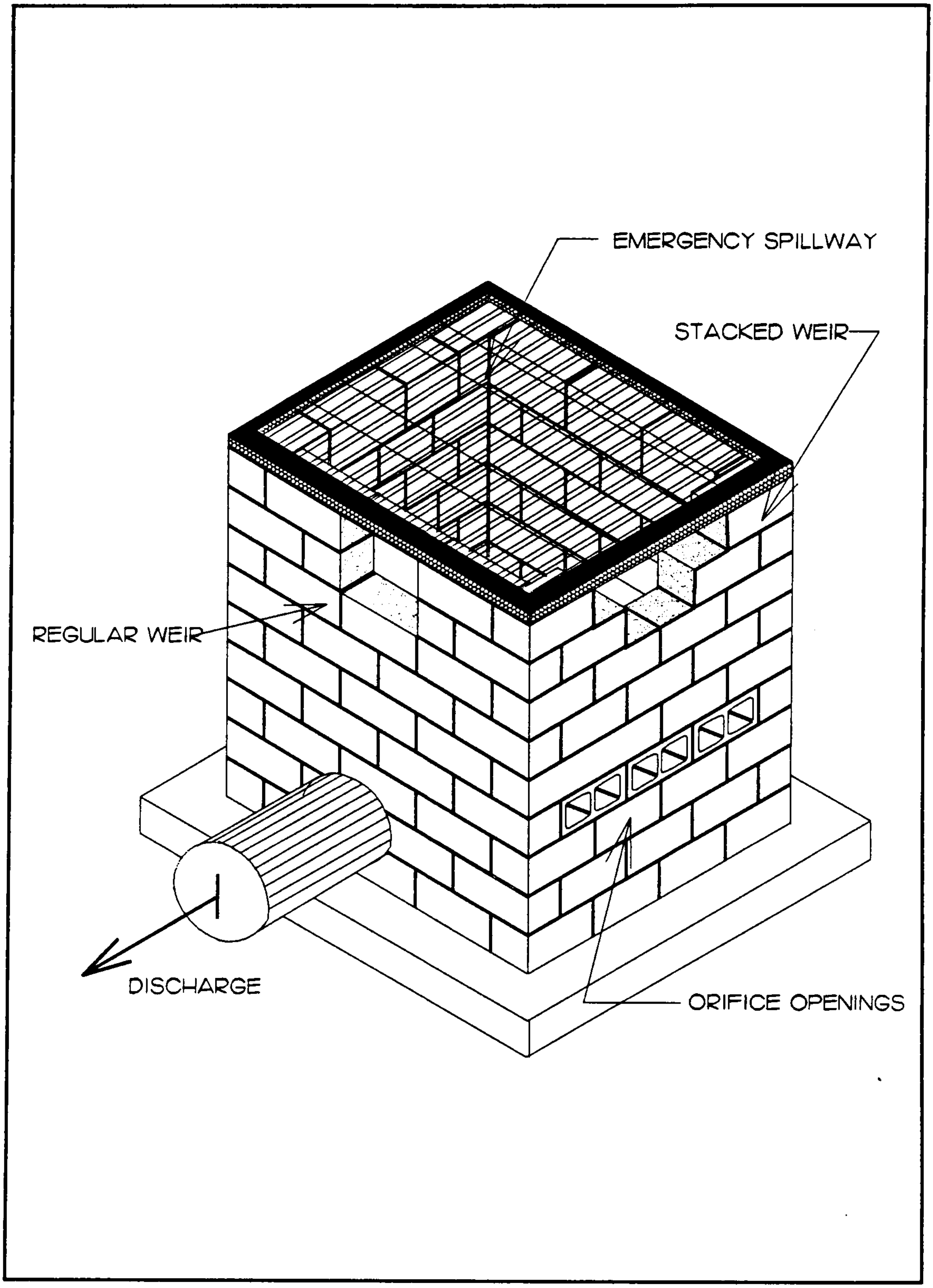


FIGURE 1

Typical Concrete Block Discharge Structure extreme flooding (e.g., 100 year storm). The top grating is removable to allow for easy maintenance. One type of emergency spillway is an opening at the top of the riser structure as shown in Figure 1. An alternate route for the flow of water is recommended in the unlikely event the pipe leading from the discharge structure becomes blocked. The alternative route should be an emergency channel spillway capable of passing the entire flow for the maximum design storm event, e.g., 100 year storm, over the top of the dam or abutment.

The major factor affecting the performance of Stormwater Detention Facilities discharge structures, such as the one shown in Figure 1, is the rate at which the various openings discharge water (34). The stage-discharge relational model is the most important aspect of the hydraulic characteristics of the outlet device (16). Technical literature discusses various methods for designing Stormwater Detention Facilities. These approaches range from simplistic approximate methods (2,3,16,23) to well developed computer storage routing (11). All of these methods have fundamental shortcomings when it comes to the design of concrete block discharge structures. No matter how complicated the mathematical approach used in the routing process, the approach will utilize standard weir and orifice equations and coefficient of discharges applying them to very complex and composite control structures (26). However, the lack of published orifice and weir coefficients of discharge, for openings formed from concrete blocks, leaves the hydraulic engineer with little choice but to design the detention storage and outlet control opening(s) using "guesstimate" coefficients of discharge. Inaccurate coefficients could result in either expensive over-designed or ineffective under-designed ponds. This thesis presents laboratory test results for determining weir and orifice coefficient of discharges and shows the sensitivity of pond storage to chosen coefficients.

BACKGROUND

Although the use of Stormwater Detention Facilities is a recent development in our culture, many of the basic principles that lead to the development and refinement of this method of controlling flooding were developed in the last century. The component of any Stormwater Detention Facilities that most affects performance is the discharge structure (16). Discharge structures can be constructed in a variety of ways out of various materials. Two components that are present in every discharge structure are openings formed by weirs and orifices (34). These two components of the discharge structure are used to control the rate at which water is released from the Stormwater Detention Facilities. The classic performance of these two components is addressed in almost any text dealing with hydraulics or elementary fluid mechanics, and they are defined as follows (7,13,18,24,25,28):

Orifice - An orifice is an opening with a closed perimeter in the wall of a tank or other structure through which water flows. The water level must be above the top of the opening in order for the opening to function as an orifice and not as a weir. A standard orifice is one with a sharp edge or an absolutely square corner with a thickness that is thin in comparison to the size of the orifice opening. One of the earliest experiments with sharp-edges orifices was performed by Hamilton Smith, Jr. and published in his book Hydraulics in 1886. The results of Smith's work were verified in 1940 by Medaugh and Johnson and found to be accurate to within a few percent (4). As the plate or wall through which the orifice penetrates becomes thicker in comparison to the size of the orifice opening the characteristics of the way the orifice passes water changes, and the orifice may start to function as a short tube. The coefficient of discharge for a short tube can exceed that of a sharp edged orifice by as much as forty percent. This amount of difference could have a dramatic effect on the design and performance of a Stormwater Detention Facilities discharge structure.

Weir - A weir is an obstruction to liquid flow that causes the liquid to be backed up behind the obstruction and then flow over or through it (28). The major use of weirs is in the measure and control the flow of liquids and, for this discussion, water. Weirs are classified according to their shape into a variety of groups. They are rectangular weirs; triangular, or V-notch, weirs; trapezoidal weirs; parabolic weirs; and linear weirs (13,29).

Much work has been done over the years regarding the performance of weirs and orifices. The results of these studies are of great interest, and until now, the results of these studies were the only rational approach to the design of these discharge components of Stormwater Detention Facilities discharge structures formed with concrete block.

An extensive literature survey was performed prior to starting the laboratory work in order to ascertain the extent of research done in the area of concrete block discharge structures. The majority of the references located and used for this thesis were obtained from ASCE publications and standard Hydraulics textbooks.

The ASCE electronic data base was checked from 1975 to the present using various key words relating to discharge structures, e.g., "concrete", "concrete block", "stormwater detention", "weir" and "orifice". The Water Resources Abstract (WRA) Data

Base was checked electronically form 1968 to 1987 followed by a manual check of the abstracts from 1988 to the present. A search of the University of Tennessee Central Library,including all electronic data bases, was accomplished using the same set of key words that were used in the ASCE and WRA search. The Municipal Technical Advisory Services (MTAS) Library and Data Base were checked with only limited success. An article referencing to a precast concrete discharge structure similar to Figure 1, Typical Concrete Block Discharge Structure, was located (10). The MTAS Library also provided an article that dealt with simplified design of detention basins (23).

Mr. Mark Prahl of the National Concrete Masonry Association was contacted in order to determine if his organization had sponsored any research related to the use of concrete block for Stormwater Detention Facilities discharge structures (20). After checking his records Mr. Prahl stated that the Association had not sponsored any research and he had no knowledge of any research in the area of concrete block discharge structures for Stormwater Detention Facilities. Conversations with Mr. James F. Ramsey and others of The U.S. Soil Conservation Service (SCS) revealed that they had done no studies pertaining to the use of concrete block for hydraulic structures (22).

Two documents, one obtained from the U.S. Army Corps of Engineers (COE), and the second, obtained from the American Public Works Association (APWA) contained information on the use of stacked-weirs (15,31). The coefficients used in the APWA manual were standard handbook coefficients obtained from King (13). The COE document was the result of a detailed model study of the Little Sioux River in

Iowa (15). This document made no direct discussion of stacked weirs as a design feature, and proposed no general concept for stacked-weirs.

The final background check was made with the Colorado State University Hydraulics Laboratory. Colorado State University is one of the leading hydraulic research institutions in the country, and is considered to be a leader in all matters related to Hydraulic Engineering. Conversations with Dr. Steve Abt, the head of the Research and Development Department, revealed that he knew of no studies related to concrete block discharge devices, but they would be interested in seeing the results of any work that might be undertaken at the University of Tennessee (1).

STUDY SCOPE AND OBJECTIVES

Due to the apparent lack of specific performance information related to concrete block weir and orifice coefficient of discharges, this thesis attempts to establish coefficients of discharge (C_d) values that can be used by the practicing engineer to make intelligent use of the concept of concrete block discharge structures. The following components are evaluated:

1. The discharge characteristics of orifice openings formed by laying concrete block on their side, rather than in the normal way, are determined. Concrete blocks laid in this way produces two symmetrical orifice openings per block. Both the single and double opening are evaluated.
2. The effect of the position of the channel bottom at the invert elevation of an orifice on orifice performance are evaluated.

3. The performance of weirs formed with concrete block are compared with the performance predicted for broad-crested weirs using standard handbook coefficients (13).

4. A method for predicting the discharge rate through stacked-weir configurations is developed.

5. Interaction between closely spaced weir and orifice devices are evaluated for possible performance impact.

A comparison is made between standard handbook values used for orifice and weir coefficient of discharges and the values developed by this thesis. The comparison demonstrates that serious errors can be expected in sizing detention ponds and estimating the discharge rate from completed Stormwater Detention Facilities. The types of errors that are possible can not only lead to property damage and increased risk to the public, but may expose the design profession to litigation that could lead to loss of the privilege to practice engineering.

In addition to the investigation of standard weir and orifice coefficients, the stacked-weir and weir/orifice combinations are investigated. Very little data has been developed concerning the performance of these types of structure except as previously discussed (15,31).

SECTION II

THEORETICAL DEVELOPMENT

GENERAL

The mathematical models used to define the characteristics of weir and orifice flow can be derived through the application of the Bernoulli Equation. This fundamental equation in the science of hydraulics was named in honor of the Swiss pioneer in the field of hydraulics Daniel Bernoulli even though the equation was first formulated and used by Leonhard Euler (5). The Bernoulli Equation summarizes the various energy forms associated with the total energy of flowing water, and as developed, the equation is only applicable for an ideal fluid. An ideal fluid is defined as one that is both frictionless and incompressible (7). The Bernoulli Equation is based on the principle of conservation of energy between any two sections of a reach of channel, such as exists in a detention pond having a concrete block discharge structure, and the equation addresses each form of energy associated with a flowing liquid, i.e., kinetic energy, potential energy, and pressure energy. The Bernoulli Equation has broad application in the field of hydraulics, and it will be used here to develop the governing equations for both weir and orifice flow. Figure 2 illustrates possible configurations for an orifice, and Figure 3 illustrates possible configuration for a weir. In each case Point 1 is upstream of the discharge device (orifice or weir) such that conditions at Point 1 are not effected by water flowing through the discharge device; Point 2 is located at the point where the minimum area of the accelerating flow occurs. Equation 1 is one of the most often used forms of the

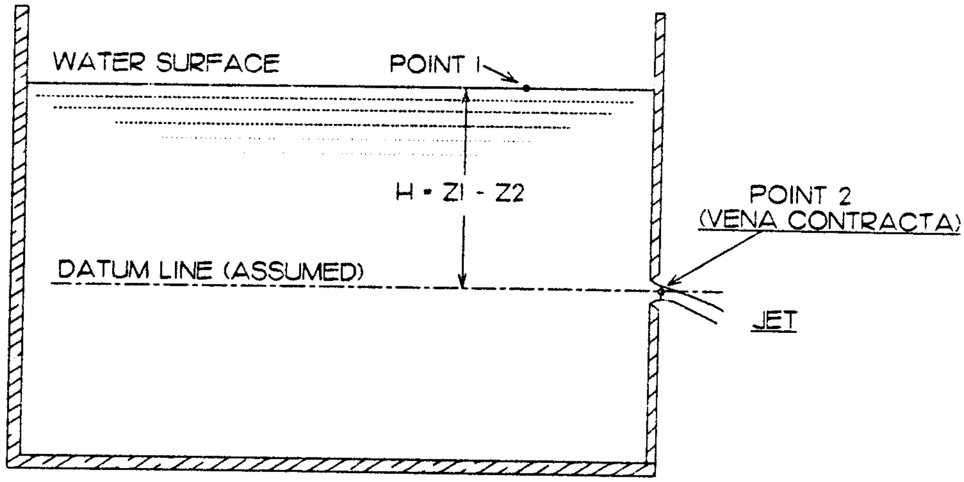
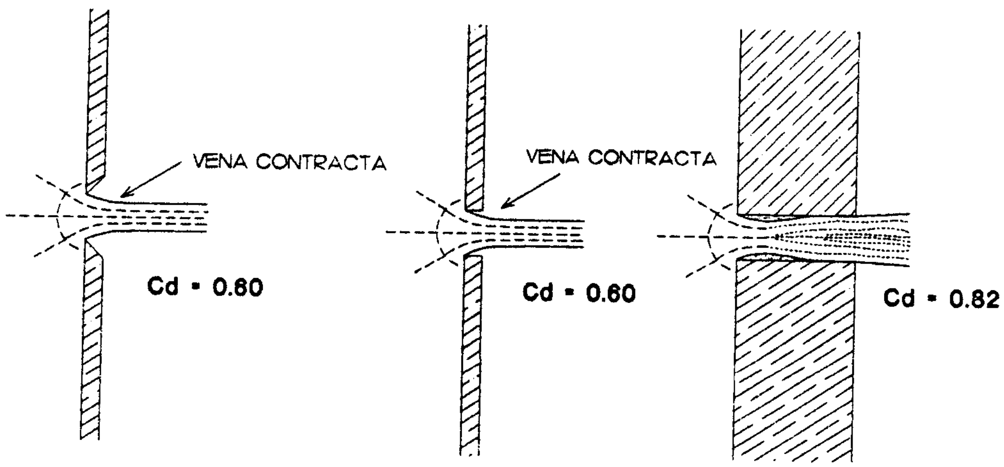


FIGURE 2A - TYPICAL SHARP-EDGE ORFICE LAYOUT



SHARP-EDGE ORIFICE THICK PLATE ORIFICE STANDARD SHORT TUBE

FIGURE 2B - POSSIBLE ORIFICE GEOMETRY (7)

FIGURE 2

Orifice Configurations

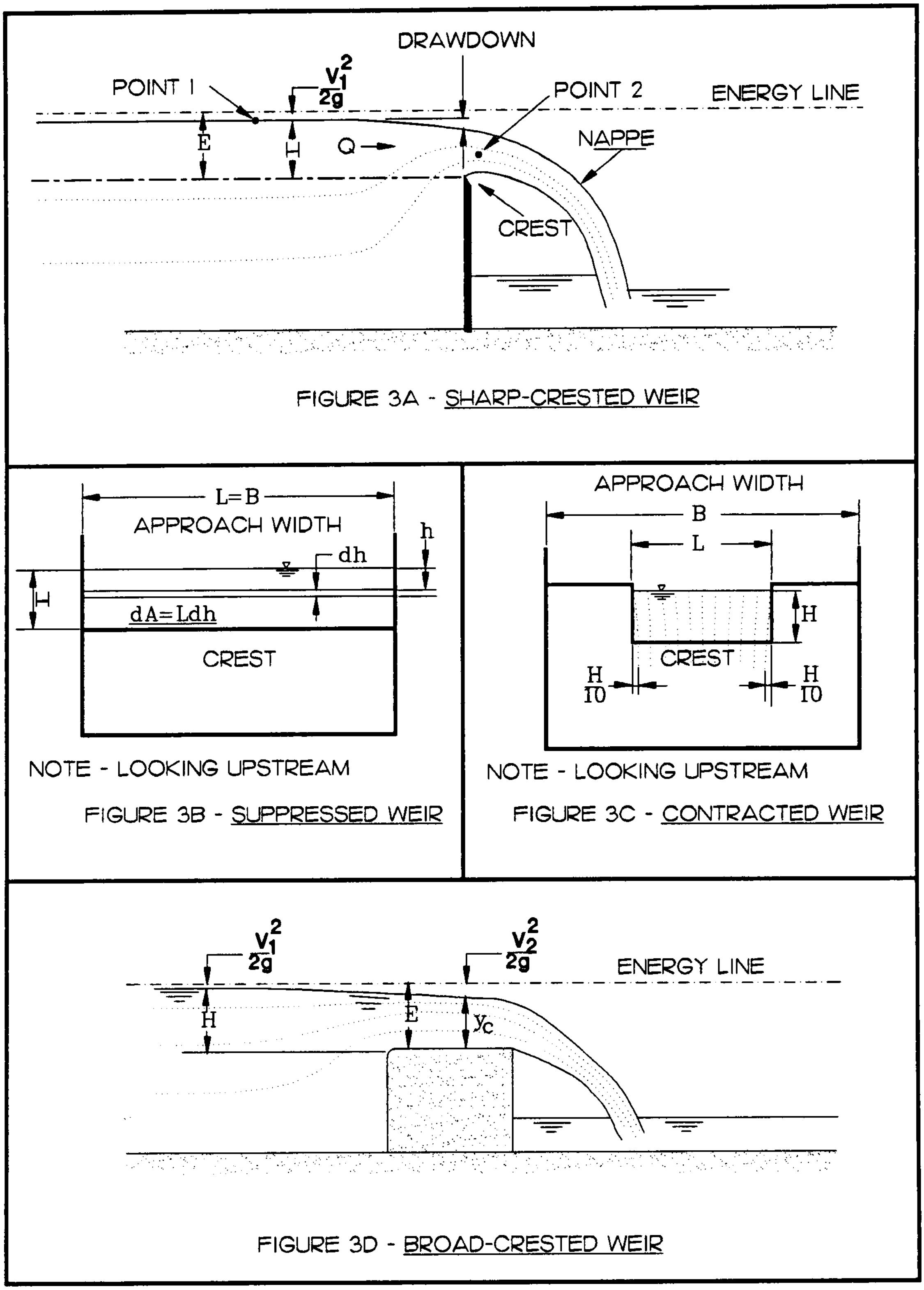


FIGURE 3A - SHARP-CRESTED WEIR

FIGURE 3B - SUPPRESSED WEIR

FIGURE 3C - CONTRACTED WEIR

FIGURE 3D - BROAD-CRESTED WEIR

FIGURE 3

Weir Configurations

Bernoulli Equation, and it will be employed here to commence a derivation of the governing equations for weir and orifice flow.

$$\frac{V_1^2}{2g}+\frac{p_1}{\gamma}+z_1=\frac{V_2^2}{2g}+\frac{p_2}{\gamma}+z_2+h_L \quad \ldots\ldots\ldots\ldots \quad (1)$$

In Equation 1, V = Velocity, with subscripts referring to points 1 and 2 (ft/s); g = Acceleration of Gravity (32.2 ft/s^2); P = Water Pressure, with subscripts referring to points 1 and 2 (lb/ft^2); γ = Weight of Water (lb/ft^3); Z = Elevation Above Datum, with subscripts referring to points 1 and 2 (ft); and h_L = Total Energy Loss (ft). The left side of Equation 1 quantifies the unit specific energy (foot-pounds per pound weight of water) for Point 1 in Figures 2 and 3. The right side of Equation 1 quantifies the specific energy at Point 2 Figures 2 and 3. At this juncture in the development of the orifice and weir equations it is appropriate to proceed with the developments separately.

ORIFICE EQUATION

Figure 2A shows the configuration of a typical sharp-edged orifice structure. Solving Equation 1 for the theoretical maximum flow at Point 2 assuming h_L = 0 yields Equation 2.

$$V_2=\sqrt{2g[(z_1+\frac{p_1}{\gamma})-(z_2+\frac{p_2}{\gamma})]} \quad \ldots\ldots\ldots\ldots \quad (2)$$

When the approach velocity of the water is low the kinetic energy term $V_1^2/2g$ approaches zero, and if the orifice discharges to the atmospheric, $P_1 = P_2 = 0$ on the gage scale, Equation 2 reduces to

$$V_2=\sqrt{2gH} \quad \text{(3)}$$

where H(ft) = $Z_1 - Z_2$. Considering the continuity equation

$$Q=AV \quad \text{(4)}$$

where the following quantities are measured at the same section: Q = Total Flow (cfs); A = Area (ft^2); and V = Velocity (ft/s). Combining terms from Equation 3 and Equation 4 to find the ideal orifice flow rate yields

$$Q_i=A_iV_i=A_oV_2=A_o\sqrt{2gH} \quad \text{(5)}$$

where Q_i = Ideal Orifice Flow Rate (cfs); A_i = Ideal Orifice Area (ft^2); V_i = Ideal Orifice Velocity (ft/s); A_o = Orifice Area (ft^2); and V_2 = Velocity at Point 2 (ft/s).

Equation 5 gives the ideal maximum flow rate through an orifice opening, and in order to be of any practical value Equation 5 must be corrected for the way orifice openings actually function. As water enters an orifice opening there is a transverse velocity component directed toward the center of the orifice opening (13). If the orifice is located such that the physical boundaries of the structure do not impede the flow of water to the orifice there will be a reduction in the area of flow at a downstream point. This reduction in area is illustrated by Point 2 in Figure 2A. The point where the reduction in area is a maximum and the flow area is a minimum is called the vena contracta. To deal with this reduced area in a general way the term coefficient of contraction C_c is introduced. In general, the area at the vena contracta is $A_2 = C_cA_o$. This contraction process combined with the friction losses associated with forcing water through the orifice along with all other losses are the components of h_L in Equation 1. Introducing the term C_v as the coefficient of velocity to account for all the energy losses associated with h_L permits the revision of Equation 3 to

$$V_2 = C_v V_I = C_v \sqrt{2gH} \qquad (6)$$

where C_v = Coefficient of Velocity.

Based on the theory of continuity, the discharge from a given orifice will be equal to the product of the area and velocity at the vena contracta. Equation 7 shows this relationship.

$$Q = A_1 V_1 = A_2 V_2 = C_c A_o V_2 \qquad (7)$$

Combining terms from Equation 6 and Equation 7 yields

$$Q_a = C_c C_v A_o \sqrt{2gH} \qquad (8)$$

where Q_a = Actual Flow Rate (cfs).

The product of the coefficient of contraction and the coefficient of velocity ($C_c x C_v$) is termed here the coefficient of discharge C_d for the orifice equation. The physical meaning of C_d is the ratio of the actual flow rate to the ideal flow rate (Q_a/Q_i) through an orifice. Equation 9 is the basic derived orifice equation that relates C_d to the flow rate and head for an orifice discharging as shown in Figure 2A.

$$Q_a = C_d A_o \sqrt{2gH} \qquad (9)$$

The best way to determine the coefficients of discharge C_d of an orifice is by experiment in the laboratory (7). In many cases, measuring the area and velocity accurately at the vena contracta may be difficult, as in the situation of a concrete block orifice. Because of this inability to make measurements, the value of C_d must be determined by some indirect method of laboratory measurement.

Figure 2B shows the arrangement of three types of orifices. Note that the performance of an orifice is sensitive to the thickness of the orifice in the direction of flow.

WEIR EQUATION

The type of weir produced by concrete block discharge structures is referred to as a broad-crested weir. Figure 3D illustrates the major components of a broad-crested weir. One advantages to this type of weir is that it is rugged and can stand up well under field conditions (7). When the head reaches one to two times the breath of the weir, the nappe becomes detached from the broad-crested weir surface and the weir becomes essentially a sharp-crested rectangular weir as shown in Figure 3A (13).

SHARP-CRESTED WEIR - The derived equation for discharge over a sharp-crested weir with suppressed contractions will be developed initially. Figure 3B illustrates the major components of a sharp-crested weir with suppressed contractions. This derived sharp-crested weir equation will then be modified to account for end contractions as shown in Figure 3C, and finally, the equation for maximum flow over a broad-crested weir will be developed as shown in Figure 3D.

Figure 3B shows an elevation of a sharp-crested weir with a width L and an elementary area $dA = L\,dh$ in the plane of the crest. This elementary area is, in effect, a horizontal slot of length L and height dh. Neglecting the velocity of approach, the ideal velocity of flow through the area will be equal to Equation 3. The apparent flow through this area is then:

$$dQ = V\,dA = L\,dh\sqrt{2gh} = L\sqrt{2g}\,h^{1/2}\,dh \quad \text{(10)}$$

This equation will be integrated over the whole area, i.e., from h = 0 to h = H in order to determine the total ideal flow Q_i over the weir. When the approach velocity of the water is low the kinetic energy term $V_1^2/2g$ approaches zero and the following equation is can be written:

$$Q_i = \sqrt{2g}\,L\int_0^H h^{1/2}\,dh = \frac{2}{3}\sqrt{2g}\,LH^{3/2} \quad \text{(11)}$$

The actual flow over the weir will be less than the ideal flow because the effective flow area is considerably smaller than L x H due to the drawdown from the top and contraction of the nappe from below as shown in Figure 3A. As discussed in the orifice development, the coefficient of discharge C_d will be introduced to correct Equation 11 for all the losses associated with flow over the weir.

$$Q_a = C_d\,\frac{2}{3}\sqrt{2g}\,LH^{3/2} \quad \text{(12)}$$

As with the orifice, the product of the coefficient of contraction C_c and the coefficient of velocity C_v is called the coefficient of discharge C_d for the weir equation. Equation 12 is the basic derived weir equation that relates C_d to the flow rate and head for an weir discharging as shown in Figure 3A. In most cases when dealing with the weir equation it is convenient to express Equation 12 as

$$Q_a = C_w LH^{3/2} \quad \text{(13)}$$

where C_w, the weir coefficient, replaces $C_d 2/3\sqrt{2g}$.

Figure 3C illustrates a weir with end contractions. Tests made by Francis (13) indicated that the effect of the end contractions could be accounted for by reducing L in Equation 13 as follows

$$Q_a = C_w(L-0.1\,nH)\,H^{3/2} \qquad (14)$$

where Q_a = Actual Flow Rate (cfs); C_w = Weir Coefficient; L = Weir Length (ft); n = Number of Contractions (0 for Suppressed Weir); and H = Head (ft). Equation 14 is the general format for the weir equation with end contractions. Not all studies that have been performed are in agreement with this approach, and in some cases, the adjustment factor for the crest width could actually be additive to the actual length as indicated in King's Figure 5-3 (13).

BROAD-CRESTED WEIR - Discharge over a broad-crested weir is controlled by the shape of the weir cross section and the area of the weir opening. The typical broad-crested weir is constructed with a horizontal or nearly horizontal crest as shown in Figure 3D. To function as a broad-crested weir the crest must be sufficiently long in the direction of flow that the nappe is supported and hydrostatic pressure is developed on the crest for at least a short distance (13). The maximum flow over a true broad-crested weir occurs when the critical depth y_c develops on the crest of the broad-crested weir. As the length and roughness of the weir increases the flow is decreased. The effect on discharge of roughness and length of the crest can be computed by applying the principles of flow in open channels (13). The derivation presented here will be for the maximum coefficient of discharge C_d of a broad-crested weir. It should be noted that when the head reaches one to two times the breath of the weir, the nappe becomes detached from the surface of the broad-crested weir and the weir becomes essentially a sharp-crested weir (13).

Equation 15 shows the governing relationships between total energy and velocity at a given section of broad-crested weir flowing at critical depth

$$y_c = \frac{2}{3}E \qquad V_c = \sqrt{g y_c} \qquad \text{(15)}$$

where y_c = Critical Depth (ft); E = Total Energy (ft); V_c = Velocity at Critical Depth; and g = Acceleration of Gravity (32.2 ft/s^2). Employing the relations in Equation 15 along with Equation 4 the following can be written

$$Q_a = A\,V = (L y_c)\sqrt{g y_c} = L\sqrt{g}\,y_c^{3/2} = L\sqrt{g}\left(\frac{2}{3}\right)^{3/2} E^{3/2} \qquad \text{(16)}$$

where Q_a = Total Flow (cfs) over the broad-crested weir. Combining Equation 16 with Equation 12 gives

$$C_d = \frac{1}{\sqrt{3}}\left(\frac{E}{H}\right)^{3/2} \qquad \text{(17)}$$

where C_d = Coefficient of Discharge for a broad-crested weir. When the approach velocity is low then total head is equal to static head, i.e., E = H, and Equation 17 reduces to $1/\sqrt{3}$ = 0.577. Combining Equation 17 and Equation 12 yields C_w = 3.087 for the discharge over a broad-crested weir (7,13). This is the maximum value of C_w that can be attained over a broad-crested weir, and if the experimental results indicate a larger value the nappe has detached from the crest of the weir and the weir is functioning as a sharp-crested weir.

