CANAL SIDE WEIRS FOR WATER DELIVERY
TO IRRIGATION FURROWS

by

Shahriar Eftekharzadeh

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SIGNED: Shahriar Esfahanizadeh

APPROVAL BY THESIS DIRECTOR

This thesis has been approved on the date shown below:

D. D. Fangmeier — March 8, 1985

D. D. Fangmier
Professor of Irrigation Engineering

Date

To my dear father who made it all possible

Preface

In irrigation, science and technology often lag behind practice. New techniques and ideas are often developed by farmers or agricultural promotors such as commercial firms, long before any scientific research is conducted, or their acclaimed improvements are independently verified. The canal side weir, as a turn-out system for furrow irrigation is no exception to the above. This research work was recognised as necessary in order to test and analyse this system, and to draw guidelines for a scientifically based practice. The work was based in the U.S. Water Conservation Laboratory in Phoenix, Arizona.

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ABSTRACT

The canal side weir system is a relatively new method for water distribution to irrigation furrows. The system consists of a concrete lined ditch (canal), equipped with small openings or weirs which deliver the design irrigation flow rate into individual furrows. This research aims at evaluating the proformance characteristics of this system through laboratory, field, and computer studies.

A laboratory model was used to establish head discharge relationships for various weirs at different channel velocities. For sharp entranced weirs, discharge for a given head was found to be inversselly proportional to the magnitude of the channel velocity. At small heads (20-30 mm), discharge at 0.3 m/s velocity was found to be about 15% less than the corresponding discharge at zero channel velocity . For normal irrigation settings (head on the weir = 60-80 mm), the difference was about 6%. Streamline entranced weirs were found to have negligible sensitivity to changes in the channel velocity.

Field evaluation studies revealed better overall proformance than conventional systems. For an operating system, an application efficiency of 86% with uniformity coefficient of 0.84 at 7% deficit was calculated. The components of loss were run-off 4% and deep percolation 10%.

CHAPTER 1

GENERAL BACKGROUND

Introduction

In this time and age, research work at the graduate level, often tends to concern itself with a specific aspect of a complex multi-component system. In order to avoid alienation of the researcher, his research work must be carried out within the context of that system as a whole. He must therefore acquire a broad overall picture of his research field and the position where he is trying to make his contribution.

For this research work, the canal side weir method is one component within the broad and complex field of irrigation science and technology. The method is a proposed solution to the problem of easy water delivery to irrigation furrows. The need for this method can not be appreciated unless one understands the history of events giving rise to the problem which it tries to solve. Furthermore, a knowledge of the history of irrigation, which is as old as civilization itself, will help to enlighten both the researcher and the reader. A brief historical review therefore follows.

Historical Background to Water Supply

Water supply is providing the right quantity of water at the right time for the required location. Ancient Egyptian civilizations realized that reliance on natural phenomenon (floods) for supply of water was too unreliable and irregular, and that some means of artificial supply was needed (Pasha, 1937). They therefore constructed man dug canals as early as 3300 BC, so that the necessary water for irrigation and drinking could be conveyed from it's river source to where it was needed. One of these canals is stated to have been 9 to 12 m (29 to 39 ft) deep and about 120 m (390 ft) wide (Gulhati and Smith, 1967). This by no means has been unique to Egyptians. Throughout history, various civilizations have devised their own means of access to water. The Persians (6000 BC), the Chinese (2200) and the Latin Americans (500 AD) all had their own appropriate methods of water conveyance (Hansen, Israelsen and Stringham, 1980), and used various sources of supply.

Persians used, and are still using quanats or underground aqueducts with periodic service wells, to tap the ground waters in highland aquifers and convey it to the lower lying agricultural areas. The quanats can be as long as 80 km (50 miles) with access wells about 50 m (165 feet) apart and up to 300 m (1000 feet) deep. The total length of the quanat system in Iran is estimated at 350,000 km (220,000 miles) all of which has been dug by hand (Bybordi, 1974, also see Olson, 1980).

Irregular supply of water by rivers and underground sources neccessitated the construction of facilities to store water in times of plenty for release in dry periods. Egyptians have the claim to the world's oldest dam. Built 5000 years ago, it spanned 108 m (356 ft) wide and stood 12 m (39 ft) high (Gulhati and Smith, 1967). The famous Tu-Kiang Dam in China was built about 200 BC and is still a successful dam today (Hansen, Isrealsen and Stringham, 1980). The main purpose for storage facilities was to ensure an adequate supply of water for drinking and irrigation throughout the year.

Today, storage facilities and conveyance systems are designed to satisfy similar needs as well as other new requirements. Designers are equipped with the vast experience of the past ages, and the results of recent scientific research. Completeness, appropriateness, efficiency and cost effectiveness are the design and construction criteria of today. In addition to water supply, storage facilities serve to provide hydroelectric power, flood control, water for industrial use and recreation and wildlife uses.

Background to Irrigation Practice

The most ancient form of irrigation is flooding of basins practiced in Egypt about 3300 BC. The idea was taken from the natural annual flooding of the land surrounding the Nile, which left a rich fertilizing deposit on the land following the disappearance of the flood waters. Seeds sown on this naturally irrigated fertile land produced excellent yields. Basins and the canals that supplied them, duplicated this natural phenomena in a controlled fashion. Gradually an elaborate network of banks was built which divided the whole countryside into interconnected basins with facilities for storage and disposal of surplus waters (Pasha, 1937). By the 8th century about 700,000hectares were irrigated annually and by the 13th century about 1.4 million, all by basin method (Gulhati and Smith, 1967).

Other areas in the world used similar methods for water application. The flooding of extensive areas of land referred to as "wild flooding" was practiced throughout Asia and Southern Europe. This technique has remained unchanged in many areas of the world today, while in other areas it has been improved and renamed as controlled flooding.

Although very widespread and common, application of water through the surface or surface irrigation, has by no means been the only technique. Persians have been using "Koozeh" or clay pot irrigation for centuries. The technique uses permeable water filled clay pots buried close to the root system. Water is then slowly released for use by the crop. The system, being simple and efficient, is being rediscovered in the developed world today.

Historical irrigation was practiced mainly with the simple understanding that the crop needs water for it's growth. There were little or no estimates of the actual quantities that the crop needed. In order to ensure sufficient water for the crop, excess water was often applied. This practice is very widespread in many areas of the world and is only restrained by the availability of water. The practice can be very hazardous especially in areas with poor drainage characteristics, resulting in high water tables, salinity and loss in production. Settlements and large agricultural communities are known to have vanished because of this practice.

Today, the practice and potentials of irrigated agriculture are much better understood. Details about the necessary environment for optimum plant growth have been identified. Accurate estimations about the quantity, quality and timeliness of the irrigation water are now available. A great deal of emphasis is placed upon efficiency of water use and its uniform application. Irrigation practice is now a commerical venture which must be cost effective to be feasible.

These advances have called for the development of compatible irrigation techniques capable of satisfying the new requirements.

Advances in Surface Irrigation Techniques

Irrigation techniques of today must be capable of uniformly applying the necessary amount of water with an acceptable efficiency. They must also be easy to operate, flexible towards system variables, appropriate to locality and cost-effective. In order to satisfy these requirements, a number of new irrigation techniques under the categories of sprinkler, trickle and sub-surface have recently been developed, whilst the old surface irrigation techniques have been modified. Comprehensive descriptions of such techniques may be found in most texts on irrigation and in particular in that by Jensen (1981). We turn our attention to developments in surface irrigation.

Surface irrigation through the many centuries of its practice, has resulted in the evolution of numerous local methods. The scientific approach of recent years has selected and developed those which are thought to be most effective. Scientifically based design criteria have been developed for the selected methods. Level basin (Erie and Dedrick, 1979), border strips (Withers and Vipound, 1980) and furrows and corrugations (Marr, 1967) are the results of the application of the design criteria to the old techniques. These techniques are known as controlled flooding.

In level basins a predetermined volume of water is allowed to flood a level area surrounded by dikes. In other methods water is guided down an irrigation slope by channels as wide as 30 meters (border strips) and as narrow as several millimeters (corrugations). In achieving uniform applications of water, effective and accurate land leveling plays an essential role. The successfull operation of all surface systems depends upon the capability of providing relaivly uniform design slopes. The design slope is, zero in the case of level basins and anywhere between 0.1% to 12% with other techniques (Withers and Vipound, 1980). The development of sophisticated levelling techniques, which incorporate lazers for accurate indications of the desired slope (Erie and Dedrick, 1979), has allowed for accurate installation ofthese design slopes.

The Importance of the Delivery Technique

An essential factor in the success of surface irrigation is the technique used for water delivery from the on-farm source to the application layout (basins, borders etc.). The success of level basins for example, depends on the fast spreading of the water over the basin area such that a uniform depth of water is applied over the whole basin. This means a large inflow rate into the basin and requires special structures to prevent erosion (Hart, Collins, Woodward and Humphrys, 1980). In all other methods, the uniformity in the advance of the water front down the field, prepared for by the elaborate land forming operations, depends on the capability of uniform water delivery along the upper end of the field. Many techniques have been developed to ensure a uniform water delivery to individual furrows or corrugations, or to provide a uniform inflow along the top end of the border strip. Uniform water delivery is now an important design objective for graded irrigation layouts.

In achieving uniformity, advantage has been taken of the hydraulic head-discharge relationship for pipes and orifices. Providing the same hydraulic head over identical pipes or orifices, which empty into consecutive furrows or corrugations, should provide identical discharge into these furrows. Indeed for many practical purposes this idea works. For a border, a uniform inflow at the upstream end, may be provided by employing a number of these orifices or pipes, equispaced as appropriate.

In putting this idea into practice a number of different techniques such as siphon tubes, spiles, gated pipes and turn out valves have developed (Marr, 1967). All of these techniques have been tried for decades with reasonable success. The nature of these delivery systems however, is such that they require much human attention and supervision for their satisfactory operation.

Problems with Current Surface Delivery Techniques

With current techniques, the labor required for applying the irrigation water is a significant part of the total man-hours required for producing the crop. Present systems require extensive degrees of supervision for their satisfactory operation. This is a major disadvantage of the existing surface delivery techniques, which initiates a chain of operational problems and performance limitations.

For irrigation to start, all the delivery points serving the field must be visited by an irrigator in order to either prime the tube (siphons), or to open the valve or the gate. Any subsequent adjustment to the delivery rate requires a similar visit. In furrow irrigation, siphon tubes seems to be the most dominant method of water delivery. Marr (1967) credits the siphon tube as the only type of outlet practical for concrete lined head ditches serving flat land furrows. Siphon tubes, however, may have inadequate discharge sensitivity to head fluctuations that result from the often unsteady nature of the supply system. If so an irrigator must try to balance the inflow by the supply network to his ditch, with the outflow through the siphons which means adding or taking out siphon tubes to maintain equilibrium. Since the adding or taking out of siphon tubes is only practical along a small portion of the supply ditch, fluctuations in inflow may result in non-uniformity in application and reduce application efficiency.

The presence of an irrigator is further required to correct for the non-uniformity in advance that results from the variability in infiltration rates of different furrows (in particular wheel and non-wheel furrows). Whether it is siphons, spiles or gated pipes, the irrigator must try to maintain a uniform advance front by increasing the stream size in the lagging furrows or reducing the inflow to the leading ones. Another problem requiring irrigator attention is the clogging of the turn-out points (siphons, spiles etc.) by the surface debris often present in irrigation water, specially if it comes from a surface source.

To provide the lower end of the furrow with sufficient intake opportunity time for acceptable moisture replacement, irrigation must continue after the advance front has reached the end (for a time equal to the necessary intake opportunity time). For maximum efficienty, the stream size must be matched with the furrow intake rate. Because of the continous reduction in furrow intake rate with time, this would require a large number of cut-backs to the initial stream size. Were this possible, the advance front would just remain at the end of the furrow with no run-off (Withers and Vipound, 1974). In practice one or two cut-backs are sometimes made. The reason for this non-ideal practice and the resulting low efficiencies due to excessive run-off, is the difficulty and labor intensiveness of cut-back practice with the present systems. The possible higher efficiencies resulting from the reduction in water use do not pay for the increased cost of labor.

The problems described above all require the presence of somebody to intervene or correct. This means that often several irrigators must be employed for irrigating one plot or a field, which means additional incurred costs. Also the nature of the problems are such that they require human skill, which means the maximum possible efficiency and uniformity depends on the knowledge of the irrigator and the extent of effort he is willing to expend.

This state of affairs has therefore motivated research towards finding a system with fewer inherent problems requiring less human attention and intervention for its operation.

Automation of Surface Irrigation

In order to eliminate the problems described above, a number of researchers have tried to develop techniques independent of labor. Although operationally favourable, such systems may not be economically feasible or socially desirable. Research in the direction of automation however, is likely to assist in the development of less labor dependent or semi-automatic systems.

Complete automation of surface irrigation requires automation of all its componants. The task can be divided into two major parts (Langley, 1968):

(1) Irrigation Scheduling, based on the prediction as to the time and amount of water needed in the plant-root zone to meet optimum evapotranspiration, plus any leaching requirements;

(2) Automation of release from storage, its conveyance and distribution to individual farms and finally delivery from a farm source to the field.

It is automation of the last part of the second task, i.e. delivery to the field, with which we are concerned.

Several researchers have chosen this task. Humphreys (1971), investigated the feasibility of using spiles (furrow tubes) in conjunction with different automatic and semi-automatic canal gates operated by timer switches. Evans (1976) came up with a timer controlled check gate and outlined an automatic cut back layout using spiles as means of water delivery to furrows. The efficiency of water application, with and without runoff re-use, using an automatically controlled gated pipe delivery system was investigated by Fischback and Somerhalder (1971). They reported an improvement of efficiency from the conventional 31 to 52 percent, to up to 73 percent without the re-use system, and up to 96.8 percent with a re-use system. The greatest contributor to the high efficiency was the savings made in the run-off volume. Haise and Whitney

(1976), worked on a remotely controlled, hydraulically operated field gate in distribution ditches feeding furrows, border strips or basins and reported its successful operation. Dedrick and Erie (1978) worked on automating the existing jack-gates, used as turn-outs to level basins in the Southwestern U.S. They modified the pneumatically-operated cylinders of Payne, Duke and Haise (1974) and improved its operating mechanism. The modified automated jack-gates are operating successfully. It must be pointed out that "automatic" refers to a system capable of being remotely operated according to some predetermined schedule, whereas "semi-automatic" refers to systems that must either be reset between irrigations or where their complete remote operation is not possible.

Use of air bellows and air pillows for automation of tile turnouts in delivery to level basins was investigated by Erie and Dedrick (1978). Automatic drop gates, utilizing a clock activated release mechanism, have been developed in New Zealand and tested by Taylor, Ryde and Aldridge (1982) in border strip irrigation.

The researchers above share the same motives for their work. They all recognize labor as an unreliable, limiting and often troublesome tool upon whom the vital operation of irrigating the crop is dependent. Reducing this dependence is a major driving force behind their work. They all report drastic improvements in efficiency levels with introduction of their less labor dependent techniques.

It must not, however, be forgotten that all the above research has been carried out in the developed world. Problems of unavailability, cost and low water application efficiency associated with labor are only strictly true in the developed world. In the underdeveloped world, availability and cost of labor is seldom a problem. Inefficient water use is mainly because of lack of proper design, poor levelling, uncontrolled water release (wild flooding) and use of ancient irrigation techniques. Here, effort should be focused on improving irrigation techniques and incorporation of appropriate designs which optimize labor use.

Other Aspects of Irrigation

The technological advances in irrigation and agriculture have had their biggest impact in the developed countries. Grain yields per hectare in North America, for example, have increased by 108% from 1938 to 1960. The figure for such increase in productivity for Asia is only about 7%. Increase in productivity has kept well ahead of population growth in the developed world. In France, for example, wheat yields are increasing at an annual rate of 2.3% compared with their population growth rate of 1% per year (Brown, 1980). Developments in agricultural techniques and irrigation have had significant impacts upon the economics of the developed world. Here wealth created through industry, has been invested in research towards better productivity in agriculture. Being simultaneous with other developments in these countries and having developed from within, the outcome of research in agriculture and in particular irrigation techniques has brought the people wealth and prosperity.

The impact of new techniques and technologies has however been totally different in the developing countries. Here, population growth rates have far exceeded rates of yield increase. Implementation of modern agricultural techniques has been too rapid and without adequate study into its social impact and side effects. Incentives provided by economies of scale and short-term profitability, lack of sufficient research into the prevailing agrarian structure and it's delicate balance, and above all the socio-political spectacle of large scale irrigation, have given rise to rapid duplication of western-style irrigation projects in the developing countries. Because of their inappropriateness, these projects are often unprofitable, low in productivity and damaging to the social structure of the country in which they are implemented. For these countries successful, permanent irrigation projects require a slow and cautious implementation of appropriate scientific methods which aim at long-term increase in agricultural productivity and not short-term profitability for the firm undertaking the "development" project. In achieving this, emphasis must be placed upon the use and incorporation of locally available resources (human power, materials, technology) in the design and operation of the systems and not upon blind import of impressive high technologies which work elsewhere. The importance of the appropriateness of design and its associated technology can never be overemphasized.

