CALIBRATION OF LONG CRESTED WEIR DISCHARGE COEFFICIENT

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ABSTRACT

Long crested weirs are used in open-channel irrigation distribution systems to minimize fluctuations in the canal water surface above canal turnouts. The object of this report was to develop an equation for the long crested weir discharge coefficient so that the weirs could be used to measure canal flowrates. Sixty-seven different weir models were tested and several general discharge equations were developed which predicted the flowrate to within plus or minus five percent.

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CHAPTER I

INTRODUCTION

The settlement of the arid and semi-arid western and southwestern United States and the development of agriculture in many of the arid regions of the world has depended largely on irrigation. As settlement and development of these areas has increased, demands for quality water have also increased. Water in these areas is generally in short supply and must be used frugally to meet the current and future needs of agriculture, industry, and municipalities (Brosz, 1971; Jensen, 1990).

Many factors influence the efficient use of water in irrigated agriculture. Water may be lost through seepage and evaporation from irrigation channels in irrigation distribution systems. Irregular field topography and poor irrigation delivery systems can cause variable application of water and excess runoff and deep percolation in some parts of the field while not providing enough water to other parts of the field. The frequency, flowrate, and duration of water delivery to the farm is also a factor in efficient on-farm water use.

For farmers to effectively use irrigation water, they must be able to request and receive water when their crop needs it. Meeting the crop's water needs requires a flexible delivery irrigation system (Merriam, 1977). Though this system potentially increases efficiency of water use by providing it to the farmers on demand, it also has the disadvantage of varying flowrates and water surface levels in the supply canal. Fluctuations in the surface level of the canal cause changes in the canal turnout discharges to the fields (Clemmens, 1984). Changes in flowrate to the fields during an irrigation "set" can lead to over-watering or under-watering in different parts of the field.

A combination of a weir and an orifice turnout can be used in the canal to help alleviate the problem of varied discharges through the turnout. An orifice turnout in the side of a canal will give a discharge that is a function of the difference between the water level in the main canal and the water level in the lateral. If the water surface level in the lateral is fairly constant, a rise in the canal water level causes increased discharge, and a drop in the canal water level will reduce the flow through the orifice. A weir may be placed in the main canal downstream of where the outlet is located in the side of the canal. The weir in the main channel can provide for a more constant water level above the orifice turnout over a range of flows in the canal. Often the width of the canal is not sufficient for a straight weir to be used and still keep the water level within the desired tolerances.

Long crested weirs can be used in the canal to provide added weir crest length for any given width of canal. The long crested weir can maintain the water surface elevation within small tolerances as the flowrate in the canal fluctuates. Therefore, a long crested weir and orifice turnout combination can be used to provide a fairly constant turnout discharge in spite of fluctuations of flowrate in the canal. One disadvantage of using long crested weirs compared to some other standard types of weirs is that other weirs can be used to measure the flowrate in the canal accurately. These standard types of weirs have been calibrated for use as flow measurement structures, and accurate discharge equations are available for the standard weirs. However, because no accurate discharge coefficients have been determined for the long crested weir, its usefulness as an accurate measuring device is still in question.

OBJECTIVE

The objective of this research is to develop an equation which can accurately predict the flowrate over a long crested weir. The procedures to accomplish this objective are as follows: (1) use scale models of long crested weirs with sharp crests in an open-channel in the University of Wyoming Civil Engineering Hydraulics Laboratory to determine the discharge characteristics of the weirs; (2) develop dimensionless Pi terms which are in terms of the weir geometry; (3) vary the weir geometry and flowrates to obtain varying values for the

Pi terms; (4) use the Pi term values in a regression analysis to develop an empirical equation for the long crested weir discharge coefficient.

Hydraulic modelling will allow results from the model tests to be analyzed to determine discharge coefficients which will be applicable to full-size long crested weirs. With known discharge coefficients, long crested weirs constructed in the future may be used not only to maintain more constant canal water levels for orifice turnouts but also to measure the flowrate in the canal.

It is anticipated that long crested weirs used for flow measurement in the canal would maintain fairly constant water levels above the canal orifice turnouts and therefore provide the farmers with constant flowrates and allow for more effective on-farm water use. In addition, the weir should also enable the operators of the canal to better control the water in the canal. By knowing the flow into and out of the canals in the distribution system, the operators may be able to minimize water wasted due to excess flows at the end of the distribution system and also avoid the problem of not providing enough water to the farms at the end of the system.

Using the flow measuring weirs in unlined canals should give canal operators values of flow into and out of various reaches of the canal between outlets and may give indications of areas where seepage losses are unacceptably high. Knowing where the high loss areas of the canal are would give the canal management an opportunity to concentrate corrective measures where they would likely be most effective in improving canal performance. These anticipated benefits of long crested weirs should be valuable to the canal management as well as help increase the effectiveness of the delivery system in providing the proper flows so that farmers can make better use of the water provided to them.

ORGANIZATION

This report contains four other chapters. Chapter Two contains a review of pertinent literature associated with weirs. Chapter Three describes the laboratory apparatus and the experimental procedures. Chapter Four presents the analysis of data obtained from the experiments and contains a discussion of the results of the data analysis. Chapter Five contains the summary and conclusions of the research.

CHAPTER II

LITERATURE REVIEW

Agricultural productivity in many arid and semi-arid areas of the world depends largely on irrigation. In 1979 it was estimated that only 13 percent of the total arable land in the world was irrigated. However, the value of the crops produced on the irrigated land was 34 percent of the total value of the world's agricultural crops (Jensen, 1983). Surface irrigation (gravity flow of water over the ground surface) is the oldest irrigation method, and it is still the most common irrigation method used throughout the world. Hillel (1989) gives an estimate that more than 95 percent of the world's irrigated land is irrigated using surface irrigation.

The effects of irrigation can be both beneficial and harmful, and the amount of harm or benefit to the land depends on the proper management and use of available water resources. Barren deserts can be made productive farms with the addition of proper amounts of irrigation water. In semi-arid areas where reliable irrigation water can complement water received from intermittent rains, crop yields can be increased significantly over what is produced with dryland farming. However, the excessive application of water without proper drainage has brought salinization, waterlogging, and a total loss of productivity to some agricultural areas. Other environmental problems also occur if the irrigation water is not carefully and efficiently used. Too much irrigation water applied to fields often results in the leaching of pesticides and fertilizers from the crop root zone down into the ground water causing possible health hazards to people who use water from nearby wells. Excess irrigation water that does not infiltrate the soil often carries chemicals and large amounts of sediment with it as it runs off of the field. As the field runoff returns to streams or rivers, it can cause water quality problems for downstream wildlife and water users (Bowman, 1971).

Water use conflicts have arisen in many irrigated areas where good quality water is in short supply. Competition for water use between agriculture and municipalities has led to an increase in the cost of irrigation water in some areas. Increased water costs, higher quality discharge requirements for agricultural runoff, and problems with groundwater pollution have prompted many farmers and irrigation districts to make improvements which will enhance water use efficiency (Jensen, 1983).

WATER MEASUREMENT

Water measurement is essential to the effective use of irrigation water. Examples of the importance of irrigation flow measurement are discussed in the following pages.

Walker (1972) describes some problems of irrigation districts in the Sevier River Basin in Utah. By 1890 all the land along the river that could be developed by direct diversion had been brought under cultivation. By 1922 enough reservoir capacity had been developed to completely control the flow of the river in all subsequent years except for two. The area is semi-arid to arid and years of drought are common.

In spite of complete river control and many years of drought, the average on-farm water use efficiency was below 45 percent. Rather than being stored for dry years, extra water in wet years is lost due to poor efficiencies and excess diversions. Canal conveyance efficiency was poor until canal lining was installed, but this still left room for improvement in canal management and on-farm water management. Good flow measurement is essential in improving the performance of the system.

Some of the problems found in the Sevier River Basin may be typical of many old canal systems in the western U.S. Most canal systems were built when water seemed cheap and abundant, and flow measurement structures were not initially installed as a part of these systems.

In the early years of the system on the Sevier River, an engineer was hired to design and install gates which could measure the flows delivered to the farmers from the system. The gates, called Cotteral gates after the engineer who designed them, were calibrated and installed, and the discharge curves for the gates were used as a basis of charging the farmers for the water delivered by the canal company. By 1960 almost all of the Cotteral gates had been replaced by different turnout gates. More recently some canal lining has been completed, and with the lining came replacement of all the outlet gates. The canal company discovered that their estimated delivery losses actually increased from 25 to 35 percent after having made the improvements to the canal! The canal lining had reduced seepage losses, but the higher velocities in the channels and the newer gates resulted in farmers getting more water than was being saved by the canal lining. The canal company was still using the Cotteral discharge curves to determine water delivered to the farmers even though there weren't any Cotteral gates in the system. There were a number of different types of outlet gates used in the system, but when the Cotteral gates had been replaced, the new gate discharge rating curves were not developed and used. Only by updating the discharge rating curves for the various gates currently in use could the canal companies get a more reasonable estimate of water deliveries and losses.

Other problems were found with the system management. One canal company had four "water masters" in five years. With each new water master came a period of high losses and low efficiency until enough experience was gained to properly manage the system. "The entire system was an experience rated system of turns, notches, pegs, holes, and threads" (Walker, 1972). However, once enough experience was gained to master the system, the system losses decreased by as much as 20 percent. This type of experience based water control system is common with older canals.

Walker (1972) estimated that using a combination of surface and underground waters (pump from the aquifers in dry years), limiting diversions in wet years, and improving efficiency by 10 percent would eliminate water shortages on the farms currently under irrigation. He also noted that adopting standard structures and methods, together with measurement controls at key points in the system, should provide a base for continuous system improvement. The farmers need to know what the flow is to their fields so they can keep track of their allotment of water and use it most effectively. Standard measurement and outlet structures could allow all farmers on the system to be charged by the same standards and allow the farmers to know what their flowrates are simply by being familiar with the discharge characteristics of the outlets. Experience indicates that farmers are not inclined to use devices which require numerous measurements and complicated mathematical formulas or volumes of discharge tables; they prefer structures gauged for reading the discharge directly and flow measurement devices which are as inexpensive and simple as possible (Walker, 1977).

Good water measurement is essential even where water shortages do not exist; the lack of good water measurement can cause drainage problems, excessive seepage losses, canal breaks, poor on-farm water practices, excessive erosion and sedimentation, and poor water quality. This is demonstrated by two older water districts in Idaho; one is a large district of 65,000 acres, and the other is only 990 acres. No flow measurement structures were used in the distribution systems of either district. After many years of operation with excess water deliveries, many farms in the 65,000 acre district show reduced productivity due to problems caused by a rising water table. The crop production of farms in this district is only about 80 percent of the production of farms in a neighboring district with similar soil and climate conditions, and the neighboring district only uses about one quarter as much water (Schaack, 1975). The canal in the 990 acre district carries about 55 cfs and is composed entirely of earthen channels with wood and concrete structures used for control. The farms all use surface irrigation to apply the water. The overall irrigation efficiency is less than 20 percent, and the area is plagued by high water tables (Busch, 1975). In these districts, excess water caused decreased productivity. Water measurement is essential to good water management for both limited and abundant supplies of water.

SHARP CRESTED WEIRS

Sharp crested weirs are relatively simple devices for open channel flow measurement. There are many types of sharp crested weirs in use, but the most common are rectangular weirs, suppressed rectangular weirs, triangular (V-notch) weirs, and trapezoidal (Cipolletti) weirs (Figure 1). All of these weirs have a number of features in common: the weir crest is a sharp metal plate, the flow jet over the crest must be fully aerated, and the discharge is proportional to the upstream head on the weir.

Many discharge equations for different types of weirs have been developed in laboratory experiments. Schoder (1929) published the results of over 2,000 discharge measurements for 1,512 different heads on rectangular suppressed weirs of different heights and with different velocity profiles. These experiments were performed at Cornell University between 1904 and 1920 and utilized an open channel with a number of weir arrangements. Although weirs of various crest heights were tested, all of the weirs extended the full width of the channel (rectangular suppressed weirs). Velocities were measured in the upstream channel, and baffles and stilling rafts were used to produce different velocity profiles. Discharge volumes were measured in a weighing tank at the end of the open channel and weir apparatus.

The experimenters arrived at a number of conclusions based on the weir measurements. The Francis formula was found

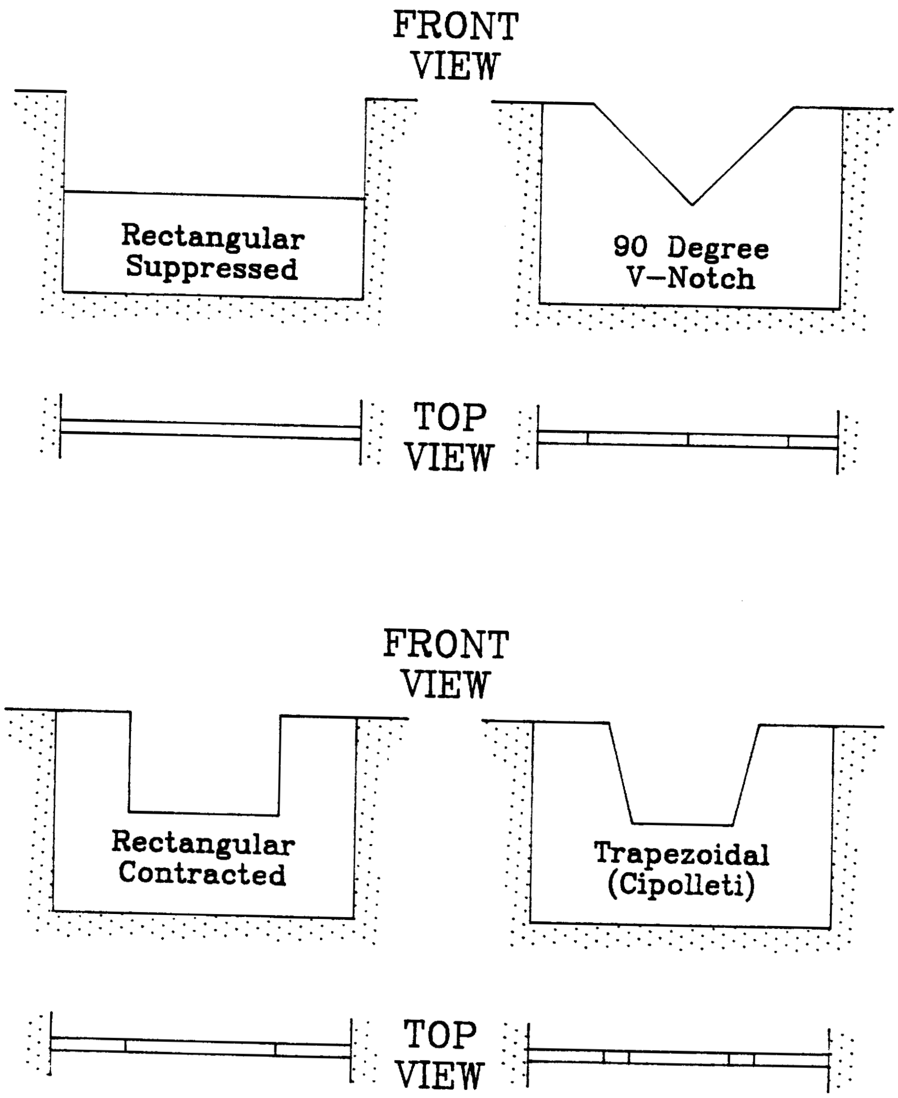

Figure 1. Views of Several Standard Sharp Crested Weirs.

acceptably accurate for weirs with sharp, square edges, a smooth vertical upstream face, a deep upstream pool behind the weir, and negligible effects of approach velocity. It is widely understood that weir discharge equations are accurate to within about two percent as long as the head on the weir is carefully measured; caution should be used when assuming this range of accuracy. The head may be measured carefully, but there are a number of other conditions which may affect discharge by several percent. Slight roundness of the upstream weir edge results in the weir discharge increasing by several percent. Roughness of the weir plate near the crest was also found to increase the discharge even when the crest was sharp. Velocity distribution was also found to influence the weir discharge. It was concluded that if standard weir discharge equations are to be used, careful consideration should be given to measurement of the head on the weir but also to velocities in the upstream channel, weir sharpness, and general conditions of the weir plate (Schoder, 1929).