COMMENTS ON THEORETICAL DEVELOPMENT

Although the basic weir and orifice are the simplest of hydraulic devices, the equations that describe their performance can not be derived exactly due to the many variables involved. The derivations presented here are not exact, but they are in keeping with the state of the art in weir and orifice coefficient of discharge knowledge. The derivations do not include the effects of viscosity, surface tension, and show the effects of gravitational forces only in an approximate way (13).

SECTION III

LABORATORY SETUP AND PROCEDURES

GENERAL

A series of twelve (12) full-scale laboratory tests was conducted to evaluate the performance and hydraulic characteristics of various orifice and weir configurations commonly used in single- and multiple-stage stormwater detention outlet control structures constructed with concrete block. The study was performed in the University of Tennessee Civil Engineering Department Water Resources Laboratory. Figure 4 shows the general layout of the laboratory with the modifications made for this study highlighted. Every effort was made to utilize existing equipment, but conducting this study required considerable modification to the laboratory facility to accommodate the test facility.

LABORATORY SETUP

The laboratory setup made in order to perform this experiment falls into three major categories, (1) concrete block wall; (2) rock diffuser; (3) piping modifications. The reader is directed to Appendix E for photographs of all the components of the laboratory setup.

Concrete Block Wall - A 5'-0" wide by 5'-4" high concrete block wall was constructed in a side channel of the laboratory reservoir. The modular block used for the construction of the wall were manufactured in accordance with ASTM C-90. This type of block has been the industry standard since being introduced in 1946. Figure 5 shows a plan and elevation of the upstream face of the wall as originally

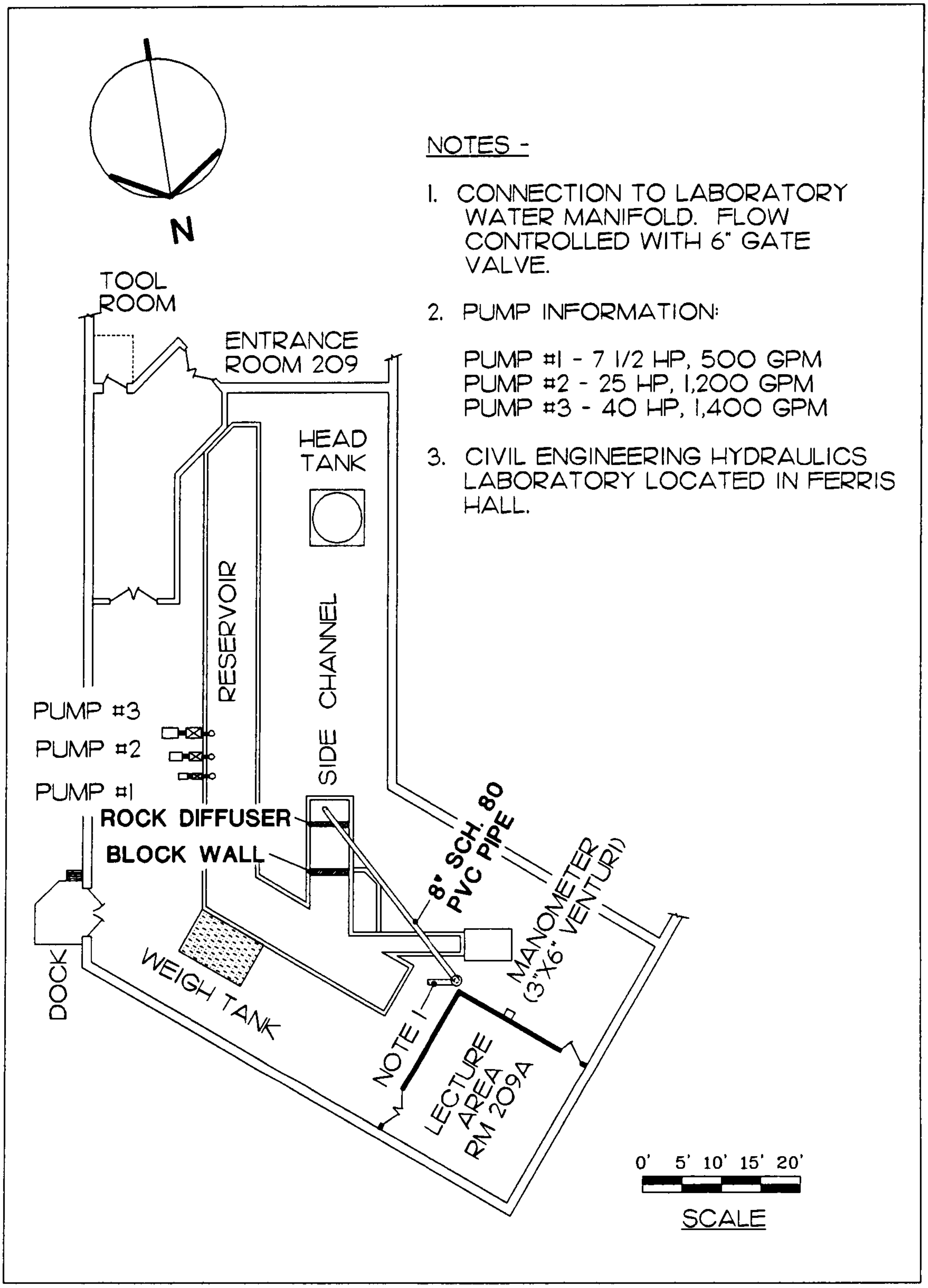


FIGURE 4

University of Tennessee Hydraulics Laboratory Layout

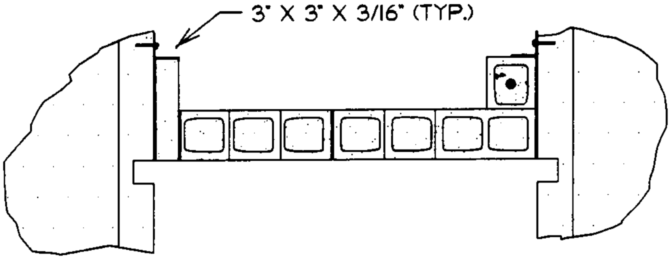


FIGURE 4A - CONCRETE BLOCK WALL PLAN

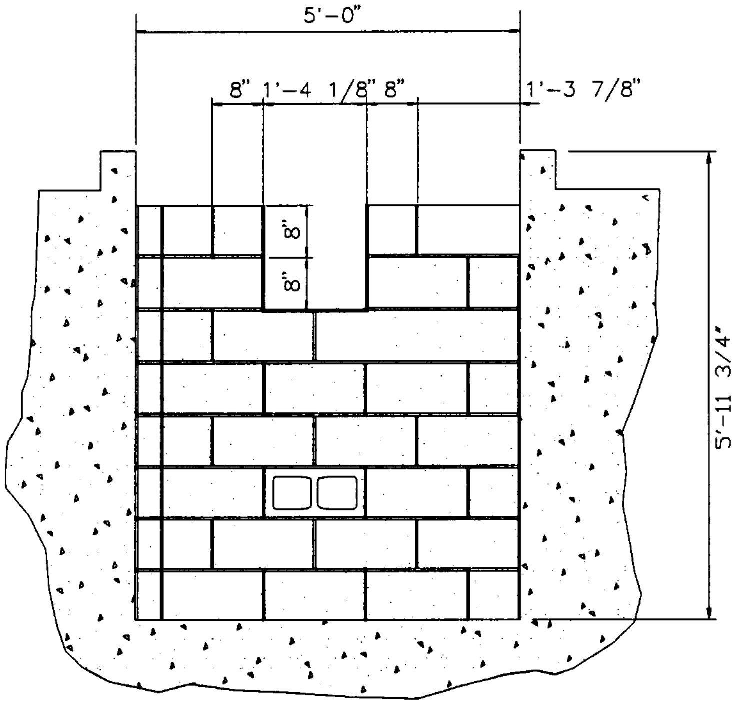
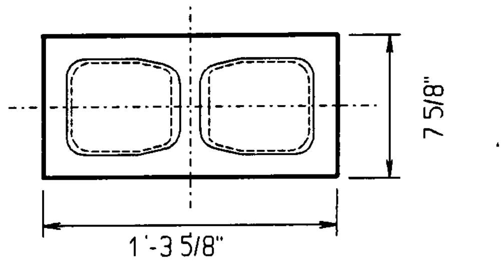


FIGURE 4B - BLOCK WALL ELEVATION

FIGURE 5

Concrete Block Wall Elevation

constructed. The wall was constructed with nominal 8" x 8" x 1'-4" concrete blocks obtained from Southern Cast Stone of Knoxville. The dimensions of the block and its openings are shown in Figure 6. Horizontal joint reinforcing (12 Gage) was installed in each course, and the wall was supported on the downstream face by two 3" x 3" x 3/16" steel angles expansion anchored to the concrete reservoir.



NOTES

1. UPSTREAM AREA 0.409 FT2

2. DOWNSTREAM AREA 0.332 FT2

FIGURE 6 - Concrete Block Detail

The block wall was constructed so that various arrangements of weir openings could be installed and later modified. In a similar manner, the orifice openings could be closed when not a part of the particular test being performed. A cementitious block sealer was applied around the perimeter of the wall to minimize leakage at the block wall reservoir interface.

Rock Diffuser - A rock diffuser wall was constructed between the point in the side channel where water was introduced and the concrete block wall. The steel fabrication drawings for the frames used to contain the rounded rocks used in the diffuser are shown in Appendix A. The frames were fabricated using 3" x 3" x 3/16" steel angles, 2" x 1/4" flat bar, and expanded metal screen. The frames were connected to the concrete reservoir wall approximately 6" apart using expansion anchors. Once the angle frames and expanded metal screens were in place, approximately 1,700 pounds of rounded river rock with a mean diameter of approximately 3" were placed between the two screens. The rock diffuser functioned to smooth out the turbulence that was generated when water was introduced into the reservoir upstream of the block wall. A stationary staff gage was installed just downstream of the diffuser in order to obtaining water level (head) readings over the orifice and weir configurations used in each test.

PVC Piping - Approximately 30' of 8" diameter Schedule 80 PVC pipe was installed to bring water from the laboratory distribution manifold for use in this study. The piping was supported at six points for resisting gravity, thrust, and vibration loads associated with the flowing water. Appendix A contains the steel fabrication details for the PVC pipe supports. All PVC piping was assembled with adhesives except for the fanged connection to the laboratory piping system (8).

Once construction activities were completed it was necessary to calibrate two components of the system to ensure that the test results obtained would accurately reflect the discharge device performance. The items that required calibration were the 3" X 6" Venturi meter for flow rate and the just constructed concrete block wall for leak rate.

3" x 6" Venturi Flow Meter - Once the source of water for the study was determined it was necessary to calibrate the flow rate of water to the laboratory setup so that a known flow rate could be supplied. Water was introduced upstream of the diffuser through the calibrated 3" x 6" Venturi meter and the 8" diameter Schedule 80 PVC piping. The existing Venturi meter was calibrated using a laboratory weigh tank to measure water flow up to the total laboratory pump capacity of 3 cubic feet per second (cfs). The weigh tank is kept in calibration by the Civil Engineering Department. The three pumps provided the flow capacity are described on Figure 4. Due to the long and circuitous route from the supply to the discharge point, there was

TABLE 1

Laboratory Test Data
3" x 6" Venturi Meter Calibration
(Date of Test - October 16, 1990)

DATA POINT	Feet (Hg)	Weight (Lb)	Time (Sec.)	Flow (cfs)
1	4.370	13,000	69.5	3.00
2	4.120	13,000	70.4	2.96
3	3.670	13,000	75.7	2.75
4	3.230	13,000	80.2	2.60
5	2.740	13,000	86.6	2.41
6	2.470	13,000	91.7	2.27
7	2.120	13,000	99.4	2.10
8	1.670	13,000	109.8	1.90
9	1.485	13,000	116.7	1.79
10	1.280	11,000	107.2	1.64
11	1.060	10,000	107.4	1.49
12	0.860	9,000	107.3	1.34
13	0.675	8,000	108.1	1.19
14	0.455	7,000	112.8	0.99
15	0.260	5,000	105.9	0.76

a considerable amount of pressure drop in the pumping system. This pressure drop limited the flow rate that could be provided to 3 cfs. The nominal discharge rating for all three pumps totaled 7 cfs. The data collected in the laboratory is shown in Table 1 for the Venturi meter calibration. The data is analyzed in Figure 7 which shows a plot of the data along with a power regression plot and the equation for the venturi flow rate for various pressure drops across the Venturi meter.

Block Wall Leakage Rate - As the water level rises behind the model discharge structure (concrete block wall) it will start to seep through the porous block wall and joints. In order to accurately evaluate the flow rate through the orifices and weirs of the concrete block wall, leakage was determined and accounted for during each laboratory test run. All openings in the block wall were closed and the water level

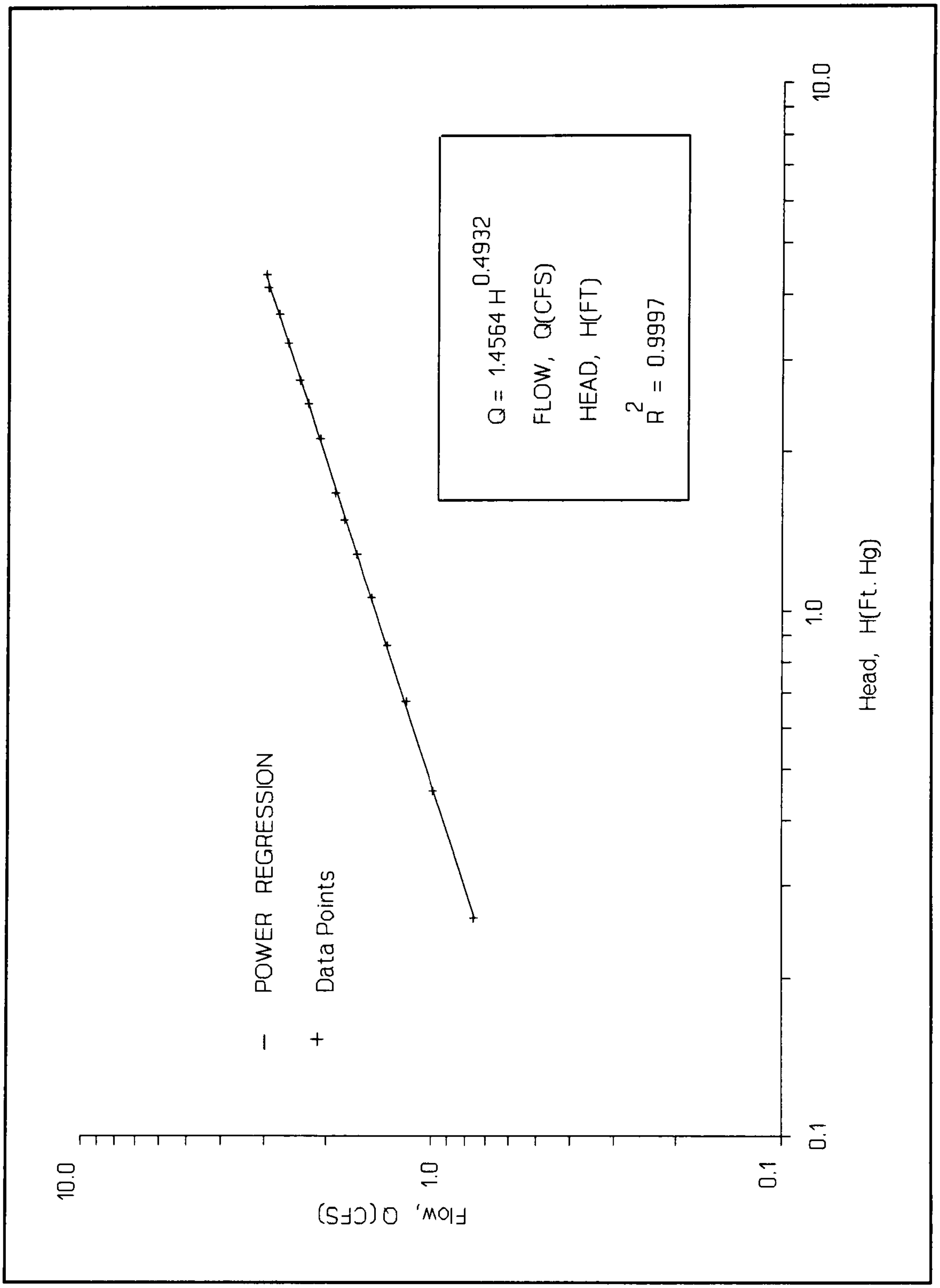


FIGURE 7

3" x 6" Venturi Meter Flow Calibration

increased to the maximum level. Staff gage readings were taken over time to determine the rate at which the water seeped thorough the concrete block wall as the head behind the wall decreased. The data is shown in Table 2 Laboratory Test Data Concrete Block Wall Leak Rate Calibration. A linear regression of the leakage rate, Q (cfs), as a function of water depth, H (ft), behind the wall is shown in Figure 8. In general terms, the leak rate through this standard hollow concrete block wall, having no water proofing or void filling, is calculated by

$$Q = 3.4A \qquad (18)$$

where Q = Leak Rate (gpm); and A = Wall Area (ft^2). The leakage was found to be a small percentage of the total flow in most test runs being typically less than 5% of the total flow as shown in Appendix C. The weir or orifice flow was adjusted for the leakage at corresponding heads using the equation given in Figure 8 Concrete Block Wall Leak Rate Calibration.

Figure 9 and Figure 10 show the arrangement of the various components of the completed laboratory setup. Note that the distance from the rock diffuser to the concrete block wall is sufficient to ensure that the staff gage readings are not effected by the changes in flow pattern as the water moves through the concrete block wall discharge structure.

EXPERIMENTAL PROCEDURES

This laboratory experiment was designed to study the flow of water through various combinations of orifice only, weir only, stacked weirs, and weir/orifice

TABLE 2

Laboratory Test Data
Concrete Block Wall Leak Rate Calibration
(Date of Test - October 16,1990)

Data Point	Head (ft)	Time(MM + SS)	Flow Rate (gpm)
1	3.11	0+00	57
2	3.02	0+30	51
3	2.94	1+00	51
4	2.86	1+30	45
5	2.79	2+00	45
6	2.72	2+30	51
7	2.64	3+00	38
8	2.58	3+30	46
9	2.51	4+00	46
10	2.44	4+30	46
11	2.37	5+00	38
12	2.25	6+00	38
13	2.13	7+00	35
14	2.02	8+00	35
15	1.91	9+00	32
16	1.81	10+00	29
17	1.72	11+00	29
18	1.63	12+00	29
19	1.54	13+00	25
20	1.46	14+00	22
21	1.39	15+00	22
22	1.32	16+00	22
23	1.25	17+00	19
24	1.19	18+00	19
25	1.13	19+00	19
26	1.07	20+00	16
27	1.02	21+00	13
28	0.98	22+00	19
29	0.92	23+00	16
30	0.87	24+00	16
31	0.82	25+00	13
32	0.78	26+00	13
33	0.74	27+00	13
34	0.70	28+00	13
35	0.66	29+00	10
36	0.63	30+00	13
37	0.59	31+00	10
37	0.56	32+00	10
39	0.53	33+00	10
40	0.50	34+00	10
41	0.47	35+00	6
42	0.45	36+00	10
43	0.42	37+00	10
44	0.39	38+00	6
45	0.51	39+00	6
46	0.49	40+00	-

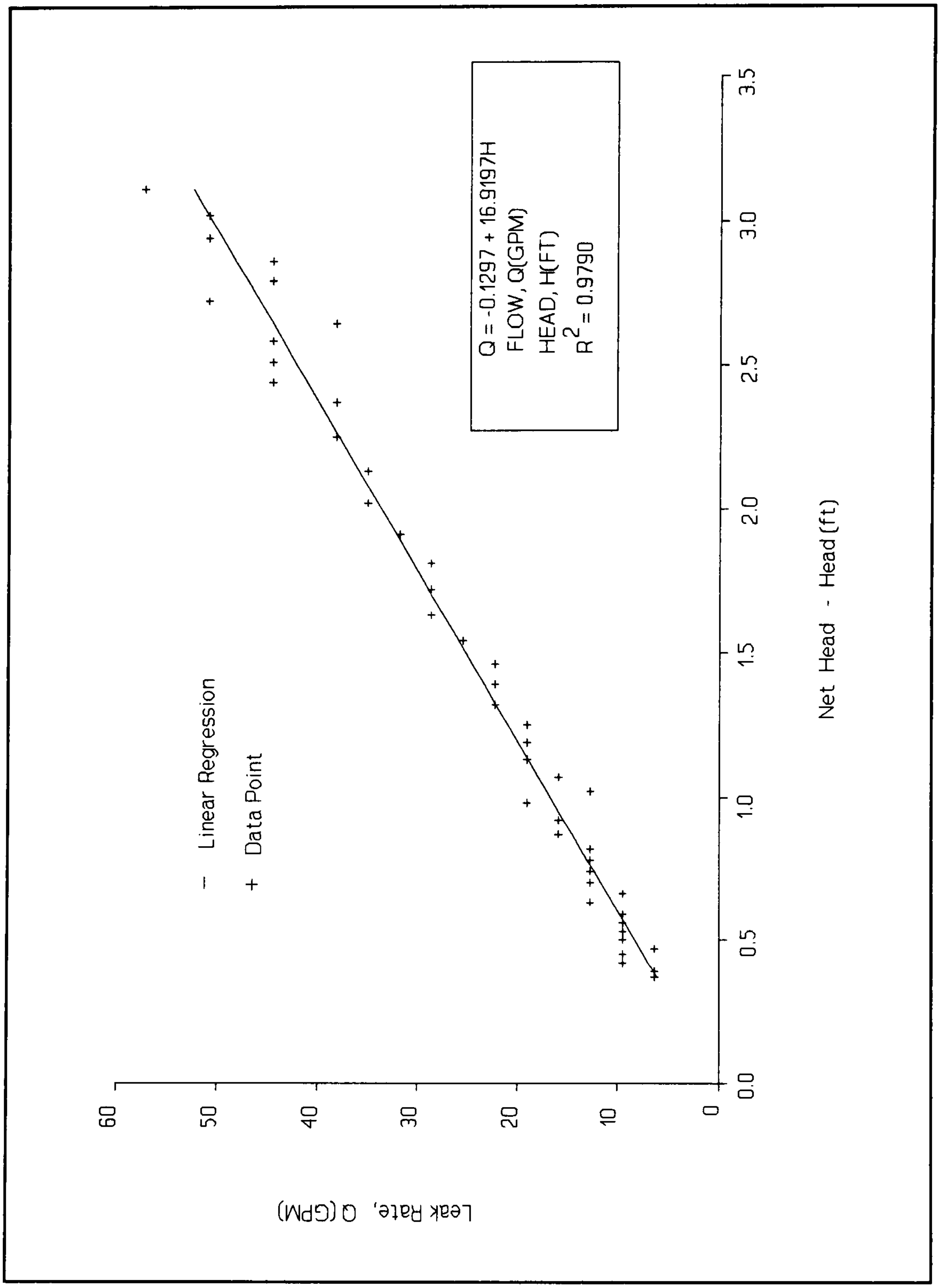


FIGURE 8

Concrete Block Wall Leak Rate Calibration

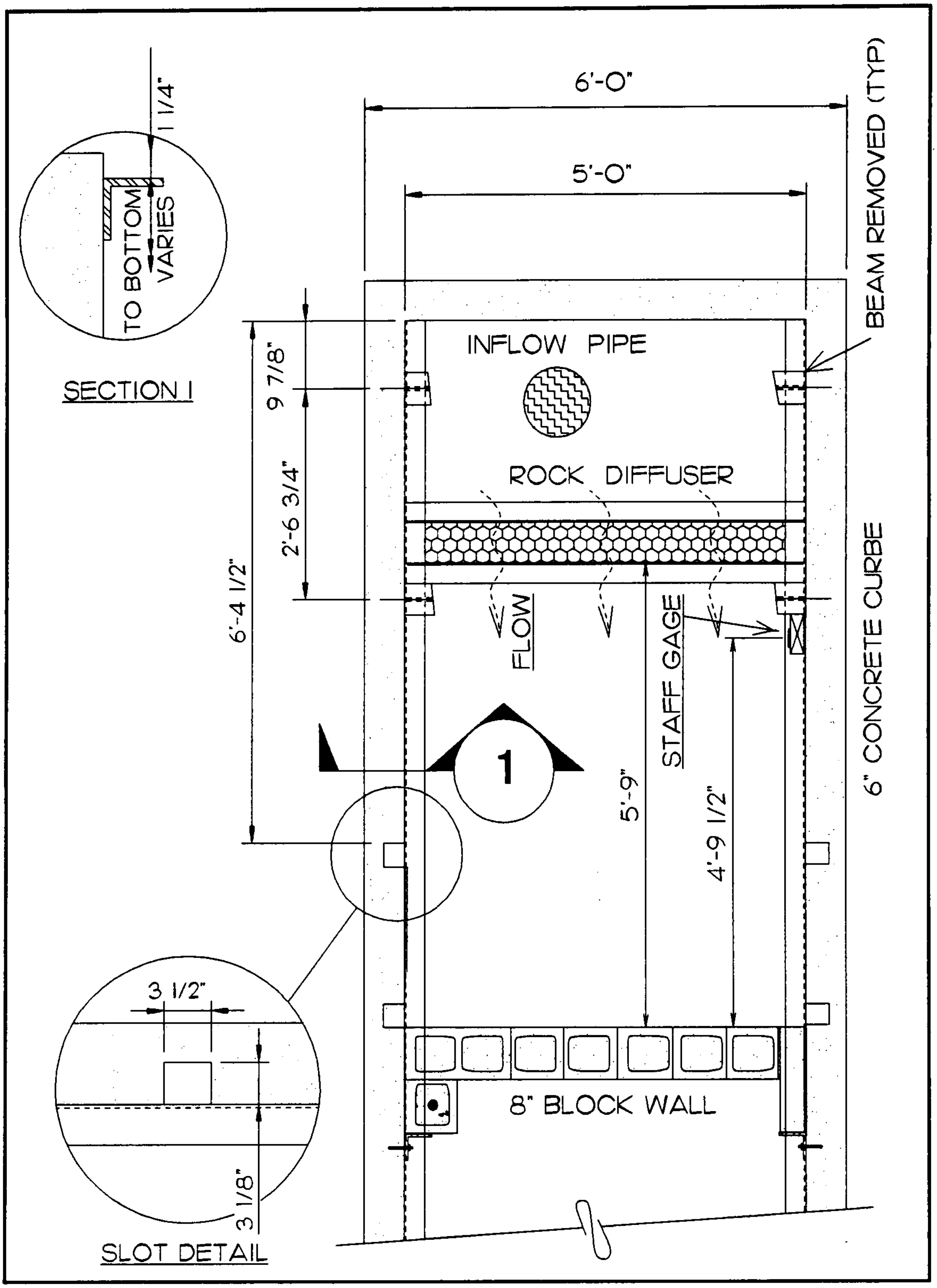


FIGURE 9

Plan Showing Test Section and Inflow Pipe

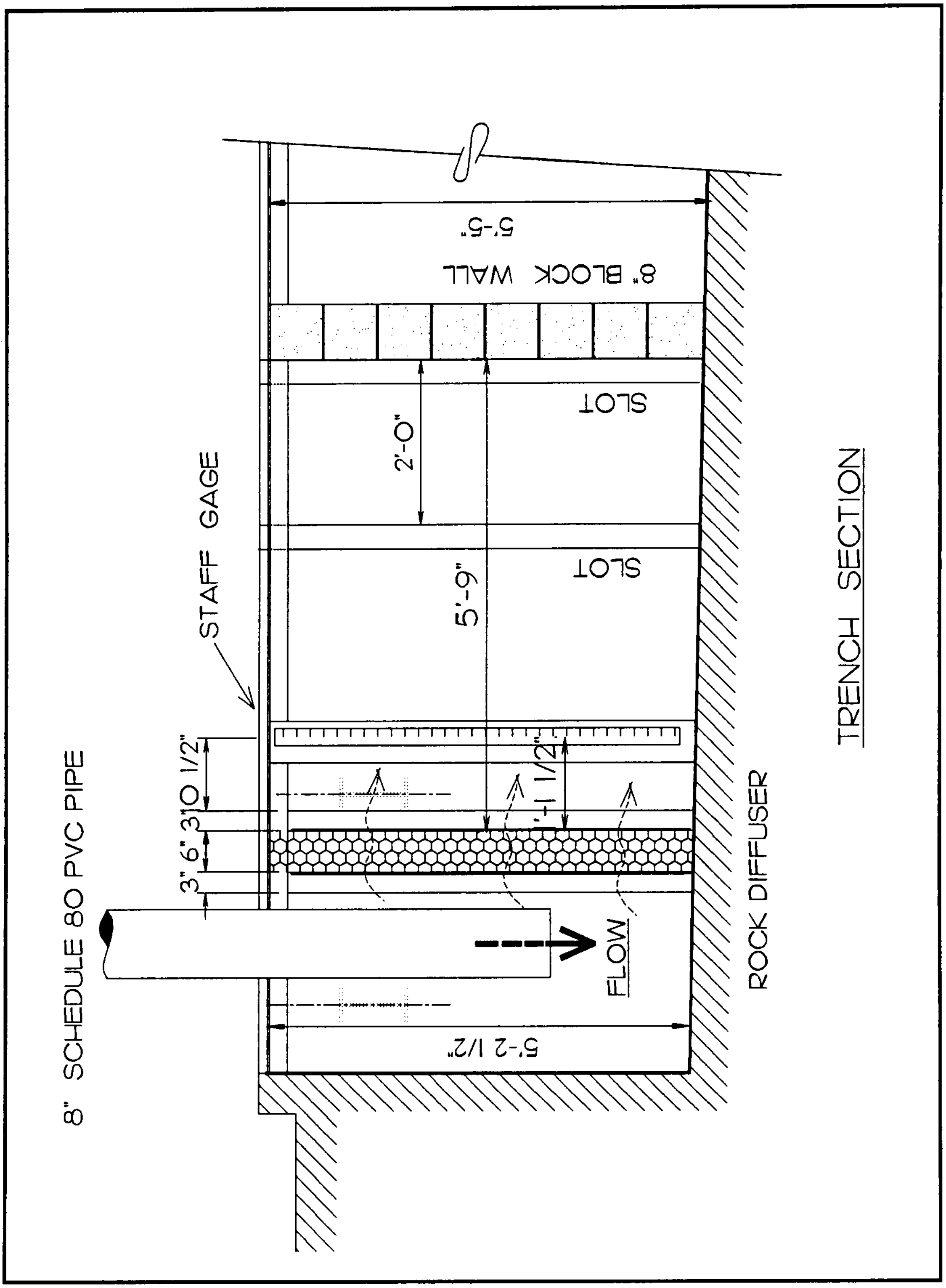


FIGURE 10

Section Reservoir Side Channel

combinations used for concrete block discharge structures. The twelve tests consisted of four orifice-only tests, four weir-only tests, two stacked weir tests, and two combined weir/orifice tests. Four of the tests also included the installation of an elevated channel bottom (raised floor) flush with the invert elevation of the orifice opening. Table 3 provides a complete listing of the arrangements tested during this study. Because each individual test required setup, i.e., construction effort, it was prudent to accomplish the individual tests in an order different from that shown in Table 3. Appendix B lists the actual sequence of tests along with the construction activities required to prepare for each test.