Irrigated agriculture is also considered to be a powerful political tool used for settlement and resettlement of people especially in border (frontier) areas. Irrigation systems always give an impression of stability and permenancy to an area. For this reason sustained claim of occupied areas is often ensured through establishment of irrigated agriculture and irrigation projects. Irrigation projects always appear good in the eyes of the masses regardless of their appropriateness, effectiveness and opportunity cost. They are therefore considered as achievements by the ruling government, encouraging governments in their undertaking.

The discoveries of today, together with the experiences of the past call for proper and sound management in all agricultural practices especially in irrigation. While great civilizations flourished as a result of higher and more reliable production through irrigation, they vanished through improper management, misuse of land and water resources. Salinity and excessive soil erosion caused by over irrigation and bad cultural practices have, in the past, taken large areas of land out of production with extremely costly consequences. Being profitable on the short-run and despite their known devastating long term effect, similar misuses of land and water resources continue to prevail in many areas of the developing and the developed world. A permenant and well established agricultural system must overlook the temptation of short-term higher profitability and ensure long-term stability in production through application of sound management practices such as provisions for adequate drainage, and incorporation of appropriate measures for erosion control. Accompanied with sound management, the outcome of research in recent times and the vast knowledge available through past experience, allow for the adequate production and provision of nutritional needs of today and centuries to come.

CHAPTER 2

THE CANAL SIDE WEIR SYSTEM

Introduction

In the previous chapter, some of the most common methods of water delivery in surface irrigation were discussed. A relatively new technique for furrow irrigation which was not discussed, and around which this research is formed, is the use of small openings constructed in the side of the distribution canal (head ditch), known as canal side weirs or ditch notches (Fig. 2.1).

The openings or side weirs are formed by pushing a pre-cast mould into the wet concrete at the time the canal is being constructed. The weirs discharge into their designated furrows once the level of the water in the canal is built above their crest. The depth of water in the canal (and hence the water level above each weir crest) is controlled through the use of check boards or a gate, situated at the end of the canal reach. Discharge rate of the weirs primarily depends on the level of the head above the crest.

Fig. 2.1 The Canal Side Weir System

The system was first adopted because of it's clogging free nature (wide enough entrance for most surface debris) and it has been in use for about 20 years, in small areas of Arizona and Wyoming.

Operational Characteristics of the Canal Side Weir System

The mode of operations of the canal side weir system is very similar to that of spiles (Evans, 1976). The side weirs are installed in canals constructed on benches with a control at the end of each bench. The system utilizes the head-discharge relationship for the particular shape weir installed. The construction elevation of the weirs is very important. Because of its head-discharge relationship, non-uniformity in elevation would result in large variations in discharge for a certain irrigation setting. A similar problem, but to a lesser extent, exists with spiles. Humphreys (1971) realized this problem with spiles, particularly in the secondary flows, and suggested great care to ensure uniform elevation during construction, and the use of the smaller diameter tubes less sensitive to head fluctuation. The use of a small diameter tube may remedy the non-uniformity problems to some extent, but it makes the tube more prone to clogging. The side weir has little clogging problem, but being more sensitive to head fluctuations, it requires a special construction technique to ensure the uniformity in the design elevation.

An important characteristic of the side weir system is that it enables collective control over the discharge to individual furrows. Unlike most systems, one adjustment to the control (check board or gate at the end of the canal reach) is all that is necessary to set the discharge level to all the furrows or to alter the discharge. This makes cut-back an easy and feasible practice with little extra labor or cost. With run-off identified as one of the major contributors to irrigation inefficiency (Schneider, New, Musick, 1976), the easy cut-back practice promises high possible efficiencies with this system.

The capacity of discharge by the side weir covers a wide range. A typical side weir (trapezoidal, 50 mm wide crest, 3:7 side slope) can discharge 0.3 - 10 l/s (4.8 - 150 gpm) for a head above the crest of 20 - 130 mm (0.8 - 5.0 inches approx). Because of this capacity, it may require provisions for prevention of erosion at the point of discharge. This can be provided through covering the discharge point with the excess concrete which extrudes when the mould is pushed into the side of the canal, at the time of weir construction.

The sensitivity of the weir discharge to variations in head, together with its large range in discharge capacity, make the canal side weir system self adjusting when the inflow rate to the canal fluctuates. Any fluctuation in the supply to the canal results in an alteration of the water level in the canal, which automatically alters the discharge by all the weirs to a new level. The fluctuation is distributed evenly to all the furrows, at no expense to uniformity of delivery, and without any intervention by labor. Overall, the nature of the system is such that it does not require much human attention for its operation.

Study Objective

The canal side weir system, because of its favorable operational characteristics described above, had been credited (by its builders and farmers using it) with being less labor dependent and easier to operate than conventional systems. It was claimed that this system, while being a viable delivery technique, can favorably compete in performance criteria (application efficiency and uniformity) with conventional systems. Investigation of these claims was the initial motive for this research work. The objectives of this study are:

1. To establish a scientifically based design and operation practice for the canal side weir system through:

    a) Investigating the hydraulics of the system and deriving appropriate design charts.

    b) Investigating the system sensitivity to prevailing variables.

2. To evaluate the performance of field systems already in operation through in situ testing.

Hydraulic Characteristics of the Canal Side Weir System

In studying the hydraulics of the canal side weir system, it is best to separate the system into its components i.e. the canal and the side weir. The hydraulics of each component is greatly affected and made more complex by the presence and interaction of the other component.

Manning's Roughness Coefficient for the Canal

The outflow of water by the side weirs along the canal constitutes a decreasing spatially varied flow regime in the canal. The hydraulics of a decreasing spatially varied flow are different from uniform flow. The effective roughness of the channel described by Manning's roughness coefficient, n, is different to that for uniform flow (Sweeten, Garton, Mink, 1969). In fact for each different flow regime such as decreasing spatially varied flow, gradually varied flow and uniform flow the Mannings roughness coefficient, n, is different. The actual roughness is important in that it plays a big role in determining the water surface profile in the canal and thereby affects the head upon consecutive weirs. A representative value for Manning's roughness coefficient in the canal is necessary before one can accurately determine the water surface profile. Sweeter, Garton and Mink (1969), defined two different roughness coefficients for spatially varied flow with siphon tubes. These were an effective roughness coefficient n_e and an average or mean roughness coefficient n.

Effective roughness coeffcient, ne. Through assuming negligible energy loss per unit weight due to lateral outflow by the siphons, Sweeten et al (1969) attributed all the energy loss per unit weight between the extremities of an irrigation bay to friction. Using the Bermouli's equation they defined the effective friction slope as:

$$S_{fe} = 1/L \left(\frac{V_1^2 - V_2^2}{2g} + Z_1 - Z_2 + Y_1 - Y_2 \right) \qquad (2.1)$$

Using the effective slope in the Manning equation;

$$n_e = 1.486 \ . \ R^{2/3} \ . \ S_{fe}^{1/2} \qquad (2.2)$$

Where V is the average velocity of flow, Y is depth of flow, Z is distance between the channel bottom and an arbitrary datum and R is the hydraulic radius. Subscripts 1 and 2 refer to upstream and downstream extremeties respectively.

Average roughness coefficient, n. This is a roughness value which results in the same calculated water surface profile as the one present. Sweeten et al (1969) used the following procedure to arrive at an average roughness coefficient for their trials. Starting from the downstream end with a known depth they:

1. Selected an aritrary trial n as a representative average value for a short reach ΔX.
2. Used Manning's and Bernouli's equation to calculate the depth at ΔX upstream.
3. Calculated the water surface profile in the bay by iterating ΔX at a time.
4. Compared calculated profile with profile actually measured. If markedly different they started from step 1

with a new trial. This procedure was repeated until satisfactory match between the calculated and measured water surface profiles was obtained. The Manning's roughness coefficient resulting in the satisfactory match was then designated as the average roughness coefficient, n.

Sweeter et al (1969) arrived at an experimental relationship between n and n_e which was

$$n = 1.728\ n_e \qquad (2.3)$$

A similar work on evaluating roughness coefficient for spatially varied flow was conducted by Sweeten and Garton (1970). They used sharp crested side weirs for lateral outflow. They defined n and n_e in a similar manner. The experimental relationship between the two was arrived at as;

$$n = 1.62\ n_e \qquad (2.4)$$

The mean values of n_e and n were 0.0096 and 0.0156 respectively.

In addition to calculating roughness values, Sweeten and Garton (1970) carried out tests on the effect of some of the prevailing variables in a distribution bay upon the discharge uniformity of consecutive weirs. They concentrated on the effect of weir spacing, weir crest length and channel inflow rates. They reported better uniformity with reduced channel inflow rate, shortened weir crest length and increased side weir spacing.

The difference in equations 2.3 and 2.4 suggests that the actual roughness coefficient for spatially varied flow is also dependent on the outflow mechanism. The outflow mechanism used in this study was broad crested side weirs whichare different to the mechanisms used in the studies described above. No work on evaluation of roughness coefficients for spatially varied flow with broad crested side weirs could be found. A subjective judgement regarding the representative value of roughness coefficient for this work was therefore necessary. The representative average roughness coefficient, n, for this study was selected as 0.015.

Solution to Water Surface Profile

In order to determine the discharge level by each side weir for a certain irrigation setting, the head above the crest of each weir must be known. This depends on the nature of the water surface profile. For a constant weir crest elevation, the head above the crest of each weir will only be constant if the water surface profile is horizontal i.e. if it has a constant zero slope throughout the distribution canal. The actual slope of the water surface and therefore its profile, depends on variables such as canal cross section area, roughness, bed slope, inflow and outflow rates, extent of lateral outflow, spacing between consecutive weirs, and the quality of the irrigation water (sediment load, temperature). Depending on the nature and interaction of these variables, a wide range of positive and negative water surface slopes are possible. It is therefore necessaryto accurately calculate the water surface profile before the actual head on consecutive weirs is known. The knowledge of the actual water surface profile, together with experimentally determined head-discharge relationships for consecutive side weirs, are essential in the design of a distribution bay (canal) capable of uniform water delivery along its reach.

Various researchers have engaged themselves with calculating the water surface profile for this type of flow regime (decreasing spatially varied flow), and have all adopted the energy approach. The unanimous assumption is that "there is no loss in energy per unit weight resulting from the lateral outflow". This is a logical assumption since there is no turbulence due to the lateral outflow. In with this assumption, the significant energy components to be considered in the energy equation are; the depth of flow, the velocity head, the friction loss and the bed elevation. All other forms of energy may be considered negligible. These forms of energy are interchaneged along the canal, resulting in an alteration of their value. It is the change in the value of the depth of flow along the canal that we are after. To calculate this through an energy approach, variations of the other significant components of energy with distance along the canal must be known. These variations depend on many hydraulic parameters and are complex to express mathematically.

In order to arrive at a mathematical closed form solution for the surface profile, many researchers have simplified the variations of the other energy components involved. Sweeten, Garton and Mink (1969) assumed a horizontal channel and therefore omitted the bed elevation variation. For their mathematical analysis, they had to further assume a horizontal water surface profile and uniform lateral outflow before they could arrive at a simplified closed form equation for the actual water surface profile. Ramamurthy et al (1978) neglected the energy loss due to friction in their energy equation, and used a canal with zero bed slope. They then theoretically investigated the extent of the necessary rise in the channel bed or the necessary contraction in the channel side, to keep a horizontal surface profile. The side contraction or bed elevation rise was such that it would keep the canal velocity constant compensating for the lateral outflow. They experimentally verified their theoretical results which in turn verified their assumption about the friction loss component.

In order to calculate the water surface profile for any condition, an approach free of simplifying assumptions is necessary. All significant energy components must therefore be considered and accounted for in the energy equation. This approach would contain a large number of variables, resulting in an equation to be solved numerically and iteratively, rather than a closed form solution. The following are the theoretical considerations necessary to arrive at such an equation.

Consider points 1 and 2 of Fig. 2.2. From Bernouli's equation

$$\frac{V_1^2}{2g} + Y_1 + Z_1 = \frac{V_2^2}{2g} + Y_2 + Z_2 + S_f \Delta X \qquad (2.5)$$

Where V is the average velocity of flow, Y is the depth of flow, Z is the bed elevation, S_f is the friction slope and Δx is a small distance between points 1 and 2. The change in the water depth is;

$$Y_2 - Y_1 = \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) + (Z_1 - Z_2) - S_f \Delta X \qquad (2.6)$$

i.e.

$$\Delta Y = -\Delta H_v - \Delta Z - S_f \Delta X \qquad (2.7)$$

or

$$\Delta Y + \Delta H_v = -\Delta Z - S_f \Delta X \qquad (2.8)$$

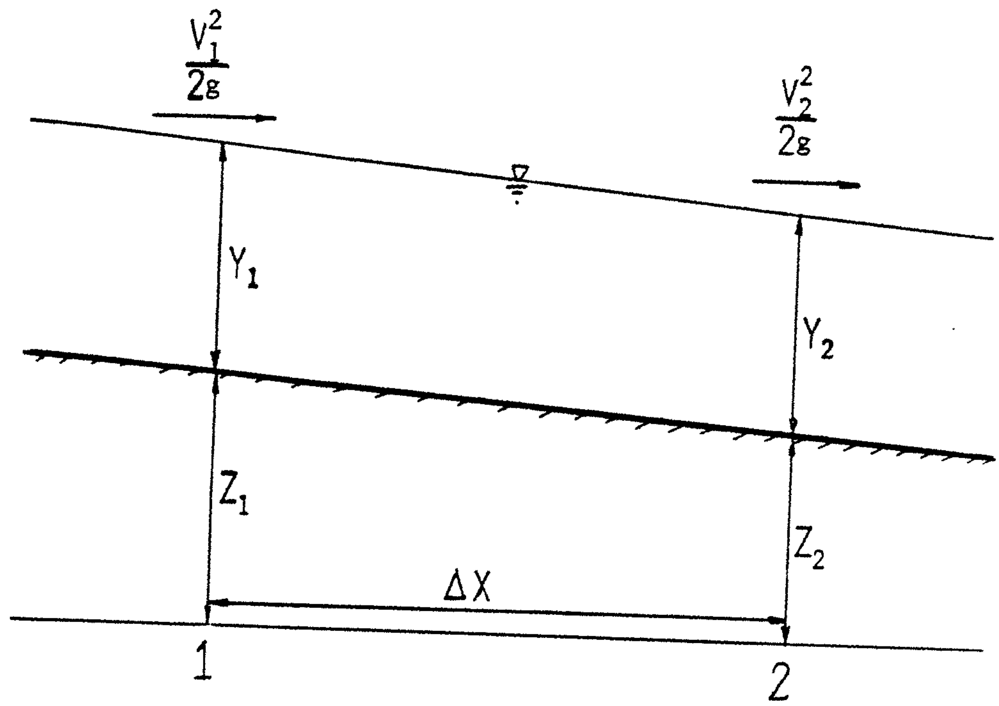

Fig. 2.2 The energy components in the canal.