Ackers (1978) gives five discharge equations for full-width weirs. Each equation is accepted in various parts of the world and for various applications. Ackers points out that no one equation is entirely right and all others wrong; the equations agree with the experimental data obtained by the various experimenters in the development of their formulas. Different weir equations seem to give better results for different conditions (eg. high heads on low weirs or low heads on high weirs). In all cases, the weirs used for flow measurement should be built to standard conditions, and the upstream channel should be straight and uniform for a distance of at least ten times the channel width so that no irregular flow conditions will be present at the weir.

Some of the standard conditions given for small measurement weirs are described by Kraatz and Mahajan (1975). The weir crest should have a square upstream edge with a top thickness of between 1 and 2 mm. The downstream face of the weir should have a camfer of at least 45° so that the water jet springs clear of the weir. For contracted weirs, the crest height should be greater than twice the depth of the maximum head on the weir, and the sides of the weir opening should be at least a distance of twice the head from the side walls of the channel. Conditions other than these could affect the shape of the nappe (the water jet over the weir) and alter the weir discharge. The head on the weir (h) should meet the following criteria: $6 \text{ cm} \leq h \leq 60 \text{ cm}$, and the head on trapezoidal and rectangular weirs should be less than or equal to one third of the crest length. Very low heads should be avoided because the jet will cling to the weir face rather than springing clear. The weir crest must be level and straight. The weir crest must be higher than the downstream water surface so that the weir can discharge freely. Sharp crested weirs are not intended to operate under submerged flow conditions.

The use of sharp crested weirs is limited by several constraints. Because weirs trap sediments very effectively, weirs should be used only where the flow is relatively free of sediments and debris. Weirs used in flows carrying sediment should have gates in the bottom that can be opened to flush the sediments from behind the weir, or other methods for periodic sediment removal should be provided to insure that the sediment does not accumulate excessively behind the weir. A free fall of the nappe is required for sharp crested weirs to operate properly, and this requires some fall in the channel. Sharp crested weirs may not be practical to use in channels with very flat slopes because the backwater curve created behind the weir may extend for miles and require expensive raising of the channel banks to contain the water. The bottom of the channel directly downstream of the weir crest is subjected to high pressures due to the impact of the falling nappe, and channel protection is required in this area (Papoutsi-Psychoudaki, 1988). Due to the extensive channel protection that would be required for weirs with high heads and large flow rates, sharp crested weirs are generally limited to relatively small-scale applications (the Bureau of Reclamation, 1974 gives a maximum flowrate in their sharp crested weir discharge tables of about 100 cfs).

LONG CRESTED WEIRS

Long crested weirs provide more weir crest length by installing the weir at some configuration other than perpendicular to the channel. These configurations may be diagonal weirs, duckbill weirs, or labyrinth weirs. Figure 2 gives a plan view of several types of long crested weirs. The benefits of long crested weirs over standard weirs is that long crested weirs will pass more flow with less head variation on the weir than standard weirs. Because the flow is spread out over more crest length, an increase in flow produces a smaller increase in head than occurs for standard weirs.

Labyrinth weirs are simply long crested weirs consisting of a series of duckbill-type weirs placed side by side across the channel. This type of weir has been used for spillway or overflow structures of large reservoirs. They have the advantage of providing long overflow crest lengths even when site constraints require that the spillway channel width be limited. The Ute Dam on the Canadian River in New Mexico uses a 14-cycle labyrinth weir in its spillway. The total crest length is 3,360 feet in a spillway width of 840 feet. The design discharge is 590,000 cfs for a head of 19 feet on the weir (Bureau of Reclamation, 1987). The Beni Behdel Dam in Algeria has a labyrinth weir crest length of 1200 m in a channel 80 m wide and is designed to pass a flood of 1000 m^3/sec at a head of only 0.5 m. A standard weir 80 m long with a head of 0.5 m would only pass a flow of about 95 m^3/sec (Hay, 1970).

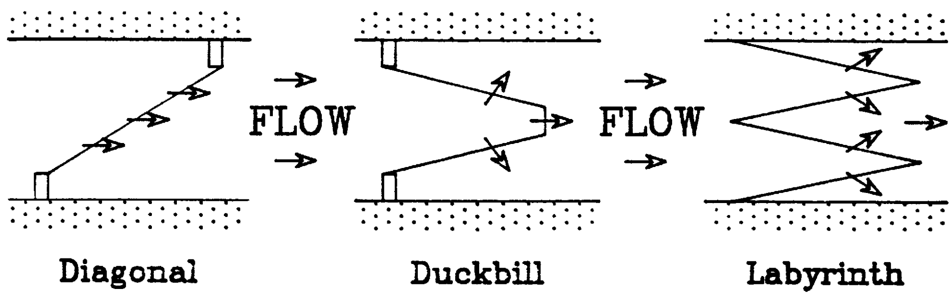

Figure 2. Plan Views of Three Types of Long Crested Weirs.

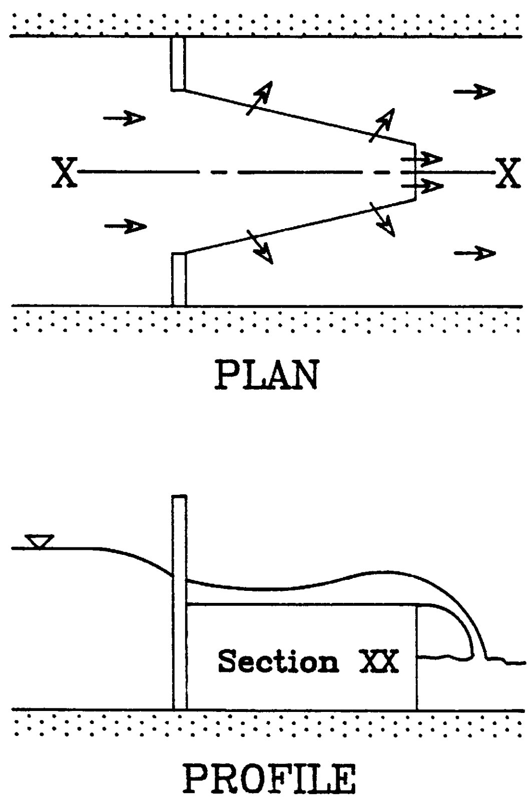

Figure 3. Water Surface Profile of Flow in a Duckbill Weir.

The performance of labyrinth weirs depends on the weir configuration, operating heads, and ratio of weir crest length to channel width. The flow patterns over the labyrinth weir can be very complex. As the flow enters one cycle of the weir, the water surface tends to drop because of the constriction caused by the weir. As the flow continues down the weir, the weir continues to constrict the flow, but the flow rate which approaches the downstream end of the weir is decreasing because of the flow which passes over the weir crest into the downstream channel. The flow passing over the weir crest tends to cause a rise in the water surface in the weir because the water in the weir reacts to the lost flow. The contraction and decreased flowrate create a gradual rise in the water surface as it approaches the downstream end of the weir (a spatially varied flow phenomena). Therefore, the head of the water passing over the weir is not constant for the entire length of the weir crest (Figure 3). This phenomena tends to reduce weir performance at high upstream heads and high approach velocities.

Hay (1970) used labyrinth weir models in a channel 16 ft long 3 ft wide and 1.2 ft deep to test the relationship of weir performance (Ql/Qn) to the following dimensionless parameters: h/p, w/p, ℓ/w, α, and n. Ql is the discharge of the long crested weir, and Qn is the discharge of a normal rectangular weir with the same crest type. The "h" is the head on the weir, and the "p" is the crest height from the bottom of the channel. The width of one cycle of the weir is "w", and "ℓ" is the length of the long crested weir's crest. "α" is the angle of the sides of the weir to the direction of flow, and "n" is the number of cycles of trapezoids or triangles in the long crested weir. The model weir tests showed that performance was independent of n, but it decreased for high ℓ/w and h/p values. Higher α values (triangular plan views) generally showed the best performance as long as there was no nappe interference between neighboring weir cycles. As long as w/p was greater than 2.5, its effect on performance appeared to be negligible.

Duckbill weirs and other long crested structures have been put to use in irrigation projects in Spain and North Africa. Their main purpose is to maintain constant water levels for turnout structures upstream from the weir. This function is very important for providing a fairly uniform discharge through the outlet structures. Canal systems designed to provide water to farmers on demand often have large variations in canal flowrate throughout the irrigation season. Changes in canal flow with water demand causes the flow depths in the canal to vary from day to day and even from hour to hour. When the canal turnout structures are submerged orifice gates, fluctuations in water surface levels in the canal causes discharges to fields through the orifice turnouts to vary. During irrigation, these varying flows reduce the efficiency of field irrigation systems (Clemmens, 1984).

The discharge of long crested weirs for design purposes can be estimated using the following equation:

$$Q = cBH^{3/2}\sqrt{2g} \quad (1)$$

where Q = discharge over the weir (m^3/sec), c = discharge coefficient, B = crest length (m), and H = height of water above the weir crest (m). Kraatz and Mahajan (1975) give the following estimates of the discharge coefficients for various types of long creste weirs and for two different crest types:

	Weir Type		
Crest Type	Diagonal	Duckbill	Labyrinth
Unrounded Crest	0.34	0.32	0.31
Rounded Crest	0.38	0.36	0.34

The authors emphasized that although these coeficients are adequate for use in estimating design flowrates and heads, the coefficients are not calibrated closely enough to use the weirs as accurate flow measurement structures. Although not currently used to measure flow, the long crested weirs have proven to be very effective structures for maintaining relatively constant (within 5 to 10 cm) upstream water surface elevations in spite of variations in the flow in the main canal. These weirs can also pass large flowrates with relatively small changes in head on the weir.

CHAPTER III

LABORATORY APPARATUS AND PROCEDURES

HYDRAULICS LABORATORY

The experiments with the long crested weir were performed in the Hydraulics Laboratory of the Civil Engineering Department at the University of Wyoming. The long crested weir was placed in a concrete open-channel in the floor of the laboratory. This channel is three feet wide and four feet deep and is about seventy-two feet long, running nearly the entire length of the laboratory. Although some sections of the channel floor appeared to be slightly depressed or elevated relative to adjacent channel sections due to the nature of the concrete finishing, the channel has no overall slope, and the elevation of the channel floor does not vary by more than about one-half inch over its entire length.

The Laboratory is equipped with a series of Bell & Gosset centrifugal pumps which pump water from a reservoir or sump in the laboratory basement and circulate it within the laboratory. An eight inch diameter pipe carries the water from the pump room to the far end of the laboratory where it passes through a manifold and series of valves before being discharged into the concrete open-channel. Once in the open-channel, the water flows back toward the sump and pump area.

At the end of the open-channel is a diverter tank which directs the flow into two Fairbanks weighing tanks which can be used to determine the flowrate in the channel. The weighing tanks empty into the reservoir or sump, so that the water can be recirculated back to the channel again.

The Fairbanks weighing tank system is an electronic weighing system. The valves on the diverter tank and the two weigh tanks are actuated by pneumatic pistons which are electronically controlled. Weights of the tanks are determined with a strain gage system. The weighing system is controlled by an electronic timer which can be set to a specified number of seconds. If the timer is set for sixty seconds, for example, then when the weighing system is started (by pushing the START button on the control panel), the valve in the diverter tank which empties into tank A will open, and the valve to tank B will simultaneously close. The valve at the bottom of tank A will be closed, and the entire flow from the open channel will pass through the diverter tank into tank A for sixty seconds. At the end of sixty seconds, the diverter tank valves will switch positions so that the valve to tank A will now be closed, and all of the flow will pass through the open valve into tank B. As the diverter tank valve to tank B opens, the valve in the bottom of tank B will shut, so that the flow is contained in the tank for the next sixty seconds. Tank A will stabilize within a few seconds, and the weight will automatically be recorded and printed out at the control box. After the weight is printed, the valve in the bottom of tank A opens to empty the water from the tank by gravity into the reservoir below. After tank B has filled for sixty seconds, the diverter tank valves will switch and start filling tank A again while tank B is weighed and emptied. For medium and low flows the tanks could be operated continuously because sixty seconds is enough time for tank A to empty while tank B is filling (and vice versa), but at high flows some problems were created which will be discussed in the procedures section.

The flowrate of the water being pumped into the open-channel could be controlled in two ways. Since there were four pumps connected in parallel, the flow could be varied simply by turning the pumps on or off. This only allows for four flowrates, with the maximum flowrate provided by all four pumps operating simultaneously and the minimum flowrate provided by just one pump. The other way to control the flowrate was to adjust the setting of the valves in the pipe which discharged into the open-channel. These flow control methods were used together during the experiments involving the long crested weir.

LONG CRESTED WEIR

The weir crest was made of aluminum plates that could be interchanged in assembly to provide different shapes of weir configuration. The side crest sections were 11.75 inches tall and 3/16 inches thick. The top edge was machined to form a sharp crest with a top thickness of about 0.08 inches (2 mm). The side crest sections were originally 3.99 feet long, but were later cut and machined to lengths of 3.00, 2.00, and 1.00 feet as the experiments progressed. Two lengths of one inch steel channel were attached to the side crest sections to reinforce and strengthen them. The downstream crest sections were also 11.75 inch tall aluminum plates and machined to have a crest which matched the sharp crest of the side sections. The downstream crest sections were attached to the side crest sections with screws and nuts (Figure 4). The holes for the screws were drilled so that the screw heads mounted flush with the upstream face of the weir; this minimized any detrimental effects the screw heads may have had on streamlines or upstream flow conditions. The screws were also located far enough below the crest so that in free-flow conditions the nuts attached on the downstream side of the weir did not interfere with the free jet of water flowing over the weir. The downstream crest sections were 18, 15, 12, 9, 6, and 3 inches wide; experiments were also conducted with no downstream crest section at all, and the two side crest sections were connected at their ends to form a weir in the shape of a "V" in plan view (Figure 5).