The procedure for accomplishing each run was as follows:

Finalize Laboratory Setup - Using the sequence described in Appendix B the concrete block wall was configured according to the requirements of the particular test. The appropriate lines were then valved off such that water could flow to the test trench through the 3" x 6" Venturi meter.

Establish Pumping Sequence - Pump #1, the smallest pump, was started first to establish a flow in the piping system and fill the constant head tank. The constant head tank (2,500 gal.) functions as a buffer for the laboratory pumping system and damped out minor pressure fluctuations in the piping system due to building power supply fluctuations. Once steady flow was established, one or both of the larger system pumps was started in order to deliver the appropriate amount of water to the piping system and laboratory setup.

Modulation of the Flow Rate - A 6" gate valve was used to control the flow of water into the trench area. The 3" x 6" Venturi meter was located upstream of the 6" gate valve. An 8-foot high mercury manometer was used to measured the pressure

TABLE 3

Laboratory Testing Program for Determining
Weir and Orifice Discharge Characteristics

Test #	8" Wide Weir	16" Wide Weir	24" Wide Weir	Single Orifice Opening	Double Orifice Opening	Elevated Channel Bottom
1				X		
2					X	
3				X		X
4					X	X
5		X		X		
6		X		X		X
7		X				X
8		X				
*9	L	U				
10	X					
*11	L		U			
12			X			

X Test Configurations
* Stacked-Weir Test
L Lower Portion of Stacked-Weir Test Configuration
U Upper Portion of Stacked-Weir Test Configuration

across the Venturi meter. The pressure drop was then used with the equation developed in Figure 7 to determine the flow rate. Each test was started with the flow rate set at the maximum, and then the rate was reduced in small increments to the minimum value. For tests using an orifice, the minimum flow was obtained when the water level dropped to the vicinity of the top of the orifice opening and ceased to be orifice flow. Minimum head for recording broad-crested weir flow data was chosen to be 0.20 feet. This value was selected due to the considerable discrepancy existing in the results of various experiments when broad-crested weir data was taken at low heads (13). The selected of a minimum head of 0.20 ft. also matched the lowest head for published coefficients of discharge generally in use with broad-crested weirs, and 0.20 ft. is the minimum value recommended by White et al. (13,33).

Collecting Data - When collecting data it was important to gather and record an adequate number of data points so that the relationship between head and flow rate could be accurately established. Changing from one experimental test to the next often required considerable construction and setup effort as indicated in Appendix B, and for this reason, it was imperative that the data be collected correctly the first time. When using the mercury manometer it was critical that any air in the lines leading from the Venturi meter to the mercury manometer be removed prior to taking readings. Since air is compressible its presence in the lines would cause errors in the manometer readings and flows obtained.

Once the maximum flow rate was established in the pumping system the data collecting began. For each reading the position of the modulating gate valve was set and then the manometer reading was taken to the nearest 0.005 foot. In most cases, the manometer reading was changed in increments of 0.2 feet of Mercury for each data point. This increment of change was sufficient to provide an adequate number of data points in order to establish the governing relationships. Once flow through the experimental setup stabilized the staff gage was read to the nearest 0.005 foot. Flow stabilization in the laboratory flow system required several minutes especially when the run being recorded included an orifice opening.

SECTION IV

LABORATORY RESULTS

GENERAL

Once collected, the laboratory data was evaluated in two ways. First, the Venturi and staff gage reading for each data point were assessed for that point only. The total flow through the discharge device(s) was determined by subtracting the wall leakage given in Figure 8 from the total flow rate given in Figure 7; once this calculation was completed the experimental C_d for each data point (head) was determined. A detailed evaluation of all the data collected is contained in Appendix C for each of the twelve test runs.

Second, the data points recorded for each test were evaluated as individual sets. This evaluation was accomplished using power regression analysis in the form

$$Y=aX^{b} \qquad (19)$$

where Y represents the discharge rate, Q (cfs); a = Power Regression Coefficient; X represents the head, H (ft); and b = the power regression exponent. The results of these analysis were then plotted in a Log-Log format. As shown on each plot of the data, a high order of agreement was obtained for all data sets with R^2 averaging 0.9987. It is noted that, the power regression orifice exponents averaged within 2.5% of the theoretical value of 0.5, and the power regression weir exponents averaged within 4.2% of the theoretical value of 1.5.

TABLE 4

Laboratory Test Data
Single Orifice Opening (Test 1)
(Date of Test - December 4, 1990)
(Single Orifice Area = 0.205 Ft^2)

DATA POINT	HEAD H(ft)	FLOW Q_a (cfs)	FLOW Q_i (cfs)	$C_d = Q_a/Q_i$
1	2.887	1.872	2.792	0.671
2	2.757	1.824	2.728	0.669
3	2.467	1.724	2.581	0.669
4	2.177	1.617	2.425	0.668
5	1.867	1.503	2.234	0.674
6	1.547	1.376	2.045	0.674
7	1.237	1.234	1.829	0.676
8	0.927	1.071	1.584	0.677
9	0.627	0.872	1.304	0.671
10	0.317	0.605	0.929	0.655

ORIFICE ONLY COEFFICIENTS

Four orifice only tests were conducted in order to determine the coefficients of discharge for various configurations of orifice openings as discussed in Section III. For all runs containing orifice openings, the orifice head was measured from the center of the orifice opening to the free water surface, and the flow rates shown in each table have been corrected for the leak rate shown in Figure 8. The C_d and C_w values shown in all tables have been calculated, as discussed Section II, using the relationship of Q_a/Q_i.

Single Orifice Opening (Test 1) - During this test the full capability of the pumping system was not utilized since the maximum head attained with a flow rate of 1.988 cfs approached the water level limits of the experimental setup. Table 4 and Appendix C show the data gathered and the evaluations made for Test 1. Figure 11

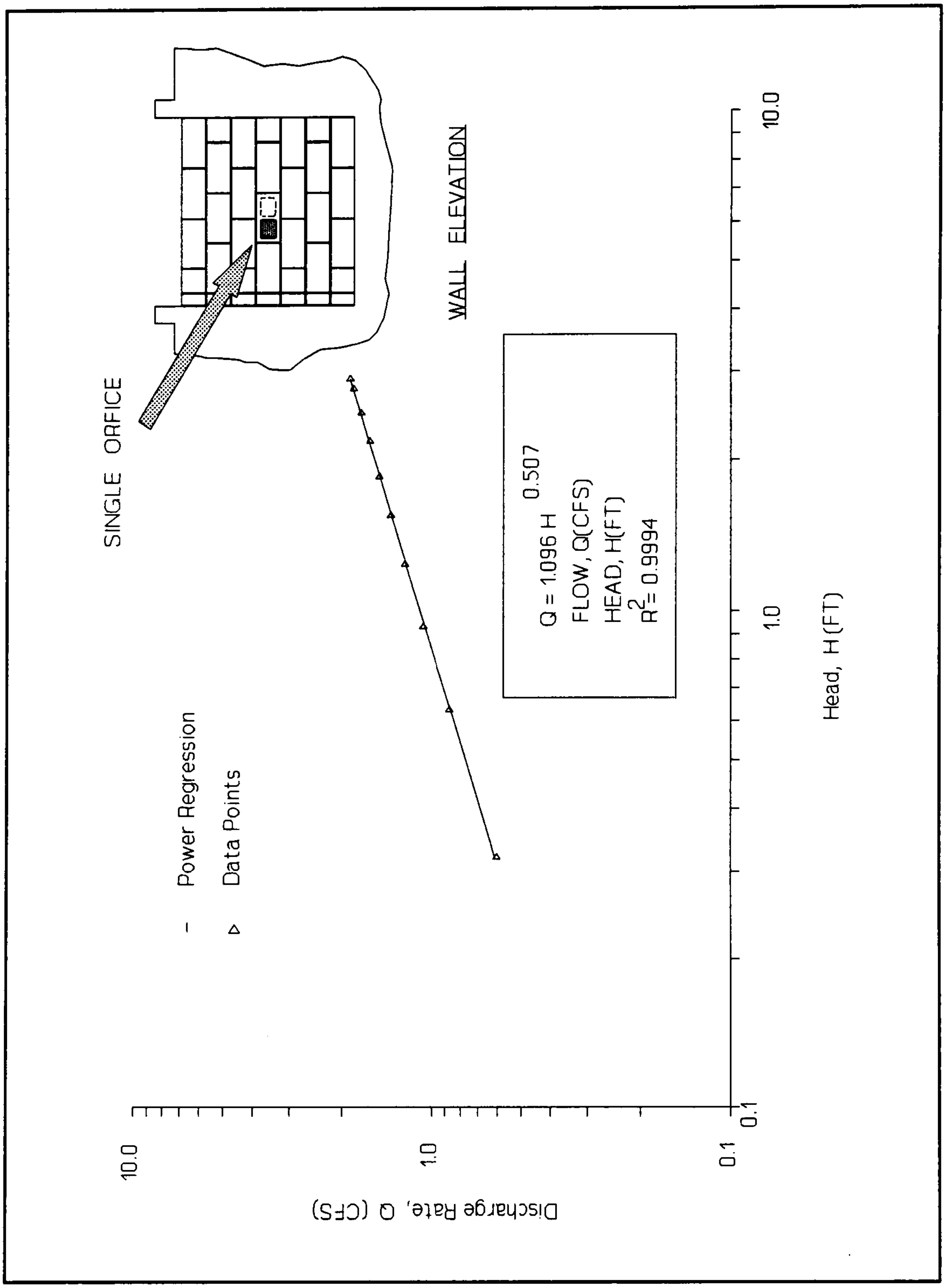


FIGURE 11

Single Orifice Opening (Test 1)

shows the data points plotted along with the power regression line and regression equation.

TABLE 5

Laboratory Test Data
Double Orifice Opening (Test 2)
(Date of Test - November 5, 1990)
(Double Orifice Area = 0.409 Ft^2)

DATA POINT	HEAD (ft)	FLOW Q_a (cfs)	FLOW Q_t (cfs)	$C_d = Q_a/Q_t$
1	1.837	2.954	4.449	0.664
2	1.737	2.865	4.326	0.662
3	1.607	2.745	4.161	0.660
4	1.532	2.678	4.063	0.659
5	1.452	2.601	3.955	0.657
6	1.337	2.527	3.795	0.666
7	1.222	2.433	3.633	0.670
8	1.132	2.361	3.492	0.676
9	1.052	2.286	3.366	0.679
10	0.942	2.144	3.186	0.673
11	0.822	2.001	2.976	0.672
12	0.722	1.852	2.789	0.664
13	0.607	1.683	2.557	0.658
14	0.487	1.480	2.291	0.646
15	0.357	1.251	1.961	0.638

Double Orifice Opening (Test 2) - During this test the full capability of the pumping system was used. Table 5 and Appendix C show the data gathered and the evaluations made for Test 2. Figure 12 shows the data points plotted along with the power regression line and regression equation.

Single Orifice Elevated Channel Bottom (Test 3) - During this test the full capability of the pumping system was not utilized since the maximum head attained with a flow rate of 1.850 cfs approached the water level limits of the experimental setup. Table 6 and Appendix C show the data gathered and the evaluations made

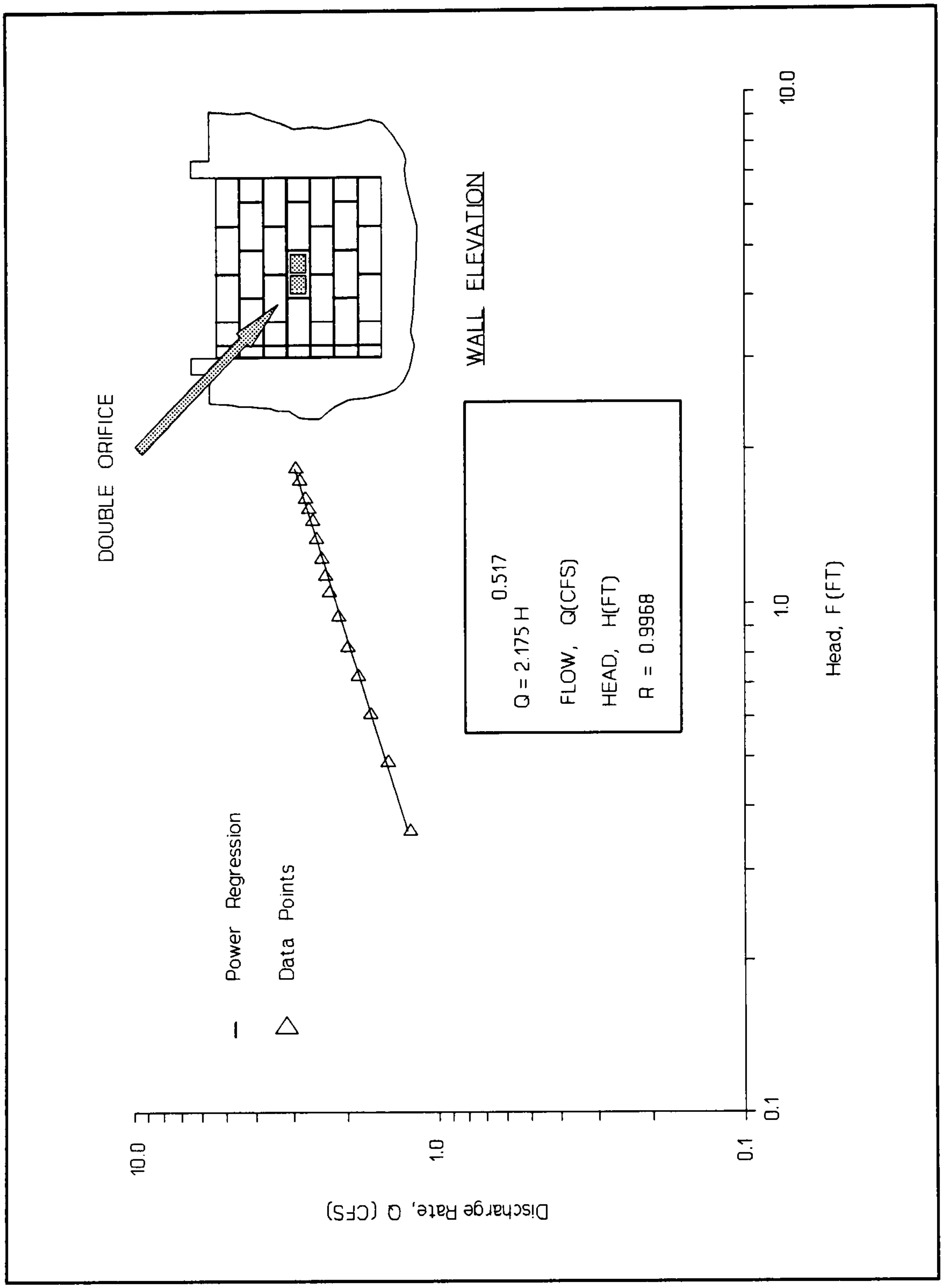


FIGURE 12

Double Orifice Opening (Test 2)

TABLE 6

Laboratory Test Data
Single Orifice Opening Elevated Channel Bottom (Test 3)
(Date of Test - April 12, 1991)
(Single Orifice Area = 0.205 Ft^2)

DATA POINT	HEAD H(ft)	FLOW Q_a (cfs)	FLOW Q_a (cfs)	$C_d = Q_a/Q_i$
1	2.747	1.748	2.722	0.642
2	2.497	1.677	2.595	0.646
3	2.367	1.635	2.527	0.647
4	2.412	1.590	2.495	0.647
5	2.117	1.538	2.390	0.644
6	1.972	1.492	2.306	0.647
7	1.847	1.435	2.232	0.643
8	1.717	1.391	2.152	0.647
9	1.582	1.315	2.066	0.637
10	1.427	1.251	1.962	0.638
11	1.257	1.166	1.841	0.634
12	1.097	1.111	1.720	0.646
13	0.927	1.022	1.581	0.647
14	0.722	0.892	1.395	0.640
15	0.582	0.801	1.253	0.639
16	0.357	0.621	0.981	0.633

for Test 3. Figure 13 shows the data points plotted along with the power regression line and regression equation. For a photograph of this layout see Appendix E.

Double Orifice Opening Elevated Channel Bottom Test (4) - During this test the full capability of the pumping system was used. Table 7 and Appendix C show the data gathered and the evaluations made for Test 4. Figure 14 shows the data points plotted along with the power regression line and regression equation.

Discussion of Orifice Only Tests - Table 8 summarizes the results of the orifice only laboratory tests performed. The C_d values shown in Table 8 were calculated based on the ratio of the actual orifice flow rate to the ideal orifice flow rate Q_a/Q_i as developed in Section II.

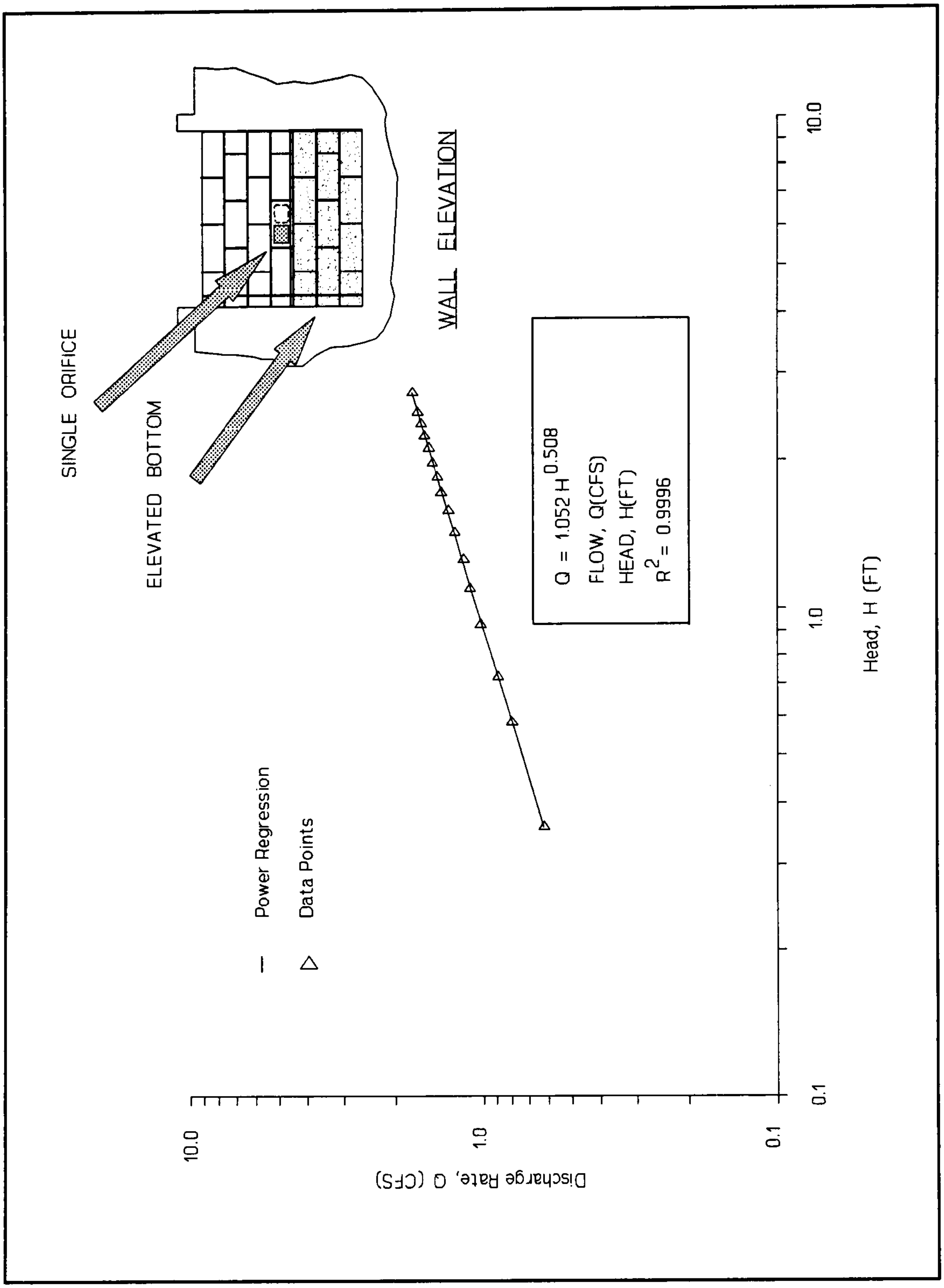


FIGURE 13

Single Orifice Opening Elevated Channel Bottom (Test 3)

TABLE 7

Laboratory Test Data
Double Orifice Opening Elevated Channel Bottom (Test 4)
(Date of Test - November 5, 1990)
(Double Orifice Area = 0.409 Ft^2)

DATA POINT	HEAD H(ft)	FLOW Q_a (cfs)	FLOW Q_t (cfs)	$Cd = Q_a/Q_t$
1	2.017	2.896	4.661	0.621
2	1.892	2.821	4.515	0.625
3	1.837	2.751	4.449	0.618
4	1.752	2.692	4.344	0.619
5	1.647	2.609	4.212	0.619
6	1.567	2.555	4.109	0.620
7	1.432	2.426	3.928	0.617
8	1.292	2.303	3.731	0.617
9	1.167	2.232	3.546	0.629
10	1.111	2.118	3.461	0.612
11	1.022	2.039	3.318	0.614
12	0.897	1.915	3.109	0.616
13	0.752	1.731	2.846	0.608
14	0.627	1.562	2.599	0.601
15	0.467	1.342	2.243	0.598
16	0.297	1.094	1.789	0.612

Since orifice flow can be effected by the viscosity of the fluid flowing through the orifice, one way to compare the performance of orifice openings is through the use of the Reynolds Number. Depending on the size of orifice opening being evaluated and the fluid being used, the effects of viscosity forces relative to inertia forces can be represented as

$$N_r = \frac{VL}{\nu} \qquad (20)$$

where N_r = Reynolds Number; V = Velocity at Section (ft/s); L = Characteristic Length (ft); and ν = Kinematic Viscosity of Water (1.08×10^{-5} ft^2/s). For evaluating an

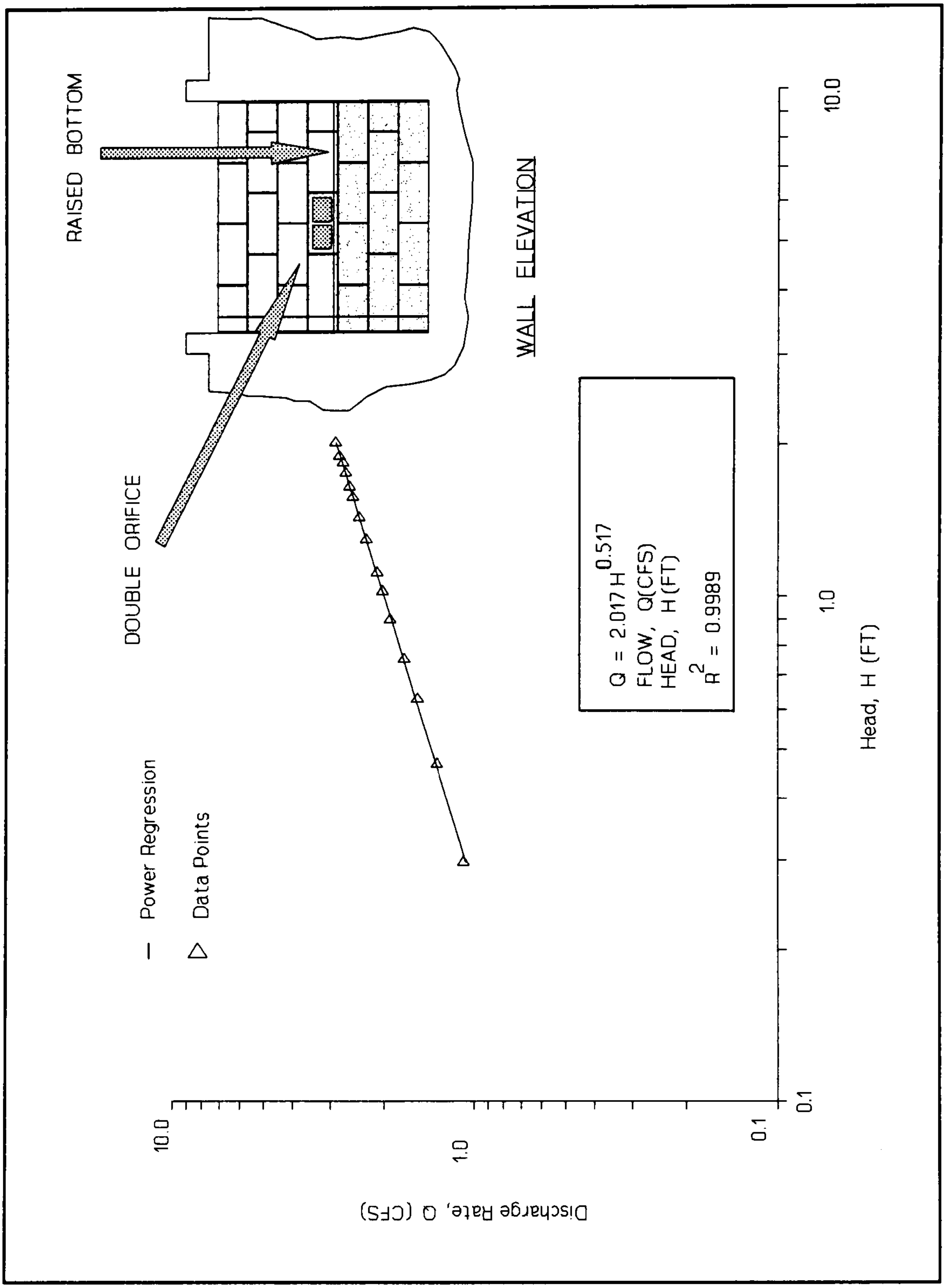


FIGURE 14

Double Orifice Opening Elevated Channel Bottom (Test 4)

TABLE 8

Laboratory Determined Coefficients of Discharge
Orifice Only Tests

Orifice Head (H)	Test 1 Single Orifice	Test 2 Double Orifice	Test 3 Single Orifice Flush Bottom	Test 4 Double Orifice Flush Bottom
H=1.00 FT	0.677	0.679	0.646	0.614
H=1.50 FT	0.674	0.659	0.637	0.620
H=2.00 FT	0.668	0.664	0.647	0.621
H=2.50 FT	0.669	-----	0.646	-----
H=3.00 FT	0.671	-----	0.642	-----

orifice opening the characteristic length is taken as the Hydraulic Radius of the individual orifice opening (6).

At Reynolds Numbers greater than approximately 4.0×10^4 orifice C_d values are essentially a constant value (13). Figure 15 shows a plot of the data gathered from Tests 1 and 2 for an unrestricted orifice openings formed with concrete block. The mean value for C_d was found to be <u>0.67</u> with a standard deviation of 0.010. Figure 16 shows a plot of the data gathered from Tests 3 and 4 for an orifice opening with a suppressed contraction at the orifice invert elevation caused by the elevated channel bottom. The mean C_d value for this configuration was found to be <u>0.63</u> with a standard deviation of 0.013.