Where ΔH_v is the change in the velocity head in the canal. But;

$$\Delta Y + \Delta H_v = \Delta E \qquad (2.9)$$

Where E is the specific energy in the canal at any point. Also;

$$\Delta Z = -S_o \, \Delta X \qquad (2.10)$$

Where S_o is the bed slope of the channel. Substituting equation (2.9) and (2.10) into (2.8) we get;

$$\Delta E = S_o \Delta X - S_f \, \Delta X \qquad (2.11)$$

or

$$\frac{\Delta E}{\Delta X} = S_o - S_f \qquad (2.12)$$

For an infinitely small ΔX, we can write the equation in the differential form.

$$\frac{dE}{dX} = S_o - S_f \qquad (2.13)$$

This is a general equation valid for any channel where the assumption of no energy loss due to lateral flow, is valid. This equation may be written in terms of the water surface slope. Using the specific energy principle;

$$E = Y + \frac{V^2}{2g} \qquad (2.14)$$

Incorporating the continuity principle, we get

$$E = Y + \frac{Q_d^{\,2}}{2gA^2} \qquad (2.15)$$

Where Q_d is the canal (ditch) flow rate at any point, and A is the corresponding cross sectional area of flow. Differentiating with respect to X;

$$\frac{dE}{dX} = \frac{dY}{dX} - \frac{Q_d^{\,2}}{2g}\left(\frac{-2}{A^3}\right)\frac{dA}{dx} + \frac{1}{2gA^2}(2Q_d)\frac{dQ_d}{dX} \qquad (2.16)$$

Substituting B dY/dX, for dA/dX and simplifying;

$$\frac{dE}{dX} = \frac{dY}{dx} - \frac{Q_d{}^2B}{gA^3}\frac{dY}{dX} + \frac{Q_d}{gA^2}\frac{dQ_d}{dX} \tag{2.17}$$

Where B is the top width of the flow cross section. Substituting for dE/dX from equation (2.13);

$$S_o - S_f = \frac{dY}{dX}\left(1 - \frac{Q_d{}^2B}{gA^3}\right) + \frac{Q_d}{gA^2}\frac{dQ_d}{dX} \tag{2.18}$$

Therefore

$$\frac{dY}{dX} = \frac{S_o - S_f - \dfrac{Q_d}{gA^2}\dfrac{dQ_d}{dX}}{1 - \dfrac{Q_d{}^2B}{gA^3}} \tag{2.19}$$

But

$$\frac{Q_d{}^2B}{gA^3} = Fr^2 \tag{2.20}$$

Where Fr is the Froude No. in the canal. Substituting equation (2.20) into (2.19);

$$\frac{dY}{dX} = \frac{S_o - S_f - \dfrac{Q_d}{gA^2}\dfrac{dQ_d}{dX}}{1 - Fr^2} \tag{2.21}$$

Equation (2.21) is similar to the universal equation for gradually varied flow which is;

$$\frac{dY}{dX} = \frac{S_o - S_f}{1 - Fr^2} \tag{2.22}$$

Equation (2.21) may be found (without derivation) in a text by Henderson (1966). If there is no lateral outflow, i.e. if we have constant flow rate in the main canal, then the term dQ_d/dX in equation (2.21) is zero. This equation will therefore be identical to equation (2.22) which means that equation (2.22) is a special case of flow described by (2.21). One can therefore use equation (2.21) whether or not there is lateral out-flow. This is very useful when trying to solve for the water surface profile in a canal with side weirs. The actual condition here is such that there is lateral out-flow where we have a side weir, and no lateral out-flow in between consecutive weirs. Equation (2.21) is the equation of the slope of the water surface at any point along the canal. To solve for the profile slope, one must calculate the friction slope using the Manning equation (with a correct and representative value for roughness coefficient n), evaluate the Froude number and know the rate of spatial variation of flow, dQ_d/dX, at the point of interest. The rate of spatial variation of flow is the same as the rate of lateral flow by the weirs. Since Q decreases with distance downstream, dQ_d/dX is negative and therefore;

$$dQ_d/dX = -Q \tag{2.23}$$

Where Q is the rate of lateral outflow by the weir which depends on the head on the weir and the head-discharge relationship for the weir. The change in depth for a small distance ΔX along the canal, can then be calculated by multiplying the calculated profile slope, dy/dx, by ΔX.

As

$$\Delta X \longrightarrow 0 \quad , \quad \Delta Y / \Delta X \longrightarrow dY/dX$$

therefore;

$$\Delta Y = dY/dX \cdot \Delta X \tag{2.24}$$

Appendix A deals with the procedure used for numerically solving the water surface profile. The computer program written for this numerical solution is provided in Appendix B. An example of the output of this program is also included in this Appendix.

The Hydraulics of the Side Weir

Introduction

There has been considerable work by various researchers on the characteristics of lateral flow through side weirs. DeMarchi (1934), Collinge (1957), Frazer (1957), El Khashab and Smith (1976), and Ramamurthy and Carballada (1980) have all investigated the flow characteristics of side weirs and have come up with discharge equations. Researchers in the Eastern world namely Aoki (1970), Yoshioka (1979) in Japan and Dombrovskii (1977) in the Soviet Union have conducted similar research. Unfortunately because their work has not been translated, their contributions could not be accessed. All the work above has been concerned with side weirs of the overflow type i.e. sharp crested side weirs. No reference on the discharge characteristics of broad crested side weirs could be found. The flow characteristics of these different types are significantly different.

With sharp crested side weirs, streamlines follow a curved flow path which means presence of centrifugal forces and subsequent non-linear pressure distribution. This curvilinear flow complicates the analysis. In addition, the issuing water jet from the side weir has two perpendicular components. One is the velocity due to the discharge caused by the hydraulic head above the weir crest, and the other is the canal velocity, as there is no side weir wall. These two components make the jet emerge making an oblique angle with the weir plane. Ramamurthy and Carballada (1980) discuss this extensively.

Broad crested side weirs on the other hand possess none of the above. Streamline path through the weir is linear introducing no complications in the energy equation. The canal velocity has no component in the direction of

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Where H is the available energy, Y_c is the critical depth and V_c is the critical velocity. Incorporating continuity,

$$H = Y_c + \frac{Q_t^2}{2gA_c^2} \qquad (2.26)$$

Q_t is the ideal (theoretical) discharge through the weir and A_c is the cross sectional area at the critical section. The definition of critical flow is that Froude Number is unity, therefore,

$$\frac{V_c}{\sqrt{g\ A_c/b_c}} = F_r = 1 \qquad (2.27)$$

Where b_c is the top width of the area Ac. Squaring both sides and incorporating continuity

$$\frac{Q_t^2\ b_c}{gA_c^3} = 1 \qquad (2.28)$$

Or

$$Q_t = \sqrt{g\ \frac{A_c^3}{b_c}} \qquad (2.29)$$

We can solve for the ideal discharge Q_t given H and the geometry of the side weirs, through a trial and error procedure using equations (2.26) and (2.29). A quick converging procedure, suitable for programmable handheld calculators is provided in Appendix C.

Q_t is the ideal discharge. The actual conditions for discharge however are far from ideal. There is discontinuity for flow at the weir entrance, resulting in convergence of stream lines, turbulence and energy loss. Furthermore the above analysis is only one dimensional whereas the actual flow situation is three dimentional. In addition the above analysis disregards energy losses between the weir control section and the canal (friction and eddies). The actual discharge is therefore different from the ideal discharge. The discrepancy between the two is corrected by using a coefficient of discharge, Cd.

$$Q = C_d \cdot Q_t \tag{2.30}$$

Where Q is the actual discharge that results for the given head H.

The Influence of Canal Velocity

Because of the spatially decreasing nature of the flow regime in the distribution canal, the canal velocity (average flow velocity in the canal) is also spatially decreasing for a constant flow depth. Since this velocity is blocked by the side weir wall and has no component in the weir flow direction, one might expect the variations in canal velocity to have no bearing on the magnitude of weir discharge. This is not true and weir discharge for a given head is a function of the canal velocity.

For flow to occur through the side weir, flow lines in the canal must change their direction by 90 degrees. There is therefore a discontinunity in flow path resulting in a separation of the streamlines. The extent of this separation is directly proportional to the abruptness of the discontinuity, which in this case is the sharpness of the entrance to the weir. Furthermore the extent of streamline separation varies directly with the original velocity of the flow lines. It was observed that because of this flow separation, two vortices form at the entrance to the side weir (Fig. 2.4).

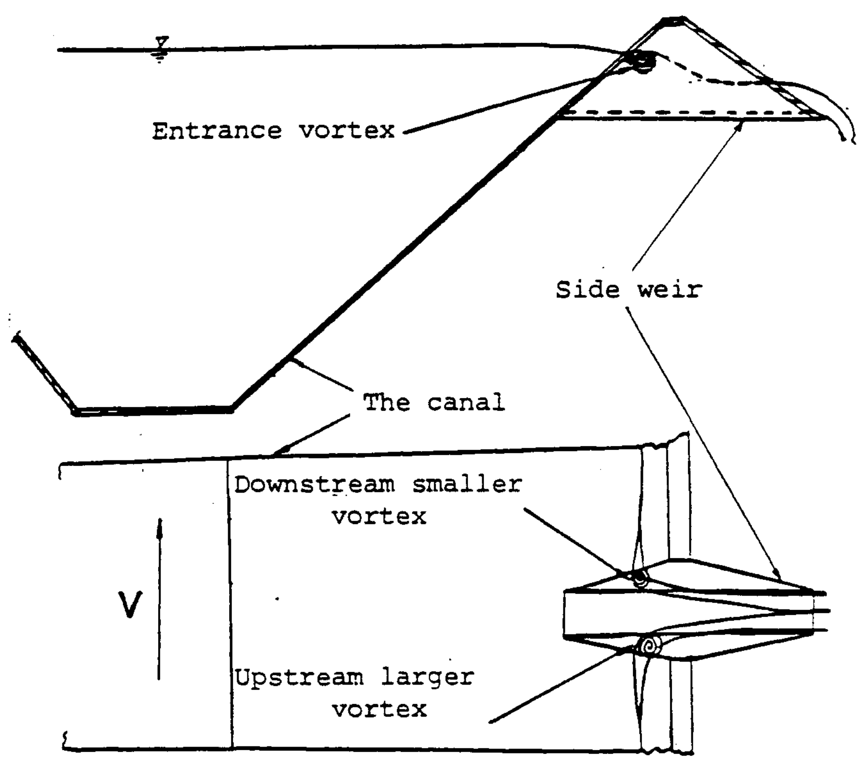


Fig. 2.4 Side Weir Entrance Vortices

The vortex at the upstream edge was observed to be larger than the one at the downstream edge.

Because these vortices form at the entrance, they alter the entrance conditions of the side weir and further remove it from being ideal. The value of the coefficient of discharge, Cd (equation 2.30), is therefore affected. But since the size of these vortices is a function of the canal velocity, and the magnitude of the canal velocity decreases down the canal, entrance conditions for each weir are affected differently. The value of Cd for consecutive weirs is therefore different which means non uniformity in weir discharge for a constant head.

In order to correct for this, the effect of the canal velocity upon the coefficient of discharge for side weirs with different entrance conditions (sharp or curved), must be quantified.

Other Factors Influencing Discharge by Individual Side Weirs

In the previous sections of this chapter, the surface profile, velocity variations in the canal, magnitude of the head on the weir, weir geometry and type were identified as the major variables influencing the discharge level by individual side weirs. Another factor very important in discharge uniformity is the uniformity of construction of side weirs.

Ideally we would like identical weirs constructed at a uniform elevation. In practice there are variations in geometry and elevation with individual weirs, which will result in non-uniformity in discharge for a given head. Every effort (within reason) should be made to minimize non-uniformity in construction.

Irrigation waters often carry some sediment load depending on their source. With the decreasing flow velocity along the distribution canal, a deposition layer, increasing in thickness with distance downstream, is often characteristic. This layer, alters the design slope of the distribution canal. This will affect the water surface profile and subsequently the discharge by individual weirs will be influenced. Periodic maintenance and clearing of the ditch will prevent this problem.

Having identified the factors affecting discharge by individual weirs, tests were designed and conducted to quantitatively evaluate the effect of these factors.

CHAPTER 3

LABORATORY EXPERIMENTS AND FIELD EVALUATION

Introduction

In the previous chapters important factors in irrigation as a whole, and in particular those that are most relevant to the canal side weir system were discussed. The variables influencing the discharge uniformity of the side weir system were identified as weir entrance condition, magnitude of the canal velocity, weir geometry, construction uniformity and the canal bed slope. It was emphasized that a quantitative knowledge of these variables is necessary when designing for discharge uniformity. For this purpose laboratory model studies and field evaluation studies were undertaken.

For in-situ testing of the system components i.e. canal conditions and construction uniformity of side weirs, and for evaluation of performance criteria such as application efficiency and uniformity, and the degree of labor involvement in the operation of the system, it was decided to carry out a field evaluation study with the procedure suggested by Merriam (1978), as a guide.

Laboratory Model Study

In order to determine the effect of the variations in the canal velocity upon weir discharge, a laboratory model study was thought to be necessary. In addition the model could serve to establish the head-discharge relationships for different weirs.

Using the facilities at the U.S. Water Conservation Laboratory in Phoenix, Arizona, a model of the system was designed, constructed and installed in the hydraulic laboratory (Fig. 3.1). The model consisted of:

- a half scale trapezoidal channel (0.23 m bottom width, 1:1 side slope, 3.66 m long, with 0.38 m maximum depth) fabricated from galvanized sheet metal. A single full scale side weir discharged water from this channel. An overflow gate at the end of the channel served as the depth control device.
- three trapezoidal side weirs of 3" (0.076m) bottom width, 3:7 slope, capable of 5" (0.13m) maximum head on their crest. The weirs differed in their bed slope (two horizontal, one with 1:10 slope) and entrance conditions (two sharp edged and one with a 45 approach angle at it's entrance). The angle entranced weir had zero bed slope. The side weirs were designed to be fitted to the side of the channel, through a slide adaptor.
- a stilling basin at the top of the channel for non-turbulent water delivery.

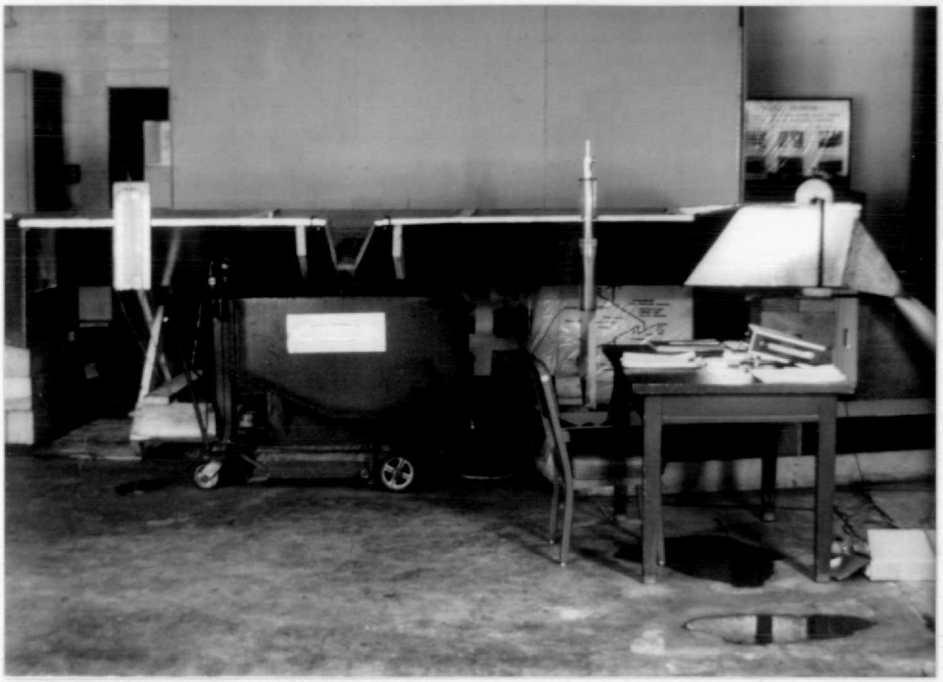


Fig. 3.1 The Laboratory model

- a regulating gate at the end of the channel for control of flow depth.
- a pipe network of 1.5", 4", 10" diameter pipes connected to a constant head tank for steady water delivery to the stilling basin. The different size pipes enabled a wide range in delivery rate to the main channel allowing for velocities between 0.0 m/s and 0.3 m/s.

The instruments used in this model were:

- propeller flow meters for measurement of flow rates in the 1.5" and 4" delivery pipes,
- venturi meter for flow measurements in the 10" delivery pipe

pipe

- weighing tank and stop watch for measurement of delivery rate by the side weir,
- stilling well and point gauge for measurement of the head on the side weir.

Overall the model formed an adequate and realistic apparatus for determining the influence of canal velocity upon side weir discharg, and establishing head-discharge relationships at different channel velocities.

Laboratory Testing Procedure

By sealing off the downstream end of the channel, the inflow to the channel was made to entirely discharge through the single side-weir. With no flow in the channel downstream of the side weir, the resulting water velocity passing the side weir was zero. By altering the delivery rate to the channel and measuring the head above the crest, a head discharge relationship for that weir at zero canal velocity was established. In addition the accuracy of the flow meters, could be checked against the weighing tank (considered most accurate). Using continuity and the weir head discharge relationship obtained with zero velocity, the necessary delivery rates by the supply network, which would result in the required channel velocities (0.1, 0.2, 0.3 m/s) passing the weir, were calculated. The end gate was then adjusted to allow through flow while maintaining a desired head on the weir. The discharge by the weir at that head, with that canal velocity, was then measured with the weighing tank. In this manner, the head-discharge relationships for the three side weirs at four different canal flow velocities (0.0, 0.1, 0.2, 0.3 m/s) were established. Sufficient information was therefore collected to enable a through analysis of the effect of canal velocity upon discharge by the different side weirs.