The upstream sidewalls of the weir were also made of interchangeable aluminum plates. These plates were of the same thickness as the crest pieces, but they were two feet tall and had varying widths to produce different throat widths

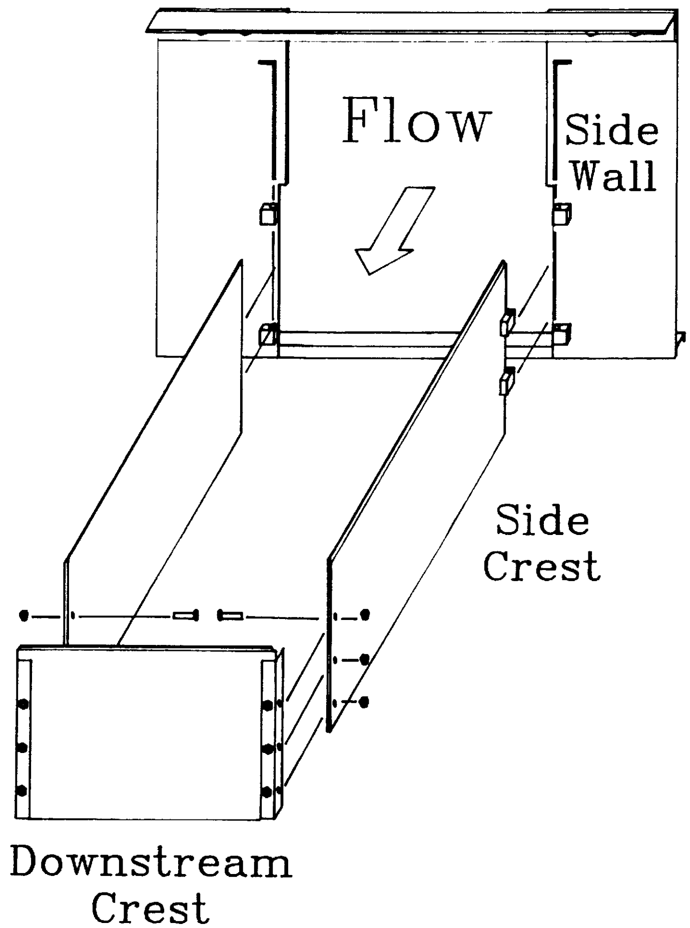

Figure 4. Exploded View of the Weir Crest Assembly.

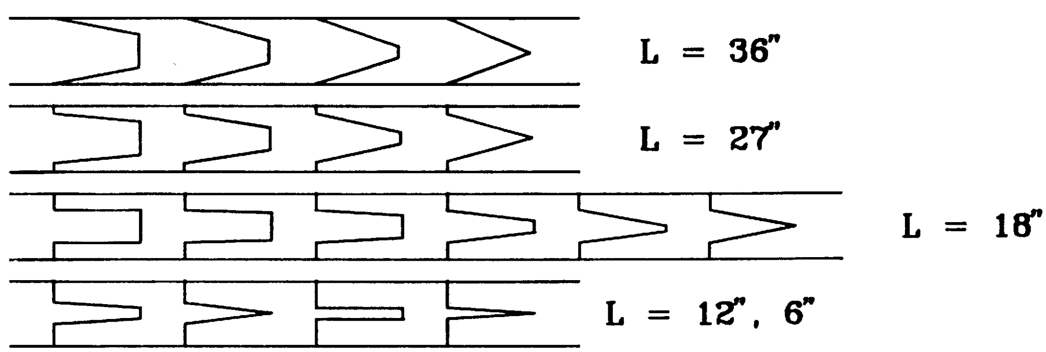

Figure 5. Plan Geometry of Weirs Tested With Sl=4 ft.

for the weir. The weir throat widths that were used in the experiment were 36 inches (no sidewalls), 27 inches, 18 inches, 15 inches, 12 inches, 9 inches, and 6 inches. The sidewalls were attached at the top and bottom to brackets fastened to the concrete floor and sides of the open channel. The side crest sections were attached to the sidewalls with hinges which allowed flexibility in testing several different downstream crest sections without having to disconnect the side sections from the sidewalls. The sides of the sidewalls which became the throat of the weir were machined to form vertical sharp crests similar to the sharp crest of the side crest sections. This produced a weir crest similar to the sharp crests on the sides of a regular sharp crested contracted weir.

Two brackets were anchored to the concrete floor and sides of the channel to support the weir and hold it in place. A three foot section of one inch steel channel was fastened to the concrete floor of the open-channel with anchor bolts; this became the main bracket to which the bottom of the sidewalls of the weir were attached. The tops of the sidewalls were attached to an aluminum angle section which extended across the channel and was fastened to the sides of the channel with anchors in the concrete. These brackets held the sidewalls in place, which in turn held the weir crest sections in place and kept the weir from being pushed down the channel by the force of the water behind it. When no sidewalls were used (testing the 36 inch throat width), the weir crest sections were placed in the channel upstream of the brackets and the downstream crest sections were fastened to the bracket on the floor of the channel. This proved sufficient to hold the weir in place for the experiments.

Because the contact of bare metal with concrete did not provide a good seal, a sealant was used to prevent excess leakage around and under the weir. Various types of caulk were tested, and of the sealants tested, GE Silicone II was the sealant that proved to be most effective for this particular application. It was easily applied, had a relatively quick curing time, and was easy to remove. However, for the silicone to stick, the surfaces had to be dry. The concrete channel could be dried sufficiently in about 45 minutes using a squeegee, a mop, rags, and blow dryers in preparation for the next silicone seal. The seals were not perfect and did not eliminate all leakage, but the leakage was very small and considered insignificant compared to the flow over the weir during the experiments. If a seal permitted what appeared to be a significant amount of leakage, attempts were made to plug the leaks; and if these proved unsuccessful, the channel was drained, dried, and a new seal installed.

DEPTH MEASUREMENT

The depth of flow was measured mechanically using three point gages with Vernier Scales. The scales read to 0.001

feet. The gages were mounted over the center of the channel at the throat entrance, 1.25 feet upstream of the throat, and 2.50 feet upstream of the throat (Figure 6). The only gage reading that was used to evaluate the discharge coefficient was the reading on the gage at 2.50 feet upstream of the weir throat (the other readings were taken for possible use in future flow studies). The literature recommended taking the depth measurement at an minimum upstream distance of four times the head on the weir (Kraatz and Mahajan, 1975). The gage 2.50 feet upstream of the throat was positioned there so that at a maximum head of 0.5 feet on the weir the gage would be at an upstream distance of five times the maximum expected head on the weir.

The depth at 2.50 feet upstream of the throat was also measured electronically using a Level Plus™ Direct Digital Access (DDA) Industrial Tank Gage produced by MTS Systems Corporation. The DDA gage was installed in a stilling well made of 6 inch diameter PVC pipe and located downstream of the weir. A metal plug with small holes drilled in it was put in the end of a 5/8 inch diameter plastic hose and fastened to the floor of the channel 2.50 feet upstream of the weir throat and directly beneath the point gage. The plastic hose extended to a pipe through one of the weir sidewalls, and another plastic hose on the downstream side of the weir connected the pipe in the sidewall to the stilling well. This arrangement allowed the water surface in the stilling well to

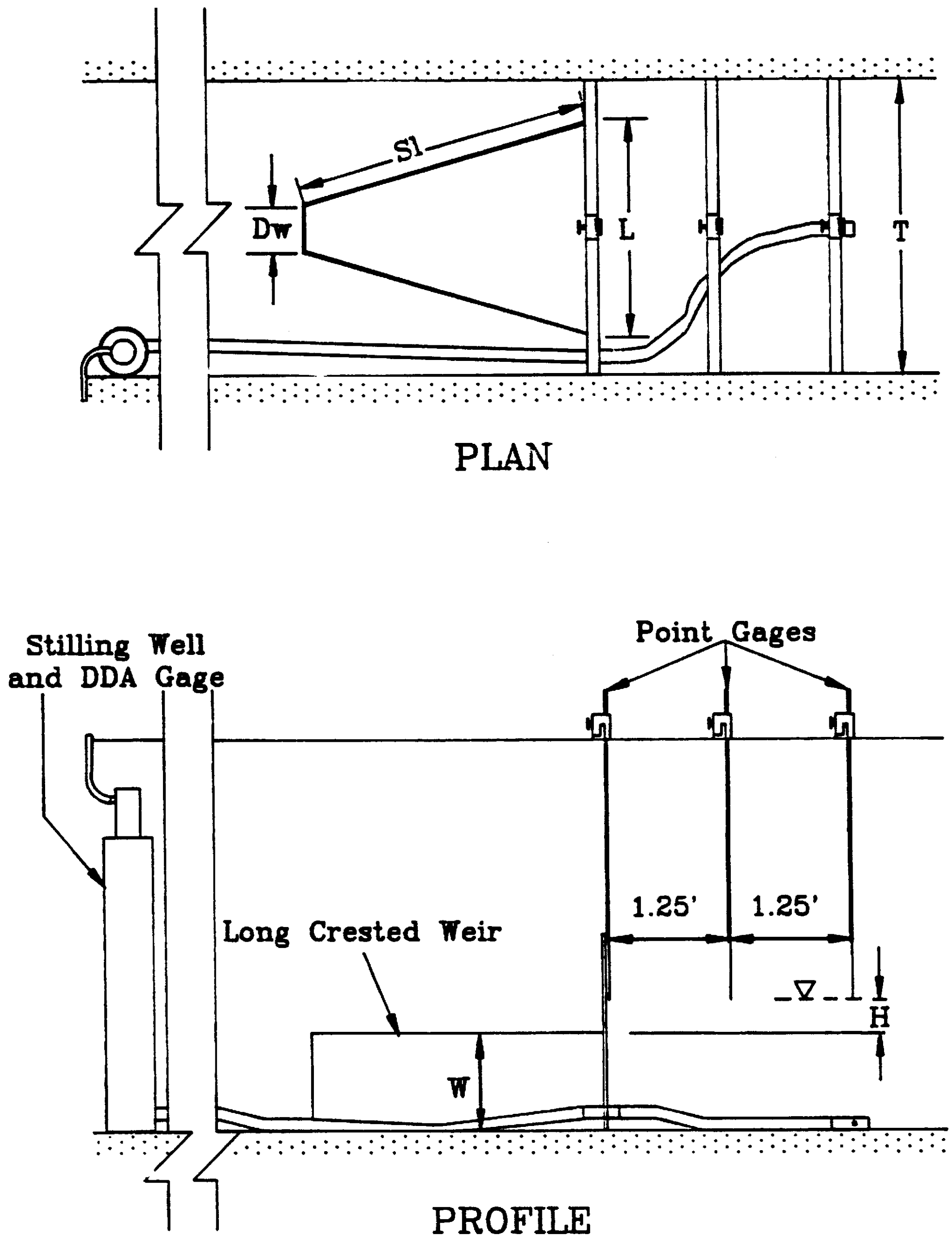

Figure 6. Views of the Weir Installation.

be the same elevation as the water surface in the channel 2.50 feet upstream of the weir throat, and it allowed the DDA gage to measure the same changes in head that were being measured with the point gage. The small size of plastic pipe and the small holes in the plug restricted the flowrate to the stilling well and dampened the influence of waves in the channel so that the water surface in the stilling well gave an average water surface rather than showing a peak and trough for every wave moving down the channel. The stilling well was fastened to the wall of the channel about 8.5 feet downstream from the weir sidewalls; this distance was enough to insure that the presence of the stilling well in the channel did not interfere with flow over the weir.

The DDA gage consisted of four main parts which functioned together to electronically determine the water depth: a stainless steel float, a hollow stainless steel tube, a rod called a waveguide with strain gages attached to one end, and the electronic components. The stainless steel float was somewhat cylindrically-shaped and hollow in the center so that it could fit over the hollow stainless steel tube. Inside the float is a permanent magnet. The tube acts as a guide for the float, and the float is free to move up and down the tube. The hollow stainless steel tube also houses the waveguide and protects it from corrosion. The waveguide is made of magnetorestrictive material and has an insulated wire that extends down through the inside of the waveguide from the electronic components and then back up to the electronic components along the outside of the waveguide. The electronic components are in a special casing to which the steel tube is attached. When the signal to measure the water level is received by the gage, the electronic components interpret the signal and send an electric pulse down the insulated wire in the waveguide. As the pulse travels down the wire, it creates a magnetic field in the waveguide. The float, riding on the water surface, also has a magnetic field created by the permanent magnet it contains. When the magnetic field from the float contacts the magnetic field in the waveguide, the interaction of the two magnetic fields causes the waveguide to twist. The strain gages detect the twist in the waveguide, and the electronic components can determine the water depth by measuring the time from sending the electric pulse to when the twist is detected by the strain gages (Figure 7).

The gage was connected to a 286 IBM compatible computer. Since the electronic signal from the gage had to be translated into a signal that the computer could interpret (and vice versa), a PC-422/485 Serial Interface was installed in the computer to allow communication between the DDA gage and the computer. A DDA-PC communications software program was provided by the MTS Systems Corporation. This program allowed a certain degree of programming of the DDA gage and provided the necessary communication commands to get the depth readings

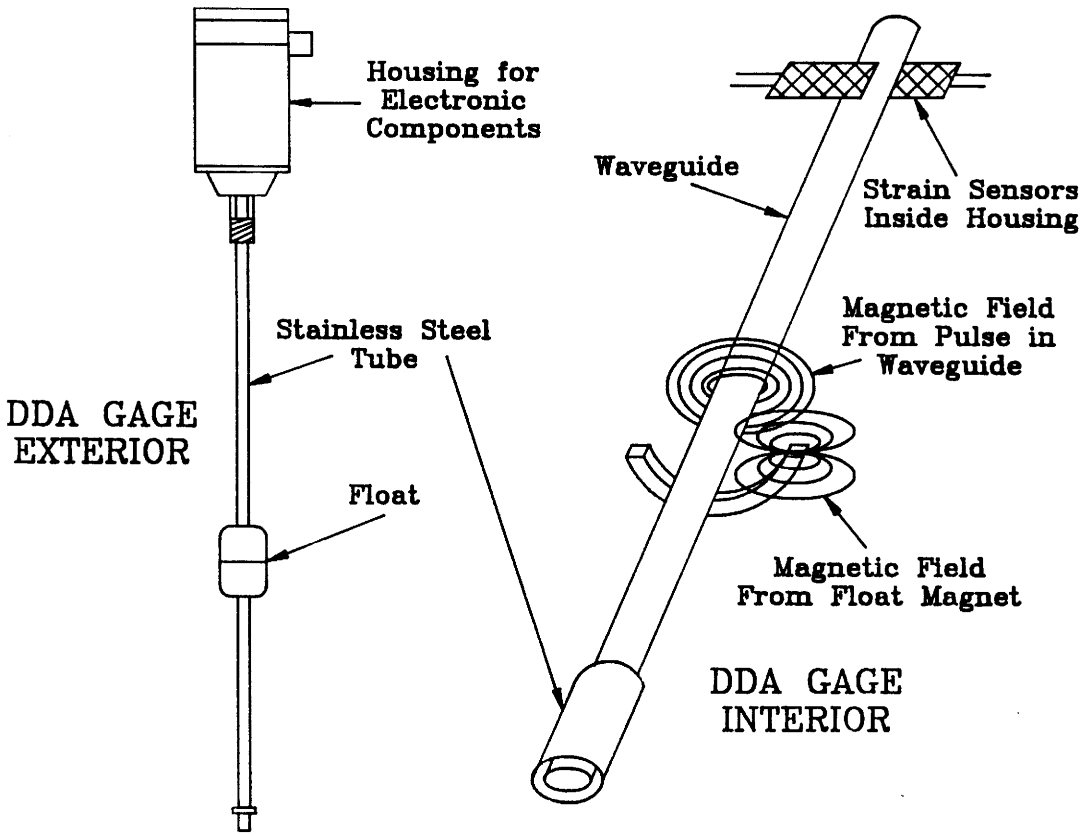

Figure 7. The DDA Gage.

from the gage. Commands to the DDA gage were issued in Hex code by direct input from the computer keyboard. The gage readings were displayed on the computer screen and recorded on data sheets. The gage reading was in inches, and the depth readings displayed on the computer were to three decimal places (0.001 inches).