The geometry of a concrete block orifice can approaches that of a short tube as shown in Figure 2C. With a short tube there is a predicted improvement in flow performance over a thin plate orifice. The standard short tube has a C_d of 0.82 (13). The individual concrete block orifice opening tested had a ratio of length to mean

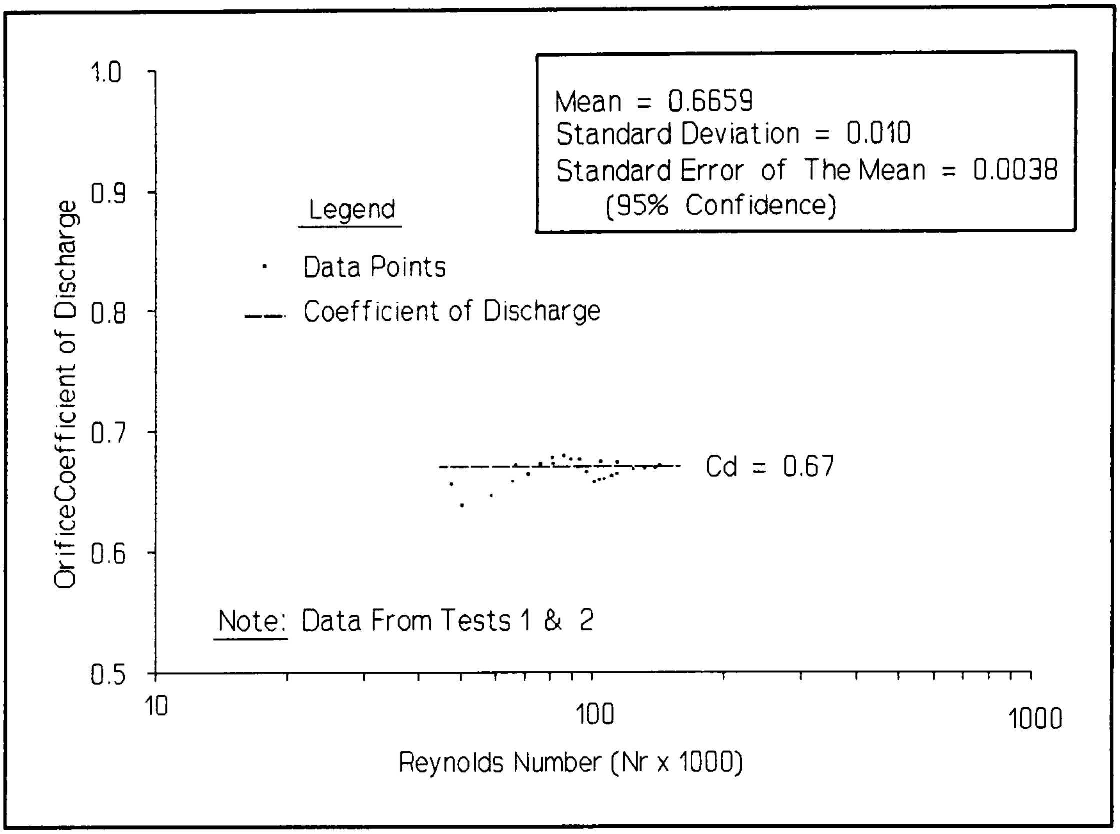


FIGURE 15

Coefficient of Discharge Determination for One and Two Orifice Openings (Test 1 & 2)

opening diameter of approximately 1.3. Schoder et al. (24) predicted a C_d of 0.78 for an orifice of this configuration while King (13) predicts a C_d of 0.80 for the same configuration. In direct contradiction to Merritt (18) and Schoder (24), the introduction of an elevated channel bottom at the invert elevation of the orifice appears to reduce, not increase, C_d. This reduction in C_d indicates that the degree of roughness and localized turbulence associated with the full scale experiment may have more impact on C_d than the suppression of the vena contracta at the bottom of the channel. It should be noted that with the opening size being evaluated King (13) did not predict

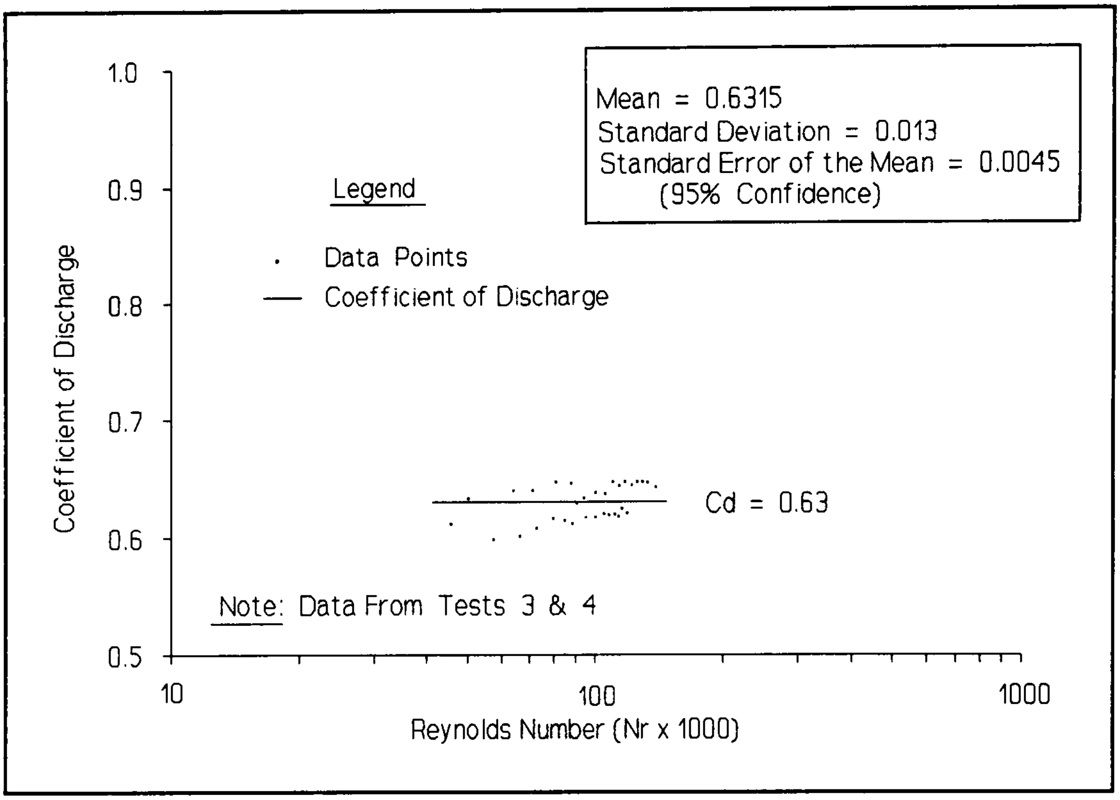


FIGURE 16

Coefficient of Discharge Determination for One and Two Orifice Openings With Elevated Channel Bottom (Test 3 & 4)

an increase in performance from a suppressed contraction at the invert elevation of the orifice.

WEIR ONLY COEFFICIENTS

Three weir only tests were conducted in order to determine the coefficients of discharge for various configurations of weir openings discussed in Section III. For all tests containing weir openings, the weir head was measured from the lowest point of the weir opening to the free water surface. As noted in Section III, the water surface measurements were made sufficiently back from the block wall to ensure that the head readings were not effected by changes in flow patters as the water moves

TABLE 9

Laboratory Test Data
8" Wide x 16" High Weir Opening (Test 10)
(Date of Test - April 30, 1991)
(Weir Only Opening - 8.0 in. Wide & 16 in. Deep)

DATA POINT	HEAD H(ft)	FLOW Q_a (cfs)	FLOW Q_t (cfs)	$C_w = Q_a/Q_t * 2/3 * (2g)^{1/2}$
1	1.230	2.979	4.864	3.275
2	1.205	2.923	4.716	3.314
3	1.200	2.892	4.687	3.301
4	1.190	2.848	4.629	3.291
5	1.175	2.803	4.541	3.301
6	1.160	2.743	4.455	3.294
7	1.150	2.693	4.397	3.276
8	1.135	2.616	4.311	3.245
9	1.120	2.540	4.226	3.214
10	1.095	2.450	4.086	3.207
11	1.075	2.369	3.974	3.188
12	1.045	2.290	3.809	3.215
13	1.020	2.198	3.673	3.201
14	1.000	2.117	3.566	3.175
15	0.975	2.028	3.433	3.159
16	0.950	1.934	3.301	3.134
17	0.915	1.821	3.121	3.120
18	0.880	1.711	2.943	3.108
19	0.850	1.624	2.794	3.108
20	0.800	1.471	2.551	3.084
21	0.750	1.337	2.316	3.087
22	0.670	1.092	1.955	2.986
23	0.630	1.001	1.783	3.002
24	0.580	0.891	1.575	3.025
25	0.540	0.802	1.415	3.031
26	0.505	0.689	1.280	2.878
27	0.440	0.556	1.041	2.856
28	0.385	0.468	0.852	2.939
29	0.340	0.385	0.707	2.915

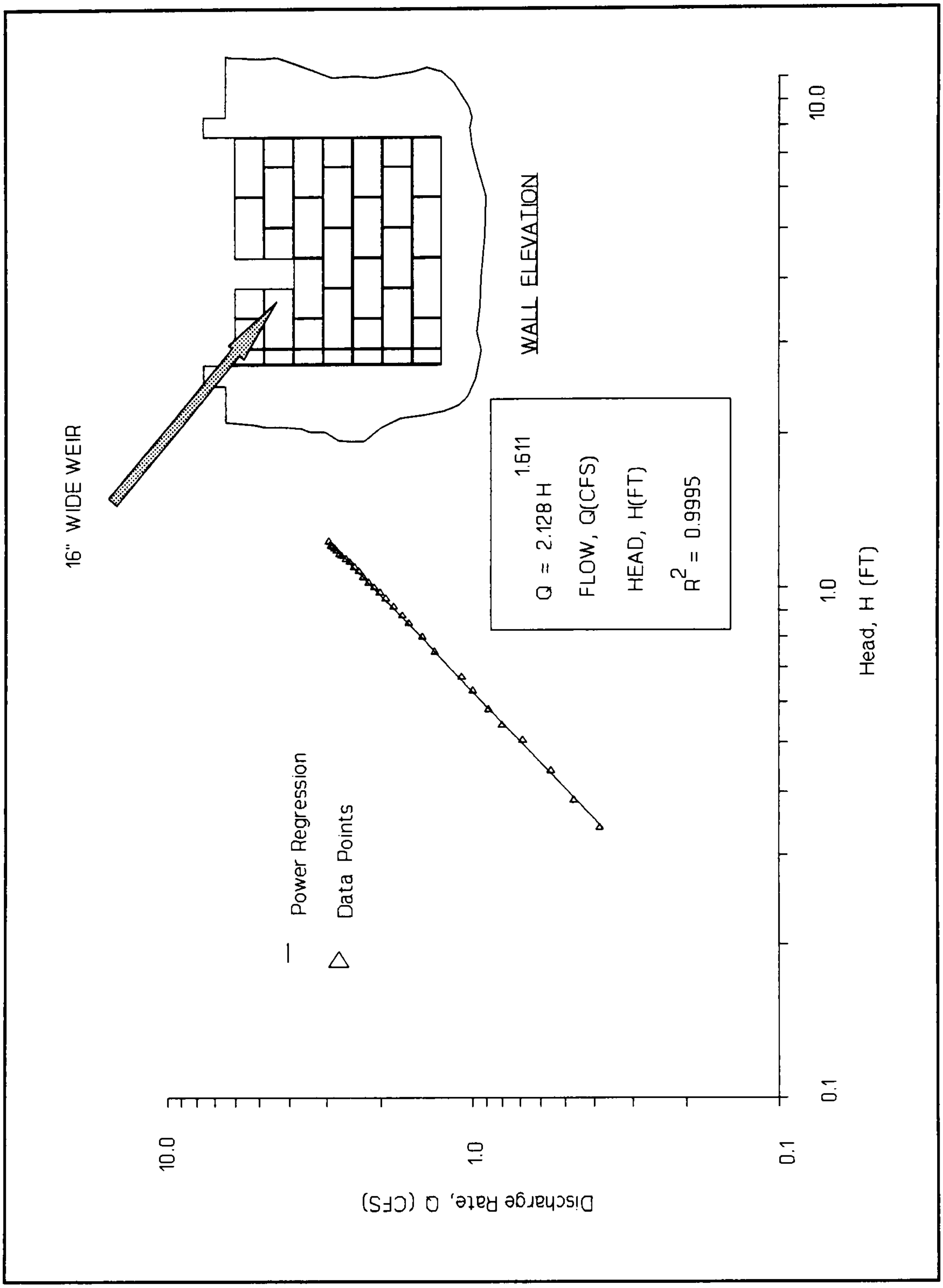


FIGURE 17

8" Wide x 16" High Weir Opening (Test 10)

through the concrete block weir. The flow rates shown in each table have been corrected for leak rate shown in Figure 8. The C_w values shown in all tables have been calculated, as discussed Section II, using the relationship Q_a/Q_i.

8" Wide x 16" High Weir (Test 10) - The head achieved in this test was the maximum for any weir tested in this program. Table 9 and Appendix C show the data gathered for Test 10. The reader is referred to Appendix E for photographs of this test setup and photographs taken during the actual testing process. Figure 17 shows the data points plotted along with the power regression line and equation.

16" Wide x 16" High Weir (Test 8) - There were two versions of this test performed. Test 7 included a raised channel bottom as shown in Appendix E; Test 8 did not include an elevated channel bottom. The purpose of Test 7 was to ascertain the impact on weir performance of an elevated channel bottom. After analyzing the date from Tests 7 and 8 it was apparat that, within the accuracy limits of this study shown in Appendix F, the raised channel bottom had no impact on weir only performance, and for this reason additional tests on the 16" wide and 24" wide weirs with elevated channel bottom were not performed. The reader is directed to Appendix E for a photographs of the Test 8 configuration. Table 10 shows the data gathered for Test 8. Figure 18 shows the data points plotted along with the power regression line and equation.

24" Wide x 8" High Weir (Test 12) - Due to the width of this weir and the limitations of the water supply, the maximum head achieved on the test was 0.618 feet. Table 11 and Appendix C show the data gathered for Test 12. Figure 19 shows the data points plotted along with the power regression line, and Appendix E contains photographs of the weir in operation.

TABLE 10

Laboratory Test Data
16" Wide x 16" High Weir Opening (Test 8)
(Date of Test - April 26, 1991)
(Weir Only Opening - 16 1/8 in. Wide & 16 in. Deep)

DATA POINT	HEAD H(ft)	FLOW Q_a (cfs)	FLOW Q_t (cfs)	$C_w = Q_a/Q_t * 2/3 * (2g)^{1/2}$
1	0.800	3.001	5.142	3.122
2	0.795	2.978	5.094	3.127
3	0.780	2.881	4.951	3.112
4	0.775	2.811	4.903	3.067
5	0.755	2.730	4.715	3.097
6	0.745	2.638	4.621	3.053
7	0.735	2.562	4.529	3.062
8	0.705	2.428	4.254	3.052
9	0.690	2.337	4.119	3.034
10	0.675	2.234	3.986	2.997
11	0.650	2.125	3.766	3.018
12	0.620	1.982	3.509	3.021
13	0.600	1.854	3.340	2.968
14	0.565	1.682	3.052	2.947
15	0.530	1.536	2.773	2.963
16	0.480	1.340	2.390	2.998
17	0.440	1.169	2.098	2.981
18	0.400	1.009	1.818	2.969
19	0.345	0.809	1.456	2.972
20	0.275	0.562	1.036	2.900
21	0.225	0.412	0.767	2.874

Discussion of Weir Only Tests - Table 12 summarizes the laboratory test results for the performance of 8", 16", and 24" wide broad-crested weirs formed with concrete block and evaluated in this study. Figure 20 shows a plot of the weir only test C_w values developed in this study. Figure 20 was used to compare the experimentally developed concrete block broad-crested C_w values with the C_w values predicted for broad-crested weirs by King (13). The comparison was made by plotting the difference between the laboratory C_w value and the predicted C_w value against the weir head. As indicated in Figure 20, the Mean difference was -0.032, the Standard

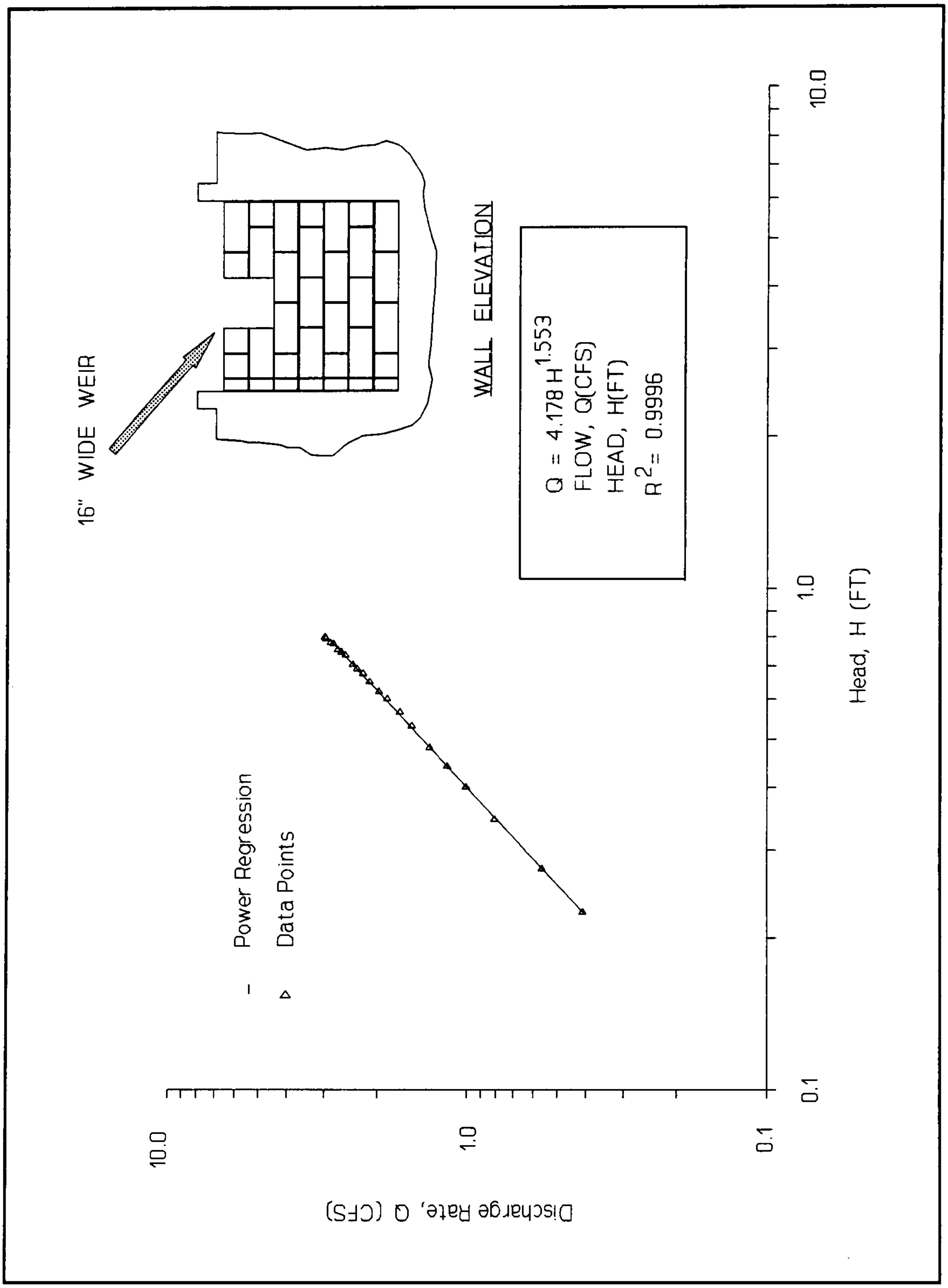


FIGURE 18

16" Wide X 16" High Weir Opening (Test 8)

TABLE 11

Laboratory Test Data
24" Wide x 8" High Weir Opening (Test 12)
(Date of Test - May 5, 1991)
(Weir Only Opening - 23 1/2 in. Wide x 8 in. Deep)

DATA POINT	HEAD H(ft)	FLOW Q_a (cfs)	FLOW Q_t (cfs)	$C_w = Q_a/Q_t * 2/3 * (2g)^{1/2}$
1	0.618	2.977	5.088	3.129
2	0.613	2.947	5.027	3.135
3	0.608	2.848	4.965	3.068
4	0.603	2.793	4.904	3.046
5	0.588	2.704	4.772	3.062
6	0.573	2.615	4.543	3.079
7	0.563	2.496	4.424	3.018
8	0.548	2.363	4.249	2.975
9	0.528	2.215	4.018	2.947
10	0.508	2.054	3.792	2.897
11	0.478	1.901	3.461	2.938
12	0.463	1.812	3.300	2.938
13	0.433	1.644	2.984	2.947
14	0.403	1.457	2.679	2.908
15	0.358	1.228	2.243	2.928
16	0.313	0.997	1.834	2.908
17	0.273	0.822	1.494	2.942

Deviation was 0.059, and the Standard error of the Mean was 0.014 for a 95% confidence level. This close agreement confirms two things about concrete block weir only performance.

1. For the nominal 8" concrete blocks used in this study, sufficient accuracy can be attained for weir performance using the broad-crested weir C_w values tabulated by King.

2. The basic derived weir equation that relates C_w to flow rate and head was developed in Equation 13 and expanded on in Equation 14. Based on the laboratory data gathered in this study, as depicted in Figure 20, and a review

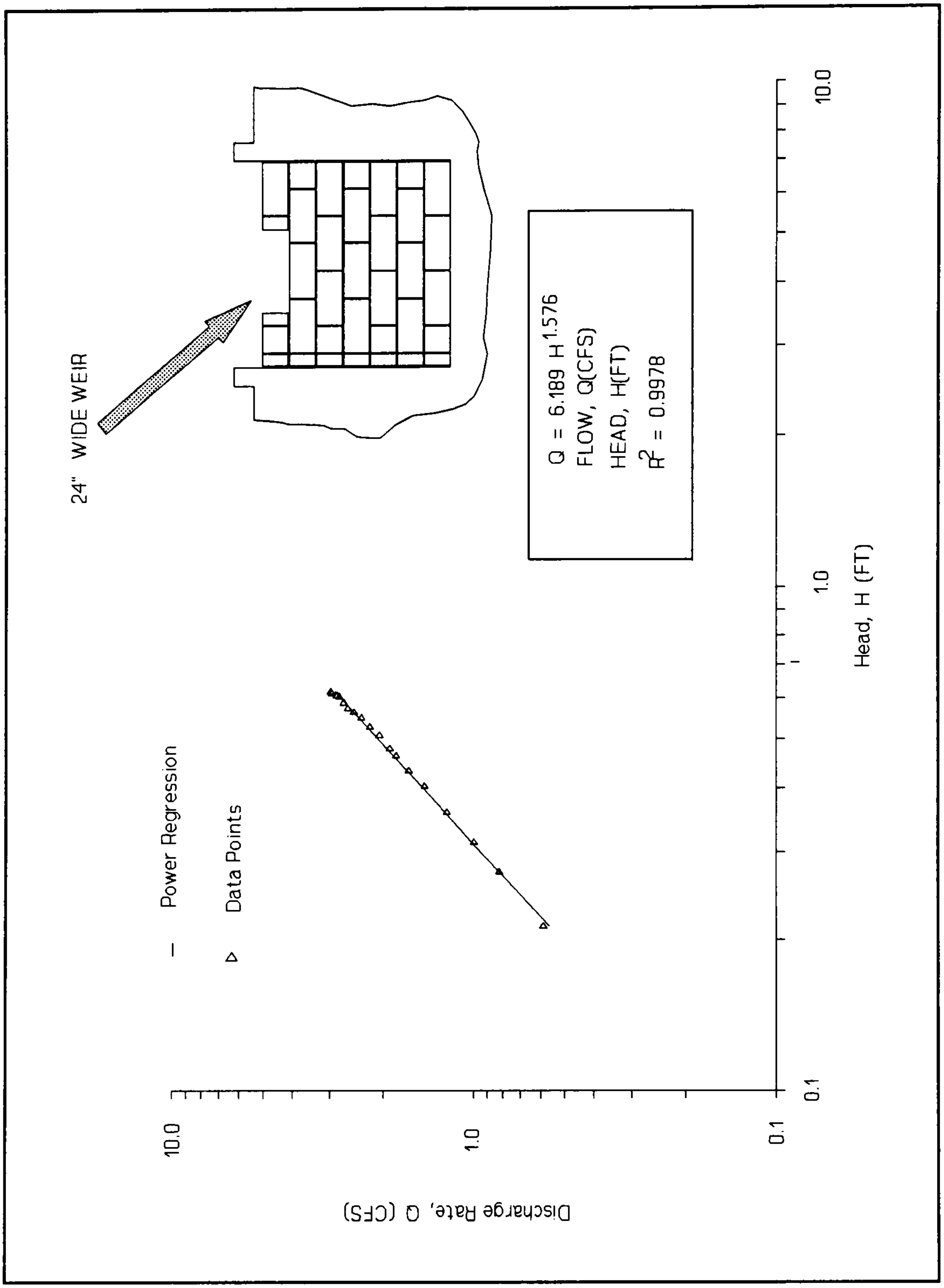


FIGURE 19

24" Wide x 8" High Weir Opening (Test 12)

TABLE 12

Laboratory Determined Coefficients of Discharge and Comparison With Handbook Values for Weir Only Tests

Weir Head (H)	Test 10 8" Wide Weir	Test 8 16" Wide Weir	Test 12 24" Wide Weir	King Broad-crested Weir
H=0.62 FT	3.002	3.021	3.129	2.971
H=0.80 FT	3.084	3.122	-----	3.127
H=1.23 FT	3.275	-----	-----	3.246

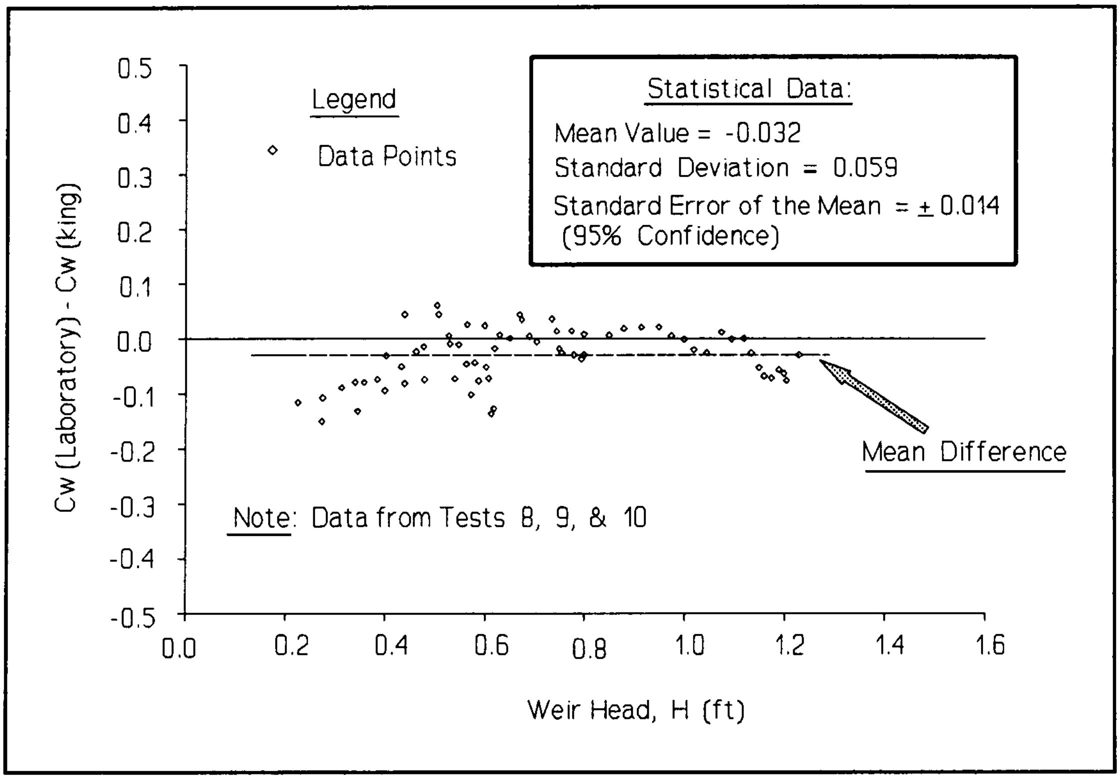


FIGURE 20

Comparison of Weir Coefficients of Discharge C_w Between Experimental and Predicted Values for Broad-Crested Weirs Constructed Using Concrete Block of literature, the reduction in flow rate predicted for end contractions (i.e., 0.1nH in Equation 14) can be neglected for broad-crested weirs.

STACKED-WEIR EVALUATION

Tests 9 and 11 were stacked-weir tests as described in Section III and shown in Figures 21 and 22. These two tests were accomplished in order to evaluate the performance characteristics of stacked-weirs, and develop a methodology for predicting the flow performance of stacked-weirs.

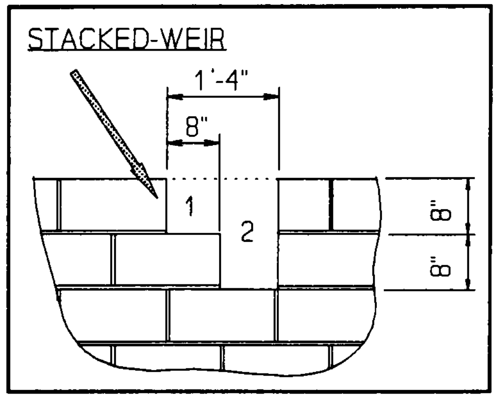


FIGURE 21

Stacked-weir Test 9

Based on the results of this study, flow through a stacked-weir arrangement can be determined by using a method of *vertical strips*. Test 9, shown in Figure 21, and Test 11, shown in Figure 22, depict discrete strips "1" and "2" for segmenting the total flow over the stacked-weirs.

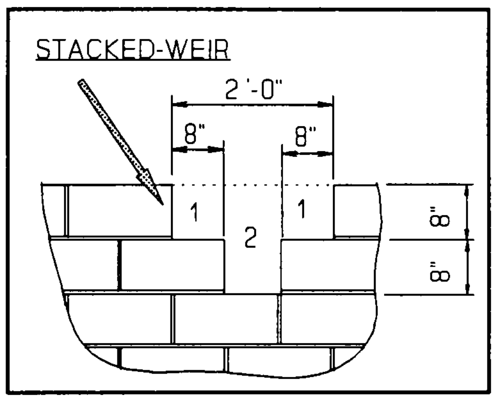


FIGURE 22

Stacked-weir Test 11

The laboratory results of Test 9 and 11 were evaluated using the power regression equations developed in Tests 8 and 10. To clarify, the full height 8" vertical "2" strip of Tests 9 and 11 shown in Figures 21 an 22 was evaluated using the power regression equation from Test 10, shown in figure 17. The particle height vertical strip "1" of Tests 9 and 11 were evaluated using the same power regression equation. When applying the Test 10 power regression equation to each vertical strip the corresponding head for the section being evaluated was used for each vertical strip. Using this method of evaluation, the predicted discharge for Test 9 was under estimated by 1.69% with a Standard Deviation of 1.52%. Using the same method, along with the power regression equations from Tests 8 and 10, Test 11 produced an under estimation of 1.33% with a Standard Deviation of 2.02%. Both of these differences are within the combined uncertainty limitations of Tests 8 and 10 which is approximately 2.0% as shown in Appendix F.

WEIR/ORIFICE EVALUATION

Figure 23 shows the weir/orifice configuration that was evaluated in Test 5. Test 6 included the addition of a elevated channel bottom at the invert elevation of the orifice as shown in Appendix E. ASCE identified this type of discharge arrangement consisting of multiple discharge devices, often used on multiple-stage outlet structures, as one needing additional research (26).