Field Evaluation

Field evaluation was thought to be necessary for several reasons. The head-discharge relationships obtained in the laboratory needed to be compared with what prevailed under field conditions. The construction variability of the side weirs installed had to be evaluated, and the feasibility for improvement investigated. The actual uniformity of discharge, efficiency of application and overall performance of the system had to be determined for comparison with other furrow irrigation systems.

For these purposes, sites in Safford, Arizona, where side wiers are in common use, were selected.

Description of the Field Location

A suitable location for field evaluation studies, Safford, is located in the Southeastern corner of Arizona. The climate here is typical of most of Arizona with little rainfall (300-450 mm average annual rainfall) and total irrigated agriculture. Here, a variety of crops, mainly cotton, alfalfa and grasses, are grown on soils ranging from light sands to heavy clays. Use of surface irrigation with the side weir delivery system is widespread. The water source in the area is mainly surface water diverted from the Gila river. In addition many farms have access to ground water.

All side weir systems in the area are installed by a single contractor whose work is supervised by the local Soil Conservation Servic (SCS) office. The SCS sets standards on the construction elevation of the side weirs. For any one distribution bay (i.e. the canal reach controlled with one gate), the construction elevation of 80% of all the installed weirs must be within plus or minus 3.0 mm (0.01') of the design elevation. Construction elevation of no weir is to deviate more than plus or minus 6.00 mm (0.02') from the design elevation. This standard which must be satisfied before farms can obtain financial assistance for installation of the system. Sites to be tested were selected through consultation with the SCS office.

Evaluation Procedure

Two sites were selected for testing. One was cotton grown on relatively heavy clay and other was grass on sandy clay. The system serving the grass field was only tested for construction and discharge uniformity of the weirs,

Fig. 3.2 Small Portable Flume for Weir Discharge Measurement.

whereas the system serving the cotton field was fully evaluated. All tests were carried out on one irrigation bay in each of these sites.

By measuring the dimensions of individual weirs, they were tested for uniformity in construction geometry. Surveying equipment was used to determine variability in construction elevation of the weirs in one irrigation bay.

For performance evaluation, the discharge level by all the weirs in one bay, at initial and cut back stream sizes were measured (Fig. 3.2) using small portable flumes (Bos et al, 1984). For the cotton field, the advance rate in each furrow being served by the metered side weirs, was carefully monitored. A large portable flume (Fig. 3.3) was then used to measure the run-off and establish the run-off hydrograph.

Fig. 3.3 Large Portable Flume for Determination of the Run-Off Hydrograph.

Using Oakfield probes, the adequacy of moisture replacement along the furrows was checked three days after the irrigation, when the field was believed to be at field capacity.

The information collected in this study, allowed for the calculation of the variables considered to be performance indicators (efficiency and uniformity). A tentative assessment of the general performance of the system was made.

CHAPTER 4

RESULTS AND DISCUSSION

Laboratory Results

Variations in the coefficient of discharge

Using the procedure outlined in Appendix C, the ideal discharge corresponding to different heads on each weir, for which the actual discharge had been measured, was calculated. Using equation (2.30) the coefficient of discharge Cd, for the weir at that head and that canal velocity was calculated. Figures 4.1 and 4.2 show plots of coefficient of discharge Cd, versus the head on the weir at different velocities for the two sharp entranced weirs tested. From these plots it is apparent that in general, with a sharp entrance to the weir, the discharge by the weir decreases as the channel velocity increases. Careful examination reveals that the nature of the decrease is somewhat a function of the weir and bed slope.

The decrease in discharge with increase in velocity comes as no surpirse. As discussed in Chapter 2, increase in through velocity results in increased turbulence at the entrance to these weirs thereby reducing the actual discharge for a given head and hence lowering the coefficient of discharge. Figures 4.1 and 4.3 quantify the variations of the coefficient of discharge Cd, with canal velocity variations for the sharp entranced weirs tested.

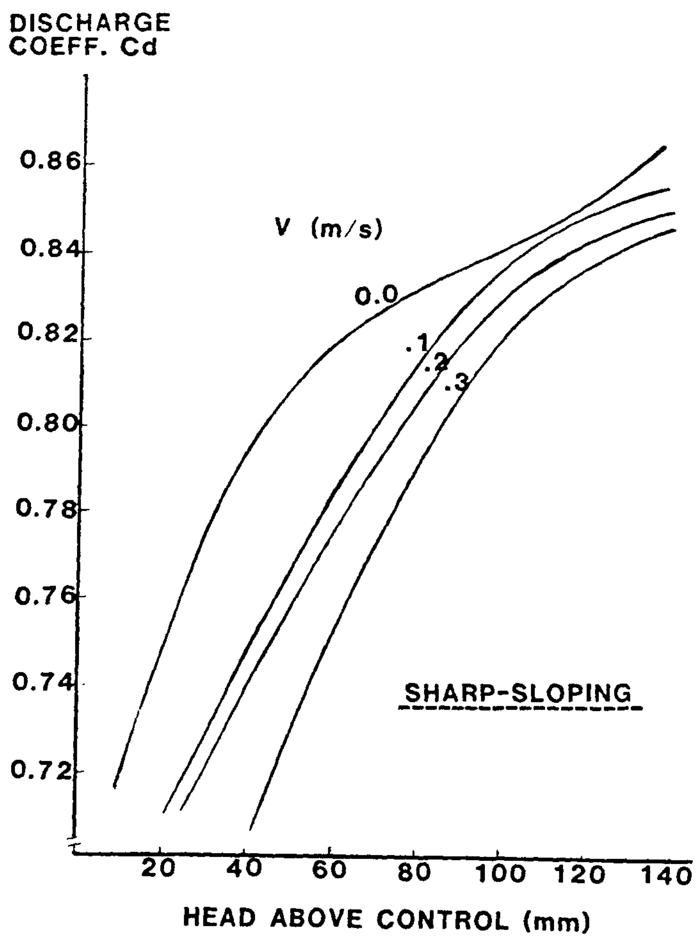


Fig. 4.1 Discharge Coefficient, Cd, Vs Head above the Control Section, H, at Different Velocities for a Sharp Entranced Weir with a Sloping Bed.

In calculating the ideal discharge for the weir with the sloping bed, the location of the control section had to be assumed (see chapter 2). After careful observation, the control section was assumed to occur directly below the location where the water surface in the canal touches the side of the canal (Fig. 4.2).

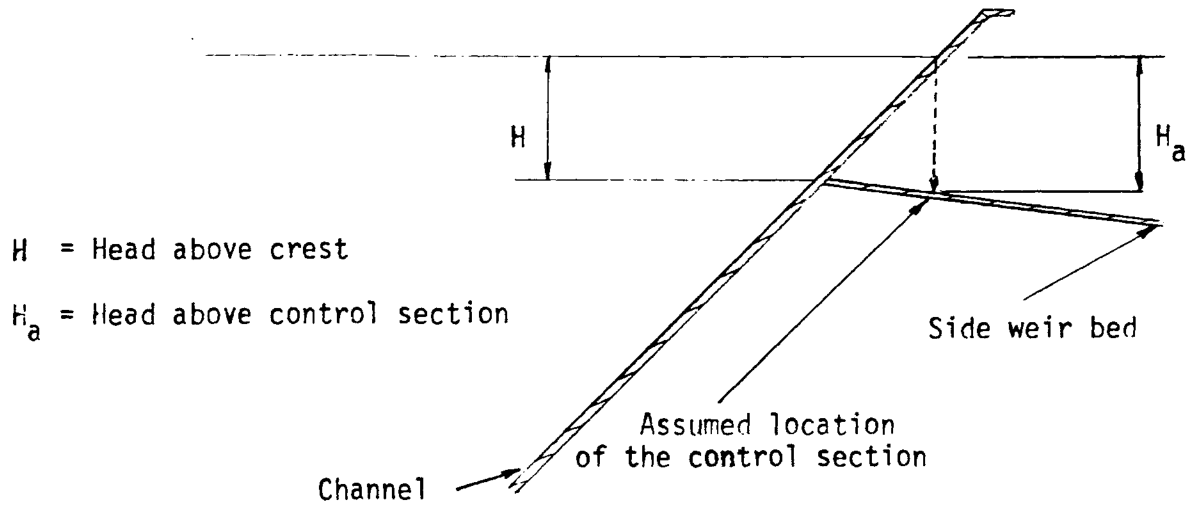


Fig. 4.2. Schematic Representation Showing the Assumed Location of the Control Section for the Side Weir with the Sloping Bed.

This assumption is good in that it makes the location of the control section a function of the head on the weir (which is known to be true), but inadequate in that it does not consider the effect that channel velocity might have on the location of critical depth occurence. The slight difference in the shape of Cd - H curves of figure 4.1 (especially for zero velocity), might be due to to the inaccuracy introduced by this assumption.

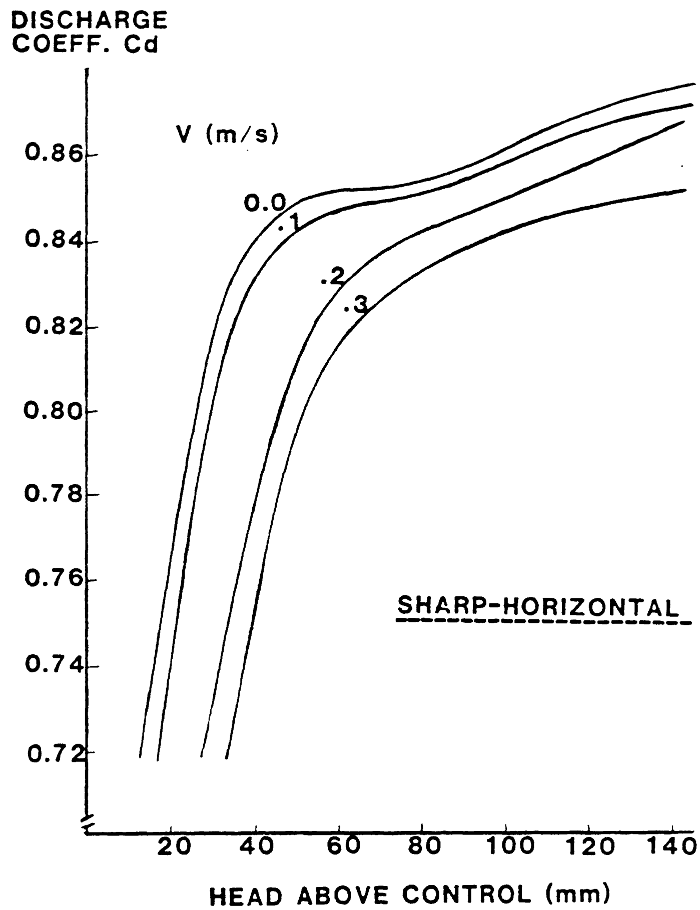
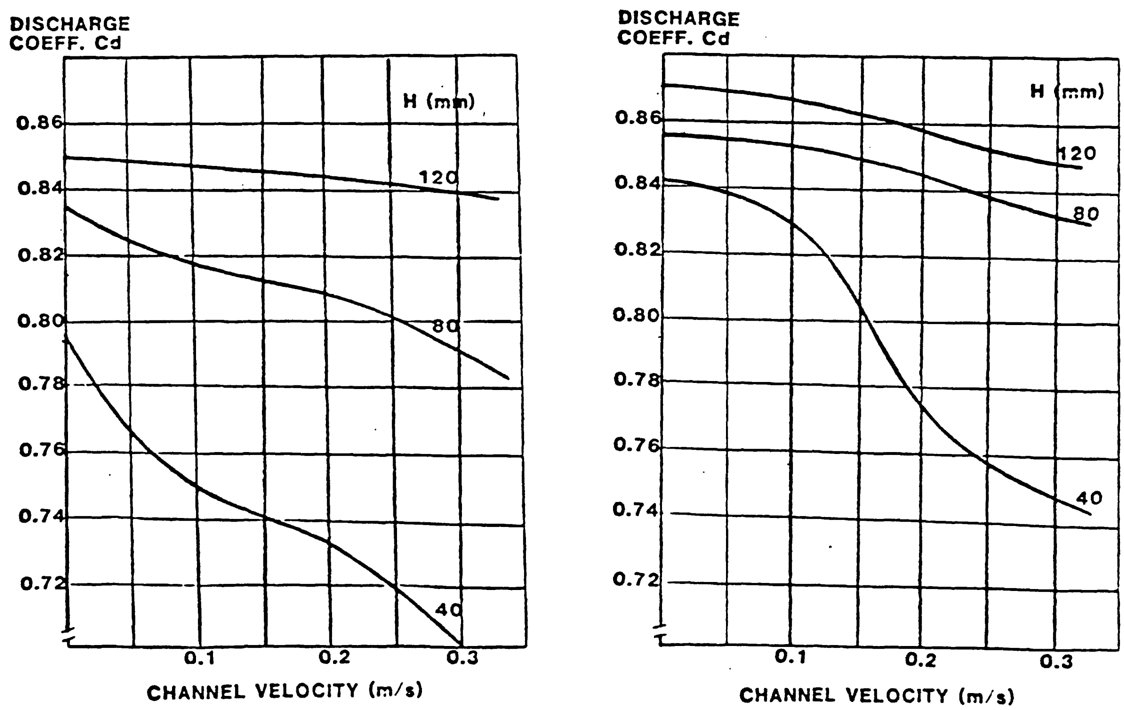


Fig. 4.3. Discharge Coefficient, Cd, Vs Head above the Control Section, H, at Different Velocities for a Sharp Entranced Weir with a Horizontal Bed.

Figs. 4.1 and 4.2 reveal that with sharp entranced weirs installed, possible variations in velocity from the upstream end of the canal (high velocity) to the downstream end, result in decrease in discharge between 3% and 15% of the discharge with zero channel velocity, depending on the operating head. For normal irrigation settings (head on the weir = 60 - 80 mm), the decrease in discharge is about 6%. The alteration in discharge is brought about solely as a result of variations in velocity, with all other factors kept constant. In figure 4.4 discharge coefficient for these sharp entranced weirs is plotted against the channel velocity at different heads. For both weirs, channel velocity of about 0.15 m/s seems to mark a significant value. A sharper incremental decrease in the coefficient of discharge results due to velocity increases beyond 0.15 m/s. This is illustrated by the increased distance between 0.1 and 0.2 m/s curves of Fig 4.3 . One speculation is that a minimum channel velocity of 0.15 m/s is necessary before the formation of the vortices is complete.



(a) sharp, sloping (b) sharp, horizontal

Fig. 4.4 Discharge Coefficient, Cd, Vs Channel Velocity at Different Heads for Different Type Weirs.

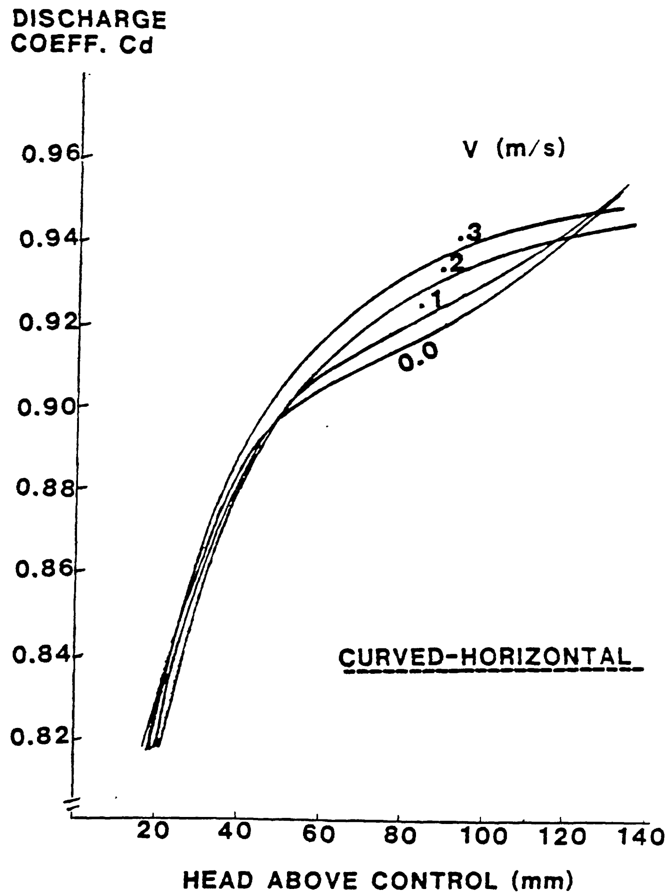


Fig. 4.5 Discharge Coefficient, Cd, Vs Head on the Weir, H, at Different Channel Velocities for a Streamlined Entrance Weir with a Horizontal Bed.

Figure 4.5 shows Cd - H curves at different velocities obtained for the weir with a streamlined (45 entrance angle) entrance and horizontal bed. Comparison with figure 4.2 reveals that in general, Cd for the zero velocity curve had increased by about 10% compared with the sharp entranced weirs. Streamlining the entrance to the weir has therefore greatly improved the weir entrance conditions.