TESTING PROCEDURE

Once the weir was assembled in the channel and the silicone caulk seals were in place, a certain experimental procedure was followed to test the weir and acquire the data. This procedure was followed while testing the weirs to assure consistency in testing and data recording conditions and also for convenience in running the experiments. There were four main parts to the procedure: preliminary measurements and gage settings, depth and weight readings, calculations, and adjustments for the next flowrate. Typically, a certain weir configuration was tested with seven different flowrates.

The initial stage in the experimental procedure was the preliminary measurements and gage settings. Before starting the experiment, measurements were made of the weir side crest length, the downstream weir crest width, and the weir throat width. These measurements defined the exact weir configuration and were used in developing the Pi terms and coefficient of discharge used in the data analysis. Since our only interest was in measuring the head on the weir--in other words, the depth from the water surface to the weir crest elevation--and not the total depth of water in the channel, the gages were initially set to the elevation of the weir crest. This was done by filling the channel with water upstream of the weir until water was flowing over the weir, and then stopping the pump. When the water stopped flowing over the weir, the zero-flow condition was set, and the water surface behind the weir was essentially at the elevation of the weir crest. The point gages were adjusted to where their points were just touching the water surface, and the gage readings were recorded on the data sheet. Accordingly, the water surface in the stilling well was also at the elevation of the weir crest, and the DDA gage reading was set to 0.000 inches. The DDA gage could then give a direct measure of head in inches; the head readings given by the point gages had to be calculated by subtracting the initial zero-flow reading from the reading taken during the experiment.

After the preliminary measurements and gage adjustments had been made, the weir was ready for flow. The pumps were turned on, and the flow in the open-channel was allowed to stabilize for several minutes. Then the point gages were readjusted so that their points were again just touching the surface of the water. During the high flows there were waves in the channel, and the point gages were set so that they would be touching the water approximately half the time and out of the water the other half of the time. This was done to try to get an average water surface with an elevation midway between the elevation of the wave's crest and trough. These point gage readings were recorded on the data sheet, and then the reading was taken from the computer for the DDA gage. Several readings were taken from the DDA gage to insure that the water surface in the stilling well was not changing significantly, and then the DDA gage reading was recorded on the data sheet. The head readings for the DDA gage and the point gage at 2.5 feet from the weir were compared to make sure they agreed. If there was a large discrepancy, the readings were taken again. This comparison helped catch any misreading of the gages or other human error before moving on to the next step in the procedure. At this time, notes were taken regarding visual observations of flow conditions over or through the weir.

The weight readings came next. The timer was set on the control panel and the weighing system was started. For the medium and low flows, tank A would fill then the flow would switch to tank B. A weight would be automatically printed out for tank A, and it would empty while tank B was filling. By the time the timer switched the flow from tank B back to tank A, tank A had emptied. Tank B would print the weight then empty while tank A was filling again. In this way, the flow could switch continuously from one tank to the other as the weights were being printed out. Occasionally one of the diverter valves would not open all the way when the system was first started, and this would reduce the flowrate into the tank and give a faulty weight reading. Therefore, if the weights from the two tanks didn't agree close enough, the weighing process was repeated until acceptable weight readings were obtained. Then two weights from tank A and two from tank B were recorded on the data sheet. The procedure was a little different for the high flows because tank A could not drain fast enough to be empty by the time the timer switched the flow back from tank B. This required that tank A be filled and weighed and tank B filled and weighed, but then the weighing system had to be stopped and the tanks allowed to drain before starting again. Otherwise, the flow would switch from tank B back to tank A, and tank A would still have water (possibly several thousand pounds) from the first weighing and give a faulty weight reading the second time.

Performing various calculations was the third part of the procedure. The channel flowrate was calculated by averaging the four recorded weights (two each from tank A and tank B), dividing this average weight by 62.4 pounds per cubic foot of water, and then dividing by the number of seconds set on the timer of the weighing system. This calculation produced a flowrate in cubic feet per second. The value of the experimental coefficient of discharge was also calculated using the head, the flowrate, and the total length of weir crest. Some dimensionless Pi terms were also calculated. Performing the calculations at this stage in the experiment took only a few minutes and gave an indication of whether or not the data were following a regular pattern or if something unusual was happening.

Once the data were taken and the calculations performed, the pumps and valves were adjusted for the next flowrate. Depending on the flowrate and the weir configuration, sometimes the only adjustment made was to turn off a pump and let the flow in the channel stabilize again. At other times a pump would be turned off and a valve closed to further restrict the flow. The depth of flow in the channel gave visual indication of the flowrate adjustment, and the flowrate was adjusted to give a fairly uniform spread of head measurements between the highest and lowest values. The flowrate was adjusted based on the change in water surface elevation in the channel; although the actual flowrate was determined using only the weigh tanks, it was obvious that a small change in head produced a change in flowrate over the weir. Careful adjustments of the valves and pumps could produce somewhat consistent changes in head on the weir and at the same time produce somewhat consistent changes in flowrate over the range of flowrates used with each experimental weir. With some experience it was possible to adjust the flow rapidly and with consistency simply by observing the change in flow depth brought about by the valve and pump adjustments. After the flow depth (and therefore, the flowrate) was adjusted satisfactorily, the gage readings, weigh tank readings, and calculations for this new flowrate were performed. This process of adjusting the flowrates and taking the data was repeated on average seven times for each weir configuration.

After the testing of the weir for the various flowrates was complete, the channel was drained and dried, and the old weir configuration was replaced with a new one to be tested. Draining the channel was accomplished by removing the downstream crest width of the weir. The silicone caulk was removed from the metal weir plates and the concrete channel, and the area of the channel where the new weir configuration would be positioned was dried using a squeegee, a sponge mop, cloth rags, and blow dryers. If the new weir configuration required only a change of the downstream crest width and not a change of throat width, then the caulk was removed only from the crest pieces, but the silicone seal at the sidewalls was left intact. The side crest pieces were repositioned and a new downstream crest piece installed. Then all of the joints of the crest pieces were sealed with silicone caulk. If the throat width was to change, the crest sections and the sidewalls were removed from the channel. The caulk was removed from the channel walls and floor, and then the new sidewalls and crest sections were installed and sealed with caulk. This procedure was done for the 67 separate weir configurations that were tested.

CHAPTER IV

DATA ANALYSIS AND DISCUSSION OF RESULTS

DIMENSIONAL ANALYSIS

The Buckingham Pi Theorem was used to develop dimensionless parameters, the Pi (Π) terms, from the variables. Five parameters are used to describe the geometry of the long crested weir: the top width of the channel (T), the width of the weir throat (L), the height of the weir crest (W), the length of the weir side crest (Sl), and the total length of the weir crest (Llcw). Variables used to describe the flow condition were the flowrate (Q), the acceleration due to gravity (g), and the head of water on the weir (H). The relationship of these variables can be described as

$$Q = f(g, H, T, L, W, Sl, Llcw) \tag{2}$$

which indicates that the flowrate is a function of all the other parameters. The dimensionless Π terms can be obtained by following the procedure outlined in Introduction to Fluid Mechanics by Fox and McDonald (1985):

1. The parameters involved are

   Q g H T L W Sl Llcw n=8 parameters

2. Select primary dimensions of mass, length and time

(m, ℓ, t)

3. Define the parameters in terms of the primary dimensions

$Q=\ell^3/t$, $g=\ell/t^2$, $H=\ell$, $T=\ell$, $L=\ell$, $W=\ell$, $Sl=\ell$, $Llcw=\ell$

$r=2$ primary dimensions (ℓ and t)

4. Select r parameters whose combined dimensions include all of the primary dimensions; these will be the repeating parameters.

$g=\ell/t^2$, $H=\ell$ $m = r = 2$ repeating parameters

5. This results in $n - m = 6$ dimensionless groups.

Π_1: $\Pi_1 = g^a H^b Q = (\ell/t^2)^a (\ell)^b (\ell^3/t) = \ell^0 t^0$

Equating the exponents of ℓ and t results in

ℓ: $a + b + 3 = 0$ ---->$a = -1/2$

t: $-2a - 1 = 0$ ---->$b = -5/2$

$$\Pi_1 = \frac{Q}{H^{5/2}\sqrt{g}} \tag{3}$$

Π_2: $\Pi_2 = g^c H^d T = (\ell/t^2)^c (\ell)^d (\ell) = \ell^0 t^0$

ℓ: $c + d + 1 = 0$ ---->$c = 0$

t: $-2c = 0$ ---->$d = -1$

$$\Pi_2 = \frac{T}{H} \tag{4}$$

Π_3: $\Pi_3 = g^e H^f L = (\ell/t^2)^e (\ell)^f (\ell) = \ell^0 t^0$

ℓ: $e + f + 1 = 0$ ---->$e = 0$

t: $-2e = 0$ ---->$f = -1$

$$\Pi_3 = \frac{L}{H} \tag{5}$$

Similarly, since all the remaining parameters have units of length only, they are of the same form as Π_2 and Π_3. The remaining Π terms are

$$\Pi_4 = \frac{W}{H} \tag{6}$$

$$\Pi_5 = \frac{Sl}{H} \tag{7}$$

$$\Pi_6 = \frac{Llcw}{H} \tag{8}$$

The functional relationship then becomes

$$\Pi_1 = f(\Pi_2, \Pi_3, \Pi_4, \Pi_5, \Pi_6) \tag{9}$$

or

$$\frac{Q}{H^{5/2}\sqrt{g}} = f\left(\frac{T}{H}, \frac{L}{H}, \frac{W}{H}, \frac{Sl}{H}, \frac{Llcw}{H}\right) \tag{10}$$

the actual form of this function must be determined by statistical analysis of the experimental data.

There are many possible combinations of the parameters to form dimensionless Π terms. The Π_1 term developed above using g and H as repeating parameters is $Q/(H^{5/2}g^{1/2})$ and is a form of the Froude Number. If a different geometric parameter is used as a repeating parameter, then it ends up in the denominator of the dimensionless terms Π_2 through Π_6. In discussing the options of which combinations of parameters can be used as Π terms, Murphy (1950, p. 37) states "... the only restrictions placed upon the Pi terms is that they be dimensionless and independent." The quality of independence means that a $\prod$ parameter considered in a problem can't be formed by the combination of the other parameters of the problem through multiplication or division. For example, the parameter L/T could not be used along with the parameters T/H and L/H because L/T = (L/H)/(T/H). However, transformations, combinations, and substitutions are allowed on the conditions of being dimensionless and independent of the other parameters considered in the problem. Therefore, L/T could replace L/H and the parameters L/T and T/H would be independent because L/H is no longer included in the problem.

The dimensional analysis indicated that six $\prod$ terms should be used to evaluate the problem, but the selection of which $\prod$ terms to use involved a degree of judgment mixed with some trial and error. The equation for the flowrate of a long crested weir given by Kraatz and Mahajan (1975) is of the form

$$Q = C_d \, Llcw \, H^{3/2} \sqrt{2g} \tag{11}$$

where C_d is a discharge coefficient, Llcw is the length of the long crested weir crest, H is the head on the weir, and g is the acceleration due to gravity. This is basically the same equation as (1) but uses different letters to represent the variables. Solving this equation for the coefficient of discharge, C_d, produces equation 12 which is dimensionless and similar in form to the Π_1 term developed through the dimensional analysis.

$$C_d = \frac{Q}{Llcw\, H^{3/2}\, \sqrt{2g}} \tag{12}$$

Because C_d is dimensionless, independent, and similar in form to Π_1, it can be substituted for Π_1 in the evaluation of the problem. This allows the evaluation of the coefficient of discharge as a function of the geometry parameters:

$$C_d = f(\Pi_2, \Pi_3, \Pi_4, \Pi_5, \Pi_6) \tag{13}$$

A multivariable linear regression was done to relate C_d to the other parameters. The repeating parameter with just the dimension of length was varied to produce different terms of Π_2 through Π_6. The r^2 fit of the different linear regressions was as follows:

$$C_d = f\left(\frac{W}{H}, \frac{L}{H}, \frac{T}{H}, \frac{Sl}{H}, \frac{Llcw}{H}\right) \qquad r^2 = 55.5\% \tag{14}$$

$$C_d = f\left(\frac{H}{W}, \frac{L}{W}, \frac{T}{W}, \frac{Sl}{W}, \frac{Llcw}{W}\right) \qquad r^2 = 77.3\% \tag{15}$$

$$C_d = f\left(\frac{H}{L}, \frac{W}{L}, \frac{T}{L}, \frac{Sl}{L}, \frac{Llcw}{L}\right) \qquad r^2 = 80.6\% \tag{16}$$

$$C_d = f\left(\frac{H}{T}, \frac{W}{T}, \frac{L}{T}, \frac{Sl}{T}, \frac{Llcw}{T}\right) \qquad r^2 = 77.3\% \tag{17}$$

$$C_d = f\left(\frac{H}{Sl}, \frac{W}{Sl}, \frac{L}{Sl}, \frac{T}{Sl}, \frac{Llcw}{Sl}\right) \qquad r^2 = 59.8\% \tag{18}$$

$$C_d = f\left(\frac{H}{Llcw}, \frac{W}{Llcw}, \frac{L}{Llcw}, \frac{T}{Llcw}, \frac{Sl}{Llcw}\right) \quad r^2 = 67.4\% \qquad \textbf{(19)}$$

It was apparent from the linear regressions that not all of the dimensionless Π parameters do an equally good job of describing C_d. For this reason, an arbitrary selection of Pi parameters would not be satisfactory. Judgment had to be used in the selection of the parameters which best described the geometry of the weir as it related to the flow conditions. Along with the judgment, there was also some trial-and-error involved in determining the final Π terms and evaluating their effects on weir performance. The dimensionless parameters that were selected to be evaluated with the coefficient of discharge are explained in the following paragraphs.

A driving force behind the flow of water over the weir is the head of water on the weir, and the dimensionless term H/W appears in the Rehbock equation for sharp crested weirs (Vennard and Street 1982). The ratio of the head to the weir height has a large influence on the shape of the streamlines in the flowfield for standard sharp crested weirs. This dimensionless parameter, H/W, was selected as Π_2 for this reason (the first dimensionless term is the coefficient of discharge, C_d).

The next dimensionless parameter selected, Π_3, was L/T. This is the ratio of the long crested weir throat width to the width of the channel in which the weir is placed. This ratio gives an indication of the lateral contraction of the flow streamlines as they enter the weir. As will be shown later, the lateral contraction of the flow had a large influence on weir performance.