Tests 5 and 6 were evaluated using the power regression equations developed for the appropriated components of the configuration tested. Test 5, shown in Figure 23, was evaluated using the power regression equations developed in Test 1, for a single orifice opening, and the power regression

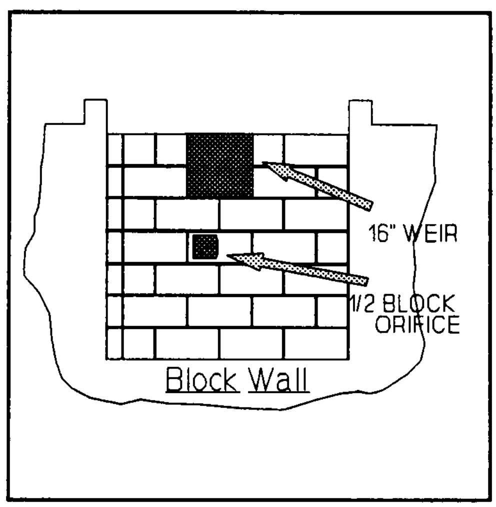


FIGURE 23

Weir/Orifice Configuration

equation developed in Test 8 for a 16" wide weir; Test 6, the same as Figure 23 with an elevated channel bottom added, used the regression equation developed in Test 3, for a single orifice opening with elevated channel bottom, and the regression equation developed in Test 7 for a 16" wide weir with an elevated channel bottom.

In Test 5 the predicted flow was 2.15% greater than the actual flow with a Standard Deviation of 0.61%; the Test 6 predicted flow was 0.24% less than the actual flow with a Standard Deviation of the Mean of 0.34%. Both of these differences are within the combined uncertainty limitations of tests used which is approximately 2.0% as shown in Appendix F.

SECTION V

DISCUSSION AND APPLICATION

DISCUSSION

In the design and construction of urban Stormwater Detention Facilities the stage-storage relation and the stage-discharge relation represent the primary components of the Stormwater Detention Facilities often used for urban flood control (16). To establish the stage-discharge relation it is necessary to have information on the C_d for the various components of the Stormwater Detention Facility discharge structure. Use of the information presented in this thesis will allow the design professional to establish the correct stage-discharge relation for a concrete block discharge structure.

The sensitivity of detention pond design storage to weir and orifice discharge coefficients for typical urban drainage areas, where use could be made of a concrete block discharge structures, was assessed and the results of this assessment are shown in Appendix D. Figure 24 shows a plot of the relationships between orifice coefficient of discharge C_d, discharge rate (cfs), and detention pond volume in acre feet (AcFt) for a typical urban drainage area (20 acres; SCS Composite Curve Number, 92; time of concentration, 0.3 hr; 25 year storm) evaluated with TR-55 (30) and HEC-1 (11). The procedure used by TR-55 is approximate and does not consider the geometry of the pond being designed, nor the configuration of the discharge structure, in choosing the required pond storage volume. In using TR-55 the required pond volume depends on the ratio of peak outflow to peak inflow (q_o/q_i) for a given SCS storm event. Pond volume is effected by the C_d selection. Curve A, Figure 24,

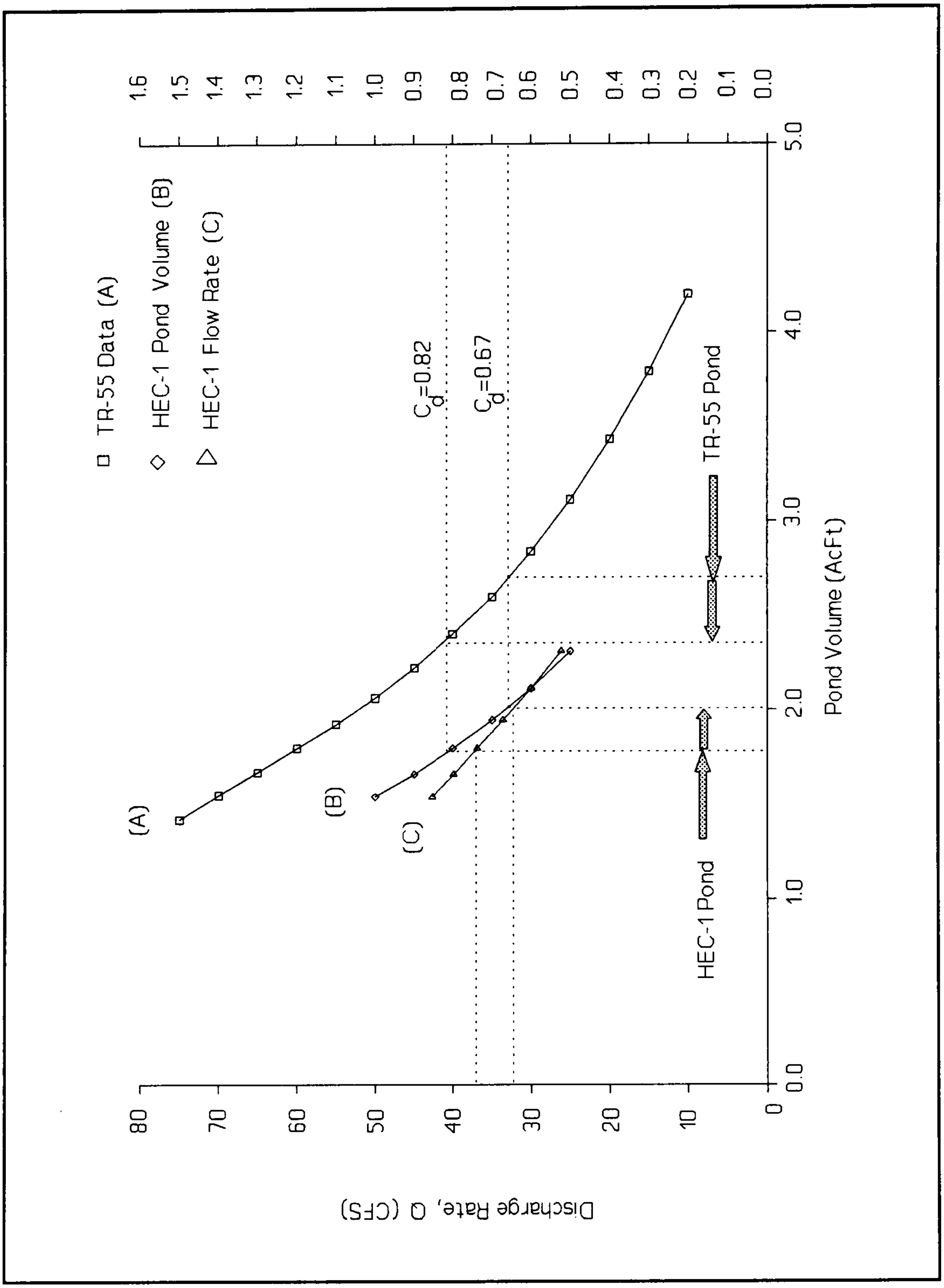


FIGURE 24

TR-55 & HEC-1 Pond Design Comparison for Assumed Orifice Discharge Coefficients

shows how sensitive TR-55 calculated pond volume is to proper C_d selection. In the typical urban drainage area, using a C_d of 0.82, which is typical for a standard short tube will cause detention pond volume to be underestimated by 12.6% when compared to the pond volume estimated by TR-55 using the experimentally-determined C_d value for elevated orifices of 0.67 (13,24). However, when designing by TR-55 this 12.6% difference may not be significant since TR-55 is only considered to be accurate for a range of 25%, and for this reason TR-55 should only be used for preliminary design (30).

The HEC-1 calculated detention pond volume Curve B and discharge rate Curve C plots in Figure 24 show the impact of the detention pond stage-storage relationships and discharge structure stage-discharge relationships on the design volume of detention storage and design pond discharge rates. For the same urban drainage area described earlier, the HEC-1 volume required by storage routing is approximately 25% less than the TR-55 design volume. The relationship between the two HEC-1 curves depends on the geometry of the detention pond and discharge structure being evaluated. Using a C_d of 0.82 causes HEC-1 designed detention pond volume to be underestimated by 13.6% when compared to the pond volume calculated by HEC-1 using the test developed C_d of 0.67. Design using any method that routes storm flow through a detention pond (6), such as HEC-1, requires the use of accurate stage-discharge and stage-storage information to ensure correct and cost effective design. Satisfactory stage- discharge relationships can only be achieved using the proper C_d values.

APPLICATION

For design purposes an accurate measurement of the orifice opening area produced by concrete block is needed prior to commencing detailed discharge structure design. Once the block geometry has been determined, the weir and orifice C_d values provided in Figure 15, Figure 16, and King (13) can be used for design. All concrete blocks are molded such that the top and bottom openings are different sizes. This configuration is to facilitate removal of the completed concrete block from the forms used in the manufacturing process. When placing a concrete block in a wall, it is important for the contractor to ensure that the larger opening is on the upstream face of the wall. Concrete block orifices are operating in a condition of inlet-control for all heads used in urban discharge structures, and for this reason the amount of water that can enter the orifice is directly related to the area of the orifice opening. Figure 6 shows the configuration of the concrete block used in this laboratory study. The orifice opening should be located above the pond bottom (e.g., 16") to allow for some sediment and water borne debris storage. Water trapped below the orifice will easily dewater itself by seeping through the concrete block wall(s) after a storm event as shown in Equation 18. If a more positive dewatering method is desired, a small perforated pipe wrapped with geotextile filter fabric and encapsulated in free-draining stone may be placed in a trench below the lowest elevation of the detention pond so that the pond will completely dewater after a storm event. The designer is advised to provide some type of trash rack(s) to prevent debris from blocking orifice opening(s) (26).

Discharge structures need to be inspected by the hydraulic engineer during construction to ensure that the design intent is being followed. Maintenance and inspection of the structure is simplified by installing the top grating, see Figure 1, in sections for easy removal. It is recommended that all concrete block structures have reinforcing steel installed to ensure structural adequacy. At a minimum, discharge structure walls should have horizontal joint reinforcing every other course, a continuous top bond beam (minimum of 2-#5 bars), and vertical steel (minimum of 1-#5 bar) at each corner doweled into the foundation and lapped into the bond beam.

With the increased use of detention ponds for urban stormwater management, it is recommended that the design engineer consider rigorous storage routing methods, such as provided in HEC-1, to prevent over-designed pond volumes (2). The amount of increased design time spent on more rigorous routing methods may be cost effective, resulting in more reasonably sized ponds, and less expensive development area dedicated to stormwater management. Whenever possible, the designer should consider the use of deep, rather than shallow, ponds for detention. Deep ponds allow higher heads that require smaller discharge structures and take up less land space. Site locations and safety considerations will often constrain optimum pond configuration, but it is incumbent on the design professional to take advantage of the information and design methods available to ensure efficient and cost effective Stormwater Detention Facilities.

CONCLUSIONS AND RECOMMENDATIONS

Concrete block discharge structures are an efficient and effective way to deal with the discharge from urban detention ponds. These structures are more attractive than most of the discharge devices now in use in this country, and they make an excellent addition to any Stormwater Detention Facility.

Further study of orifice openings formed with concrete block needs to be accomplished in order to make this potentially practical discharge structure configuration more useful to the engineering and construction industry. This thesis dealt with only one type of concrete block orifice opening. The blocks used in this study were placed such that the largest block opening faced upstream. How would the coefficient of discharge change if the smaller area was turned upstream? What would be the coefficient of discharge if the orifice opening was the same size at each end? It would be valuable to peruse the obtaining of answers to these questions in future laboratory study.

During this laboratory study it was determined that the presence of a rough elevated channel bottom at the invert elevation of the orifice decreased the value of C_d. More study is needed to isolate the factors that lead to this decrease in C_d. How large an impact does the texture of the channel bottom have on the value of C_d? How does the relationship between the orifice invert and the channel bottom impact the value C_d? Knowledge in these two areas could be of value in the design of Stormwater Detention Facilities made from materials other than concrete block.

Although not directly evaluated in this laboratory study, one area that is in need of much more study is the design and construction of cost effective and efficient trash racks. Coefficients of discharge that have been carefully determined in the laboratory are of little value if the first piece of debris that enters the detention pond moves to block the outflow and render the discharge device totally ineffective.

This thesis validates the use of the broad-crested weir coefficients provided by King (13). Additional study needs to be undertaken in the area of slots formed with concrete block or reinforced concrete. This type of device is often used for the long term draw-down of detention ponds. Often government agencies have requirements for how much time it should take to drain a pond after a specified storm event. Increased knowledge of the performam of these slots could assist the designer in complying with these regulations. As with an orifice, slots are very susceptible to clogging by debris. Recommendations on how to provide protection from this clogging is needed in order to make effective use of slots in the construction of discharge structures,

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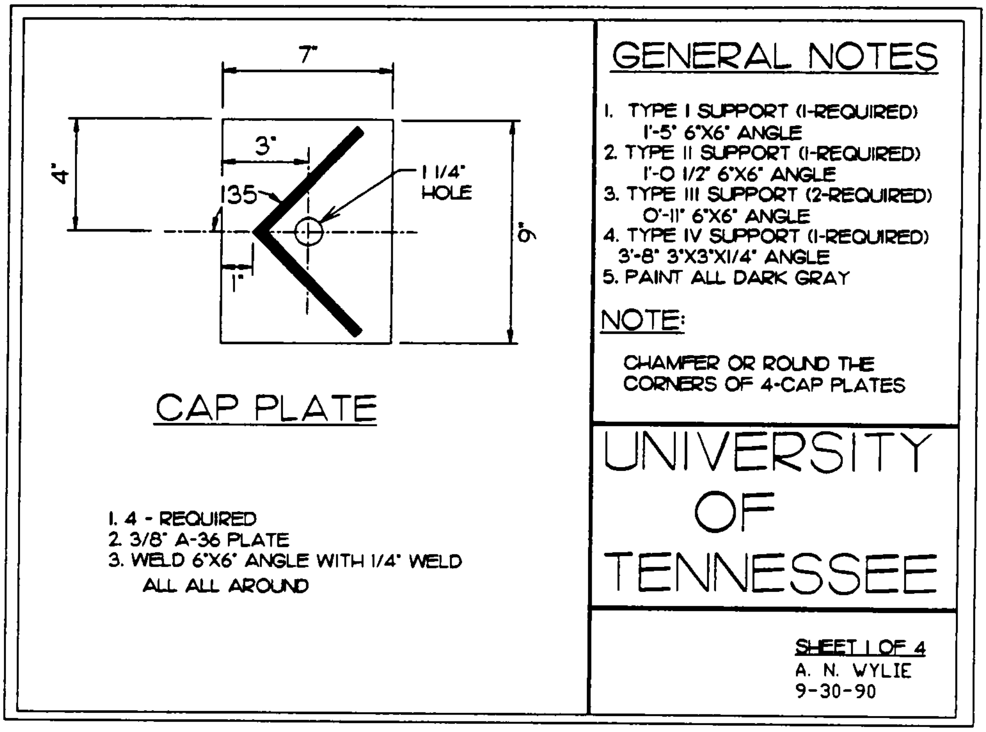
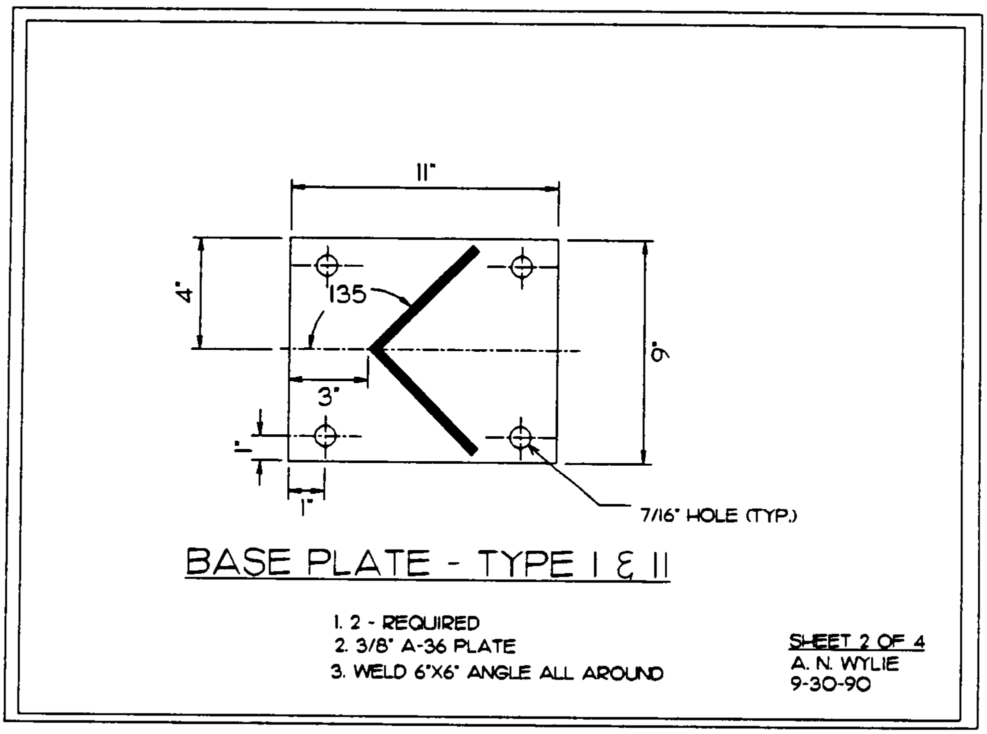
APPENDICES

APPENDIX A

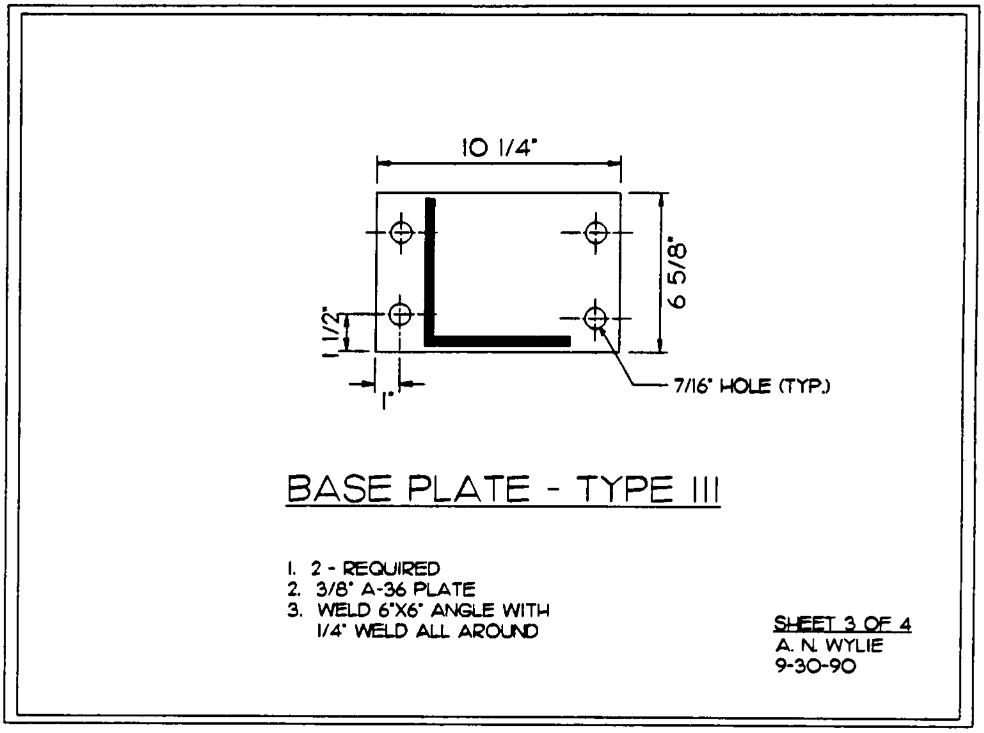
STEEL FABRICATION SHOP DRAWINGS

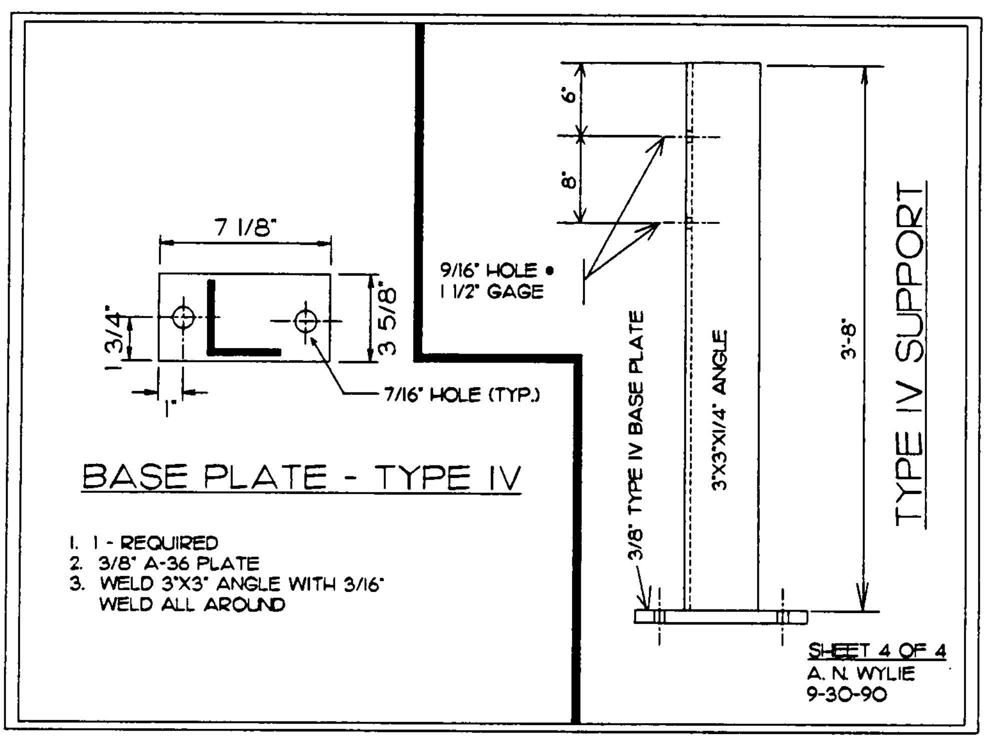
All miscellaneous steel used for this project was fabricated from ASTM A-36 by Towe Iron Works, Knoxville, Tennessee. Field measurements were made in order to prepare the shop drawings. All components were fabricated with over sized holes and approximately 1/8" shorter than the field measurements indicated. This was done to insure that any errors would be of such a nature that no field fabrication (cutting) would be required. No field problems were encountered in the assembly of the frames and supports.





Pipe Support Fabrication Details (Sheet 1 of 2)





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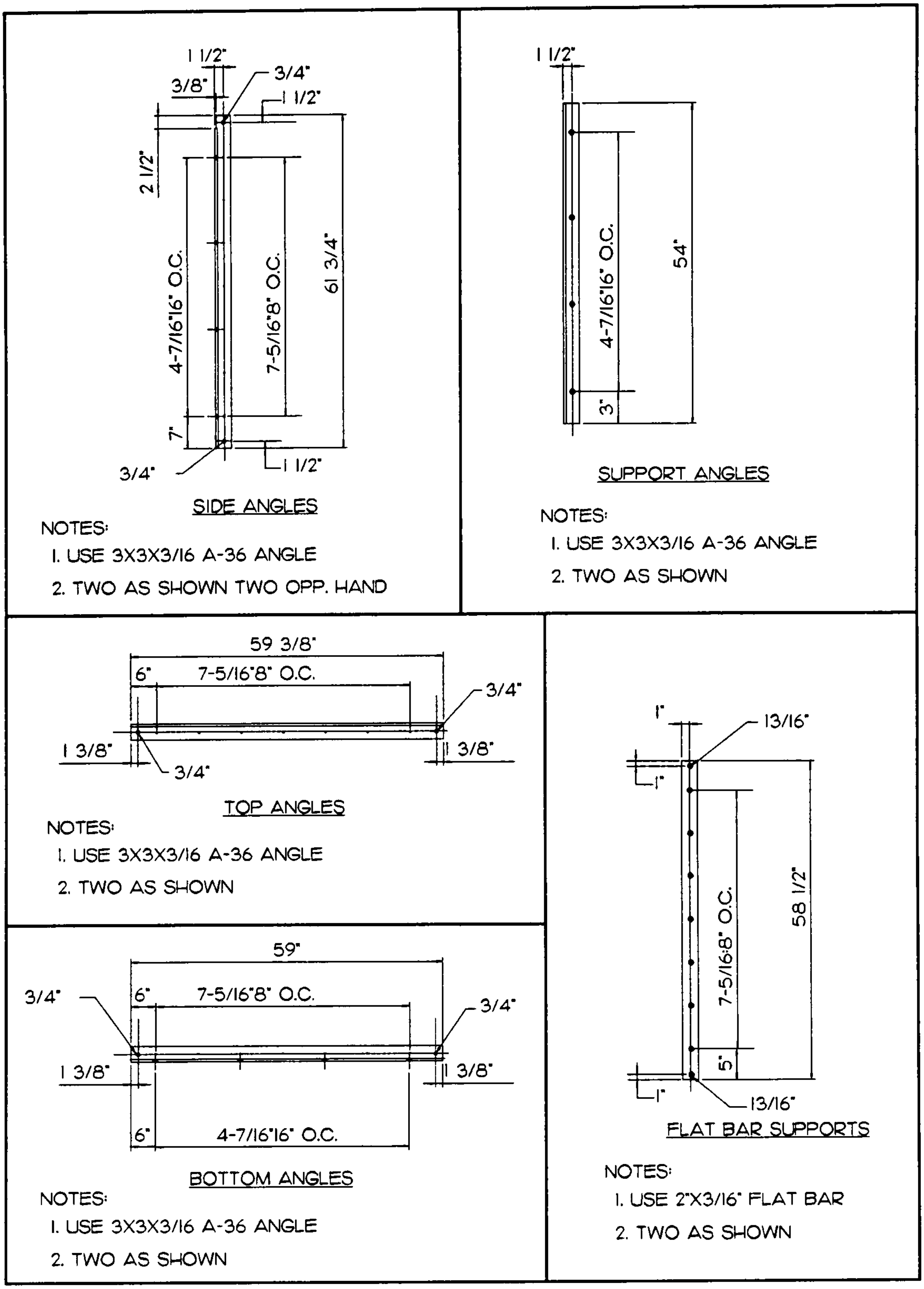

Diffuser Shop Drawing

APPENDIX B

DETAILED TESTING PROGRAM PROCEDURES

Due to the amount of construction required to configure for each test it was prudent to develop a sequence of testing that minimized duplication of effort. Particularly labor intensive items were as follows:

Elevated Channel Bottom - To configure this element of a test it was necessary to install a 6' x 4' sheet of plywood and approximately 40 concrete block in the reservoir trench. This installation could be carried out "in the dry" if the reservoir was drained. In most cases the draining process could not be accomplished due to laboratory schedule conflicts. In these cases the elevated channel bottom was installed standing in two feet of water.

Weir Changes - Each time the weir shape changed concrete block had to be removed and mortared into the new configuration. Usually water remained in the trench complicating the process.

Orifice Changes - The tests that required two orifice openings were accomplished first (Test 2 and Test 4). After they were completed one orifice openings was mortared closed for the remainder of the testing program. The remaining orifice opening was closed with a removable plywood plug that was installed or removed as appropriate for test run being accomplished (Appendix E).

The numbering system used for the detail procedure utilized a two number field for test identification, e.g., Run 4-1. The first number corresponds to the test numbering in Table 3, e.g., Test 4. The second number identifies the sequence in which the tests were run. In this example Run 4-1 was Test 4 and it was the first test run.

INDIVIDUAL TEST DESCRIPTION

October 21, 1990 - Revised 4/22/91

Note: A series of data runs were made for each test indicated below.

Test No.	Test Description
1-3.	ORIFICE ONLY - 1 OPENING
2-2.	ORIFICE ONLY - 2 OPENINGS
3-4.	ORIFICE ONLY - 1 OPENING - ELEVATED CHANNEL BOTTOM
4-1.	ORIFICE ONLY - 2 OPENINGS - ELEVATED CHANNEL BOTTOM
5-6.	16" x 16" WEIR & 1 ORIFICE OPENING
6-5.	16" x 16" WEIR & 1 ORIFICE OPENING-ELEVATED CHANNEL BOTTOM
7-8.	16" X 16" WEIR - ELEVATED CHANNEL BOTTOM
8-7.	16" X 16" WEIR
9-9.	STACKED-WEIR 8" X 8" BELOW & 8" HIGH X 16" WIDE ABOVE
10-10.	8" WIDE X 16" HIGH WEIR
11-11.	STACKED-WEIR 8" X 8" BELOW & 8" HIGH X 16" WIDE ABOVE
12-12.	24" WIDE X 8" HIGH WEIR

SEQUENCE OF ACTIVITIES

Note: Start with elevated channel bottom installed, 16" x 16" weir closed, and double orifice open.

1. **RUN TEST 4-1** - ORIFICE ONLY - 2 OPENINGS - ELEVATED CHANNEL BOTTOM
2. REMOVE ELEVATED CHANNEL BOTTOM
3. **RUN TEST 2-2** - ORIFICE ONLY - 2 OPENINGS
4. DRAIN WATER
5. CLOSE ONE SIDE OF ORIFICE
6. **RUN TEST 1-3** - ORIFICE ONLY - 1 OPENING
7. ADD ELEVATED CHANNEL BOTTOM
8. **RUN TEST 3-4** - ORIFICE ONLY - 1 OPENING - ELEVATED CHANNEL BOTTOM
9. OPEN 16" WEIR
10. **RUN TEST 6-5** -16" WEIR & 1 ORIFICE OPENING-ELEVATED CHANNEL BOTTOM
11. REMOVE ELEVATED CHANNEL BOTTOM
12. **RUN TEST 5-6** - 16" WEIR & 1 ORIFICE OPENING
13. DRAIN WATER
14. CLOSE SECOND SIDE OF ORIFICE (TEMPORARY PLUG)
15. **RUN TEST 8-7** - WEIR ONLY 16" OPENING
16. ADD ELEVATED CHANNEL BOTTOM
17. **RUN TEST 7-8** - WEIR ONLY 16" OPENING - ELEVATED CHANNEL BOTTOM
18. REMOVE ELEVATED CHANNEL BOTTOM
19. ADD 8" X 8" LOWER WEIR OPENING FOR STACKED-WEIR

SEQUENCE OF ACTIVITIES (Cont.)