Streamlining the entrance, was observed to inhibit the formation of entrance vortices (Fig. 2.3), which means a smaller entrance contraction compared with sharp entranced weirs. In addition eddy currents induced by channel velocity are smaller in extent and therefore cause a smaller energy loss. Nevertheless there are still eddies and turbulence which increase with velocity and cause energy loss. One would therefore expect a decrease in coefficient

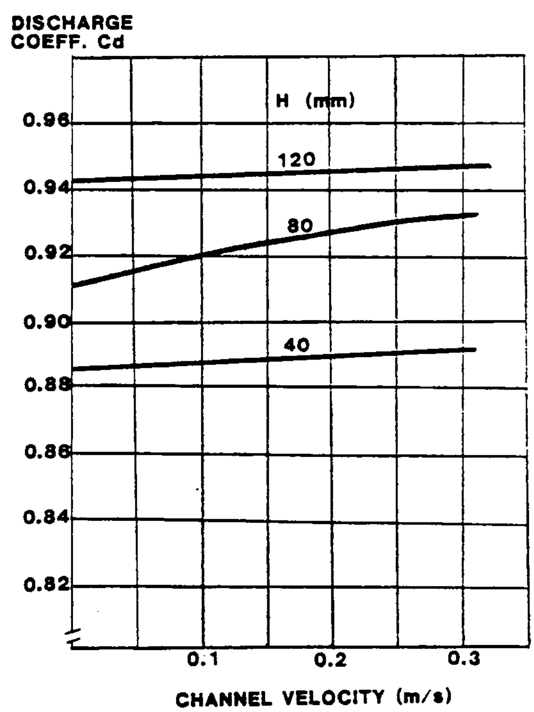


Fig. 4.6 Discharge Coefficient, Cd, Vs Channel Velocity at Different Heads for a Curved Entranced Weir with a Horizontal Bed.

of discharge with increase in channel velocity. A decrease in coefficient of discharge however, is not the case here. Fig. 4.5 shows that over most of the operating head range (H = 40 - 120 mm), the discharge coefficient for a given head actually increases. Figure 4.6 shows the nature of this increase at different heads on the weir.

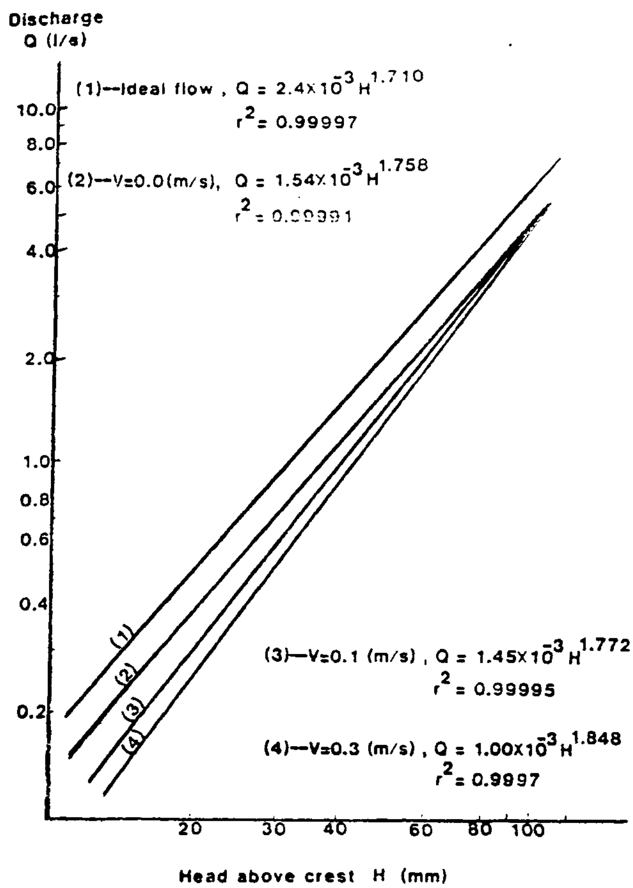


Fig. 4.7 Head-Discharge Relationships at Different Channel Velocities for the Sharp Entranced Weir with the Sloping Bed.

The reason for the increase in Cd with velocity is that, when the entrance to the weir is curved, the curvature results in the diversion of some of the velocity head in the canal towards the direction of flow through the weir. The energy available for discharge by the weir is now the static head above the control section, plus the component that the velocity head in the canal has in the direction of weir flow. Increase in channel velocity also increases energy loss through increased turbulence at the entrance. If the

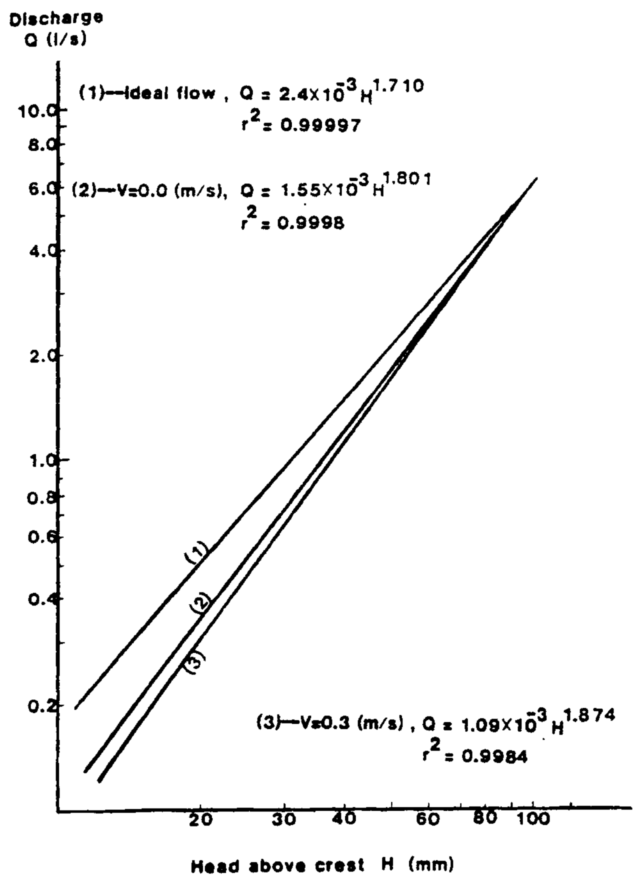


Fig. 4.8 Head-Discharge Relationships at Different Channel Velocities for the Sharp Entranced Weir with the Horizontal Bed.

energy increase is greater than the energy loss, then an increase in Cd results. This seems to be strictly the case when head above the weir is between 40 - 120 mm. The increase is, at most, (H = 80 mm) about 2%. This variation is insignificant. The discharge by the side weir with a streamlined entrance may therefore be assumed to be independent of canal velocity for practical purposes.

It is therefore apparent that because of the canal velocity influence, the head discharge relationship for a side weir is not unique and depends on the magnitude of the canal velocity.

Figures 4.7 and 4.8 show logarithmic plots of head vs discharge for the two sharp entranced weirs at different velocities, obtained by linear regression. The ideal discharge curve has also been shown. From these the discrepancy between the ideal (theoretically calculated) and the actual discharge that takes place at zero canal velocity is apparent. The influence of increase in canal velocity upon actual discharge for a given head is clearly demonstrated. Conditions are further removed from being ideal resulting in a lower discharge for a given head.

The head-discharge relationships derived are in the general form of;

$$Q = KH^{x} \qquad (4.1)$$

With this form of equation, the cross sectional area of flow

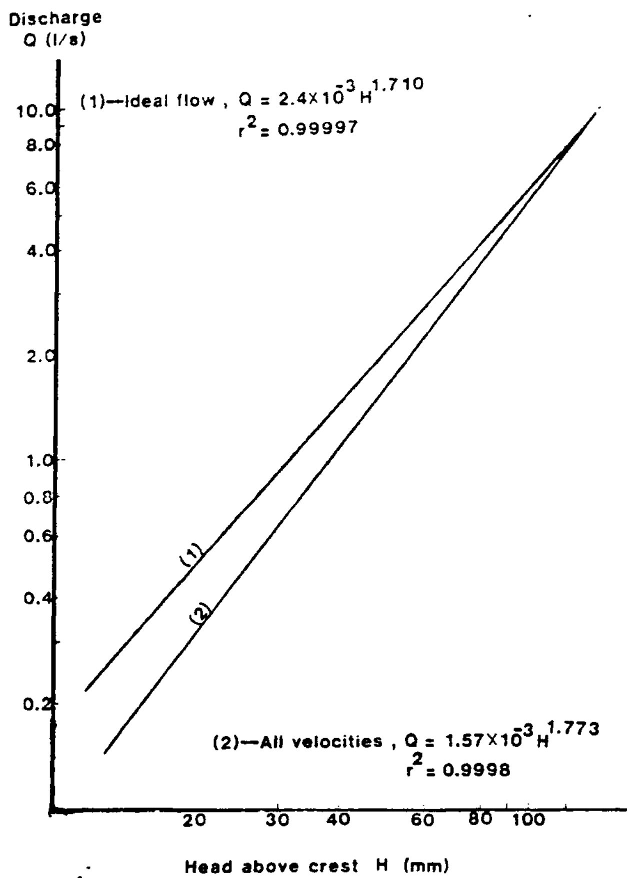


Fig. 4.9 Head-discharge relationship at all velocities for the curved entranced weir with horizontal bed.

is accounted for by the numerical values of K and x. The value of K also includes the conversion factor for the units used. The relationships in Figs. 4.7 and 4.8 show that with increase in velocity the value of K decreases while the value of x is increased. The discharge by the weir therefore becomes a stronger function of the head. The combination of lower K and higher x however, result in a lower discharge by the weir for a given head in the practical operating head range ($20 < H < 110$).

Linear regression could not significantly distinguish between head-discharge relationships at different velocities for the weir with a streamlined entrance (Fig. 4.9). A unique head discharge relationship, independent of velocity is therefore developed for this weir. With this type of weir the discharge level for a given head is closer to the ideal discharge, compared with sharp entranced weirs.

Extrapolation of Laboratory Results

The weirs used in the laboratory had 3.0" bed widths, whereas field weirs may have a range of bed widths (2.0", 2.5", 3.0", etc.). In order to make use of the results obtained in the laboratory for general design purposes one must extrapolate the laboratory results such that they could be applicable to weirs with different bed widths. This can be done using Froude modelling. If subscript p refers to prototype, m refers to the model and r to the ratio p to m, then for the model to represent the prototype;

$$F_p = F_m \text{ ------------- } F_r = 1.0 \qquad (4.2)$$

$$g_p = g_m \text{ ------------- } g_r = 1.0 \qquad (4.3)$$

Also

$$L_p/L_m = C = L_r \qquad (4.4)$$

And

$$A_r = (L_r)^2 \tag{4.5}$$

Where F is the Froude number, g is the gravitational constant and L is a length dimension and C is the scaling factor. For a prototype weir bed width of 2.5" therefore:

$$C = 2.5/3.0 = 0.833 \tag{4.6}$$

It follows that

$$F_r = V_r \ / \ (g_r \ L_r)^{1/2} \tag{4.7}$$

Therefore

$$V_r = (L_r)^{1/2} \tag{4.8}$$

But

$$V_r = V_p \ / \ V_m \tag{4.9}$$

Therefore

$$V_p = V_m \ (L_r)^{1/2} \tag{4.10}$$

Also

$$Q_r = A_r \ . \ V_r \tag{4.11}$$

Therefore

$$Q_r = L_r^2 \cdot L_r^{1/2} \quad 4.12)$$

Or

$$Q_r = L_r^{5/2} \quad (4.13)$$

i.e.

$$Q_p/Q_m = L_r^{5/2} \quad (4.14)$$

Or

$$Q_p = Q_m \cdot L_r^{5/2} \quad (4.15)$$

The head on the prototype side weir must also be scaled by the length ratio i.e.

$$H_p = H_m \cdot L_r \quad (4.16)$$

Equations 4.15 and 4.16 indicate that in order to extrapolate a given model head and discharge reading to a prototype, they must be multiplied by the appropriate factors. Using these, the head discharge relationships for the field weirs (b = 2.5") were obtained from the laboratory results. The relationships derived for the weir with a 1/10 bed slope (field weirs had the same bed slope) at zero velocity was;

$$Q_m = 1.55 \times 10^{-3} H_m^{1.801} \quad (4.17)$$

Using equation 4.15 and 4.16 to obtain the discharge equation for weirs with 2.5" bed width;

$$Q_p = (0.833)^{5/2} \cdot 1.55 \cdot 10^{-3} \cdot (H_p/0.833)^{1.801} \quad (4.18)$$

Or

$$Q_p = 1.363 \cdot 10^{-3} H_p^{1.801} \quad (4.19)$$

Similarly for V = 0.3 m/s

$$Q_p = 0.971 \cdot 10^{-3} H_p^{1.874} \quad (4.20)$$

Fig. 4.10 shows head-discharge data obtained from measurements on side weirs operating under actual field conditions (3). Equations 5.16 and 5.17 are also plotted (Lines 1 and 2 resp.). Using the experimental data, the head-discharge relationship was found to be;

$$Q = 1.29 \times 10^{-3} H^{1.740} \quad (4.21)$$

The band of error due to data scatter associated with this relationship was plus or minus 10%. The band of error in the laboratory was at the most plus or minus 1.0%. Equations (4.19) and (4.21) predict a higher discharge than actually measured in the field over the whole operating range of the weirs. The maximum difference (equation 4.19, H = 100 mm) is about 28%. It must not, however, be forgotton that the field weirs were made of concrete whereas the laboratory weir was made of galvanized sheet metal which is a much smoother material. Also since the laboratory

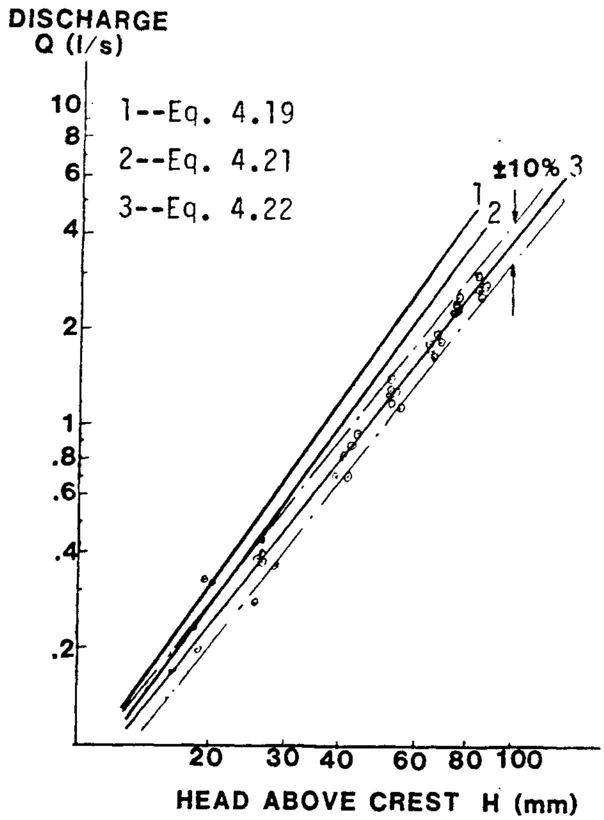


Fig. 4.10 Plot of Head Vs Discharge Using the Data Obtained in the Field.

model was constructed by bending a sheet metal, there inevitabily was a small radius of curvature at the entrance. These two differences result in a higher coefficient of discharge for the model at any given head, and account for the 28% discrepency between the model and the prototype.

Field Evaluation Results

Side Weir Construction Uniformity

For the sites tested, little variation in geometry was found between weirs along the canal. This was because a mould had been used for their construction. The data collected on the crest elevation of the weir are presented in Appendix D. Statistical analysis on these showed standard deviations of 3 mm about the mean for the weirs serving the cottom field, and 8 mm for the weirs serving the grass field. The weirs serving the cotton field had been installed using conventional surveying equipment, whereas the weirs serving the grass field had been installed using a laser level indicator in place of the telescope. For both fields the weir forming technique was identical i.e. an operator pushed a mould of the weir into the wet concrete to the indicated elevation (by the telescope or the laser). The large variations in the construction elevation of the weirs serving the grass field (8mm S.D) is thought to be because of the inexperience of the crew with the new level indicator. Furthermore, using a human operated mold in conjunction with a laser level indicator is employing incompatible technologies. The full potential of the laser technique may only be realized if the human operated mold is replaced by a laser sensitive machine capable of pushing the mould to the exact indicated elevation.

The variations in weir construction elevation measured (especially in the grass field) would contribute greatly to discharge non-uniformity along the distribution bay. The construction elevation uniformity of the weirs serving the grass field is clearly unacceptable.