The dimensionless parameter H/L was selected to be Π_4. This was a ratio of the head on the weir to the throat width of the weir. Along with Π_2 and Π_3, this parameter gives an indication of the contraction of the flow entering the weir. A value of H/L near 1.0 indicated a relatively high head with a very narrow throat, and resulted in reduced performance of the weir.

The parameter Sl/Llcw was selected as Π_5 because it defines some of the geometry of the weir crest. The total length of the crest, Llcw, is given by the equation

$$Llcw = 2Sl + Dw. \tag{20}$$

The term Sl is the side crest length, and there are two sides to each weir. The term Dw is the downstream crest width of the weir. In the case of a weir that is V-shaped in plan view, there is no downstream crest width (Dw = 0), and the total crest length is equal to twice the length of the side crest.

The final parameter, Π_6, is Llcw/L. This is a ratio of the total crest length to the width of the throat. This term is described by Walker (1987) and Hay and Taylor (1970) as a length magnification. Since a standard sharp crested contracted weir would have a crest length equal to the width of the throat, this parameter indicates how many times longer the crest of the long crested weir is than the crest of the contracted weir with the same throat width. If the throat width is equal to the channel width, then the parameter is the same (Llcw/L), but now L = T, and the comparison of weir crest lengths would be between the long crested weir and a standard suppressed weir. Under ideal operating conditions, the flowrate of the long crested weir would be Llcw/L times greater than the flowrate for the regular weir with the same head; for example, if the crest of the long crested weir is ten times longer than the crest of the regular weir (Llcw/L = 10), then ideally the flowrate over the long crested weir should be ten times greater than the flowrate over the regular weir with the same head. The experimental results showed that the flow magnification was often much less than the length magnification, but this parameter was still useful in describing the behavior of the weir.

EQUATION FOR C_d

Once the Π terms were determined, the data could be analyzed and an equation for C_d be developed. As shown in (13), the symbolic relationship of the parameters is

$$C_d = f(\Pi_2, \Pi_3, \Pi_4, \Pi_5, \Pi_6) ,$$

where the actual terms as defined by the combinations of the geometry parameters are shown in equation 21.

$$\frac{Q}{Llcw\ H^{3/2}\ \sqrt{2g}} = f\left(\frac{H}{W}, \frac{L}{T}, \frac{H}{L}, \frac{Sl}{Llcw}, \frac{Llcw}{L}\right) \qquad \textbf{(21)}$$

The objective was to develop an equation for C_d, and several forms of equations were tried. The first form of equation was a simple linear relationship between C_d and the rest of the parameters:

$$C_d = a + b\Pi_2 + c\Pi_3 + d\Pi_4 + e\Pi_5 + f\Pi_6\ , \qquad \textbf{(22)}$$

where a through f are regression coefficients from a multivariable linear regression of the data. The second form of equation is a second-order polynomial equation which starts the same as the linear equation but adds the Π terms raised to the second power:

$$\begin{aligned} C_d = {} & a+b\Pi_2 + c\Pi_3 + d\Pi_4 + e\Pi_5 + f\Pi_6 \\ & + g\Pi_2^{\,2} + h\Pi_3^{\,2} + i\Pi_4^{\,2} + j\Pi_5^{\,2} + k\Pi_6^{\,2}\ , \end{aligned} \qquad \textbf{(23)}$$

where the letters a through k represent regression constants (the constants a through f in this polynomial will be different from those given in the linear equation above). The other form of equation is a power function:

$$C_d = a\Pi_2^{\ b}\Pi_3^{\ c}\Pi_4^{\ d}\Pi_5^{\ e}\Pi_6^{\ f}\ , \qquad \textbf{(24)}$$

where again the letters a through f represent constants. The constants of the power function are found by taking the logs of the data and by doing a linear regression on the logs as shown in equation 25:

$$logC_d = loga + b\log\prod_2 + c\log\prod_3 + d\log\prod_4 + e\log\prod_5 + f\log\prod_6 \quad . \tag{25}$$

Although the number of terms involved makes the equations somewhat long, they are all simple to use.

The analysis of the data was limited to these three types of equations for several reasons. When starting the data analysis, it was not known what type of equation would best fit the data. One of the characteristics desired in the prediction equation for C_d was simplicity of both form and use. The linear and power functions are both simple in form and simple to use once the coefficients are known. The second-order polynomial equation is a little more cumbersome because of the 11 terms, but it is still simple to use. Of course, another characteristic that was desired was a high correlation coefficient (r^2) which indicates how well the data fit the relationship. As the following data analysis shows, all of these three types of equations produced r^2 values above 80 %. The third-order polynomial was also tried during the data analysis, and it produced a better r^2 than any of the other three types of equations. However, the cubic type equation was very cumbersome, and the one or two percent improvement in the r^2 appeared to be the result of the added degrees of freedom rather than actually being a better prediction equation.

The regressions were first performed on the entire data set. All of the data regressions and analysis was performed using MINITAB statistical software. The multivariable linear regression yielded the following equation which had an r^2 value of 85.5 %:

$$C_d = 0.618 - 0.346\prod_2 + 0.0143\prod_3 - 0.188\prod_4 - 0.181\prod_5 - 0.0107\prod_6 \ . \tag{26}$$

The second-order polynomial equation gave an r^2 of 92.0 % with:

$$\begin{aligned} C_d = {} & 0.296 - 0.482\prod_2 - 0.236\prod_3 - 0.808\prod_4 + 2.29\prod_5 - 0.0223\prod_6 \\ & + 0.797\prod_2{}^2 + 0.104\prod_3{}^2 + 0.485\prod_4{}^2 - 2.93\prod_5{}^2 + 0.000694\prod_6{}^2 \ . \end{aligned} \tag{27}$$

The evaluation of a power function for the data gave and r^2 fit of 81.5 %:

$$\begin{aligned} \log C_d = {} & -0.554 - 0.2691\log\prod_2 + 0.2231\log\prod_3 \\ & -0.2061\log\prod_5 - 0.1471\log\prod_6 \ . \end{aligned}$$

The $\log\prod_4$ term was highly correlated with the other variables, and MINITAB automatically removed it from the equation. These values correspond to the following power function:

$$C_d = 0.2795\prod_2{}^{-0.269}\prod_3{}^{0.223}\prod_5{}^{-0.206}\prod_6{}^{-0.147} \ . \tag{28}$$

These initial equations resulted from the analysis of the entire set of experimental data and may be considered general equations for the entire set of experiments.

Some recommendations by French (1985) for various

Some recommendations by French (1985) for various standard sharp crested weirs indicate that limitations should be placed on the operation of the weir. One of the recommendations is that the lower limit of head on the weir be about 0.10 feet because viscous and surface tension effects start to have a large influence at heads lower than this. In the experiments with the long crested weirs in the laboratory, the measurements of minimum head and flowrate were taken at a flow just greater than the flow at which the nappe would start clinging to the weir crest. These measurements were at heads less than 0.10 feet, and some of the experimental C_d values were very high which agrees with French's comments. The upper limit of the head to crest height ratio for fully contracted weirs was recommended to be 0.5 (French 1985). Most of the laboratory measurements were made for $H/W \leq 0.5$, but there were a few measurements that were greater than this upper limit. If the limitations of $H/W \leq 0.5$ and the minimum $H = 0.10$ feet are applied to the experimental data by deleting all the data that does not fall within these bounds, the following equations result and are applicable to all the weirs tested. For linear regression the $r^2 = 93.4$ %, and the equation is

$$C_d = 0.543 - 0.217\prod_2 + 0.0597\prod_3 - 0.163\prod_4 - 0.193\prod_5 - 0.00953\prod_6 \tag{29}$$

After the linear regression was done, the data was fit to a second-order polynomial equation which gave an $r^2 = 96.6$ %:

$$C_d = 0.366-0.193\Pi_2-0.247\Pi_3-0.727\Pi_4+1.65\Pi_5-0.0212\Pi_6 +0.351\Pi_2^2+0.129\Pi_3^2+0.413\Pi_4^2-2.17\Pi_5^2+0.00065\Pi_6^2 \,. \tag{30}$$

The power function had an r^2 = 84.8 % with:

$$C_d = 0.2793\Pi_2^{-0.289}\Pi_3^{0.247}\Pi_5^{-0.256}\Pi_6^{-0.179} \,. \tag{31}$$

The polynomial again provides the best fit of the data, and all of these equations showed an improvement in the r^2 values over the general equations produced from the entire data set.

The following pages contain plots of the actual test flowrates in the weirs versus the flowrates that are predicted by applying equations (29) and (30) to the experimental data. Figures 8, 10, 12, and 14 show the plots of Qa vs. Qp where Qp, the predicted flowrate, is predicted using equation (11) with equation (29) being used to calculate the value of C_d, and Qa is the actual flowrate taken from the experimental data. Figures 9, 11, 13, and 15 show the plots of Qa vs. Qp where Qp is predicted using equation (11) with equation (30) used to calculate the value of C_d. The two graphs on each page give a comparison of the predicted flowrates using the C_d values determined by the linear and the second order polynomial equations. The figure at the top of the page used the linear equation to predict C_d, while the values in the figure at the bottom of the page use the second-order polynomial equation to calculate C_d for the same weirs.

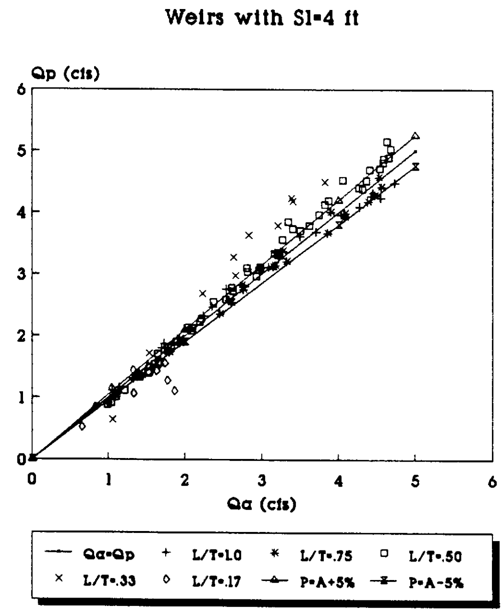

Figure 8. Actual vs Predicted Flowrate Using the Linear Equation for All Weirs (Sl=4 ft.).

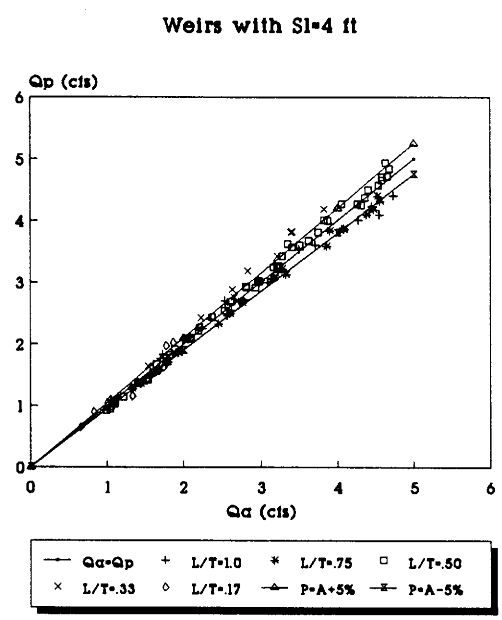

Figure 9. Actual vs Predicted Flowrate Using the 2nd Order Equation for All Weirs (Sl=4 ft.).

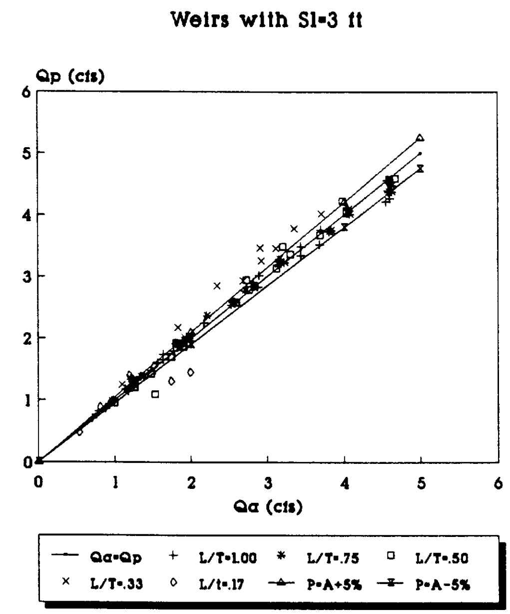

Figure 10. Actual vs Predicted Flowrate Using the Linear Equation for All Weirs (S1=3 ft.).

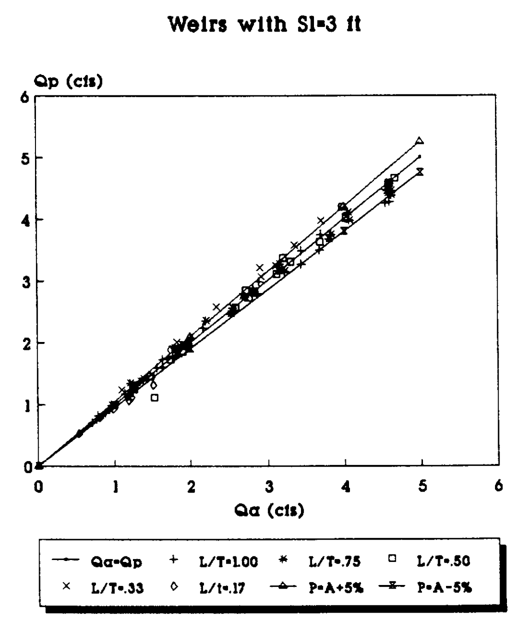

Figure 11. Actual vs Predicted Flowrate Using the 2nd Order Equation for All Weirs (S1=3 ft.).

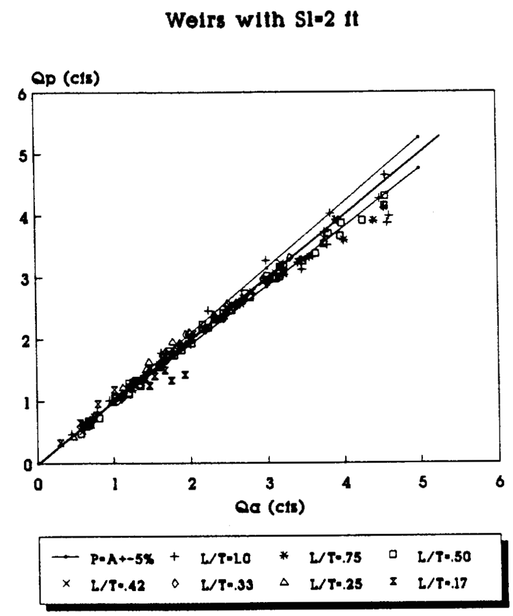

Figure 12. Actual vs Predicted Flowrate Using the Linear Equation for All Weirs (Sl=2 ft.).