20. **RUN TEST 9-9** - STACKED-WEIR 8" X 8" BELOW; 16" WIDE X 8" HIGH ABOVE

21. CLOSE 8" X 8" OF UPPER WEIR

22. **RUN TEST 10-10** - WEIR ONLY 8" WIDE X 16" HIGH OPENING

23. OPEN UPPER WEIR TO 24" WIDE

24. **RUN TEST 11-11** - STACKED-WEIR 8" X 8" BELOW; 24" WIDE X 8" HIGH ABOVE

25. CLOSE LOWER 8" X 8" OPENING

26. **RUN TEST 12-12** - WEIR ONLY 24" WIDE X 8" HIGH OPENING

APPENDIX C

EVALUATION OF LABORATORY TEST DATA

The data collected during each of the twelve (12) tests was evaluated using the computer program Lotus 123 Version 2.3. Each of the four types of test runs (weir only, orifice only, stacked-weir, and weir/orifice) was evaluated is a slightly different way. For a description of how each test was configured see Table 3, Section III.

Orifice Only (Tests 1-4) and Weir Only (Tests 7,8,10 & 12) - The equations used to evaluate the performance of the orifice only tests and weir only tests were developed in Section III. The flow delivered to the test setup was determined using

$$Q = 1.4564\, H^{0.4932} \qquad (21)$$

where Q = Measured Flow (cfs); and H = Pressure Drop Across the 3" x 6" Venturi meter (ft-Hg). The concrete block wall leak rate was determined using

$$Q = -0.130 + 16.920\, H \qquad (22)$$

where Q = Block Wall Leak Rate (gpm); and H = Depth of Water Behind the Block Wall (ft). Once the values for Equations 21 and 22 were determined for each data point it was possible to calculate the device flow rate, wall leak rate percentage, and $C_d = Q_a/Q_i$ for both the orifice only and weir only tests.

Stacked-Weir (Tests 9 & 11) - The results of these two tests were evaluated using the power regression equations developed in the appropriate weir only test arrangements. For example, the full height 8" vertical strip of Test 9, shown in Figure 21, was evaluated using the power regression equation from Test 10. The partial height vertical strip of Test 9 was evaluated using the same power regression equation. When applying the Test 10 power regression equation to each vertical strip the corresponding head was used for each vertical strip. Using this method of evaluation, the predicted discharge for Test 9 was under estimated by a mean difference of 1.69% with a Standard Deviation of the Mean of 1.52%. Using the same method, along with the power regression equations from Tests 8 and 10, with Test 11 produced and under estimation of 1.33% with a Standard Deviation of the Mean of 2.02%.

Weir/Orifice Configuration (Tests 5 & 6) - These two tests were evaluated using the power regression equations developed for the appropriated components of the configuration tested. Test 5, shown in Figure 23, used the equation developed in Test 1 for a single orifice opening along with the equation developed in Test 8 for a 16" wide weir; Test 6, the same as Figure 23 with an elevated channel bottom, used the equation developed in Test 3 for a single orifice opening with elevated channel bottom along with the equation developed in Test 7 for a 16" wide weir with an elevated channel bottom.

In Test 5 the predicted flow was 2.15% greater than the actual flow with a Standard Deviation of the Mean of 0.61%. The Test 6 predicted flow was 0.24% less than the actual flow with a Standard Deviation of the Mean of 0.34%.

Test Run No. 1

December 4, 1990

NOTE: SEE TABLE 3

NOTE: Qa = Qv−Ql

DATA POINT	ORIFICE HEAD (ft)	IDEAL ORIFICE FLOW RATE Qi (CFS)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	ACTUAL FLOW RATE Qa (cfs)	% LEAK RATE	Cd (Qa/Qi)
1	2.89	2.792	1.988	0.116	1.871	5.9	0.671
2	2.76	2.728	1.935	0.111	1.823	5.8	0.669
3	2.47	2.581	1.824	0.100	1.724	5.5	0.668
4	2.18	2.424	1.707	0.089	1.617	5.2	0.667
5	1.85	2.233	1.58	0.077	1.503	4.8	0.673
6	1.55	2.044	1.441	0.065	1.375	4.5	0.673
7	1.24	1.828	1.288	0.054	1.234	4.2	0.675
8	0.93	1.583	1.113	0.042	1.07	3.8	0.677
9	0.63	1.303	0.903	0.031	0.872	3.4	0.671
10	0.32	0.929	0.625	0.019	0.605	3.1	0.655

TABLE C1
Test No. 1 Data for
Single Orifice Opening

Test Run No. 2

NOVEMBER 5, 190

NOTE: SEE TABLE 3

NOTE: Qa = Qv−Ql

DATA POINT	ORIFICE HEAD (ft)	IDEAL ORIFICE FLOW RATE Qi (CFS)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	ACTUAL FLOW RATE Qa (cfs)	% LEAK RATE	Cd (Qa/Qi)
1	1.837	4.448	3.031	0.076	2.954	2.5	0.663
2	1.737	4.325	2.938	0.073	2.865	2.5	0.662
3	1.607	4.16	2.813	0.068	2.745	2.4	0.659
4	1.532	4.062	2.743	0.065	2.677	2.3	0.658
5	1.452	3.955	2.663	0.062	2.600	2.3	0.657
6	1.337	3.795	2.584	0.058	2.526	2.2	0.665
7	1.225	3.632	2.487	0.053	2.433	2.1	0.669
8	1.132	3.492	2.411	0.050	2.361	2.0	0.675
9	1.052	3.366	2.333	0.047	2.285	2.0	0.678
10	0.942	3.185	2.186	0.043	2.143	1.9	0.672
11	0.822	2.975	2.039	0.038	2.001	1.8	0.672
12	0.722	2.788	1.886	0.034	1.851	1.8	0.663
13	0.607	2.557	1.713	0.03	1.682	1.7	0.657
14	0.487	2.29	1.505	0.026	1.479	1.7	0.645
15	0.357	1.961	1.272	0.021	1.25	1.6	0.637

TABLE C2
Test No. 2 Data for
Double Orifice Opening

Test Run No. 3

APRIL 12, 1991

NOTE: SEE TABLE 3

Note: Qa = Qi−Ql

DATA POINT	ORIFICE HEAD (ft)	IDEAL ORIFICE FLOW RATE Qi (CFS)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	ACTUAL FLOW RATE Qa (cfs)	% LEAK RATE	Cd (Qa/Qi)
1	2.747	2.721	1.977	0.111	1.747	5.9	0.642
2	2.497	2.595	1.934	0.101	1.677	5.7	0.646
3	2.367	2.526	1.909	0.096	1.634	5.5	0.647
4	2.242	2.459	1.882	0.092	1.590	5.4	0.647
5	2.117	2.389	1.850	0.087	1.538	5.3	0.644
6	1.972	2.306	1.821	0.081	1.491	5.2	0.647
7	1.847	2.231	1.786	0.077	1.435	5.1	0.643
8	1.717	2.151	1.757	0.072	1.391	4.9	0.646
9	1.582	2.065	1.708	0.067	1.315	4.8	0.637
10	1.427	1.961	1.665	0.061	1.251	4.6	0.638
11	1.257	1.841	1.607	0.055	1.166	4.5	0.633
12	1.097	1.72	1.566	0.049	1.110	4.2	0.645
13	0.927	1.581	1.502	0.042	1.022	4.0	0.646
14	0.722	1.395	1.402	0.034	0.891	3.7	0.639
15	0.582	1.252	1.328	0.029	0.800	3.5	0.639
16	0.357	0.981	1.17	0.021	0.620	3.2	0.633

TABLE C3
Test No. 3 Data for
Single Orifice Opening Elevated Channel Bottom

Test Run No. 4

NOVEMBER 5, 1991

NOTE: SEE TABLE 3

NOTE: Qa = Qv−Ql

DATA POINT	ORIFICE HEAD (ft)	IDEAL ORIFICE FLOW RATE Qi (CFS)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	ACTUAL FLOW RATE Qa (cfs)	% LEAK RATE	Cd (Qa/Qi)
1	2.017	4.661	2.979	0.083	2.896	2.8	0.621
2	1.892	4.514	2.899	0.078	2.82	2.7	0.624
3	1.837	4.448	2.827	0.076	2.751	2.7	0.618
4	1.752	4.344	2.765	0.073	2.691	2.6	0.619
5	1.647	4.212	2.678	0.069	2.608	2.6	0.619
6	1.567	4.108	2.616	0.066	2.549	2.5	0.620
7	1.432	3.927	2.487	0.061	2.425	2.4	0.617
8	1.292	3.73	2.359	0.056	2.303	2.3	0.617
9	1.167	3.545	2.283	0.051	2.232	2.2	0.629
10	1.112	3.461	2.167	0.049	2.118	2.2	0.611
11	1.022	3.318	2.085	0.046	2.038	2.2	0.614
12	0.897	3.108	1.956	0.041	1.915	2.1	0.615
13	0.752	2.846	1.767	0.036	1.731	2.0	0.608
14	0.627	2.598	1.593	0.031	1.562	1.9	0.600
15	0.467	2.242	1.367	0.025	1.342	1.8	0.598
16	0.297	1.788	1.113	0.018	1.094	1.6	0.611

TABLE C4
Test No. 4 Data for
Double Orifice Opening Elevated Channel Bottom

Test Run No. 5

April 26, 1991

NOTE: SEE TABLE 3 NOTE: Qa=Qv−QL

DATA POINT	ORIFICE HEAD (ft)	WEIR HEAD (ft)	IDEAL TOTAL FLOW Qi (cfs)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	% LEAK RATE	Cd (Qa/Qi)
1	2.146	0.500	N/A	3.101	0.089	2.8	----
2	2.141	0.495	N/A	3.061	0.088	2.9	----
3	2.126	0.480	N/A	3.01	0.088	2.9	----
4	2.116	0.470	N/A	2.941	0.087	2.9	----
5	2.106	0.460	N/A	2.892	0.087	3.0	----
6	2.091	0.445	N/A	2.813	0.086	3.0	----
7	2.076	0.460	N/A	2.758	0.085	3.1	----
8	2.051	0.405	N/A	2.682	0.085	3.1	----
9	2.041	0.395	N/A	2.600	0.084	3.2	----
10	2.021	0.375	N/A	2.503	0.083	3.3	----
11	1.996	0.350	N/A	2.428	0.082	3.4	----
12	1.971	0.325	N/A	2.319	0.081	3.5	----
13	1.926	0.280	N/A	2.163	0.08	3.7	----
14	1.896	0.250	N/A	2.065	0.079	3.8	----
15	1.861	0.215	N/A	1.940	0.077	3.9	----

COMBINED WEIR/ORIFICE PERFORMANCE CHECK

DATA POINT	ACTUAL FLOW RATE Qa (cfs)	PREDICTED FLOW Qp (cfs)	% DIFFERENCE
1	3.012	3.053	1.3
2	2.973	3.029	1.3
3	2.922	2.959	1.2
4	2.854	2.914	2.1
5	2.805	2.868	2.2
6	2.726	2.801	2.7
7	2.672	2.734	2.3
8	2.597	2.626	1.1
9	2.516	2.583	2.6
10	2.419	2.500	3.3
11	2.345	2.398	2.2
12	2.237	2.300	2.7
13	2.082	2.131	2.3
14	1.985	2.024	1.9
15	1.862	1.907	2.3

STATISTICS

MEAN %	2.149
SD %	0.614

TABLE C5
Test No. 5 Data for
Combined Single Orifice Opening and 16" Wide Weir

Test Run No. 6

April 12, 1991

NOTE: SEE TABLE 3 — NOTE: Qa=Qv−QL

DATA POINT	ORIFICE HEAD (ft)	WEIR HEAD (ft)	IDEAL TOTAL FLOW Qi (cfs)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	% LEAK RATE	Cd (Qa/Qi)
1	2.146	0.500	N/A	3.064	0.088	2.8	----
2	2.141	0.495	N/A	3.044	0.088	2.9	----
3	2.131	0.485	N/A	2.990	0.087	2.9	----
4	2.121	0.475	N/A	2.948	0.087	2.9	----
5	2.116	0.470	N/A	2.906	0.087	3.0	----
6	2.096	0.450	N/A	2.831	0.086	3.0	----
7	2.081	0.435	N/A	2.765	0.086	3.1	----
8	2.056	0.410	N/A	2.667	0.085	3.1	----
9	2.041	0.395	N/A	2.592	0.084	3.2	----
10	2.021	0.375	N/A	2.516	0.083	3.3	----
11	2.001	0.355	N/A	2.437	0.083	3.4	----
12	1.981	0.335	N/A	2.333	0.082	3.5	----
13	1.936	0.290	N/A	2.172	0.080	3.7	----
14	1.906	0.260	N/A	2.06	0.079	3.8	----
15	1.881	0.235	N/A	1.972	0.078	3.9	----

COMBINED WEIR/ORIFICE PERFORMANCE CHECK

DATA POINT	ACTUAL FLOW RATE Qa (cfs)	PREDICTED FLOW Qp (cfs)	% DIFFERENCE		
1	2.976	2.974	-0.1		
2	2.956	2.950	-0.2	STATISTICS	
3	2.902	2.903	0.0		
4	2.861	2.856	-0.2	MEAN %	-0.241
5	2.819	2.833	0.5	SD %	0.336
6	2.744	2.741	-0.1		
7	2.679	2.673	-0.2		
8	2.581	2.563	-0.7		
9	2.508	2.499	-0.4		
10	2.432	2.414	-0.7		
11	2.353	2.333	-0.9		
12	2.250	2.253	0.1		
13	2.091	2.082	-0.4		
14	1.98	1.975	-0.2		
15	1.894	1.891	-0.2		

TABLE C6
Test No. 6 Data for
Combined Single Orifice Opening, 16" Wide Weir,
and Elevated Channel Bottom

Test Run No. 7

APRIL 15, 1991

NOTE: SEE TABLE 3

NOTE: Qa = Qv−Ql

DATA POINT	WEIR HEAD (ft)	IDEAL WEIR FLOW RATE Qi (CFS)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	ACTUAL FLOW RATE Qa (cfs)	% LEAK RATE	Cd (Qa/Qi)	Cw
1	0.800	5.142	3.074	0.099	2.974	3.2	0.578	3.094
2	0.785	4.998	2.990	0.099	2.89	3.3	0.578	3.093
3	0.770	4.855	2.899	0.098	2.800	3.4	0.576	3.084
4	0.755	4.714	2.798	0.098	2.700	3.5	0.572	3.063
5	0.735	4.528	2.678	0.097	2.581	3.6	0.569	3.048
6	0.705	4.254	2.524	0.096	2.428	3.8	0.570	3.052
7	0.670	3.941	2.333	0.094	2.238	4.1	0.567	3.037
8	0.640	3.679	2.177	0.093	2.083	4.3	0.566	3.028
9	0.600	3.34	1.988	0.092	1.896	4.6	0.567	3.035
10	0.560	3.011	1.755	0.090	1.664	5.2	0.552	2.955
11	0.500	2.540	1.498	0.088	1.410	5.9	0.555	2.968
12	0.450	2.169	1.238	0.086	1.151	7.0	0.530	2.839
13	0.405	1.852	1.094	0.084	1.009	7.8	0.544	2.913
14	0.340	1.424	0.855	0.082	0.772	9.6	0.542	2.901
15	0.260	0.952	0.625	0.0790	0.545	12.7	0.572	3.062
16	0.215	0.716	0.511	0.077	0.434	15.2	0.605	3.230

TABLE C7
Test No. 7 Data for
16" Wide x 16" High Weir
Elevated Channel Bottom

Test Run No. 8

APRIL 15, 1991

NOTE: SEE TABLE 3

NOTE: Qa = Qv−Ql

DATA POINT	WEIR HEAD (ft)	IDEAL WEIR FLOW RATE Qi (CFS)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	ACTUAL FLOW RATE Qa (cfs)	% LEAK RATE	Cd (Qa/Qi)	Cw
1	0.800	5.142	3.101	0.099	3.001	3.2	0.583	3.121
2	0.795	5.094	3.078	0.099	2.978	3.2	0.584	3.126
3	0.780	4.950	2.979	0.099	2.880	3.3	0.581	3.112
4	0.775	4.903	2.910	0.098	2.811	3.3	0.573	3.066
5	0.755	4.714	2.827	0.098	2.729	3.4	0.578	3.096
6	0.745	4.621	2.735	0.097	2.637	3.5	0.570	3.052
7	0.735	4.528	2.659	0.097	2.561	3.6	0.565	3.025
8	0.705	4.254	2.524	0.096	2.428	3.8	0.570	3.052
9	0.690	4.119	2.432	0.095	2.337	3.9	0.567	3.034
10	0.675	3.985	2.328	0.095	2.233	4.0	0.560	2.997
11	0.650	3.766	2.219	0.094	2.125	4.2	0.564	3.018
12	0.620	3.508	2.075	0.093	1.982	4.4	0.564	3.021
13	0.600	3.340	1.946	0.092	1.853	4.7	0.555	2.968
14	0.565	3.052	1.772	0.0910	1.681	5.1	0.551	2.947
15	0.530	2.772	1.625	0.089	1.536	5.5	0.553	2.962
16	0.480	2.389	1.427	0.087	1.339	6.1	0.560	2.997
17	0.440	2.097	1.255	0.086	1.169	6.8	0.557	2.98
18	0.400	1.818	1.094	0.084	1.009	7.7	0.555	2.969
19	0.345	1.456	0.891	0.082	0.809	9.2	0.555	2.971
20	0.275	1.036	0.642	0.080	0.561	12.4	0.542	2.899
21	0.225	0.767	0.490	0.078	0.412	15.9	0.537	2.873

TABLE C8
Test No. 8 Data for
16" Wide x 16" High Weir

Test Run No. 9

April 28, 1991

NOTE: SEE TABLE 3 NOTE: Qa=Qv−Ql

DATA POINT	WEIR #1 HEAD (ft)	WEIR #2 HEAD (ft)	IDEAL TOTAL FLOW Ql (cfs)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	% LEAK RATE	Cd (Qa/Ql)
1	1.075	0.406	N/A	3.091	0.110	3.5	----
2	1.070	0.403	N/A	3.074	0.110	3.5	----
3	1.060	0.393	N/A	3.007	0.109	3.6	----
4	1.050	0.383	N/A	2.945	0.109	3.7	----
5	1.040	0.373	N/A	2.904	0.108	3.7	----
6	1.030	0.363	N/A	2.838	0.108	3.8	----
7	1.014	0.348	N/A	2.761	0.107	3.9	----
8	1.004	0.338	N/A	2.693	0.107	3.9	----
9	0.995	0.328	N/A	2.604	0.107	4.1	----
10	0.975	0.308	N/A	2.520	0.106	4.2	----
11	0.960	0.293	N/A	2.445	0.105	4.3	----
12	0.940	0.273	N/A	2.35	0.105	4.4	----
13	0.930	0.263	N/A	2.274	0.104	4.6	----
14	0.910	0.243	N/A	2.177	0.103	4.7	----
15	0.885	0.218	N/A	2.06	0.103	5.0	----
16	0.845	0.178	N/A	1.875	0.101	5.4	----
17	0.835	0.168	N/A	1.819	0.101	5.5	----
18	0.815	0.148	N/A	1.743	0.100	5.7	----
19	0.800	0.133	N/A	1.694	0.099	5.8	----
20	0.780	0.113	N/A	1.606	0.099	6.1	----
21	0.750	0.083	N/A	1.484	0.097	6.5	----
22	0.730	0.063	N/A	1.412	0.097	6.8	----
23	0.710	0.043	N/A	1.351	0.096	7.1	----
24	0.700	0.033	N/A	1.288	0.096	7.4	----
25	0.670	0.003	N/A	1.177	0.094	8.0	----
26	0.590	----	N/A	1.003	0.091	9.1	----
27	0.505	----	N/A	0.804	0.088	11.0	----
28	0.430	----	N/A	0.658	0.085	13.0	----
29	0.385	----	N/A	0.552	0.084	15.2	----
30	0.325	----	N/A	0.444	0.081	18.4	----

STACKED-WEIR PERFORMANCE CHECK

DATA POINT	ACTUAL FLOW FLOW Qa (cfs)	PREDICTED FLOW Qp (cfs)	% DIFFERENCE
1	2.981	2.895	−2.8
2	2.964	2.867	−3.2
3	2.897	2.812	−2.9
4	2.836	2.757	−2.7
5	2.795	2.703	−3.3
6	2.730	2.649	−2.9
7	2.653	2.570	−3.1
8	2.586	2.517	−2.6
9	2.497	2.465	−1.2
10	2.413	2.363	−2.0
11	2.339	2.288	−2.1
12	2.245	2.190	−2.4
13	2.170	2.142	−1.2
14	2.073	2.047	−1.2
15	1.957	1.932	−1.2
16	1.773	1.755	−1.0
17	1.718	1.712	−0.3
18	1.642	1.629	−0.8
19	1.595	1.569	−1.6
20	1.507	1.490	−1.1
21	1.386	1.378	−0.6
22	1.315	1.307	−0.6
23	1.255	1.239	−1.2
24	1.192	1.207	1.2
25	1.082	1.116	3.1
26	0.911	0.909	−0.1
27	0.715	0.708	−1.0
28	0.572	0.546	−4.5
29	0.468	0.457	−2.2
30	0.362	0.348	−3.8

STATISTICS

MEAN %	−1.685
SD %	1.524

TABLE C9

Test No. 9 Data for Stacked-Weir; 8" Wide x 8" High Below and 16" Wide x 8" High Above

Test Run No. 10

APRIL 30, 1991

NOTE: SEE TABLE 3

NOTE: $Qa = Qv - Ql$

DATA POINT	WEIR HEAD (ft)	IDEAL WEIR FLOW RATE Qi (CFS)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	ACTUAL FLOW RATE Qa (cfs)	% LEAK RATE	Cd (Qa/Qi)	Cw
1	1.230	4.863	3.094	0.116	2.978	3.7	0.612	3.275
2	1.205	4.716	3.037	0.115	2.922	3.7	0.619	3.314
3	1.200	4.687	3.007	0.114	2.892	3.8	0.617	3.300
4	1.190	4.628	2.962	0.114	2.848	3.8	0.615	3.291
5	1.175	4.541	2.917	0.113	2.803	3.9	0.617	3.301
6	1.160	4.454	2.856	0.113	2.743	3.9	0.615	3.293
7	1.150	4.397	2.806	0.113	2.693	4.0	0.612	3.275
8	1.135	4.311	2.728	0.112	2.615	4.1	0.606	3.244
9	1.120	4.226	2.651	0.111	2.539	4.2	0.600	3.213
10	1.095	4.085	2.56	0.110	2.449	4.3	0.599	3.206
11	1.075	3.974	2.478	0.110	2.368	4.4	0.596	3.187
12	1.045	3.808	2.398	0.109	2.289	4.5	0.601	3.214
13	1.020	3.673	2.306	0.108	2.198	4.6	0.598	3.200
14	1.000	3.565	2.224	0.107	2.116	4.8	0.593	3.175
15	0.975	3.432	2.134	0.106	2.027	4.9	0.59	3.159
16	0.950	3.301	2.039	0.105	1.934	5.1	0.585	3.133
17	0.915	3.120	1.924	0.104	1.820	5.4	0.583	3.12
18	0.880	2.943	1.813	0.102	1.710	5.6	0.581	3.108
19	0.850	2.794	1.725	0.101	1.623	5.8	0.581	3.107
20	0.800	2.551	1.571	0.099	1.471	6.3	0.576	3.083
21	0.750	2.315	1.434	0.097	1.336	6.8	0.577	3.086
22	0.670	1.955	1.186	0.094	1.091	8.0	0.558	2.985
23	0.630	1.782	1.094	0.093	1.001	8.5	0.561	3.001
24	0.580	1.574	0.982	0.091	0.891	9.3	0.565	3.024
25	0.540	1.414	0.891	0.09	0.801	10	0.566	3.03
26	0.505	1.279	0.777	0.088	0.688	11.4	0.538	2.878
27	0.440	1.041	0.642	0.086	0.555	13.4	0.534	2.856
28	0.385	0.851	0.552	0.084	0.468	15.2	0.549	2.938
29	0.340	0.706	0.467	0.082	0.385	17.6	0.545	2.915

TABLE C10
Test No. 10 Data for
8" Wide x 16" High Weir

Test Run No. 11

May 2, 1991

NOTE: SEE TABLE 3

DATA POINT	WEIR #1 HEAD (ft)	WEIR #2 HEAD (ft)	IDEAL TOTAL FLOW Qi (cfs)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	% LEAK RATE	Cd (Qa/Qi)
1	1.004	0.338	N/A	3.101	0.107	3.4	----
2	1.000	0.333	N/A	3.061	0.107	3.5	----
3	0.995	0.328	N/A	3.020	0.107	3.5	----
4	0.990	0.323	N/A	2.976	0.107	3.5	----
5	0.985	0.318	N/A	2.934	0.106	3.6	----
6	0.975	0.308	N/A	2.838	0.106	3.7	----
7	0.960	0.293	N/A	2.750	0.105	3.8	----
8	0.950	0.283	N/A	2.678	0.105	3.9	----
9	0.935	0.268	N/A	2.600	0.104	4.0	----
10	0.925	0.258	N/A	2.516	0.104	4.1	----
11	0.915	0.248	N/A	2.428	0.104	4.2	----
12	0.895	0.228	N/A	2.306	0.103	4.4	----
13	0.875	0.208	N/A	2.205	0.102	4.6	----
14	0.855	0.188	N/A	2.080	0.101	4.8	----
15	0.830	0.163	N/A	1.924	0.100	5.2	----
16	0.800	0.133	N/A	1.749	0.099	5.7	----
17	0.765	0.098	N/A	1.113	0.068	6.1	----
18	0.735	0.068	N/A	0.982	0.065	6.6	----
19	0.710	0.043	N/A	0.879	0.062	7.1	----
20	0.670	0.003	N/A	0.642	0.051	8.0	----
21	0.640	----	N/A	0.532	0.044	8.4	----
22	0.585	----	N/A	0.363	0.033	9.3	----
23	0.550	----	N/A	1.351	0.138	10.2	----
24	0.440	----	N/A	1.288	0.173	13.4	----
25	0.380	----	N/A	1.177	0.185	15.7	----
26	0.275	----	N/A	0.444	0.097	22.0	----

STACKED-WEIR PERFORMANCE CHECK

DATA POINT	ACTUAL FLOW RATE Qa (CFS)	PREDICTED FLOW Qp (CFS)	% DIFFERENCE
1	2.994	2.888	-3.5
2	2.954	2.853	-3.4
3	2.914	2.818	-3.3
4	2.869	2.784	-3.0
5	2.828	2.750	-2.8
6	2.732	2.682	-1.8
7	2.645	2.583	-2.3
8	2.573	2.517	-2.2
9	2.496	2.421	-3.0
10	2.412	2.358	-2.2
11	2.324	2.295	-1.2
12	2.203	2.174	-1.3
13	2.103	2.056	-2.2
14	1.978	1.942	-1.8
15	1.824	1.806	-1.0
16	1.649	1.651	0.1
17	1.495	1.484	-0.8
18	1.359	1.352	-0.5
19	1.256	1.253	-0.2
20	1.092	1.117	2.3
21	1.019	1.034	1.5
22	0.891	0.895	0.5
23	0.789	0.810	2.6
24	0.556	0.566	1.8
25	0.448	0.447	-0.4
26	0.284	0.265	-6.4

STATISTICS

MEAN %	-1.332
SD %	2.016

TABLE C11

Test No. 11 Data for
Stacked-Weir; 8" Wide x 8" High Below and
24" Wide x 8" High Above

Test Run No. 12

MAY 5, 1991

NOTE: SEE TABLE 3

NOTE: Qa = Qv−Ql

DATA POINT	WEIR HEAD (ft)	IDEAL WEIR FLOW RATE Qi (CFS)	VENTURI FLOW RATE Qv (cfs)	WALL LEAK RATE Ql (cfs)	ACTUAL FLOW RATE Qa (cfs)	% LEAK RATE	Cd (Qa/Qi)	Cw
1	0.618	5.088	3.094	0.118	2.976	3.8	0.584	3.128
2	0.613	5.026	3.064	0.117	2.946	3.8	0.586	3.135
3	0.608	4.965	2.966	0.117	2.848	3.9	0.573	3.068
4	0.603	4.904	2.91	0.117	2.792	4.0	0.569	3.045
5	0.588	4.722	2.82	0.116	2.703	4.1	0.572	3.062
6	0.573	4.542	2.731	0.116	2.615	4.2	0.575	3.079
7	0.563	4.424	2.612	0.116	2.496	4.4	0.564	3.017
8	0.548	4.248	2.478	0.115	2.363	4.6	0.556	2.975
9	0.528	4.018	2.328	0.114	2.214	4.9	0.55	2.946
10	0.508	3.792	2.167	0.113	2.053	5.2	0.541	2.896
11	0.478	3.461	2.014	0.112	1.901	5.6	0.549	2.938
12	0.463	3.299	1.924	0.112	1.812	5.8	0.549	2.937
13	0.433	2.984	1.755	0.111	1.644	6.3	0.55	2.946
14	0.403	2.679	1.567	0.11	1.456	7.0	0.543	2.908
15	0.358	2.243	1.336	0.108	1.228	8.1	0.547	2.927
16	0.313	1.834	1.103	0.106	0.997	9.6	0.543	2.907
17	0.273	1.493	0.926	0.105	0.821	11.3	0.55	2.941

TABLE C12
Test No. 12 Data for
24" Wide x 8" High Weir

APPENDIX D

DETENTION POND SENSITIVITY ANALYSIS

USING TR-55 & HEC-1

The sensitivity of detention pond design storage to variations in weir and orifice discharge characteristics was evaluated for a typical urban drainage area. The characteristics of the assumed drainage area were as follows:

1. Watershed Area - 20 Acres
2. SCS Curve Number - 92
3. Time of Concentration - 0.3 Hours
4. SCS Type II 25 Year 24 Hour Storm Event of 5.5 in/hr

TR-55 Analysis - TR-55 selects the required pond volume based on the ratio of peak outflow to peak inflow (q_o/q_i) for a given SCS storm event (30). Weir and orifice discharge coefficients are not a direct input to the TR-55 program. The input parameter used to determine pond volume by TR-55 is peak outflow (q_o), which is a function of the weir and orifice coefficients. The value of peak inflow (q_i) is determined by the TR-55 program based on the watershed characteristics. TR-55 will not provide a predicted pond volume for q_o/q_i values less than 0.1 or greater than 0.8.