Because of the sensitivity of discharge by the weir to head, uniform weir crest elevation is one of the essentials in the success of the side weir system. Better installation of weirs with present techniques must be achieved.

Discharge Variability of Field Weirs

The discharge data for the sites tested are presented in Appendix D. For the weirs serving the cottom field, analysis of the data showed that the band of discharge scatter by different weirs about the mean was initially at 11.2% (1 x standard deviation). The scatter increased to 19.7% when the stream size was cut-back. Variations in construction elevation of the weir was assessed to be the major contributor to this discharge scatter accounting for 3 - 9% of the standard deviation. Variations in velocity along the canal and the non-horizontal water surface profile accounted for the remainder of the scatter.

For the weirs serving the grass field, only one discharge setting was measured. For this the band of weir discharge scatter was 28.8%. Variations in the construction elevation of the weirs account for most of this of discharge scatter. The average of the measured discharges and the standard deviation about the mean were 2.36 l/s (37.42 GPM), 0.68 l/s (10.78 GPM) respectively.

Application Efficiency and Componants

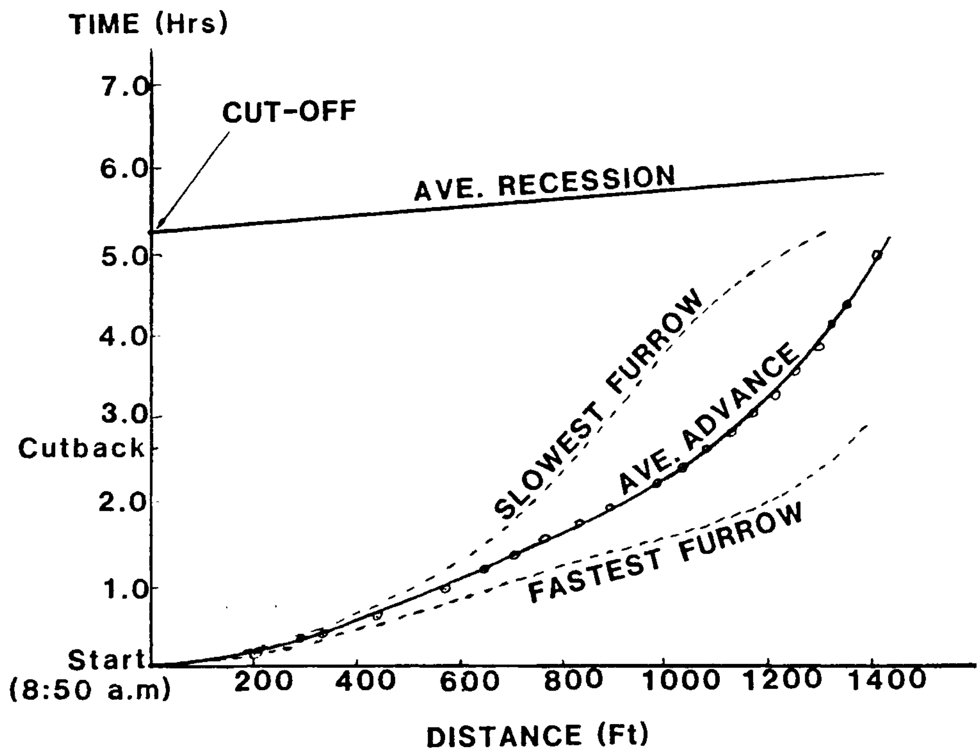


Fig. 4.11 Average Advance and Recession Curves for the Field Tested.

Figure 4.11 shows the average advance curve obtained for 25 furrows in the cotton field. The figure shows the timing of the irrigation practice (start time, cut-back time and cut-off time) and the range of advance rates. At full flow, the average of the measured discharges by individual weirs was 3.39 l/s (53.81 GPM) with a standard deviation of 0.37 l/s (6.01 GPM). This flow was cut back to an average of 1.94 l/s (30.72 GPM) with a standard deviation of 0.38 l/s (6.04 GPM).

Testing for moisture replacement adequacy (using probes) three days following the irrigation, showed that the lower 1/3 of the field had not received adequate water.

From the average advance and recession curves of figure 4.11, the intake opportunity time corresponding to the point 2/3 of the length down the furrow (distance 930 ft) is 3.75 hours. This is therefore assumed to be the time required for adequate moisture replacement.

Before one can determine the depth of water infiltrated, one requires an infiltration function representative of the field tested. No infiltration tests were carried out on the site. Soil Conservation Service (SCS) have developped a series of intake family functions covering a wide range of soils. Using a volume balanced approach to modify the closest fitting (SCS) cumulative infiltration function, an infiltration function for the field tested was derived. This was;

$$Z = 33.5\ t^{0.689} \quad \text{(intake family 0.5)} \qquad (4.22)$$

Where Z is the total depth of water infiltrated in mm, after t hours of contact.

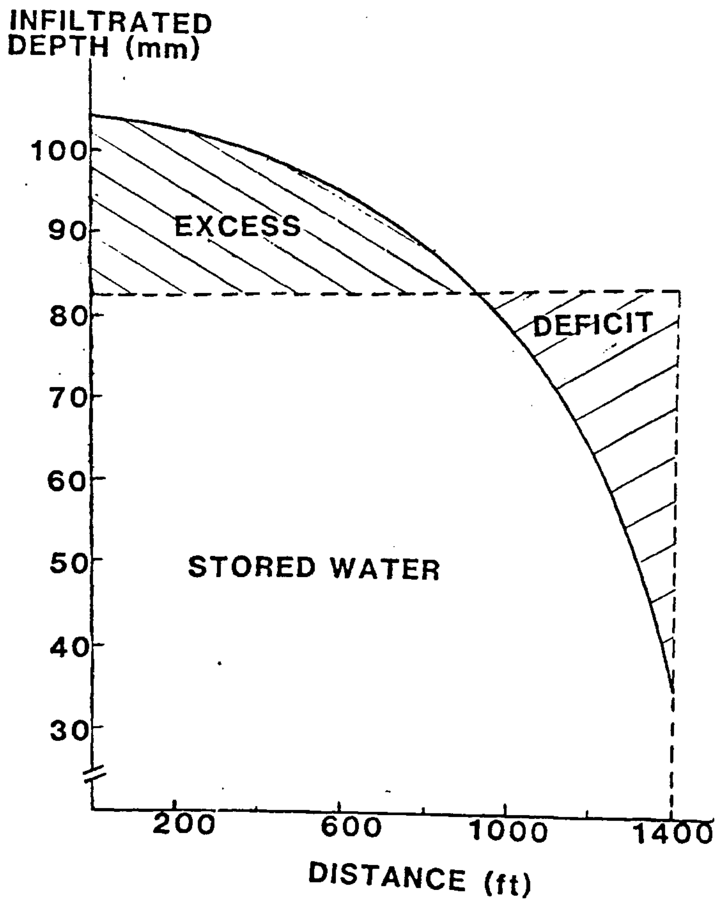


Fig. 4.12 Average Infiltrated Depth Down the Furrows.

Use of intake opportunity time (recession time minus advance time) at points along the furrow in equation 4.23 resulted in the curve of figure 4.12. With the depth of infiltration necessary for adequate replacement known, the deep percolation loss (EXCESS) and deficit in application (DEFICIT) were calculated. Also having calculated the run-off volume from the run-off hydrograph (Fig. 4.13), an accurate figure for the efficiency of application could be obtained.

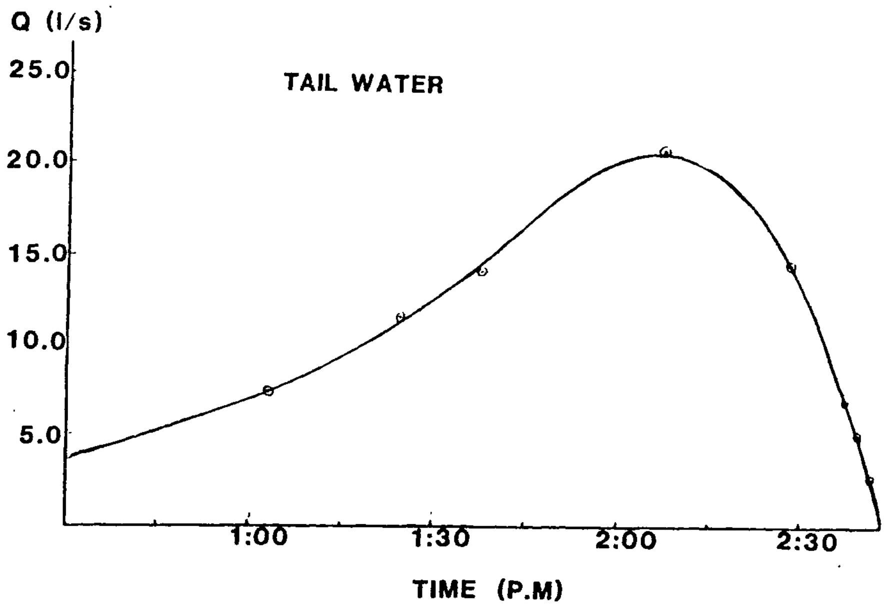


Fig 4.13 The Run-Off Hydrograph

Using Figures 4.12 and 4.13 the following volumes were obtained;

Total delivered volume = 1243.8 m^3

Total infiltrated volume = 1201.0 m^3

Total run-off volume = 42.8 m^3

Total deep percolation volume (EXCESS) = 122.8 m^3

Total deficit volume = 86.0 m^3

In calculating percentage values for efficiency, deficit, deep percolation and run-off, the following definitions were used;

$$\text{Efficiency} = \frac{\text{Total volume stored in the root zone}}{\text{Total volume disharged by the weirs}}$$

$$\text{Deficit} = \frac{\text{Volume of root zone capacity unfilled}}{\text{Root zone capacity}}$$

$$\text{Deep percolation} = \frac{\text{Volume percolated below root zone}}{\text{Total volume discharged by the weirs}}$$

$$\text{Run-off} = \frac{\text{Volume of tail water discharge}}{\text{Total volume discharged by the weirs}}$$

Using these definition we get;

$$\text{Efficiency} = \frac{1201 - 122.8}{1243.8} \times 100 = 86\%$$

$$\text{Deficit} = \frac{86.0}{1201.0 - (122.8 + 86.0)} \times 100 = 7\%$$

$$\text{Deep percolation} = \frac{122.8}{1243.8} \times 100 = 10\%$$

$$\text{Run-off} = \frac{42.8}{1243.8} \times 100 = 4\%$$

From the water applied, 86% was usefully stored in the root zone, 4% was lost as run-off and 10% percolated below the root zone. Seven precent of the total storage capacity in the root zone remained unfilled at the end of the irrigation.

The efficiency level achieved with this system is much higher than usually feasible with conventional furrow irrigation systems. This is primarily because of the easy control of the discharge to all the furrows, and the subsequent control over the advance rate and the run-off quantity. The discharge level is set to give the maximum safe advance rate minimizing deep percolation losses, and cut back for minimum run-off, which results in a high efficiency of application.

Uniformity of Application

Fig. 4.14 is a redrawing of Fig. 4.12, arranged to give a better pictorial representation of the uniformity of application. The depth of water necessary for adequate replacement, D, together with the average depth of water actually applied, d, have also been shown. A coefficient of uniformity may be described by (Schwab, Frevert, Edminster and Barnes, 1981).

$$C.U. = 1 - Y/d \qquad (4.23)$$

Where

Y = Average of the absolute values of the deviations from the average depth of water stored.

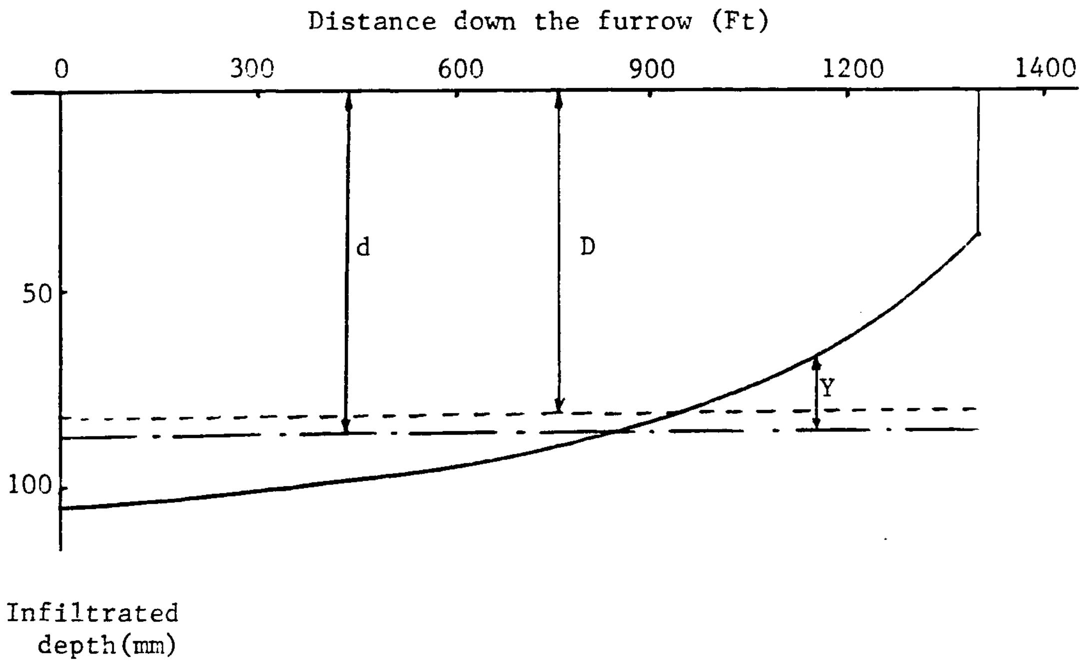


Fig. 4.14 Moisture Replacement Profile.

and

d = Average depth of water stored.

For the irrigation set tested;

$$Y = 13.9 \text{ mm}$$

$$d = 85.3 \text{ mm}$$

Therefore

C.U. = 0.84 or 84%

Computer Model Results

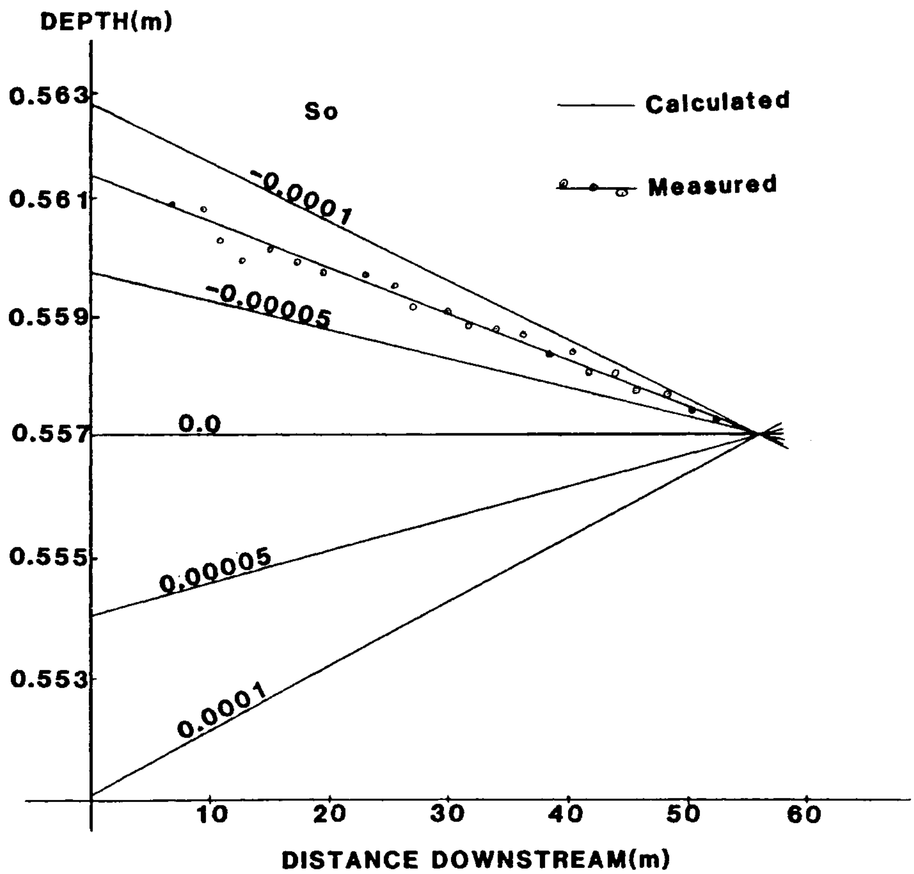


Fig. 4.15 Computer Calculated Water Surface Profiles at Different Channel Bed Slopes, and the Measured Srofile.