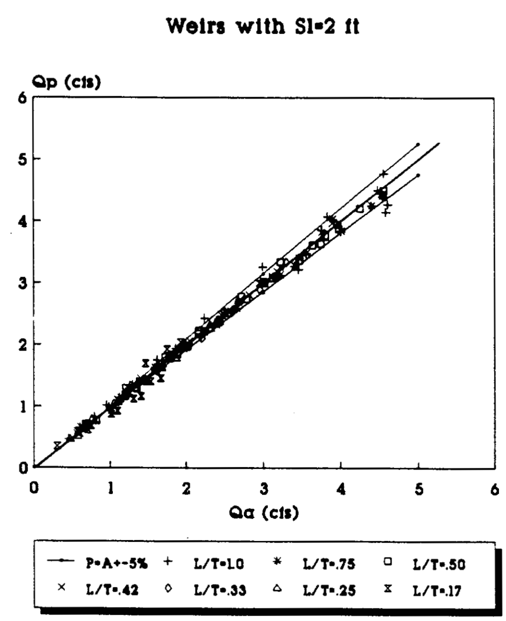

Figure 13. Actual vs Predicted Flowrate Using the 2nd Order Equation for All Weirs (Sl=2 ft.).

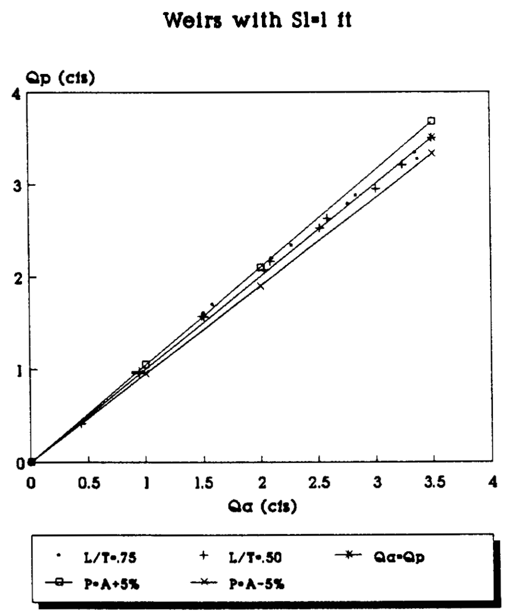

Figure 14. Actual vs Predicted Flowrate Using the Linear Equation for All Weirs (Sl=1 ft.).

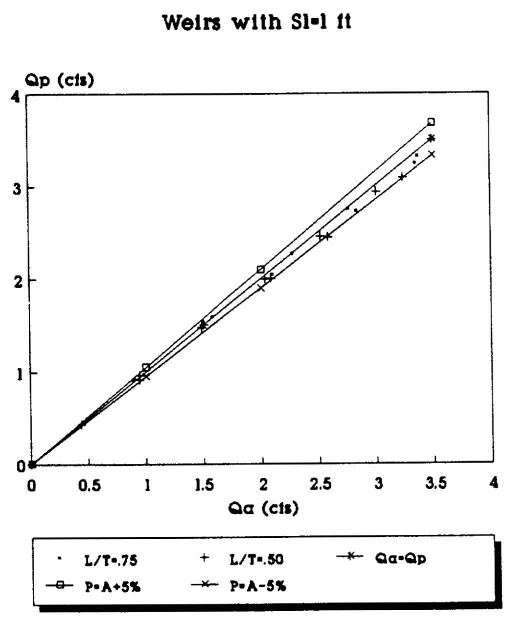

Figure 15. Actual vs Predicted Flowrate Using the 2nd Order Equation for All Weirs (Sl=1 ft.).

To plot the graphs, the data were segregated according to the length of the weir side crest, Sl. Weirs with different throat widths were plotted as a different series and are represented by a different symbol in the legend of the figures. For example, in Figure 6 the data points (Qa,Qp) for all of the suppressed weirs (L/T=1.0) with Sl=4 ft are represented by the "+" symbol. Each figure has a line which represents where the predicted flowrate would be equal to the actual measured flowrate. The objective is, of course, to get the predicted flowrate as close to the actual flowrate as possible. The figures also have a curve above and below the Qa=Qp line. The top curve (P=A+5%) shows the curve of predicted values 5 percent above the actual flowrate. The bottom curve (P=A-5%) shows the curve of the predicted values 5 percent below the actual flowrate. The area between the curves shows the range of the actual flowrate plus or minus 5 percent.

By plotting the data in groups according to Sl and the L/T values, some trends in the predicted flowrates become apparent. The predictions made with the second order polynomials consistently provided the predictions closest to the Qa=Qp line. As the L/T ratio decreases, the flowrate predictions tend to be further from the Qa=Qp line for both the linear and second-order polynomial equations; however, the fit improves with the second-order polynomial predicted flowrates even for the small L/T ratios. With the second-order polynomial equations, the bulk of the data points consistently fall on or within 5 percent of the actual values. Another trend appears to be that as the weir side length decreases, the predictions seem to be consistently closer to the actual values.

The range of the data represented by the graphs in Figures 8 - 15 can be defined by the ranges of the Π terms. The term Π_2 = H/W had a range of 0.5 to 0.1. The term Π_3 = L/T had a range of 1.0 with a fully suppressed weir to 0.17 with the 6 inch throat width. The term Π_4 = H/L had a range of approximately 1.0 (6 inches of head in a 6 inch throat) to approximately 0.033 (0.1 ft of head in the 3.0 ft throat of a suppressed weir). The Π_5 = Sl/Llcw term had a high value of 0.5 for triangular plan view weirs and a low of 0.286 for the weirs with a Sl = 1 ft and Llcw = 3.5 ft. The final term, Π_6 = Llcw/L, had a high of 17 for the weir with Llcw=8.5 ft and L=0.5 ft and a low of 1.44 for the weir with Llcw=3.25 ft and L=2.25 ft. The laboratory data in the appendix provide the actual weir geometries.

Although the linear and second-order polynomial equations show a good fit of the data taken from all of the weirs tested, the validity of some of the data obtained while testing the suppressed long crested weirs may be questionable. The curves for the coefficient of discharge for these weirs did not follow the general pattern of the Cd curves for the contracted weirs (Figure 16).

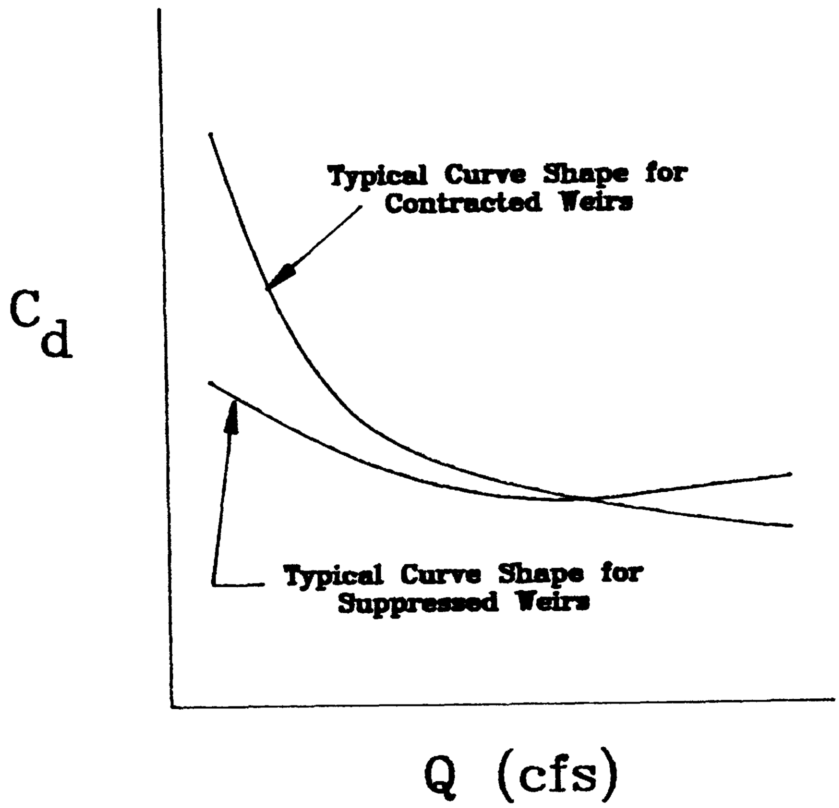

Figure 16. Comparison of Contracted and Suppressed C_d Curves.

This difference in the C_d curves may be due to a phenomena similar to that which caused the humps in the performance curves of the labyrinth weirs where the initial loss of ventilation in the nappe due to downstream interference gave a small increase in performance and then a consistent decrease in performance with increasing head (Hay and Taylor 1970). More flow interference with the channel sidewalls was observed with the suppressed weirs than any of the other weirs because the weir side crests were against the walls of the channel. However, limitations in the laboratory facilities did not allow the testing to see if there was an actual hump in the performance curve similar to the labyrinth weirs. Another aspect of the experiment that may or may not have caused error in the data was the aeration of the nappe for the suppressed weirs. Ventilation of the nappe was provided by a series of plastic tubes arranged in such a way as to avoid disturbing the flow over the crest. Air behind the nappe was readily apparent during the experiments, but occasionally the ventilation tubes filled with water and the air supply was limited. Although visual observation of the nappe was enough to verify if it was aerated or not, the degree of aeration was not readily apparent. If during various experiments with the suppressed weirs, the ventilation was not adequate to provide full aeration, then the pressure on the underside of the nappe would have been less than atmospheric. Such a decrease in pressure behind the nappe would have caused increased curvature of the nappe and increased discharge over the weir (French 1985).

In cases where long crested weirs that would be used for flow measurement are not suppressed weirs, it may be preferrable to use an equation developed from only the contracted weir data. The range of contractions that were tested are $.167 \le L/T \le .75$, where L/T is the ratio of the weir throat width to the channel width. If the suppressed weir data is deleted from the entire data set, as well as the data for $H < 0.10$ ft and $H/W > 0.5$, then the analysis of the remaining data for all of the contracted weirs provides the following equations:

$$C_d = 0.558 - 0.277\Pi_2 + 0.0839\Pi_3 - 0.123\Pi_4 - 0.232\Pi_5 - 0.00983\Pi_6 \tag{32}$$

This linear regression equation has an $r^2 = 93.7$ %.
The second-order polynomial below gives an $r^2 = 97.4$ %:

$$C_d = 0.287 - 0.350\Pi_2 - 0.211\Pi_3 - 0.585\Pi_4 + 2.06\Pi_5 - 0.0217\Pi_6 + 0.475\Pi_2^{\,2} + 0.136\Pi_3^{\,2} + 0.323\Pi_4^{\,2} - 2.72\Pi_5^{\,2} + 0.000642\Pi_6^{\,2} \tag{33}$$

The power function had an $r^2 = 87.7$ %.

$$C_d = 0.2983\,\Pi_2^{-0.342}\,\Pi_3^{0.293}\,\Pi_5^{-0.197}\,\Pi_6^{-0.201} \tag{34}$$

Plots of Qa vs. Qp for the data using equations (32) and (33) appear on the following pages. Figures 17 - 24 use the contracted weir data, meaning that all of the weirs have a

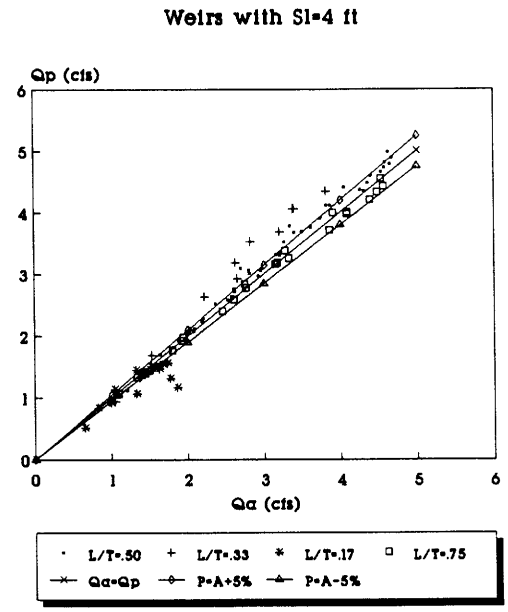

Figure 17. Actual vs Predicted Flowrate Using the Linear Equation for Contracted Weirs (Sl=4 ft.).

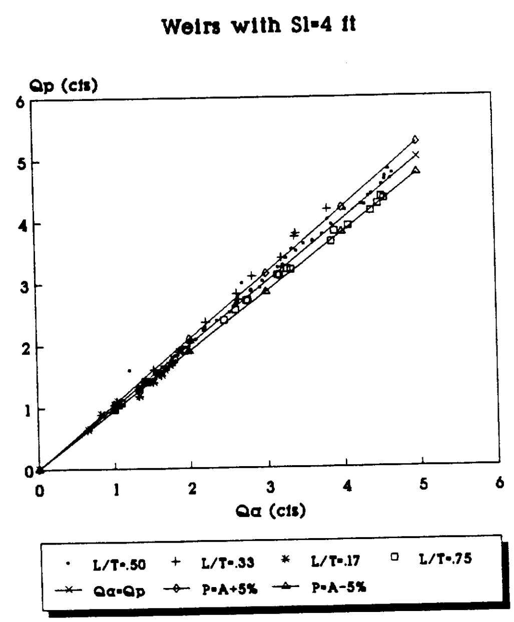

Figure 18. Actual vs Predicted Flowrate Using the 2nd Order Equation for Contracted Weirs (Sl=4 ft.).

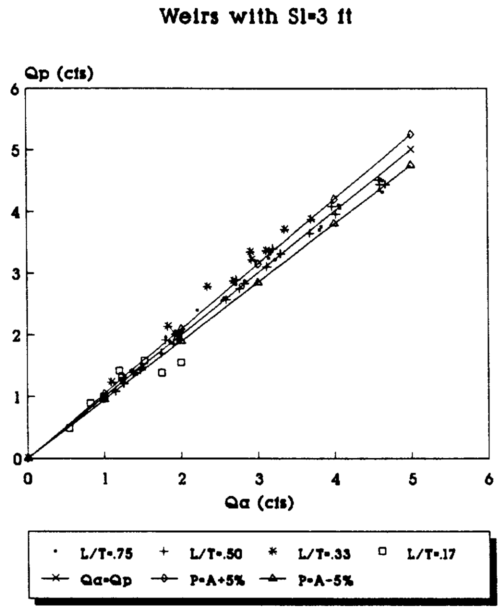

Figure 19. Actual vs Predicted Flowrate Using the Linear Equation for Contracted Weirs (S1=3 ft.).

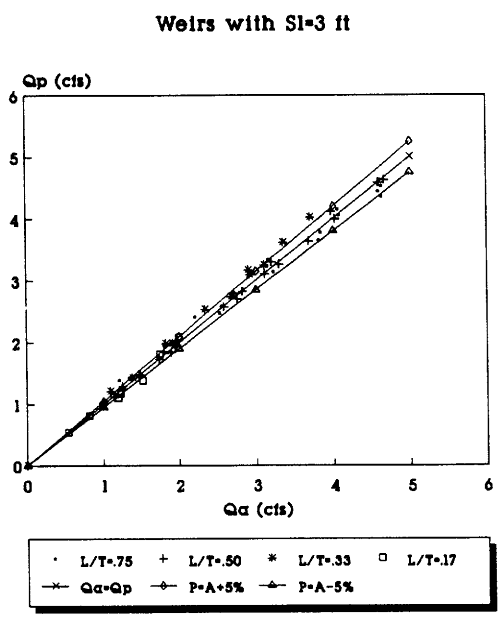

Figure 20. Actual vs Predicted Flowrate Using the 2nd Order Equation for Contracted Weirs (S1=3 ft.).