For this sensitivity analysis a series of peak outflow values was input into the Storage Volume for Detention Basins section of the TR-55 program. The watershed characteristics were held constant for each run in order to ascertain the impact on pond volume of varying outflows. It was assumed, for the purposes of this study, that the only element that caused peak outflow to change was a variation in C_d. It can be seen in Figure 24 that there is a direct relationship between C_d and peak discharge rate for TR-55. This CAN be expected based on Equation 9 Section II.

TABLE D1

TR-55 Detention Pond Volume Determination for
Volume Sensitivity Comparison

Discharge Rate (cfs)	Pond Size (Acft)	C_d
10	4.20	----
15	3.79	----
20	3.43	----
25	3.11	0.50
30	2.84	0.60
35	2.60	0.70
40	2.40	0.80
45	2.22	0.90
50	2.06	1.00
55	1.92	----
60	1.79	----
65	1.66	----
70	1.54	----
75	1.41	----

HEC-1 Analysis - For this analysis HEC-1 was used for level-pool reservoir routing to route the inflow hydrograph through the detention pond. In order to use this method, some additional information over and above that used by TR-55 had to be provided for HEC-1. The following parameters were used in the HEC-1 analysis:

1. The same watershed information as the TR-55 analysis.
2. The HEC-1 pond used had an area of 0.35 acres uniform throughout its depth.
3. The HEC-1 low level orifice outlet had an area of 2.5 $Ft.^2$.

Discussion - Improper selection of C_d can have an adverse effect on detention pond storage volume and anticipated pond discharge rate. The two possible errors in the selection of C_d are as follows:

Selected C_d Higher Than the True Value - As indicated in the TR-55 and HEC-1 studies, this situation will lead to an undersized pond volume and an

TABLE D2

HEC-1 Detention Pond Volume Determination for
Volume Sensitivity Comparison

Discharge Rate (cfs)	Pond Size (ac-ft)	C_d
25.9	2.41	0.50
29.8	2.21	0.60
33.4	2.04	0.70
36.8	1.90	0.80
40.0	1.77	0.90
42.9	1.65	1.00

over estimated discharge rate when compared to the true value. Both the TR-55 and HEC-1 pond volumes will be under estimated by approximately 13% if a C_d of 0.82 (Short Tube) is used as the orifice C_d instead of the value recommended here of $C_d = 0.67$. In the case of TR-55 this pond volume error may not be of any concern since the accuracy of TR-55 is only considered to be within 25%, and the underestimated discharge rate is considered to be conservative.

Selected C_d Lower Than the True Value - In this situation the detention pond storage volume would be over-estimated while the discharge rate would be under-estimated. Since most regulatory agencies base their zoning requirements on the comparison between pre-development discharge rate and post-development discharge rate this situation could lead to noncompliance with applicable regulations (17,19).

Errors in selecting C_d for weirs and orifices could result in either expensive over-designed or ineffective under-designed ponds, and improper estimation of the discharge rate from a developed site. For these reasons it is important to make the selection of C_d as accurately as possible and to utilize all available information.

APPENDIX E

PHOTOGRAPHS

Photographs are included in order to give the reader a feel for the total project. The photographs fall into the following three categories:

1. Laboratory Equipment and Setup - Plates 1 to 10
2. Testing Program - Plates 11 to 18
3. Concrete Block Discharge Structures - Plates 19 & 20

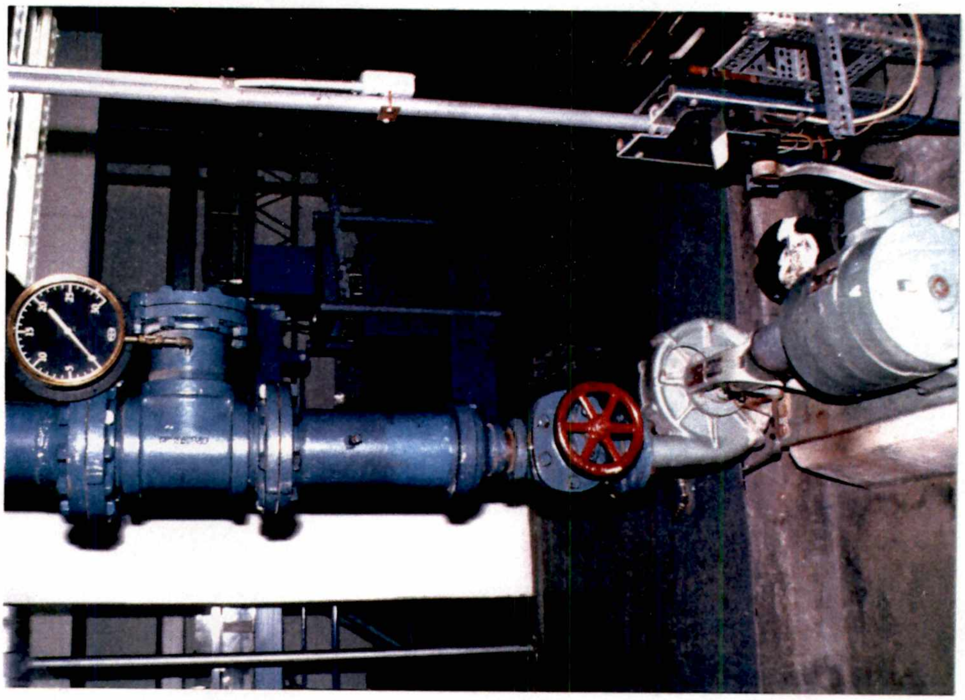

Plate No. 1 - 7 1/2 HP System Pump

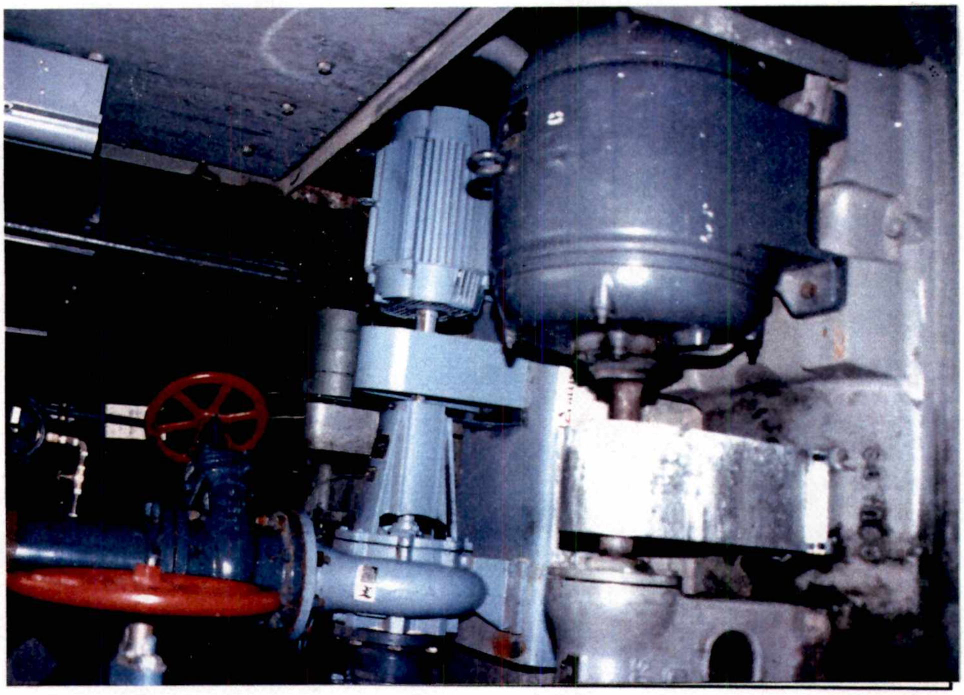

Plate No. 2 - 40 HP & 25 HP System Pumps

Plate No. 3

Concrete Block Wall; Closed 16" Weir Opening; Single Orifice Open; Elevated Channel Bottom

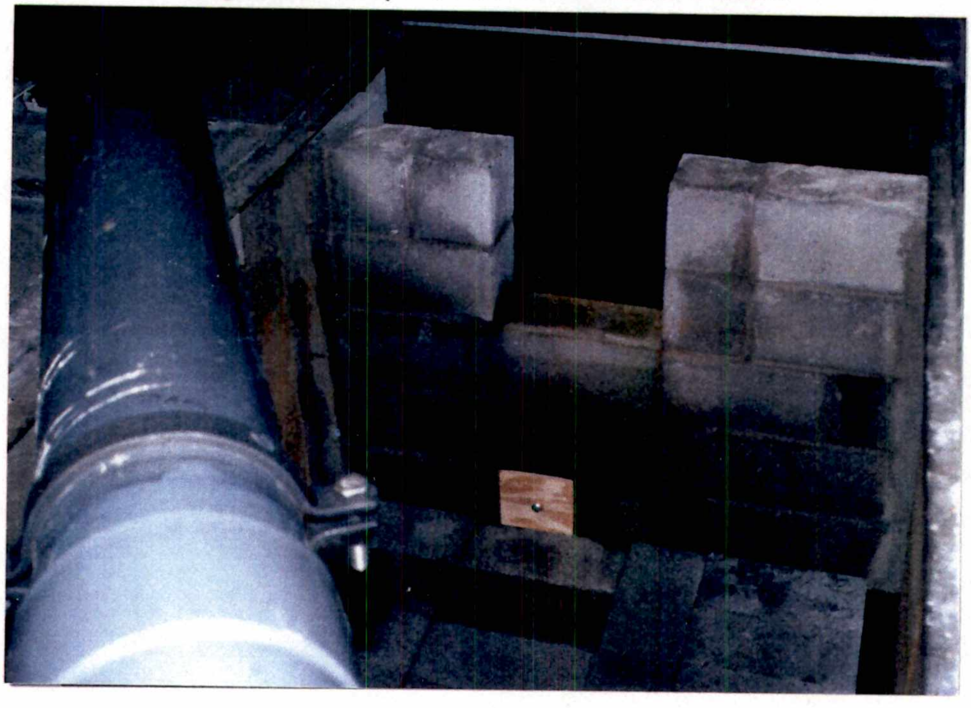

Plate No. 4

Concrete Block Wall; 16" Weir Opening; Orifice Closed; Elevated Channel Bottom

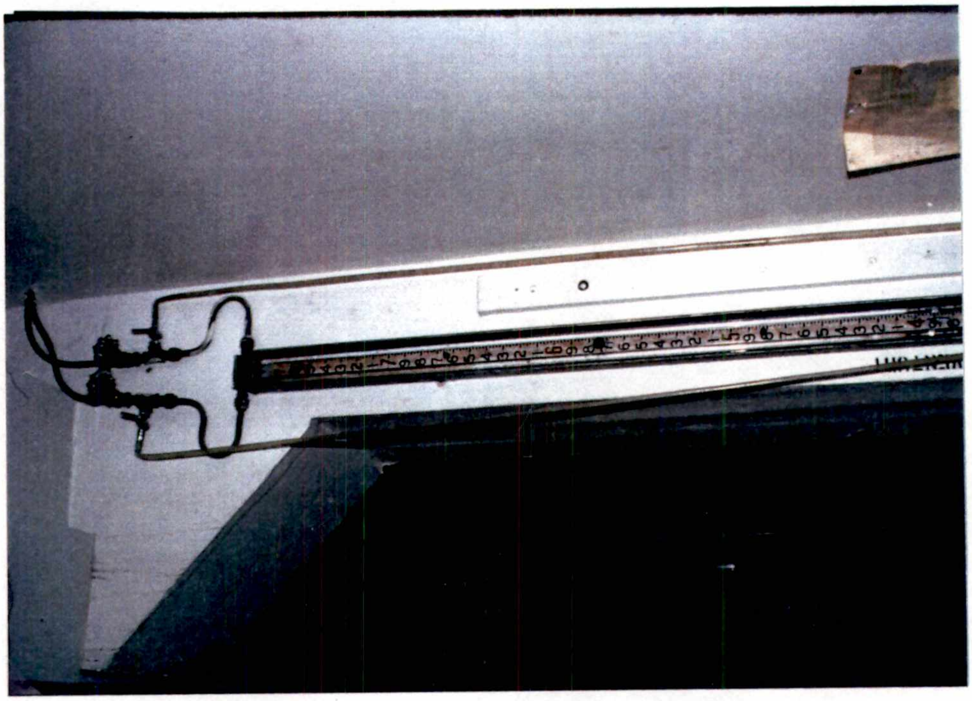

Plate No. 5 - 8 Foot Mercury Manometer

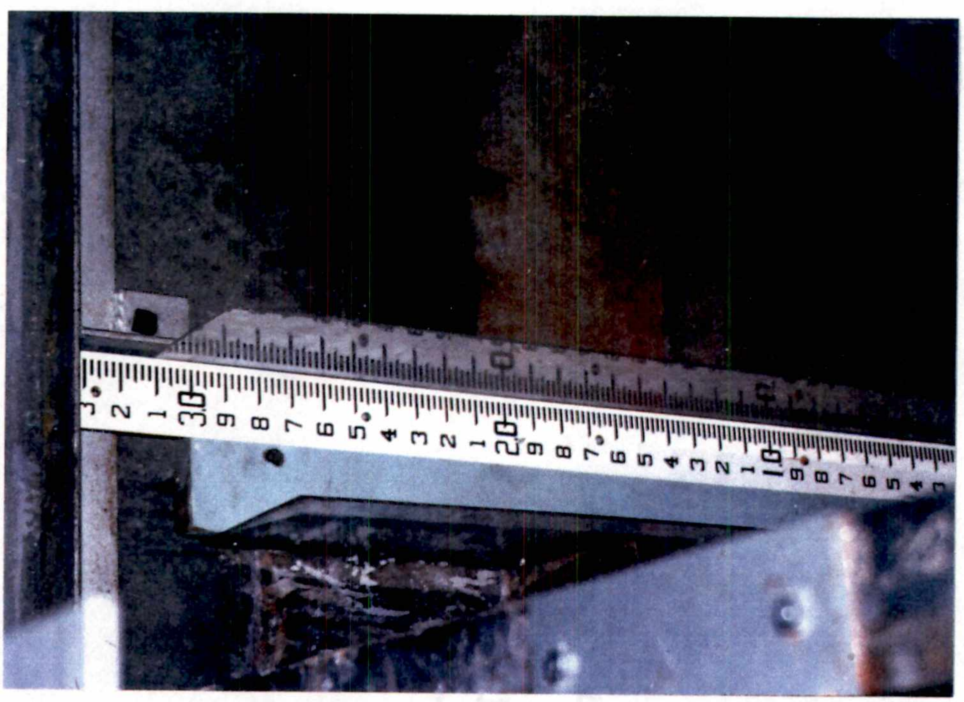

Plate No. 6 - Staff Gage & Stilling Plates

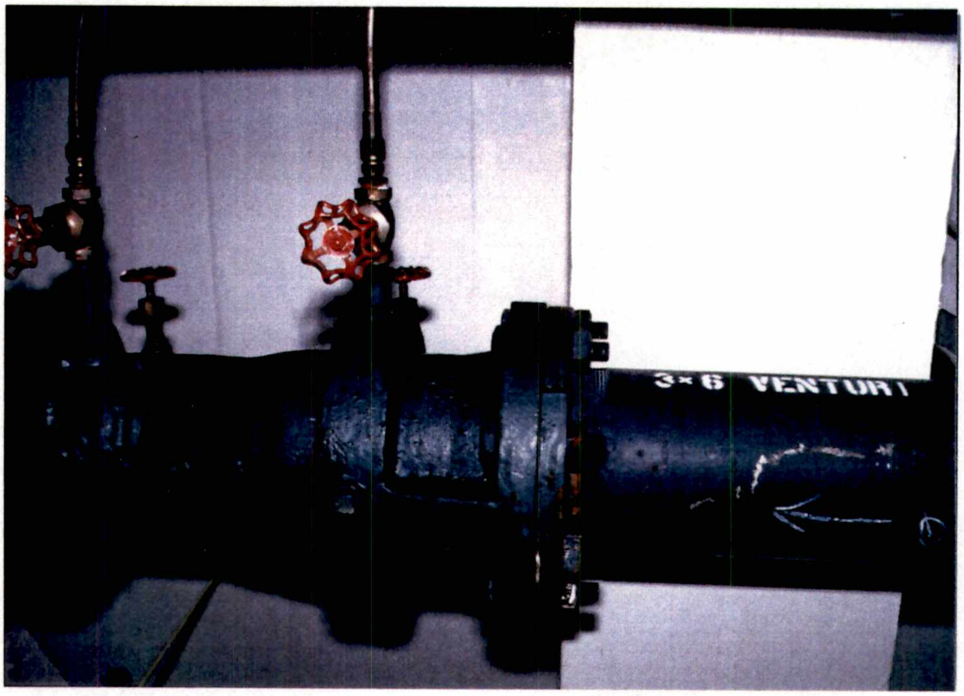

Plate No. 7 - 3" x 6" Venturi

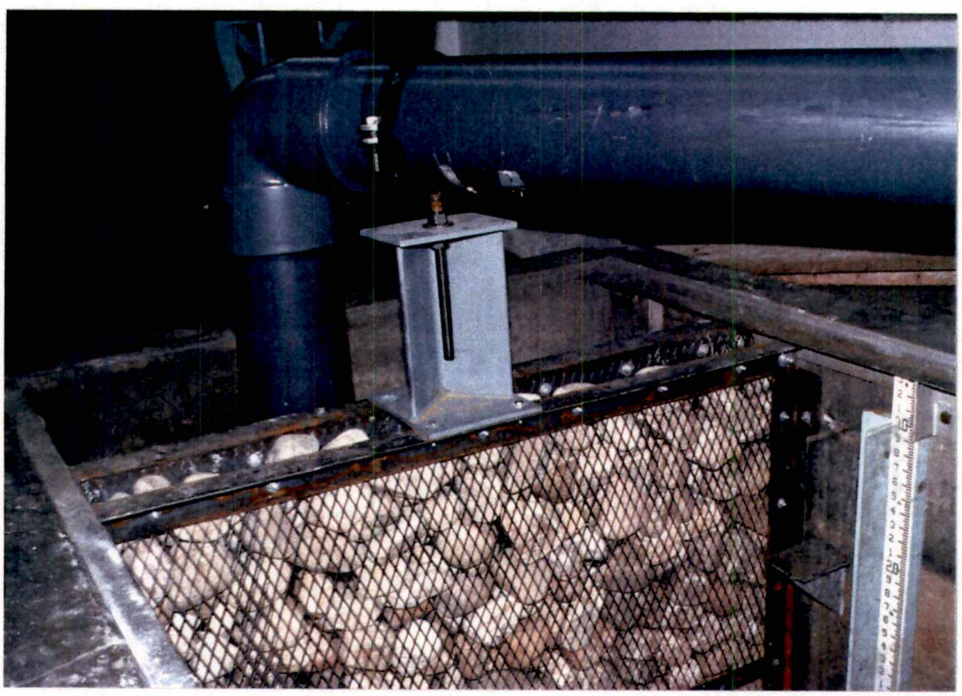

Plate No. 8 - Rock Diffuser & Staff Gage

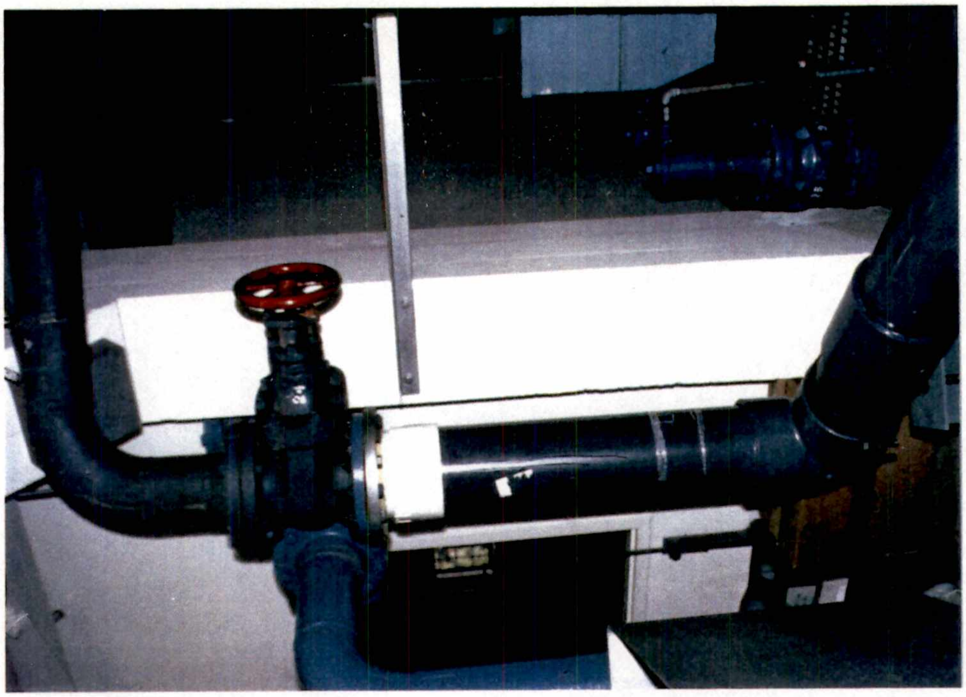

Plate No. 9

Modulating Gate Valve & Manifold Connection

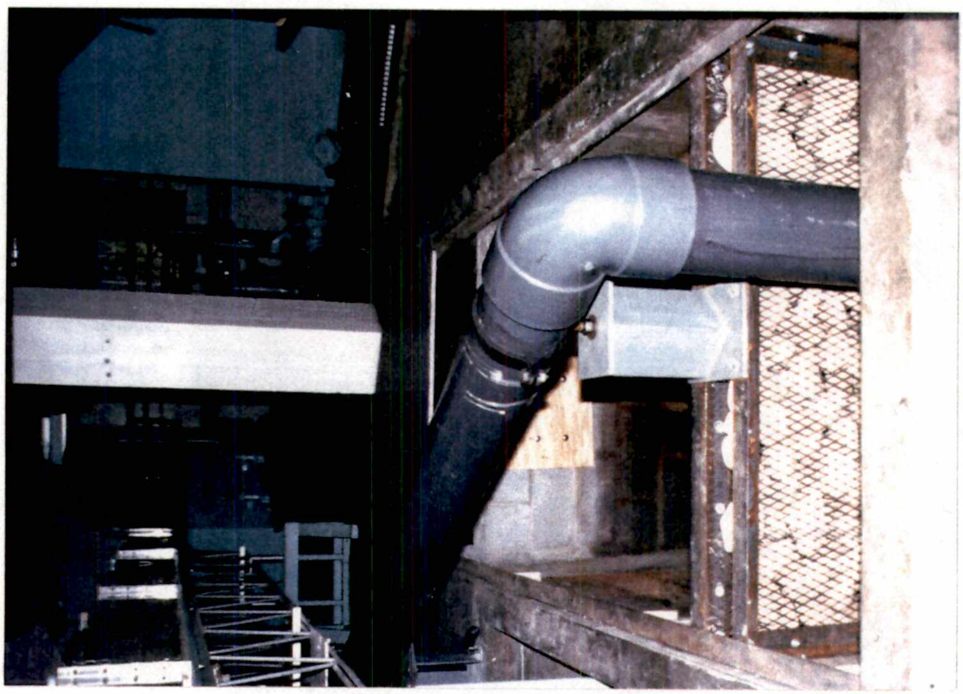

Plate No. 10

8" Schedule 80 PVC Piping

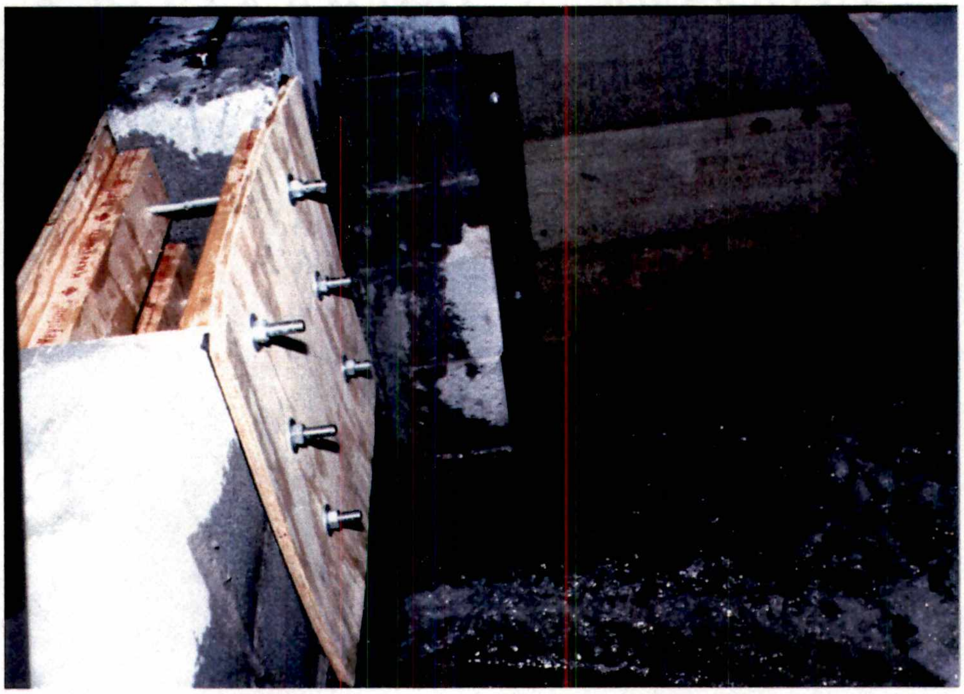

Plate No. 11 - Single Orifice (Test 1)

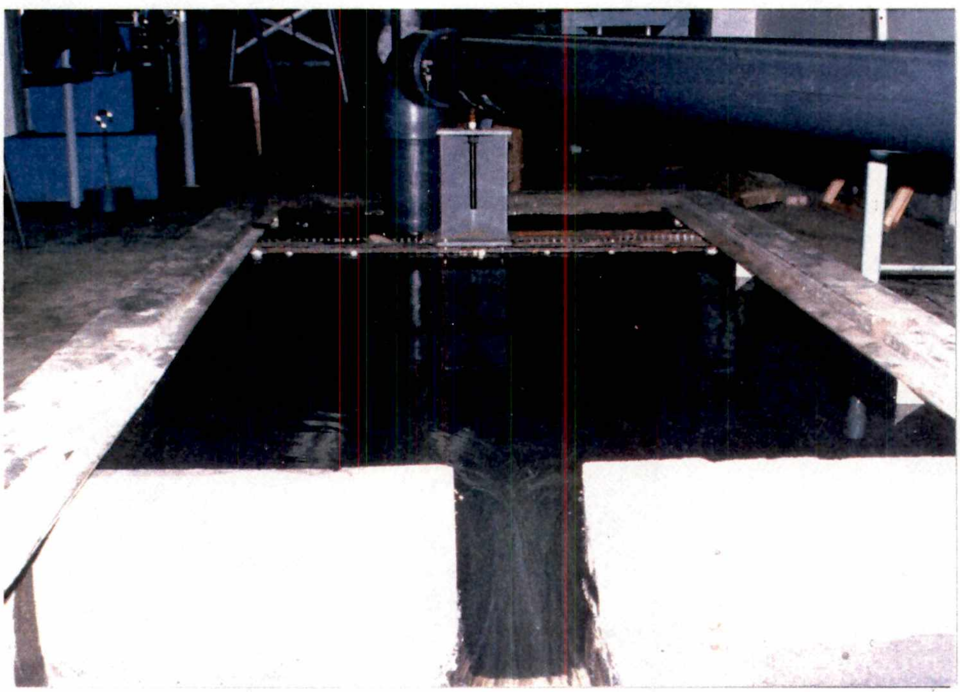

Plate No. 12

8" Wide x 16" High Weir (Test 10)

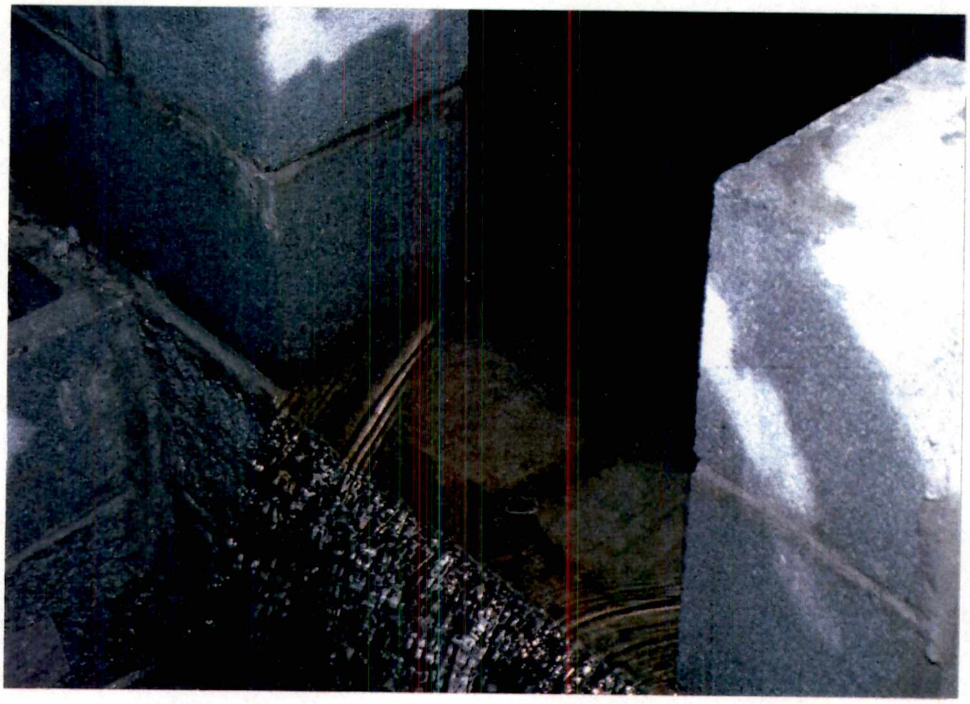

Plate No. 13

16" Wide x 16" High Weir (Test 8)