Using the computer model developmed for this study (see chapter 2, also see appendix B), water surface profiles for different channel slopes and irrigation settings, for the canal side weir system were calculated. The discharge equation for weirs used was;

$$Q = 8 \times 10^{-4} H^{1.800} \qquad (4.24)$$

This is an average of the different relationships derived, at different canal velocities.

The model calculates the profile slope as being a variable along the canal reach. The variations in slope from one point to another and the absolute values of the calculated slopes along are, however, rather small. So small that when these slope variations are converted to depth change, it is difficult to graphically distinguish between the different slopes. The overall surface profile is therefore approximated as having a constant average slope.

Fig. 4.15 shows the calculated water surface profile for different canal bed slopes. The actual measured profile is also plotted. The computer outputs corresponding to these plots are provided in Appendix B. The results clearly demonstrate the effect that bed slope has upon the water surface profile. With the adverse canal bed slopes (negative slopes), the water surface slope is calculated as being constantly negative (decrease in depth with distance), more so in locations where there is lateral outflow (near side-weirs). With positive canal bed slopes, the surface profile slope is positive at all points, more so where there is no lateral out-take (in between weirs). For a horizontal canal bed, the profile slope is positive near the weirs (lateral out-take), and negative in between consecutive weirs, resulting in a zero average slope.

CHAPTER 5

SUMMARY AND CONCLUSIONS

Summary

This study investigated the operational and hydraulic characteristics of small trapezoidal side weirs for use in furrow irrigation. The system had been credited with desirable operational characteristics such as feasible collective control over discharge to furrows, minimal labor requirement for operation, self adjusting in situations of variable canal inflow, minimal clogging in the presence of surface debris and therefore being capable of achieving higher application uniformities and efficiencies. These merits were experimentally verified through a field evaluation.

The hydraulic characteristics of the system were researched using a laboratory model. The effect of the significant variables namely weir entrance conditions, canal velocity variations and the construction uniformity of individual weirs, were experimentally established. The water surface profile for the system under various conditions was theoretically determined using a computer model. The calculated water surface profiles were compared with field data.

Conclusions

The conclusions of this study are as follows:

1. Head-discharge relationship for a side weir is not unique and depends on the magnitude of the velocity in the main canal. This dependence is most significant with sharp entranced weirs. With curved entranced weirs the velocity influence is small enough to be neglected.

2. For a given head on a sharp entranced weir, the discharge increases as canal velocity decreases. The percentage increase in discharge is inversely proportional to the magnitude of the head on the weir. Typical decrease in canal velocity between the upstream and the downstream extremities of a distribution bay (0.3 m/s - 0.0 m/s), can result in an increase in discharge of up to 15% compared with zero velocity (at low heads). For normal irrigation setting (H = 60-80 mm), the increase is about 6%.

3. Because of the head-discharge relationship of the weirs, construction elevation uniformity of individual weirs is very important. A 3 mm difference in bed elevation of two, side by side weirs, can typically result in a 10% difference in their discharge.

4. The average water profile slope was found to be a function of the canal bed slope. For a given canal bed slope the water surface profile slopes by approximately the same degree but in the opposite direction. To obtain a horizontal average surface profile, a horizontal canal bed

should be used.

5. For discharge uniformity, side weirs must have a streamlined entrance and be precision leveled to the design elevation. The canal from which they discharge must have zero bed slope.

APPENDIX A

CALCULATION OF THE WATER SURFACE PROFILE

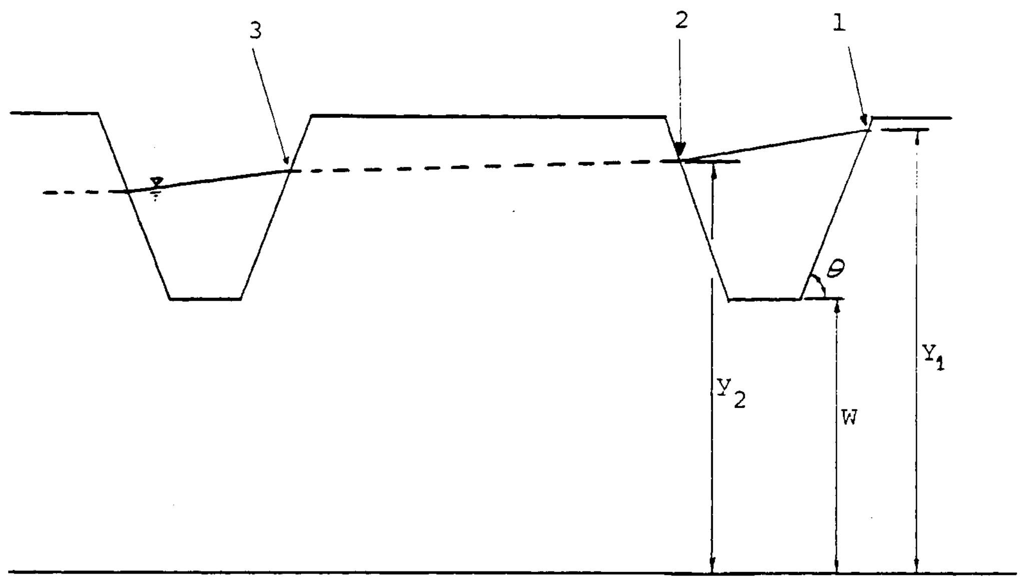


Fig. A.1. Schematic Water Surface Profile of the Canal Side Weir System.

Fig. A.1 shows a schematic, profile of the canal side weir system. In this figure, Y_1 is the control depth at the end of the canal reach which is set by a gate or check boards, W is the elevation of the weir crest above the canal bed and 0 is the side inclination of the weir. Calculations of water surface profile start at point 1.

(a) Calculate the head on the weir, $H = Y_1 - W$.

(b) Using the head discharge relationship for the weir (Q =

KH^x), determine the extent of lateral out-flow at 1. (note: $Q = - dQ_d/dx$).

(c) Calculate the cross sectional area, A, and the Froude Number, Fr, for the canal at point 1.

(d) Using Manning's equation, calculate the friction slope, S_f where:

$$S_f = \frac{n^2 \; V_1{}^2}{(1.49)^2 \; R_1{}^{4/3}} \tag{A.1}$$

(e) Using equation derived for the profile slope (see equation 2.21), calculate dY/dX at point 1.

At this stage, the most accurate step would be to chose a small value of X, multiply it by the calculated dY/dX,in order to arrive at the depth X upstream of 1. The procedure (a) through to (e) would then be repeated until we arrive at point 2. Because of the small value of dY/dX at 1, and the short distance between 1 and 2, it is reasonable to assume a linear water surface slope between 1 and 2 without significant error. The change in depth from 1 to 2 may then be found by multiplying the calculated slope at 1, by the horizontal distance between 1 and 2.

The distance between 1 and 2 however, is unknown. The depth at 2, assuming a linear water surface slope, dY/dX, between 1 and 2, may be found as follows:

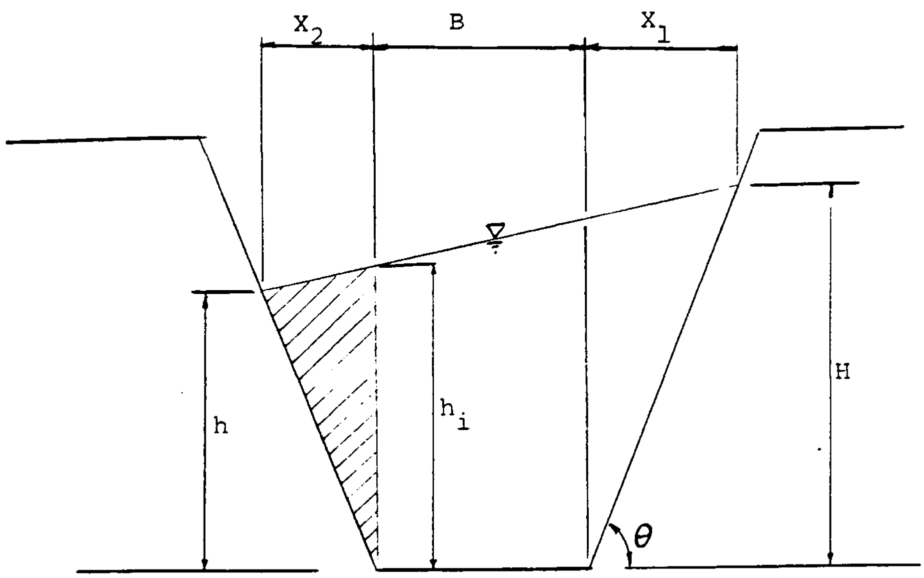


Fig. A.2. Schematic Water Surface Profile in the Side Weir.

Refering to figure A.2

(f) Calculate $X_1 = H/\tan\theta$

(g) Find $h_i = H + dY/dX\ (X_1 + B)$

Fig. A.3 shows the shaded triangle of figure A.2.

For this triangle;

(h) $a = \arctan dY/dX$

(i) $b = 90 - a$

(j) $d = 90 - \theta$

(k) $c = 180 - (b + d)$

Using the sine rule

(L) $h_i/\sin c = S/\sin b$

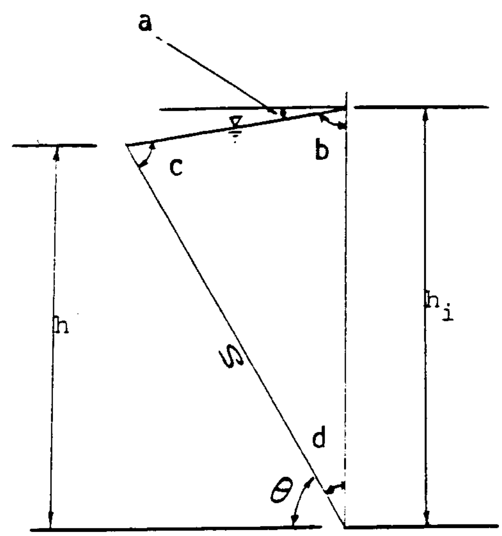


Fig. A.3. Triangle for Profile Calculation.

Therefore

(M) $S = h_i \text{ Sin } b / \text{Sin } c$

(N) $h = S \text{ Cos } d$

(O) $Y_2 = h + W$

Steps (L) through to (O) are only necessary because distance X_2 of figure A.2 is unknown. Because of the small value of dY/dX, a is very small and little error is introduced by assuming $X_1 = X_2$. This assumption does away with steps (L) through to (O), simplifying the procedure with negligible error.

With this assumption,

$$D_{1-2} = 2X_1 + B$$

$$y_2 = y_1 \quad dy/dx \; . \; D_{1-2}$$

Where D_{1-2} is the distance between points 1 and 2. (NOTE: Since we are starting from downstream, any distance upstream is negative i.e. D_{1-2}, is a negative distance).

The depth at 2 is now unknown. Using this depth, the water surface slope at 2 may be calculated in the same manner as before, with one exception. There is no lateral outflow at 2, therefore dQ_d/dX is zero. In the same way as before, the slope between 2 and 3 (Fig. A.1) may be assumed linear. With a similar assumption as for before, the distance between 2 and 3 is estimated. The depth at 3 is therefore;

$$Y_3 = Y_2 + (dY/dx)_2 \cdot D_{2-3}$$

Where D_{2-3} is the distance between points 2 and 3.

This procedure is continued to the uppermost point of the canal reach, thereby calculating the depth of water at upstream and downstream edges of all the side weirs, giving a fair picture of the surface profile.

APPENDIX B

THE COMPUTER MODEL AND SAMPLE OUTPUTS

The Computer Program

```
      PROGRAM NOTCH
C     Program calculates water surface profile in a trapizoidal or
C     rectangular channel with trapizoidal or rectangular sideweirs
C     discharging water from it. The profile is calculated by assuming
C     that no loss in energy results by the lateral out-take of water.
C     Hence dE/dx = So - Sf and from this the equation for varied flow
C        dY/dX = (So - Sf - (Q/g*A**2) ) / 1 - Fr**2
C     is derived . The program numerically solves this equation using
C     an explicit approach.

C     Variable decleration

       REAL B,m,W,bn,n,L,Qd(1000),D(1000),So,Sf(1000),T(1000),A(1000)
     &,Fr(1000),R(1000),mn,H(1000),g,qn(1000),K,x,DSTNCE(1000),DELH,
     &DELD,dYdX(1000),LEN,NUM(1000),DIN(1000),VHD(1000),E(1000)
     &,FRCLS,LENG,SE(1000),PE(1000),SD(1000)

C     Information on data input format is provided and the specifications
C     of the channel and the notch are demanded from input file.

      WRITE(6,*) 'If you are familiar with the format and the order
     & of the input data(and you have the input file ready) type 1
     &, otherwise type 0 for information'
      READ(5,*)INDIC
      IF(INDIC.EQ.1)THEN
       GO TO 5
      ELSE
      END IF
      WRITE(6,*)
      WRITE(6,*)
      WRITE(6,*) 'Input data must be in a file numbered  TWO (2) with
     & extention .DAT, eg. FOR002.DAT. Data is read in free format form
     & and in the following order '
      WRITE(6,*)
      WRITE(6,*)'First line ___  main channels Side inclination to
     & Horizontal(degrees),Bottom width,Bottom slope(ft/ft)
     &,Length,Manning roughness n '
      WRITE(6,*)
      WRITE(6,*)'Second line  is notch data ___ Bottom width,Side
     & inclionation to horizontal(degrees),Height of the
     & crest above channel bed,Spacing between notch centers '
      WRITE(6,*)
      WRITE(6,*)'Third line is ___ Flow rate passed the last notch,Depth
     & of flow at the downstream edge of the last notch '
      WRITE(6,*)
      WRITE(6,*)'Fourth line is ___ Manning conversion factor K,Value
     & of the acceleration due to gravity g (for the system of units
     & being used)'
      WRITE(6,*)
      WRITE(6,*)'Last line is ___ values for constants C , x
     & in the notch discharge equation q = C * H**x '
      WRITE(6,*)
      WRITE (6,*) 'If the data is correctly in the file then type 1
     & for the calculations to commence, otherwise type 0 to stop the
     & program'
      READ(5,*)INDIC
      IF(INDIC.EQ.0)THEN
       GO TO 1000
      ELSE
      END IF

    5 READ(2,*)m,B,So,LEN,mn
      m = m * (22/7) / 180
      READ(2,*) bn,n,W,L
```

```
      n = n * (22/7) / 180
      L = L - bn
      READ(2,*)Qd(1),D(1)
      READ(2,*)K,g
      READ(2,*)C,X

C     All variables have been read in . Some variables are initialised

      J = 2
      DSTNCE(1) = 0
      FRCLS = 0
      LENG = 0
      IF(So.LT.0.0)THEN
      PEL = -(So * LEN)
      ELSE
      PEL = 0.0
      END IF

C     Calculation of notch discharge.

   10 H(J-1) = D(J-1) - W
      qn(J-1) = C * H(J-1)**X

C     Calculation of hydraulic parameters.

      T(J-1) = D(J-1) * 2/TAN(m) + B
      A(J-1) = (T(J-1) + B) * D(J-1)/2
      Fr(J-1) = (Qd(J-1)**2 * T(J-1)) / (A(J-1)**3 * g)
      Fr(J-1) = Fr(J-1) ** 0.5
      R(J-1) = 2 * D(J-1)/SIN(m) + B
      Sf(J-1) = ((mn*Qd(J-1)) / (K*R(J-1)**0.667*A(J-1)))**2
      VHD(J-1) = (Qd(J-1)**2) / (2*g*A(J-1)**2)

C     Substitution of the parameters in the water surface slope eqtn.

      NUM(J-1) = (Qd(J-1) / (g*A(J-1)**2)) * (-qn(J-1))
      NUM(J-1) = So - Sf(J-1) - NUM(J-1)
      DIN (J-1)= 1 - Fr(J-1)**2
      dYdX(J-1) = NUM(J-1) / DIN(J-1)
      IF(J.EQ.2)THEN
      ELSE
      FRCLS = FRCLS + ((Sf(J-1) + Sf(J-2))/2) * LENG
      END IF
      SE(J-1) = D(J-1) + VHD(J-1) + FRCLS
      PE(J-1) = PEL + So * DSTNCE(J-1)
      E(J-1) = D(J-1) + VHD(J-1) + FRCLS + PE(J-1)

C     PEL is the potential energy at end of the canal.

C     Projection of the water surface slope to the upstream edge
C     of the notch and calculation of the depth of flow there.

      SD(J-1) = H(J-1) / TAN(n)
      DELH = -(2*SD(J-1) + bn) * dYdX(J-1)
      D(J) = D(J-1) + DELH

C     Calculation of distance where the new depth occures.

      DSTNCE(J) = DSTNCE(J-1) + (2*SD(J-1) + bn)

C     upstream edge of the notch.

      Qd(J) = Qd(J-1) + qn(J-1)
```

```
C     Between the upstream edge of one notch and the downstream
C     edge of the next one up, there is no lateral outflow from
C     the main channel. The water surface profile equation is
C     therfore dYdX = (So - Sf) / (1-Fr**2). This equation is
C     hence used for all such reaches in the main channel.

C     Calculation of the hydraulic parameters.

      T(J) = D(J) * 2/TAN(m) + B
      A(J) = (T(J) + B) * D(J)/2
      Fr(J) = (Qd(J)**2 * T(J)) / (A(J)**3 * g)
      Fr(J) = Fr(J) ** 0.5
      R(J) = 2 * D(J)/SIN(m) + B
      Sf(J) = ((mn*Qd(J)) / (K*R(J)**0.667*A(J)))**2
      VHD(J) = (Qd(J)**2) / (2*g*A(J)**2)

C     Substitution of the parameters in the water surface slope eqtn.

      dYdX(J) = (So - Sf(J)) / (1 - Fr(J)**2)
      FRCLS =FRCLS + ((Sf(J) + Sf(J-1))/2) * (2*SD(J-1)+bn)
      SE(J) = D(J) + VHD(J) + FRCLS
      PE(J) = PEL + So * DSTNCE(J)
      E(J) = D(J) + VHD(J) + FRCLS + PE(J)

C     Projection of the water surface slope to the downstream edge
C     of the next notch up and calculation of the depth of flow there.

      SD(J) = (D(J) - W) / TAN(n)
      DELD = -(L - SD(J)*2) * dYdX(J)
      J = J + 1
      D(J) = D(J-1) + DELD
      Qd(J) = Qd(J-1)
      SD(J) = (D(J) - W) / TAN(n)
      LENG = L - (SD(J) + SD(J-1))
      DSTNCE(J) = DSTNCE(J-1) + LENG

C     Checking to see wether we are at the end of the channel or not.

      IF(DSTNCE(J).LT.LEN)THEN
        J = J + 1
        GO TO 10
      ELSE
      END IF

C     The entered cahnnel and notch specifications are written to
C     the screen and to file # 1.

      m = m * 180 / (22/7)
      L = L + bn
      n = n * 180 / (22/7)

C     The calculated values are written to the screen and file # 1.

      WRITE(1,20)B,m,mn,LEN,So
      WRITE(1,30)bn,n,L,C,x
      WRITE(1,34)
      IF(K.EQ.1)THEN
      WRITE(1,36)
      ELSE
      WRITE(1,37)
      END IF
      DO 25 I = 1 , J-1
```

```
      WRITE(1,50)DSTNCE(I),D(I),Qd(I),dYdX(I),SE(I),E(I)
25    CONTINUE

      WRITE(6,*)'Results have been written to the output file
     &No.1 (FOR001.DAT). Do you also want the results written
     &to the screen?(1 yes/0 no)'
      READ(5,*)INDIC

      IF(INDIC.EQ.0)THEN
       GO TO 1000
      ELSE
      END IF
      WRITE(6,20)B,m,mn,LEN,So
      WRITE(6,30)bn,n,L,C,x
      WRITE(6,34)
      IF(K.EQ.1)THEN
       WRITE(6,36)
      ELSE
       WRITE(6,37)
      END IF
      DO 40 I = 1 , J-1
      WRITE(6,50)DSTNCE(I),D(I),Qd(I),dYdX(I),SE(I),E(I)
40    CONTINUE

20    FORMAT(1X,'The channel specifications are as follows',/,/,3X
     &,'Channel bottom width = ',F7.5,/,3X,'Channel side inclination
     & to horizontal = ',F5.2,/,3X,'Channel roughness coeff.  = ',F5.4
     &,/,3X,'Channel length = ',F8.3,/,3x,'Channel bed slope = ',F8.6
     &,/,/)

30    FORMAT(1X,'The notch dimensions are as follows',/,/,3X
     &,'Notch crest width = ',F7.4,/,3X,'Notch side inclination to
     &horizontal = ',F5.2,/,3X,'Spacing between notch centers = '
     &,F7.4,/,3X,'Vallue of C in the notch discharge eqtn = ',F5.3
     &,/,3X,'Value of power x in the notch discharge eqtn = ',F6.4
     &,/,/)
34    FORMAT(1X,'The water surface profile in the channel is
     & as follows',/,/,1X,'Distance upstream',4X,'Depth of flow'
     &,4X,'Channel flow rate',4X,'Profile slope',4X,
     &'Specific Energy',4X,'Total energy',/)
36    FORMAT(8X,'(M)',16X,'(M)',14X,'(CuM/S)',13X,'(M/M)',
     &14X,'(M)',15X,'(M)',/,/)
37    FORMAT(8X,'(Ft)',16X,'(Ft)',14X,'(CuFt/S)'
     &,11X,'(Ft/Ft)',11X,'(Ft)',14X,'(Ft)',/,/)
50    FORMAT(5X,F8.4,11X,F8.6,11X,F8.5,11X,F9.7,9x,F8.6,10x,F8.6)
1000  STOP
      END
```