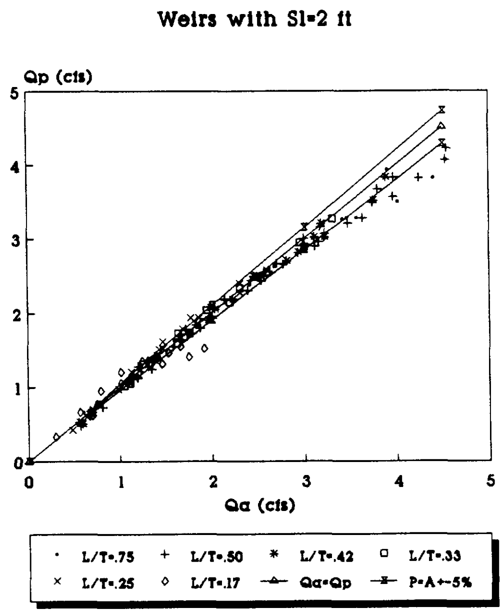

Figure 21. Actual vs Predicted Flowrate Using the Linear Equation for Contracted Weirs (Sl=2 ft.).

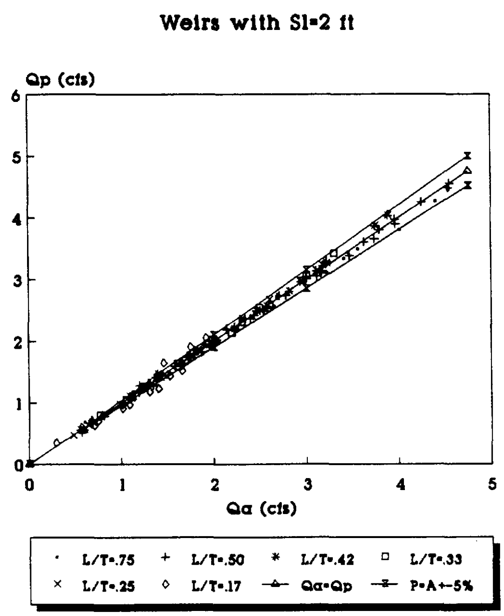

Figure 22. Actual vs Predicted Flowrate Using the 2nd Order Equation for Contracted Weirs (Sl=2 ft.).

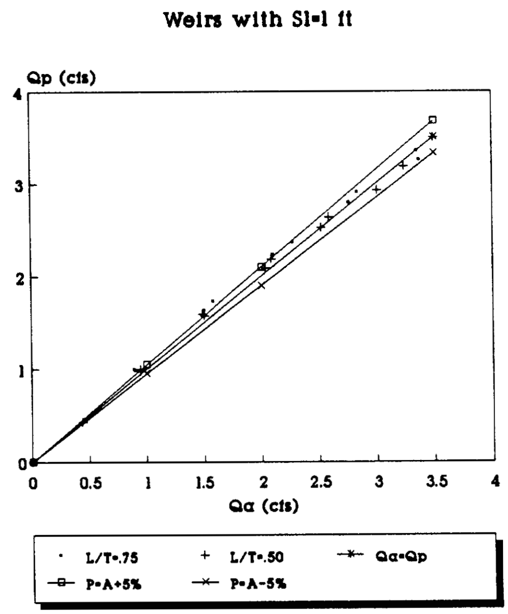

Figure 23. Actual vs Predicted Flowrate Using the Linear Equation for Contracted Weirs (Sl=1 ft.).

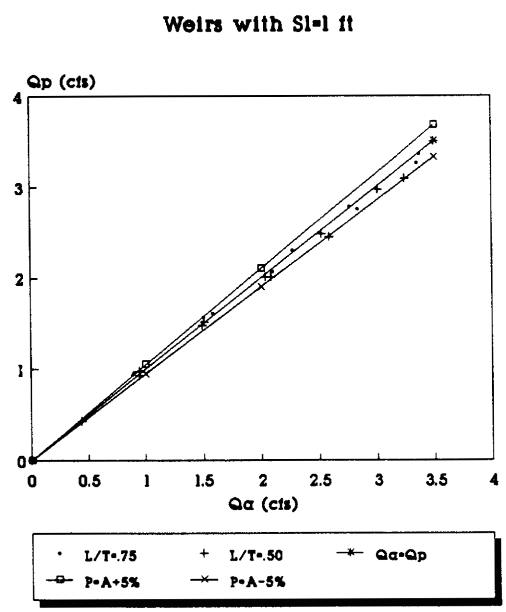

Figure 24. Actual vs Predicted Flowrate Using the 2nd Order Equation for Contracted Weirs (Sl=1 ft.).

contraction at the throat so that $L/T < 1.0$. Similar to the graphs shown previously, the Qa=Qp line represents the set of values where the predicted and actual flowrates would be equal. The range of plus or minus 5 percent is also shown on these graphs

It appears that for all of the data groups considered, the polynomial equations provided the best fit based on the r^2 values and the Qa vs. Qp graphs; the linear equations had the next best fit, and the power function had the lowest r^2 values of the three types of equations. The r^2 values also increased as the application of the equations became more specific and limitations were placed on which data were used in the analysis. The predictions generally tended to be better for the higher L/T values and the lower Llcw/L values. The weirs with the higher values of L/T were more effective at discharging larger flowrates with lower heads; for example, the weirs with $L/T = 0.75$ discharged almost double the flow of the weirs with $L/T = 0.17$ with nearly the same head and crest length. This is apparently due to the degree of contraction of the flow streamlines and how much of the flow has to make 90^0 changes in direction to discharge over the weir crest.

APPLICATION OF THE EQUATIONS

Because the coefficient of discharge is expressed in terms of head and the geometric parameters, the equations allow the flexibility of being applicable to many different configurations of weirs, so long as they are of the general shape of the so-called duckbill wiers. No diagonal or labyrinth weirs were tested, so the equations do not apply to these types of long crested weirs. Also, weirs with a crest that is rounded or something other than sharp would have different discharge characteristics than the sharp crested weirs tested, so the equations from this thesis would not apply. Although the first set of equations were developed from the entire set of data from the 67 weirs tested, it is recommended that the the second set of equations for all the weirs be used for general design purposes for suppressed weirs. The reason for selecting the second set of equations rather than the first is because of the limitations placed on the data of having H not less than 0.10 feet and of not operating the weir under H/W ratios greater than 0.5. These restrictions make the range of the data more specific, give a better r^2 fit, and would likely be more consistent with the actual operating limits of a weir used in the field. If only contracted weirs are being designed, it is recommended that only the last set of equations be used because they were developed specifically from just the contracted weir data.

A procedure similar to that presented by Walker (1987) could be used to design long crested weirs. The first step is to determine the maximum allowable water surface elevation either by an analysis of the effects of a backwater curve on upstream structures in the channel or by taking the total depth of the canal and subtracting the required allowance for freeboard. The minimum weir height may then be determined by the minimum head required on the turnout and by the depth of flow downstream of the weir. The weir must be tall enough to maintain the required head on the turnout even at minimum flow in the canal and also be tall enough to operate under free flow conditions with the nappe fully aerated when the downstream water surface is highest during peak canal flows.

The third step is determining the maximum weir head, which is done by subtracting the weir height from the maximum allowable water surface elevation. According to the limitations given by French (1985), the recommended minimum weir head on a weir used for flow measurement should not be less than 0.10 foot. The design head should be somewhere between these maximum and minimum weir heads. The designer should know the maximum and minimum flows that are likely to occur in the canal, and the designer should also determine how much fluctuation in the turnout is acceptable, as this value will influence the limiting range of design heads on the long crested weir.

The designer should then decide on the throat width of the weir (which will give the term L/T); the larger values of L/T tended to give better performance. For the first trial, a triangular shaped weir in plan view is recommended because then the parameter Sl/Llcw is known to be 0.5. The length of the long crested weir can be determined by using the equation

$$Q = C_d \, Llcw \, H^{3/2} \sqrt{2g}$$

and one of the C_d equations (the simple linear ones) to determine the C_d value. Use the maximum flowrate in the canal for Q and the maximum allowable head for H. The minimum weir crest height can be used for W; then the parameters H/W, L/T, H/L, and Sl/Llcw in the Cd equation are known. The only unknown is the Llcw which appears in only the Llcw and Llcw/L terms of the equation. The length of the weir is determined by solving the equation for Llcw. This value of Llcw tends to be the minimum which will allow for the maximum flow with the maximum head. If it is desired to pass the maximum flow over the weir with less than the maximum head, then the weir crest will have to be longer. Using the calculated value of Llcw and the minimum flowrate, solve the equations for H to determine if the minimum H is greater than 0.10 feet. This will determine if the weir is capable of measuring the entire range of flows. If the weir can not measure the entire range of flows, adjustments may be made in the weir geometry to improve the range of flow measurement. This calculated Llcw and assumed geometry give a starting value for the designer to test other weir geometries and lengths as desired.

The object of the design is usually to minimize variations in flow through the turnout while at the same time minimizing weir construction costs. A weir with a long crest will tend to provide more consistent head for the turnout, but it also costs more to build. The shorter crest may cost less, but it will not provide the same degreee of control of the flow through the turnout. The designer must use an iterative process to develop a weir design that balances performance and costs to match the specific needs of the canal system. The C_d equation that best suits the designed weir could then be used to develop rating curves for measuring flow over the structure. Once the structure is designed, the parameters Llcw, (2g), L/T, Sl/Llcw, and Llcw/L are all constants; and by varying H in the H/W and H/L terms the predicted flowrate, Q, can be calculated for various head values.

RECOMMENDATIONS

There are a number of conditions that would be present in a field installation which were not specifically addressed in the laboratory weir tests. As noted by several authors (Kraatz and Mahajan 1975; Ackers 1978; Walker 1987), the sharp crested weirs are effective sediment traps. Especially for high weirs where the approach velocity is very small, there is great tendency for heavy sand and silt particles to settle in the channel upstream of the weir. For this reason, it is recommended that in areas where the water carries a lot of sediment either a sluice gate of some type be installed with the weir to allow the flushing of sediments from the upstream side of the weir, otherwise the canal company must install a regular schedule for clearing the accumulated sediments from behind the weir. Large deposits of sediments will certainly affect the flowfield upstream of the weir and alter the discharge characteristics. However, this research did not involve the study of the effects of a sluice gate or sediment deposits on weir performance, so their actual effects on weir discharge are unknown.

The major purpose of the long crested weirs used in irrigation canals is to maintain a fairly constant water surface elevation at turnouts. The tests of the 67 weirs in the laboratory channel involved only the weirs in the channel. There were no turnouts upstream of the weir to remove water from the channel. If the turnouts in the sides of the irrigation canal were closed, then presumably the flow conditions affecting the long crested weir in the field canal would be similar to those affecting the weirs in the laboratory channel. However, if flow is being removed from the canal through the turnouts directly upstream from the long crested weir, this removal of flow may significantly alter the flowfield and head behind the weir in such a way as to change the weir discharge characteristics. The extent of influence of the flow through the turnouts on the weir performance is unknown. This situation was not tested in these long crested weir laboratory studies, and it is recommended that any application of the C_d equations be made with this limitation in mind. Accordingly, the locations of the turnouts must be far enough upstream from the weir so that the influence of the turnouts on the flowfield in the main channel must be negligible by the time it reaches the weir.

As with any other type of flow measurement structure, whenever possible it is preferable to be able to check the calibration of the long crested weir after it has been constructed. This would allow for verification of the accuracy with which the C_d equations can predict the flowrate on a full-scale long crested weir. Once the structure has been installed and used in canal operation, a program of regular inspection and maintenance is recommended to insure proper and consistent operating conditions for the weir.

CHAPTER V

SUMMARY AND CONCLUSIONS

A constant flowrate to farmers when they need it and accurate flow measurement are important factors in making efficient distribution of limited supplies of water in agriculture. The long crested weir has proven successful in providing minimal fluctuations in flowrate through canal turnouts so that farmers can maintain a fairly constant flowrate. This research used data from experiments with 67 different long crested weir models to develop equations which will permit long crested weirs to accurately measure the flowrate in canals.

Three types of equations were tested to see which gave the best prediction of the coefficient of discharge. The first type of equation was a multivariable linear regression. The second type of equation was a second-order polynomial, and the final type of equation was the power function. Of these three, the second-order polynomial provided the best prediction of the discharge coefficient, as shown by the r^2 values of fitting the data to the C_d equation and the graphs showing the actual flowrate versus the predicted flowrate.

By applying certain limitations to the data used to develop the equations to predict C_d for all of the weirs tested, the bulk of the values of predicted flowrates fell within plus or minus 5 percent of the actual flowrates measured in the Laboratory. Equations (29) and (30) are general equations to predict C_d for all of the weirs tested, and both equations have r^2 values greater than 90 percent. These can be used for suppressed long crested weirs as well as the full range of the contracted weirs that were tested.

If the long crested weir to be designed is a contracted weir, equations (32) and (33) are recommended to calculate the C_d. They both have r^2 values greater than 90 percent and have improved r^2 values from the equations used for all of the weirs. Because these equations deal specifically with the contracted weirs, they are able to give a better prediction of the flowrate than the more general equations.

Some standard weirs can measure the flowrate to within plus or minus 2 percent, but these long crested weir equations sacrifice some accuracy to provide a general application to many weir shapes. The C_d equations can give a flowrate prediction within plus or minus 5 percent of the actual for the weir models tested in the Laboratory. However, this research did not involve any tests on full-scale field weirs, so the applicability of the equations for full-scale weirs remains to be determined. The application of the equations is limited to use with the duckbill type of long crested weirs since they were the only type of weir that was tested; labyrinth and diagonal weirs have different flow characteristics than the duckbill weirs, so the equations should not be applied to them. The 67 laboratory models all had sharp crests, so the equations apply for sharp crested weirs. Crests that are rounded or square have different flow characteristics, and the sharp crested C_d equations would not apply directly to other types of weir crests.