Plate No. 14

Water Entering Reservoir Side Channel

Plate No. 15

No Contraction for Broad-Crested Weir(Test 8)

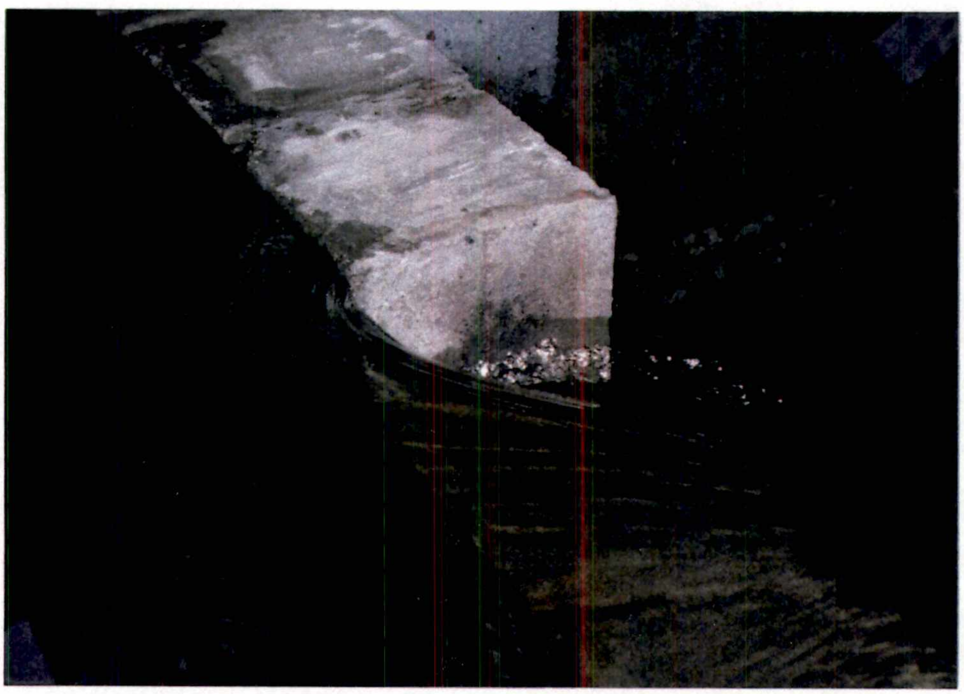

Plate No. 16

No Contraction for Broad-Crested Weir (Test 10)

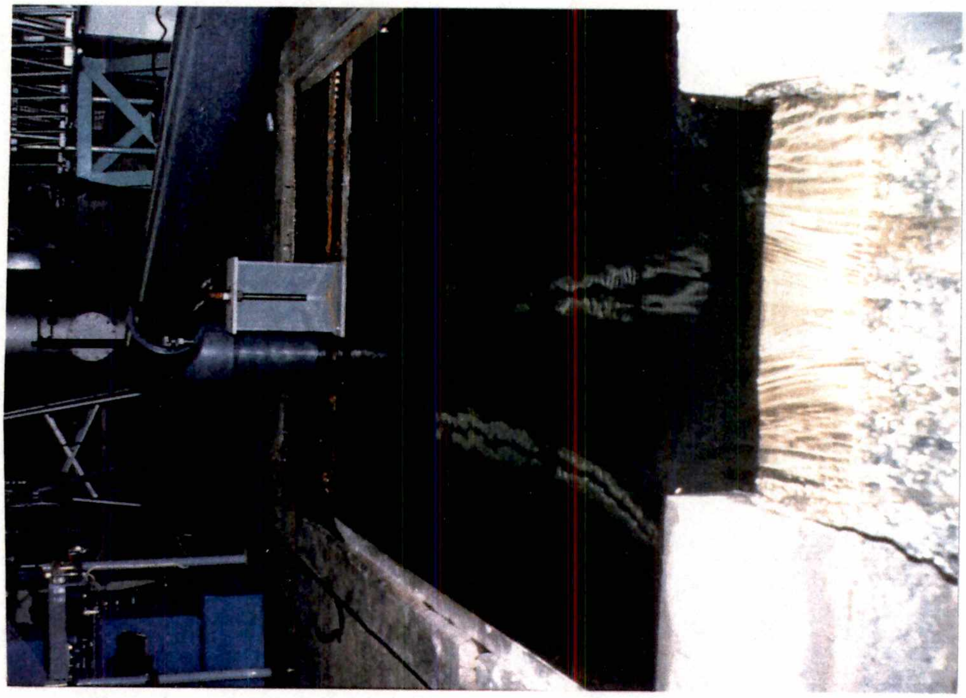

Plate No. 17

24" Wide x 8" High Weir (Test 12)

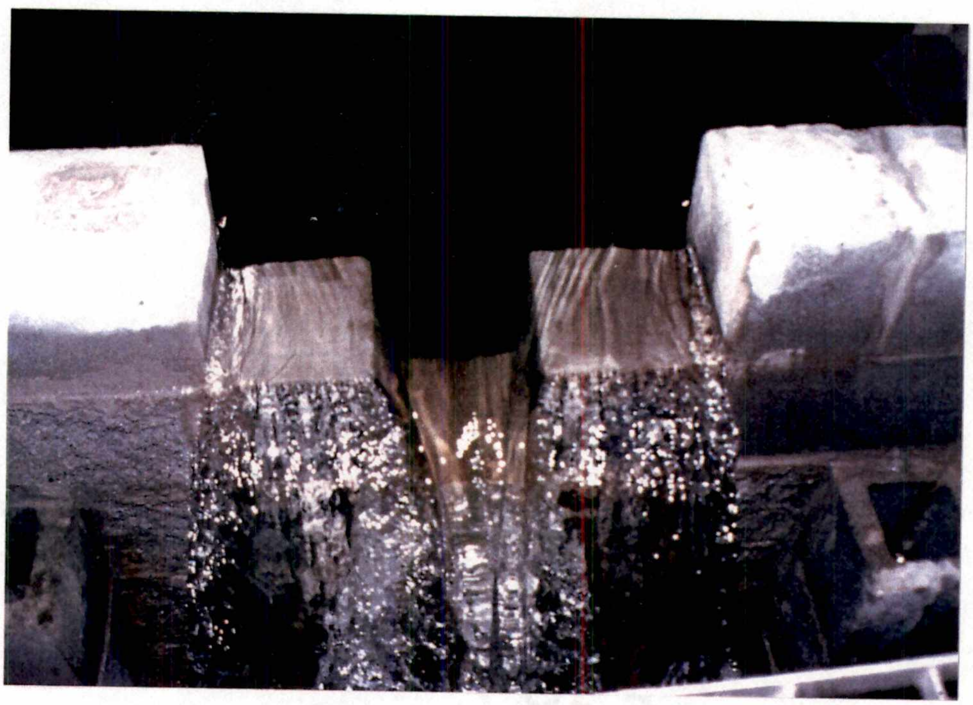

Plate No. 18 - Stacked-Weir (Test 11)

Plate No. 19

Concrete Block Discharge Structure No. 1

Plate No. 20

Concrete Block Discharge Structure No. 2

APPENDIX F

EXPERIMENTAL UNCERTAINTIES

When performing an experiment involving the gathering of laboratory data it is advisable to develop a rational method of evaluating the overall uncertainty of the experimental setup. One way to accomplish a quantitative evaluation of the data is provided by combining each of the known uncertainties associated with the components of the experimental setup into a single combined uncertainty value (33). For the study presented in this thesis the combiner uncertainty is given as

$$X_U = \pm\sqrt{X_V^2 + X_S^2 + X_L^2} \qquad (23)$$

where X_U = Combined Uncertainty of C_d (%); X_V = Venturi Reading Uncertainty (%); X_S = Staff Gage Reading Uncertainty (%); and X_L = Leak Rate Uncertainty (%).

Venturi Reading Uncertainty (X_V) - The precision with which the Mercury manometer can be read depends on the Venturi pressure drop (VPD) being evaluated. At high VPD (flow rates) it is difficult to read the manometer scale precisely due to fluctuations in the Mercury column. As discussed in Section III, the manometer can be read with a precision 0.005 ft when the Mercury column is stable. Based on a subjective evaluation of the ability of an observer to read the manometer the following uncertainties values are assumed:

VPD $\geq$ 4.000 ft	- Venturi Reading Uncertainty = 0.040 ft
4.000 > VPD $\geq$ 3.000 ft	- Venturi Reading Uncertainty = 0.030 ft
3.000 > VPD $\geq$ 2.000 ft	- Venturi Reading Uncertainty = 0.020 ft
2.000 > VPD $\geq$ 1.000 ft	- Venturi Reading Uncertainty = 0.010 ft
VPD < 1.000 ft	- Venturi Reading Uncertainty = 0.005 ft

These uncertainty values are used in conjunction with Equation 21 to evaluate the percentage change in C_d produced by an improper reading of the Mercury manometer by the amount indicated.

Staff Gage Uncertainty (X_s) - Much like the Mercury manometer, the precision with which the staff gage can be read depends on the rate at which water is flowing through the experimental setup. Based on a subjective evaluation of the ability of an observer to read the staff gage the following uncertainty values are assumed:

VPD $\geq$ 4.000 ft	- Staff Gage Reading Uncertainty = 0.030 ft
4.000 > VPD $\geq$ 3.000 ft	- Staff Gage Reading Uncertainty = 0.020 ft
3.000 > VPD $\geq$ 2.000 ft	- Staff Gage Reading Uncertainty = 0.010 ft
VPD < 2.000 ft	- Staff Gage Reading Uncertainty = 0.005 ft

These uncertainty values are used in conjunction with the weir and/or orifice equations shown in Section II to evaluate the percentage change in C_d caused by an improper reading of the staff gage by the amount indicated.

Wall Leak Rate Uncertainty (X_L) - Based on a detailed linear regression analysis of the leak rate data presented in Table 2, the maximum single point reading error in leak rate is approximately 2.25 gpm throughout the range of heads evaluated in this thesis. At each data point the percentage change in C_d caused by an error in leak rate estimation of 2.25 gpm is determined.

Combined Uncertainty (X_U) - Tables F1 to F8 provide detailed evaluations of the combined uncertainty for all weir only and orifice only tests. The stacked-weir and weir/orifice tests are not evaluated for combined uncertainty since the evaluation of these test is based on the results of the weir only and/or orifice only tests, as discussed in Section IV. When reviewing the results of the combined uncertainties, it is important to remember that the combined uncertainty values provided are based on the assumptions developed for each of the individual uncertainties.

EXPERIMENTAL UNCERTAINTIES									
RUN 1		STATISTICS	MEAN(%) –	0.5	STANDARD DEVIATION(%) –		0.4		
DATA POINT	ORIFICE HEAD (ft)	STAFF GAGE READING UNCERTAINTY (ft)	% STAFF GAGE UNCERTAINTY	VENTURI PRESSURE DROP (ft)	VENTURI READING UNCERTAINTY (ft)	% VENTURI UNCERTAINTY	POSSIBLE LEAK RATE ERROR (gpm)	% LEAK RATE ERROR	% TOTAL UNCERTAINTY
1	2.89	0.005	0.1	1.880	0.010	0.3	2.25	0.06	0.3
2	2.76	0.005	0.1	1.780	0.010	0.3	2.25	0.06	0.3
3	2.47	0.005	0.1	1.580	0.010	0.3	2.25	0.06	0.3
4	2.18	0.005	0.1	1.380	0.010	0.4	2.25	0.06	0.4
5	1.85	0.005	0.1	1.180	0.010	0.4	2.25	0.05	0.4
6	1.55	0.005	0.2	0.980	0.005	0.3	2.25	0.05	0.3
7	1.24	0.005	0.2	0.780	0.005	0.3	2.25	0.05	0.4
8	0.93	0.005	0.3	0.580	0.005	0.4	2.25	0.04	0.5
9	0.63	0.005	0.4	0.380	0.005	0.6	2.25	0.04	0.8
10	0.32	0.005	0.8	0.180	0.005	1.4	2.25	0.04	1.6

TABLE F1

Test No. 1 Combined Uncertainty for
Single Orifice Opening

EXPERIMENTAL UNCERTAINTIES									
RUN 2	STATISTICS:		MEAN(%) -	0.9	STANDARD DEVIATION(%) -		0.1		
DATA POINT	ORIFICE HEAD (ft)	STAFF GAGE READING UNCERTAINTY (ft)	% STAFF GAGE UNCERTAINTY	VENTURI PRESSURE DROP (ft)	VENTURI READING UNCERTAINTY (ft)	% VENTURI UNCERTAINTY	POSSIBLE LEAK RATE ERROR (gpm)	% LEAK RATE ERROR	% TOTAL UNCERTAINTY
1	1.837	0.030	0.8	4.420	0.04	0.4	2.25	0.03	0.9
2	1.737	0.030	0.8	4.150	0.04	0.4	2.25	0.03	0.9
3	1.607	0.030	0.9	3.800	0.04	0.4	2.25	0.03	1.0
4	1.532	0.030	0.9	3.610	0.04	0.4	2.25	0.03	1.0
5	1.452	0.020	0.6	3.400	0.03	0.3	2.25	0.03	0.7
6	1.337	0.020	0.7	3.200	0.03	0.3	2.25	0.02	0.8
7	1.225	0.020	0.8	2.960	0.03	0.4	2.25	0.02	0.9
8	1.132	0.020	0.8	2.780	0.03	0.4	2.25	0.02	0.9
9	1.052	0.020	0.9	2.600	0.03	0.4	2.25	0.02	1.0
10	0.942	0.020	1.0	2.280	0.03	0.4	2.25	0.02	1.1
11	0.822	0.010	0.6	1.980	0.02	0.3	2.25	0.02	0.6
12	0.722	0.010	0.6	1.690	0.02	0.3	2.25	0.02	0.7
13	0.607	0.010	0.8	1.390	0.02	0.3	2.25	0.02	0.9
14	0.487	0.010	1.0	1.070	0.02	0.4	2.25	0.02	1.1
15	0.357	0.005	0.6	0.760	0.01	0.2	2.25	0.02	0.7

TABLE F2

Test No. 2 Combined Uncertainty for
Double Orifice Opening

EXPERIMENTAL UNCERTAINTIES									
RUN 3		STATISTICS:	MEAN(%) –	0.4	STANDARD DEVIATION(%) –		0.1		
DATA POINT	ORIFICE HEAD (ft)	STAFF GAGE READING UNCERTAINTY (ft)	% STAFF GAGE UNCERTAINTY	VENTURI PRESSURE DROP (ft)	VENTURI READING UNCERTAINTY (ft)	% VENTURI UNCERTAINTY	POSSIBLE LEAK RATE ERROR (gpm)	% LEAK RATE ERROR	% TOTAL UNCERTAINTY
1	2.747	0.005	0.1	1.640	0.010	0.2	2.25	0.06	0.3
2	2.497	0.005	0.1	1.500	0.010	0.3	2.25	0.06	0.3
3	2.367	0.005	0.1	1.420	0.010	0.3	2.25	0.06	0.3
4	2.242	0.005	0.1	1.340	0.010	0.3	2.25	0.06	0.3
5	2.117	0.005	0.1	1.250	0.010	0.3	2.25	0.06	0.3
6	1.972	0.005	0.1	1.170	0.010	0.3	2.25	0.06	0.3
7	1.847	0.005	0.1	1.080	0.010	0.3	2.25	0.06	0.3
8	1.717	0.005	0.1	1.010	0.010	0.3	2.25	0.05	0.3
9	1.582	0.005	0.1	0.900	0.010	0.3	2.25	0.05	0.3
10	1.427	0.005	0.1	0.810	0.010	0.3	2.25	0.05	0.3
11	1.257	0.005	0.1	0.700	0.010	0.3	2.25	0.05	0.4
12	1.097	0.005	0.2	0.630	0.010	0.3	2.25	0.05	0.4
13	0.927	0.005	0.2	0.530	0.010	0.3	2.25	0.04	0.4
14	0.722	0.005	0.3	0.400	0.010	0.4	2.25	0.04	0.5
15	0.582	0.005	0.4	0.320	0.010	0.4	2.25	0.04	0.6
16	0.357	0.005	0.7	0.190	0.010	0.4	2.25	0.04	0.8

TABLE F3

Test No. 3 Combined Uncertainty for
Single Orifice Opening Elevated Channel Bottom

EXPERIMENTAL UNCERTAINTIES									
RUN 4		STATISTICS:	MEAN (%) –	0.7	STANDARD DEVIATION (%) –		0.2		
DATA POINT	ORIFICE HEAD (ft)	STAFF GAGE READING UNCERTAINTY (ft)	% STAFF GAGE UNCERTAINTY	VENTURI PRESSURE DROP (ft)	VENTURI READING UNCERTAINTY (ft)	% VENTURI UNCERTAINTY	POSSIBLE LEAK RATE UNCERTAINTY (gpm)	% LEAK RATE UNCERTAINTY	% TOTAL UNCERTAINTY
1	2.017	0.030	0.7	4.270	0.040	0.5	2.25	0.03	0.9
2	1.892	0.030	0.8	4.040	0.040	0.5	2.25	0.03	0.9
3	1.837	0.020	0.5	3.840	0.040	0.4	2.25	0.03	0.7
4	1.752	0.020	0.6	3.670	0.040	0.4	2.25	0.03	0.7
5	1.647	0.020	0.6	3.440	0.030	0.4	2.25	0.03	0.7
6	1.567	0.020	0.6	3.280	0.030	0.5	2.25	0.03	0.7
7	1.432	0.010	0.3	2.960	0.020	0.3	2.25	0.03	0.5
8	1.292	0.010	0.4	2.660	0.020	0.4	2.25	0.03	0.5
9	1.167	0.010	0.4	2.490	0.020	0.4	2.25	0.03	0.5
10	1.112	0.010	0.4	2.240	0.020	0.4	2.25	0.03	0.6
11	1.022	0.010	0.5	2.070	0.020	0.5	2.25	0.02	0.7
12	0.897	0.005	0.3	1.820	0.010	0.3	2.25	0.02	0.4
13	0.752	0.005	0.3	1.480	0.010	0.3	2.25	0.02	0.5
14	0.627	0.005	0.4	1.200	0.010	0.4	2.25	0.02	0.6
15	0.467	0.005	0.5	0.880	0.005	0.3	2.25	0.02	0.6
16	0.297	0.005	0.8	0.580	0.005	0.4	2.25	0.01	0.9

TABLE F4

Test No. 4 Combined Uncertainty for
Double Orifice Opening Elevated Channel Bottom

EXPERIMENTAL UNCERTAINTIES									
RUN 7		STATISTICS:	MEAN(%) –	3.2	STANDARD DEVIATION(%) –		1.7		
DATA POINT	ORIFICE HEAD (ft)	STAFF GAGE READING UNCERTAINTY (f	% STAFF GAGE UNCERTAINTY	VENTURI PRESSURE DROP (ft)	VENTURI READING UNCERTAINTY (ft)	% VENTURI UNCERTAINTY	POSSIBLE LEAK RATE ERROR (gpm)	% LEAK RATE ERROR	% TOTAL UNCERTAINTY
1	0.800	0.030	5.7	4.550	0.040	0.4	2.25	0.03	5.7
2	0.785	0.030	5.8	4.300	0.040	0.5	2.25	0.03	5.8
3	0.770	0.030	5.9	4.040	0.040	0.4	2.25	0.04	5.9
4	0.755	0.020	4.0	3.760	0.030	0.4	2.25	0.04	4.0
5	0.735	0.020	4.1	3.440	0.030	0.4	2.25	0.04	4.1
6	0.705	0.020	4.3	3.050	0.030	0.4	2.25	0.04	4.3
7	0.670	0.010	2.2	2.600	0.020	0.4	2.25	0.04	2.2
8	0.640	0.010	2.4	2.260	0.020	0.4	2.25	0.05	2.4
9	0.600	0.005	1.3	1.880	0.010	0.3	2.25	0.05	1.3
10	0.560	0.005	1.4	1.460	0.010	0.3	2.25	0.05	0.3
11	0.500	0.005	1.5	1.060	0.010	0.5	2.25	0.06	1.5
12	0.450	0.005	1.7	0.720	0.005	0.3	2.25	0.07	1.7
13	0.405	0.005	1.9	0.560	0.005	0.4	2.25	0.08	1.9
14	0.340	0.005	2.2	0.340	0.005	0.7	2.25	0.10	2.3
15	0.260	0.005	2.9	0.180	0.005	1.4	2.25	0.14	3.2
16	0.215	0.005	3.5	0.120	0.005	2.0	2.25	0.16	4.0

TABLE F5

Test No. 7 Combined Uncertainty for
16" Wide x 16" High Weir
Elevated Channel Bottom

EXPERIMENTAL UNCERTAINTIES									
RUN 8		STATISTICS:	MEAN(%) -	3.1	STANDARD DEVIATION(%) -		1.7		
DATA POINT	ORIFICE HEAD (ft)	STAFF GAGE READING UNCERTAINTY (ft)	% STAFF GAGE UNCERTAINTY	VENTURI PRESSURE DROP (ft)	VENTURI READING UNCERTAINTY (ft)	% VENTURI UNCERTAINTY	POSSIBLE LEAK RATE ERROR (gpm)	% LEAK RATE ERROR	% TOTAL UNCERTAINTY
1	0.800	0.030	5.7	4.630	0.040	0.4	2.25	0.03	5.7
2	0.795	0.030	5.7	4.560	0.040	0.4	2.25	0.03	5.7
3	0.780	0.030	5.8	4.270	0.040	0.5	2.25	0.03	5.8
4	0.775	0.030	5.9	4.070	0.040	0.5	2.25	0.04	5.9
5	0.755	0.020	4.0	3.840	0.030	0.4	2.25	0.04	4.0
6	0.745	0.020	4.1	3.590	0.030	0.4	2.25	0.04	4.1
7	0.735	0.020	4.1	3.390	0.030	0.4	2.25	0.04	4.1
8	0.705	0.020	4.3	3.050	0.030	0.5	2.25	0.04	4.3
9	0.690	0.010	2.2	2.830	0.020	0.3	2.25	0.04	2.2
10	0.675	0.010	2.2	2.590	0.020	0.4	2.25	0.04	2.2
11	0.650	0.010	2.3	2.350	0.020	0.4	2.25	0.04	2.4
12	0.620	0.010	2.4	2.050	0.020	0.5	2.25	0.05	2.5
13	0.600	0.005	1.3	1.800	0.010	0.3	2.25	0.05	1.3
14	0.565	0.005	1.3	1.490	0.010	0.3	2.25	0.05	1.4
15	0.530	0.005	1.4	1.250	0.010	0.4	2.25	0.06	1.5
16	0.480	0.005	1.6	0.960	0.005	0.3	2.25	0.07	1.6
17	0.440	0.005	1.7	0.740	0.005	0.3	2.25	0.07	1.7
18	0.400	0.005	1.9	0.560	0.005	0.4	2.25	0.08	1.9
19	0.345	0.005	2.2	0.370	0.005	0.7	2.25	0.10	2.3
20	0.275	0.005	2.7	0.190	0.005	1.3	2.25	0.13	3.0
21	0.225	0.005	3.4	0.110	0.005	2.2	2.25	0.17	4.0

TABLE F6

Test No. 8 Combined Uncertainty for
16" Wide x 16" High Weir

EXPERIMENTAL UNCERTAINTIES									
RUN 10		STATISTICS:	MEAN(%) –	2.1	STANDARD DEVIATION(%) –		1.0		
DATA POINT	ORIFICE HEAD (ft)	STAFF GAGE READING UNCERTAINTY (ft)	% STAFF GAGE UNCERTAINTY	VENTURI PRESSURE DROP (ft)	VENTURI READING UNCERTAINTY (ft)	% VENTURI UNCERTAINTY	POSSIBLE LEAK RATE ERROR (gpm)	% LEAK RATE ERROR	% TOTAL UNCERTAINTY
1	1.230	0.030	3.7	4.610	0.040	0.4	2.25	0.04	3.7
2	1.205	0.030	3.8	4.440	0.040	0.4	2.25	0.04	3.8
3	1.200	0.030	3.8	4.350	0.040	0.5	2.25	0.04	3.8
4	1.190	0.030	3.8	4.220	0.040	0.5	2.25	0.04	3.8
5	1.175	0.030	3.9	4.090	0.040	0.5	2.25	0.04	3.9
6	1.160	0.020	2.6	3.920	0.030	0.4	2.25	0.04	2.6
7	1.150	0.020	2.6	3.780	0.030	0.4	2.25	0.04	2.6
8	1.135	0.020	2.7	3.570	0.030	0.4	2.25	0.04	2.7
9	1.120	0.020	2.7	3.370	0.030	0.4	2.25	0.04	2.7
10	1.095	0.020	2.8	3.140	0.030	0.5	2.25	0.05	2.8
11	1.075	0.010	1.4	2.940	0.020	0.3	2.25	0.05	1.4
12	1.045	0.010	1.4	2.750	0.020	0.4	2.25	0.05	1.5
13	1.020	0.010	1.5	2.540	0.020	0.4	2.25	0.05	1.5
14	1.000	0.010	1.5	2.360	0.020	0.4	2.25	0.05	1.6
15	0.975	0.010	1.5	2.170	0.020	0.5	2.25	0.05	1.6
16	0.950	0.005	0.8	1.980	0.010	0.2	2.25	0.05	0.8
17	0.915	0.005	0.8	1.760	0.010	0.3	2.25	0.06	0.9
18	0.880	0.005	0.9	1.560	0.010	0.3	2.25	0.06	0.9
19	0.850	0.005	0.9	1.410	0.010	0.3	2.25	0.06	1.0
20	0.800	0.005	0.9	1.160	0.010	0.4	2.25	0.07	1.0
21	0.750	0.005	1.0	0.970	0.005	0.3	2.25	0.07	1.0
22	0.670	0.005	1.1	0.660	0.005	0.4	2.25	0.08	1.2
23	0.630	0.005	1.2	0.560	0.005	0.4	2.25	0.09	1.3
24	0.580	0.005	1.3	0.450	0.005	0.5	2.25	0.10	1.4
25	0.540	0.005	1.4	0.370	0.005	0.7	2.25	0.11	1.5
26	0.500	0.005	1.5	0.280	0.005	0.9	2.25	0.12	1.7
27	0.440	0.005	1.7	0.190	0.005	1.3	2.25	0.14	2.1
28	0.380	0.005	2.0	0.140	0.005	1.7	2.25	0.16	2.6
29	0.340	0.005	2.2	0.100	0.005	2.4	2.25	0.19	3.3

TABLE F7

Test No. 10 Combined Uncertainty for
8" Wide x 16" High Weir

EXPERIMENTAL UNCERTAINTIES									
RUN 12	STATISTICS:		MEAN(%) –	4.0	STANDARD DEVIATION(%) –		2.2		
DATA POINT	ORIFICE HEAD (ft)	STAFF GAGE READING UNCERTAINTY (ft)	% STAFF GAGE UNCERTAINTY	VENTURI PRESSURE DROP (ft)	VENTURI READING UNCERTAINTY (ft)	% VENTURI UNCERTAINTY	POSSIBLE LEAK RATE ERROR (gpm)	% LEAK RATE ERROR	% TOTAL UNCERTAINTY
1	0.618	0.030	7.4	4.610	0.040	0.4	2.25	0.04	7.4
2	0.613	0.030	7.4	4.520	0.040	0.4	2.25	0.04	7.4
3	0.608	0.030	7.5	4.230	0.040	0.5	2.25	0.04	7.5
4	0.603	0.030	7.6	4.070	0.040	0.5	2.25	0.04	7.6
5	0.588	0.020	5.1	3.820	0.030	0.4	2.25	0.04	5.2
6	0.573	0.020	5.3	3.580	0.030	0.4	2.25	0.04	5.3
7	0.563	0.020	5.4	3.270	0.030	0.5	2.25	0.05	5.4
8	0.548	0.010	2.7	2.940	0.020	0.3	2.25	0.05	2.8
9	0.528	0.010	2.9	2.590	0.020	0.4	2.25	0.05	2.9
10	0.508	0.010	3.0	2.240	0.020	0.4	2.25	0.05	3.0
11	0.478	0.005	1.6	1.930	0.010	0.3	2.25	0.06	1.6
12	0.463	0.005	1.6	1.760	0.010	0.3	2.25	0.06	1.6
13	0.433	0.005	1.7	1.460	0.010	0.3	2.25	0.07	1.8
14	0.403	0.005	1.9	1.160	0.010	0.4	2.25	0.07	1.9
15	0.358	0.005	2.1	0.840	0.005	0.3	2.25	0.08	2.1
16	0.313	0.005	2.4	0.570	0.005	0.4	2.25	0.10	2.4
17	0.273	0.005	2.8	0.400	0.005	0.6	2.25	0.12	2.8

TABLE F8

Test No. 12 Combined Uncertainty for
24" Wide x 8" High Weir

VITA

Anthony Nelson Wylie was born October 4, 1943 in Kingsport, Tennessee. He attended school in Oak Ridge, Tennessee where he graduated from Oak Ridge High School in June 1962. In January 1963 he entered the University of Tennessee, Knoxville where he graduated with a BS in Civil Engineering in December 1967. While at the University he was a member of Chi Epsilon, a Distinguished Graduate of ROTC, and the winner of the American Military Engineers Gold Medal for Excellence as the outstanding engineering student in ROTC.

Mr. Wylie first attended Graduate School at the University of Tennessee, Knoxville in January 1974. He was selected as a member of Phi Kappa Phi in May 1979 and, after a long delay, graduated with a Master's degree in Environmental Engineering in December 1991.

Mr. Wylie has served 24 years in the US Air Force and the Tennessee Air National Guard, and he is currently Commander of the 151st Air Refueling Squadron Tennessee Air National Guard. He has worked in consulting engineering in the East Tennessee area since resigning his regular commission in July 1972, and he is currently a Department Head in Waste Management Design for Martin Marietta Energy Systems in Oak Ridge, Tennessee.