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APPENDIX--LABORATORY DATA

L = Weir Throat Width in Feet
Dw = Downstream Weir Crest Width in Feet
Sl = Side Length of Weir Crest in Feet
Llcw = Total Length of Long Crested Weir Crest in Feet
H = Head on the Weir in Feet
Q = Flowrate in Cubic Feet per Second

L = 1.5
Dw = 1.526
Sl = 3.990
Llcw = 9.506

H	Q
0.342	4.57
0.321	4.36
0.298	3.82
0.272	3.50
0.251	3.23
0.234	2.96
0.208	2.58
0.183	2.18
0.153	1.78
0.126	1.40
0.096	0.99
0.074	0.70
0.046	0.41

L = 1.5
Dw = 1.25
Sl = 3.990
Llcw = 9.230

H	Q
0.342	4.52
0.321	4.30
0.284	3.61
0.255	3.22
0.234	2.94
0.178	2.08
0.160	1.81
0.133	1.50
0.113	1.21
0.081	0.81
0.046	0.42

L = 1.5
Dw = 1.005
Sl = 3.990
Llcw = 8.985

H	Q
0.357	4.56
0.331	4.26
0.291	3.62
0.262	3.16
0.224	2.63
0.195	2.20
0.171	1.88
0.137	1.52
0.098	0.99
0.053	0.43

L = 1.5
Dw = 0.505
Sl = 3.990
Llcw = 8.485

H	Q
0.384	4.65
0.319	3.75
0.291	3.28
0.261	2.97
0.226	2.53
0.194	2.08
0.143	1.52
0.105	1.03
0.072	0.65
0.055	0.47

L = 1.5
Dw = 0.255
Sl = 3.990
Llcw = 8.235

H	Q
0.407	4.66
0.382	4.41
0.345	3.87
0.312	3.40
0.263	2.80
0.245	2.61
0.200	2.04
0.159	1.65
0.156	1.61
0.115	1.09
0.045	0.36

L = 1.5
Dw = 0.000
Sl = 3.990
Llcw = 7.980

H	Q
0.432	4.62
0.381	4.05
0.330	3.34
0.275	2.79
0.235	2.35
0.173	1.64
0.121	1.11
0.071	0.58
0.051	0.36
0.044	0.31

L = 1.0
Dw = 0.505
Sl = 3.990
Llcw = 8.485

H	Q
0.510	4.35
0.451	3.82
0.410	3.39
0.366	3.21
0.290	2.66
0.217	2.02
0.151	1.36
0.117	1.06
0.077	0.61
0.057	0.42

L = 1.0
Dw = 0.000
Sl = 3.990
Llcw = 7.980

H	Q
0.616	4.59
0.573	4.29
0.520	3.78
0.455	3.37
0.376	2.83
0.337	2.63
0.280	2.22
0.192	1.54
0.096	0.74
0.052	0.36

L = 0.5
Dw = 0.505
Sl = 3.990
Llcw = 8.485

H	Q
0.328	1.74
0.304	1.66
0.279	1.56
0.242	1.40
0.184	1.13
0.159	1.00
0.098	0.65
0.063	0.42

L = 0.5
Dw = 0.000
Sl = 3.990
Llcw = 7.980

H	Q
0.451	1.85
0.425	1.78
0.382	1.64
0.343	1.52
0.285	1.32
0.207	1.04
0.154	0.82
0.091	0.51
0.064	0.38

L = 3.0
Dw = 1.500
Sl = 3.990
Llcw = 9.480

H	Q
0.264	4.45
0.231	3.50
0.206	2.99
0.175	2.34
0.144	1.72
0.100	1.05
0.054	0.44

L = 3.0
Dw = 1.010
Sl = 3.990
Llcw = 8.990

H	Q
0.284	4.72
0.265	4.27
0.229	3.19
0.198	2.55
0.146	1.70
0.107	1.13
0.069	0.59

L = 3.0
Dw = 0.505
Sl = 3.990
Llcw = 8.485

H	Q
0.288	4.42
0.258	3.71
0.235	3.23
0.210	2.78
0.183	2.25
0.158	1.86
0.123	1.31
0.074	0.65

L = 3.0
Dw = 0.000
Sl = 3.990
Llcw = 7.980

H	Q
0.303	4.55
0.288	4.04
0.240	3.09
0.208	2.56
0.157	1.77
0.104	1.02
0.046	0.34

L = 2.229
Dw = 1.505
Sl = 3.990
Llcw = 9.485

H	Q
0.278	4.48
0.247	3.84
0.223	3.33
0.178	2.46
0.142	1.79
0.120	1.44
0.083	0.90
0.034	0.29

L = 2.229
Dw = 1.010
Sl = 3.990
Llcw = 8.990

H	Q
0.286	4.38
0.273	4.08
0.229	3.16
0.196	2.60
0.158	1.93
0.103	1.08
0.037	0.26

L = 2.229
Dw = 0.505
Sl = 3.990
Llcw = 8.485

H	Q
0.315	4.55
0.290	4.09
0.242	3.18
0.218	2.76
0.166	1.93
0.133	1.41
0.045	0.33

L = 2.229
Dw = 0.000
Sl = 3.990
Llcw = 7.980

H	Q
0.344	4.53
0.309	3.90
0.270	3.28
0.235	2.75
0.179	1.93
0.134	1.33
0.051	0.36

L = 2.229
Dw = 1.5
Sl = 3.000
Llcw = 7.5

H	Q
0.338	4.63
0.310	4.06
0.260	3.14
0.233	2.72
0.181	1.91
0.140	1.33
0.056	0.41

L = 2.229
Dw = 1.010
Sl = 3.000
Llcw = 7.010

H	Q
0.363	4.60
0.334	4.07
0.281	3.16
0.219	2.20
0.188	1.79
0.145	1.21
0.064	0.38

L = 2.229
Dw = 0.505
Sl = 3.000
Llcw = 6.505

H	Q
0.379	4.58
0.334	3.82
0.269	2.81
0.249	2.55
0.206	1.97
0.123	0.96
0.063	0.40

L = 2.229
Dw = 0.000
Sl = 3.000
Llcw = 6.000

H	Q
0.412	4.63
0.360	3.80
0.320	3.23
0.265	2.52
0.210	1.85
0.156	1.24
0.038	0.22

L = 1.500
Dw = 1.250
Sl = 3.000
Llcw = 7.250

H	Q
0.395	4.58
0.326	3.68
0.263	2.82
0.195	1.94
0.156	1.47
0.117	0.99
0.045	0.31

L = 1.500
Dw = 0.750
Sl = 3.000
Llcw = 6.750

H	Q
0.423	4.59
0.378	4.02
0.305	3.12
0.260	2.59
0.188	1.73
0.146	1.26
0.049	0.32

L = 1.500
Dw = 0.250
Sl = 3.000
Llcw = 6.250

H	Q
0.467	4.66
0.414	4.01
0.349	3.30
0.297	2.77
0.216	1.90
0.145	1.15
0.041	0.24

L = 1.500
Dw = 0.000
Sl = 3.000
Llcw = 6.000

H	Q
0.503	4.60
0.451	3.97
0.375	3.20
0.324	2.72
0.230	1.81
0.171	1.25
0.089	0.56

L = 1.000
Dw = 1.000
Sl = 3.000
Llcw = 7.000

H	Q
0.571	4.59
0.508	4.10
0.410	3.35
0.351	2.92
0.230	1.91
0.172	1.38
0.067	0.39

L = 1.000
Dw = 0.505
Sl = 3.000
Llcw = 6.505

H	Q
0.491	3.70
0.405	3.11
0.342	2.68
0.247	1.96
0.191	1.49
0.076	0.49
0.048	0.28

L = 1.000
Dw = 0.000
Sl = 3.000
Llcw = 6.000

H	Q
0.454	2.91
0.363	2.34
0.282	1.82
0.179	1.09
0.080	0.37
0.061	0.27

L = 0.500
Dw = 0.505
Sl = 3.000
Llcw = 6.505

H	Q
0.456	1.98
0.316	1.51
0.240	1.22
0.184	0.99
0.102	0.53
0.064	0.32

L = 0.500
Dw = 0.000
Sl = 3.000
Llcw = 6.000

H	Q
0.473	1.74
0.296	1.19
0.179	0.81
0.064	0.29

L = 3.000
Dw = 1.250
Sl = 3.000
Llcw = 7.250

H	Q
0.327	4.57
0.287	3.71
0.243	2.90
0.163	1.64
0.124	1.13
0.100	0.87
0.040	0.28

L = 3.000
Dw = 0.760
Sl = 3.000
Llcw = 6.760

H	Q
0.345	4.57
0.288	3.46
0.251	2.82
0.208	2.16
0.163	1.55
0.102	0.78
0.046	0.28

L = 3.000
Dw = 0.255
Sl = 3.000
Llcw = 6.255

H	Q
0.359	4.55
0.311	3.68
0.263	2.88
0.188	1.77
0.137	1.18
0.098	0.71
0.045	0.27

L = 3.000
Dw = 0.000
Sl = 3.000
Llcw = 6.000

H	Q
0.377	4.61
0.310	3.45
0.255	2.56
0.184	1.63
0.123	0.92
0.066	0.38
0.050	0.29

L = 3.000
Dw = 1.495
Sl = 2.000
Llcw = 5.495

H	Q
0.409	4.56
0.365	3.82
0.310	2.99
0.250	2.24
0.197	1.62
0.116	0.79
0.046	0.22

L = 3.000
Dw = 1.010
Sl = 2.000
Llcw = 5.010

H	Q
0.417	4.49
0.373	3.76
0.314	2.96
0.226	1.85
0.173	1.26
0.079	0.40
0.050	0.23

L = 3.000
Dw = 0.505
Sl = 2.000
Llcw = 4.505

H	Q
0.437	4.60
0.394	3.79
0.340	3.04
0.298	2.51
0.236	1.79
0.155	0.95
0.051	0.22

L = 3.000
Dw = 0.000
Sl = 2 w/overlap
Llcw = 3.974

H	Q
0.485	4.57
0.405	3.47
0.320	2.45
0.249	1.69
0.171	0.98
0.100	0.45
0.042	0.13

L = 2.219
Dw = 1.250
Sl = 2.000
Llcw = 5.250

H	Q
0.396	3.89
0.338	3.41
0.263	2.42
0.158	1.24
0.084	0.54
0.046	0.29

L = 2.219
Dw = 0.750
Sl = 2.000
Llcw = 4.750

H	Q
0.455	4.55
0.375	3.45
0.316	2.69
0.215	1.59
0.166	1.12
0.115	0.65
0.038	0.14

L = 2.219
Dw = 0.255
Sl = 2.000
Llcw = 4.255

H	Q
0.490	4.41
0.423	3.55
0.347	2.69
0.269	1.89
0.206	1.30
0.122	0.60
0.047	0.15

L = 2.219
Dw = 0.000
Sl = 2 w/overlap
Llcw = 3.990

H	Q
0.488	4.00
0.420	3.22
0.363	2.65
0.295	1.98
0.206	1.20
0.131	0.67
0.040	0.14

L = 1.500
Dw = 1.505
Sl = 2.000
Llcw = 5.505

H	Q
0.411	3.97
0.344	3.17
0.277	2.44
0.169	1.31
0.105	0.69
0.043	0.22

L = 1.500
Dw = 1.250
Sl = 2.000
Llcw = 5.250

H	Q
0.479	4.55
0.415	3.79
0.345	3.00
0.264	2.16
0.186	1.38
0.104	0.67
0.047	0.22

L = 1.500
Dw = 1.010
Sl = 2.000
Llcw = 5.010

H	Q
0.488	4.54
0.417	3.73
0.355	3.00
0.277	2.23
0.211	1.59
0.121	0.81
0.035	0.20

L = 1.500
Dw = 0.750
Sl = 2.000
Llcw = 4.750

H	Q
0.489	4.26
0.409	3.47
0.344	2.78
0.263	2.01
0.175	1.19
0.083	0.47
0.040	0.19

L = 1.500
Dw = 0.505
Sl = 2.000
Llcw = 4.505

H	Q
0.488	3.96
0.402	3.16
0.321	2.41
0.253	1.78
0.181	1.20
0.094	0.51
0.048	0.21

L = 1.500
Dw = 0.250
Sl = 2.000
Llcw = 4.250

H	Q
0.483	3.63
0.424	3.12
0.358	2.53
0.277	1.89
0.207	1.34
0.103	0.57
0.050	0.22

L = 1.500
Dw = 0.000
Sl = 2.000
Llcw = 4.000

H	Q
0.489	3.22
0.421	2.71
0.350	2.16
0.292	1.72
0.224	1.20
0.140	0.68
0.048	0.14

L = 1.250
Dw = 1.255
Sl = 2.000
Llcw = 5.255

H	Q
0.466	3.90
0.386	3.18
0.308	2.46
0.240	1.83
0.197	1.40
0.109	0.67
0.041	0.19

L = 1.250
Dw = 0.760
Sl = 2.000
Llcw = 4.760

H	Q
0.485	3.76
0.409	3.12
0.343	2.57
0.270	1.95
0.206	1.38
0.090	0.47
0.040	0.19

L = 1.250
Dw = 0.250
Sl = 2.000
Llcw = 4.250

H	Q
0.478	3.22
0.421	2.81
0.341	2.24
0.278	1.75
0.190	1.10
0.111	0.58
0.043	0.16

L = 1.250
Dw = 0.000
Sl = 2.000
Llcw = 4.000

H	Q
0.482	2.94
0.417	2.50
0.347	2.04
0.286	1.65
0.201	1.10
0.119	0.55
0.048	0.18

L = 1.000
Dw = 1.000
Sl = 2.000
Llcw = 5.000

H	Q
0.468	3.31
0.413	2.95
0.326	2.31
0.244	1.70
0.164	1.05
0.052	0.27

L = 1.000
Dw = 0.500
Sl = 2.000
Llcw = 4.500

H	Q
0.469	3.00
0.402	2.61
0.338	2.20
0.258	1.62
0.185	1.11
0.073	0.34
0.039	0.15

L = 1.000
Dw = 0.000
Sl = 2.000
Llcw = 4.000

H	Q
0.473	2.49
0.370	1.94
0.315	1.63
0.246	1.25
0.163	0.76
0.045	0.16

L = 0.750
Dw = 0.755
Sl = 2.000
Llcw = 4.755

H	Q
0.469	2.60
0.416	2.32
0.324	1.86
0.256	1.47
0.194	1.10
0.121	0.62
0.049	0.23

L = 0.750
Dw = 0.250
Sl = 2.000
Llcw = 4.250

H	Q
0.482	2.30
0.414	2.00
0.341	1.68
0.287	1.43
0.197	0.98
0.104	0.47
0.046	0.17

L = 0.750
Dw = 0.000
Sl = 2.000
Llcw = 4.000

H	Q
0.467	1.98
0.413	1.77
0.333	1.45
0.250	1.11
0.161	0.70
0.087	0.34
0.045	0.15

L = 0.500
Dw = 0.505
Sl = 2.000
Llcw = 4.505

H	Q
0.482	1.93
0.402	1.66
0.327	1.40
0.241	1.09
0.148	0.70
0.086	0.36
0.049	0.22

L = 0.500
Dw = 0.255
Sl = 2.000
Llcw = 4.255

H	Q
0.487	1.75
0.414	1.53
0.345	1.31
0.249	1.01
0.181	0.74
0.089	0.33
0.059	0.21

L = 0.500
Dw = 0.000
Sl = 2.000
Llcw = 4.000

H	Q
0.484	1.46
0.406	1.25
0.317	1.01
0.242	0.79
0.172	0.56
0.101	0.29
0.064	0.17

L = 2.250
Dw = 1.500
Sl = 1.000
Llcw = 3.500

H	Q
0.458	3.35
0.404	2.82
0.325	2.10
0.267	1.57
0.177	0.89
0.090	0.36
0.040	0.11

L = 2.250
Dw = 1.260
Sl = 1.000
Llcw = 3.260

H	Q
0.482	3.37
0.420	2.77
0.364	2.27
0.271	1.50
0.186	0.90
0.091	0.31
0.036	0.08

L = 1.510
Dw = 1.500
Sl = 1.000
Llcw = 3.500

H	Q
0.488	3.23
0.406	2.59
0.344	2.09
0.267	1.49
0.184	0.95
0.096	0.38
0.040	0.13

L = 1.510
Dw = 1.255
Sl = 1.000
Llcw = 3.255

H	Q
0.486	3.01
0.421	2.52
0.355	2.04
0.282	1.51
0.198	0.95
0.107	0.44
0.032	0.08