Outputs

The channel specifications are as follows

Channel bottom width = 0.45700
Channel side inclination to horizontal = 45.00
Channel roughness coeff. = .0150
Channel length = 55.960
Channel bed slope = 0.000050

The notch dimensions are as follows

Notch crest width = 0.0635
Notch side inclination to horizontal = 66.80
Spacing between notch centers = 0.9300
Vallue of C in the notch discharge eqtn = 0.080
Value of power x in the notch discharge eqtn = 1.8000

The water surface profile in the channel is as follows

Distance upstream (M)	Depth of flow (M)	Channel flow rate (CuM/S)	Profile slope (M/M)	Specific Energy (M)	Total energy (M)
0.0000	0.557000	0.09000	0.0000721	0.558196	0.558196
0.1442	0.556990	0.09089	0.0000463	0.558209	0.558217
0.9300	0.556952	0.09069	0.0000723	0.558173	0.558220
1.0742	0.556941	0.09177	0.0000463	0.558187	0.558241
1.8601	0.556903	0.09177	0.0000725	0.558151	0.558244
2.0042	0.556893	0.09266	0.0000463	0.558165	0.558265
2.7901	0.556855	0.09266	0.0000726	0.558129	0.558269
2.9342	0.556844	0.09354	0.0000482	0.558143	0.558290
3.7201	0.556807	0.09354	0.0000728	0.558107	0.558293
3.8642	0.556796	0.09443	0.0000482	0.558122	0.558315
4.6501	0.556758	0.09443	0.0000730	0.558086	0.558318
4.7941	0.556746	0.09531	0.0000481	0.558101	0.558340
5.5802	0.556710	0.09531	0.0000732	0.558065	0.558344
5.7241	0.556699	0.09619	0.0000481	0.558080	0.558366
6.5102	0.556661	0.09619	0.0000734	0.558044	0.558369
6.6541	0.556651	0.09707	0.0000481	0.558059	0.558392
7.4402	0.556613	0.09707	0.0000735	0.558023	0.558395
7.5841	0.556603	0.09795	0.0000480	0.558038	0.558418
8.3703	0.556565	0.09795	0.0000737	0.558003	0.558421
8.5141	0.556554	0.09883	0.0000480	0.558018	0.558444
9.3003	0.556516	0.09883	0.0000739	0.557983	0.558448
9.4441	0.556506	0.09971	0.0000480	0.557998	0.558470
10.2303	0.556468	0.09971	0.0000741	0.557963	0.558474
10.3740	0.556457	0.10058	0.0000479	0.557979	0.558497
11.1603	0.556420	0.10058	0.0000742	0.557943	0.558501
11.3040	0.556409	0.10146	0.0000479	0.557959	0.558524
12.0904	0.556371	0.10146	0.0000744	0.557924	0.558528
12.2340	0.556361	0.10233	0.0000479	0.557940	0.558552
13.0204	0.556323	0.10233	0.0000746	0.557905	0.558556
13.1640	0.556312	0.10321	0.0000478	0.557921	0.558579
13.9504	0.556275	0.10321	0.0000748	0.557886	0.558583
14.0940	0.556264	0.10408	0.0000478	0.557902	0.558607
14.8805	0.556227	0.10408	0.0000749	0.557867	0.558611
15.0239	0.556216	0.10495	0.0000477	0.557884	0.558635
15.8105	0.556178	0.10495	0.0000751	0.557849	0.558640
15.9539	0.556167	0.10582	0.0000477	0.557866	0.558664
16.7405	0.556130	0.10582	0.0000753	0.557831	0.558668
16.8839	0.556119	0.10669	0.0000477	0.557848	0.558692
17.6706	0.556082	0.10669	0.0000755	0.557813	0.558697

Note: There was a flow through of 0.09 CuM/s for irrigation of a second bench.

17.8139	0.556071	0.10756	0.0000476	0.557830	0.558721
18.6006	0.556033	0.10756	0.0000756	0.557796	0.558726
18.7439	0.556023	0.10843	0.0000476	0.557813	0.558750
19.5306	0.555985	0.10843	0.0000758	0.557778	0.558755
19.6738	0.555974	0.10930	0.0000475	0.557796	0.558780
20.4606	0.555937	0.10930	0.0000760	0.557761	0.558784
20.6038	0.555926	0.11017	0.0000475	0.557779	0.558809
21.3907	0.555889	0.11017	0.0000761	0.557745	0.558814
21.5338	0.555878	0.11103	0.0000475	0.557763	0.558839
22.3207	0.555840	0.11103	0.0000763	0.557728	0.558844
22.4638	0.555829	0.11190	0.0000474	0.557746	0.558869
23.2507	0.555792	0.11190	0.0000765	0.557712	0.558874
23.3938	0.555781	0.11276	0.0000474	0.557730	0.558900
24.1808	0.555744	0.11276	0.0000766	0.557696	0.558905
24.3238	0.555733	0.11362	0.0000473	0.557714	0.558931
25.1108	0.555696	0.11362	0.0000768	0.557680	0.558936
25.2537	0.555685	0.11448	0.0000473	0.557699	0.558962
26.0408	0.555647	0.11448	0.0000769	0.557665	0.558967
26.1837	0.555636	0.11534	0.0000473	0.557684	0.558993
26.9708	0.555599	0.11534	0.0000771	0.557649	0.558998
27.1137	0.555588	0.11620	0.0000472	0.557669	0.559024
27.9009	0.555551	0.11620	0.0000773	0.557635	0.559030
28.0437	0.555540	0.11706	0.0000472	0.557654	0.559056
28.8309	0.555503	0.11706	0.0000774	0.557620	0.559062
28.9737	0.555492	0.11792	0.0000471	0.557640	0.559088
29.7609	0.555455	0.11792	0.0000776	0.557606	0.559094
29.9036	0.555444	0.11878	0.0000471	0.557625	0.559121
30.6910	0.555407	0.11878	0.0000777	0.557591	0.559126
30.8336	0.555396	0.11964	0.0000470	0.557611	0.559153
31.6210	0.555359	0.11964	0.0000779	0.557578	0.559159
31.7636	0.555348	0.12049	0.0000470	0.557598	0.559186
32.5510	0.555310	0.12049	0.0000781	0.557564	0.559192
32.6936	0.555299	0.12134	0.0000469	0.557584	0.559219
33.4811	0.555262	0.12134	0.0000782	0.557551	0.559225
33.6236	0.555251	0.12220	0.0000469	0.557571	0.559252
34.4111	0.555214	0.12220	0.0000784	0.557538	0.559258
34.5536	0.555203	0.12305	0.0000469	0.557558	0.559286
35.3411	0.555166	0.12305	0.0000785	0.557525	0.559292
35.4835	0.555155	0.12390	0.0000468	0.557546	0.559320
36.2711	0.555118	0.12390	0.0000787	0.557512	0.559326
36.4135	0.555107	0.12475	0.0000468	0.557533	0.559354
37.2012	0.555070	0.12475	0.0000788	0.557500	0.559360
37.3435	0.555059	0.12560	0.0000467	0.557521	0.559388
38.1312	0.555022	0.12560	0.0000790	0.557488	0.559395
38.2735	0.555011	0.12645	0.0000467	0.557509	0.559423
39.0612	0.554974	0.12645	0.0000791	0.557476	0.559429
39.2035	0.554963	0.12730	0.0000466	0.557498	0.559458
39.9913	0.554926	0.12730	0.0000793	0.557465	0.559464
40.1334	0.554915	0.12815	0.0000466	0.557487	0.559493
40.9213	0.554878	0.12815	0.0000794	0.557454	0.559500
41.0634	0.554867	0.12899	0.0000465	0.557476	0.559529
41.8513	0.554830	0.12899	0.0000796	0.557443	0.559535
41.9934	0.554819	0.12984	0.0000465	0.557465	0.559564
42.7813	0.554782	0.12984	0.0000797	0.557432	0.559571
42.9234	0.554771	0.13068	0.0000464	0.557454	0.559600
43.7114	0.554734	0.13068	0.0000799	0.557422	0.559607
43.8534	0.554723	0.13153	0.0000464	0.557444	0.559637
44.6414	0.554686	0.13153	0.0000800	0.557411	0.559644
44.7834	0.554675	0.13237	0.0000463	0.557434	0.559673
45.5714	0.554639	0.13237	0.0000802	0.557401	0.559680
45.7133	0.554627	0.13321	0.0000463	0.557424	0.559710
46.5015	0.554591	0.13321	0.0000803	0.557392	0.559717
46.6433	0.554579	0.13405	0.0000462	0.557415	0.559747

47.4315	0.554543	0.13405	0.0000805	0.557383	0.559754
47.5733	0.554531	0.13489	0.0000462	0.557406	0.559784
48.3615	0.554495	0.13489	0.0000806	0.557374	0.559792
48.5033	0.554484	0.13573	0.0000461	0.557397	0.559822
49.2915	0.554447	0.13573	0.0000808	0.557365	0.559829
49.4333	0.554436	0.13657	0.0000461	0.557388	0.559860
50.2216	0.554399	0.13657	0.0000809	0.557356	0.559867
50.3633	0.554388	0.13741	0.0000460	0.557380	0.559898
51.1516	0.554352	0.13741	0.0000810	0.557348	0.559905
51.2932	0.554340	0.13824	0.0000460	0.557372	0.559936
52.0816	0.554304	0.13824	0.0000812	0.557340	0.559944
52.2232	0.554292	0.13908	0.0000459	0.557364	0.559975
53.0117	0.554256	0.13908	0.0000813	0.557332	0.559963
53.1532	0.554245	0.13991	0.0000459	0.557356	0.560014
53.9417	0.554209	0.13991	0.0000815	0.557325	0.560022
54.0832	0.554197	0.14074	0.0000458	0.557349	0.560053
54.8717	0.554161	0.14074	0.0000616	0.557317	0.560061
55.0132	0.554149	0.14158	0.0000458	0.557342	0.560093
55.8016	0.554113	0.14158	0.0000817	0.557310	0.560100
55.9431	0.554102	0.14241	0.0000457	0.557335	0.560132

The channel specifications are as follows

Channel bottom width = 0.45700
Channel side inclination to horizontal = 45.00
Channel roughness coeff. = .0150
Channel length = 55.960
Channel bed slope = 0.000100

The notch dimensions are as follows

Notch crest width = 0.0635
Notch side inclination to horizontal = 66.80
Spacing between notch centers = 0.9300
Vallue of C in the notch discharge eqtn = 0.080
Value of power x in the notch discharge eqtn = 1.8000

The water surface profile in the channel is as follows

Distance upstream (M)	Depth of flow (M)	Channel flow rate (CuM/S)	Profile slope (M/M)	Specific Energy (M)	Total energy (M)
0.0000	0.557000	0.09000	0.0001224	0.558196	0.558196
0.1442	0.556982	0.09089	0.0000987	0.558202	0.558217
0.9301	0.556905	0.09089	0.0001226	0.558127	0.558220
1.0742	0.556887	0.09177	0.0000986	0.558133	0.558241
1.8601	0.556810	0.09177	0.0001226	0.558058	0.558244
2.0042	0.556792	0.09266	0.0000986	0.558065	0.558265
2.7902	0.556714	0.09266	0.0001229	0.557990	0.558269
2.9341	0.556697	0.09354	0.0000986	0.557997	0.558290
3.7202	0.556619	0.09354	0.0001231	0.557921	0.558293
3.8641	0.556602	0.09442	0.0000986	0.557929	0.558315
4.6503	0.556524	0.09442	0.0001233	0.557853	0.558318
4.7940	0.556506	0.09529	0.0000985	0.557861	0.558340
5.5803	0.556429	0.09529	0.0001234	0.557786	0.558344
5.7240	0.556411	0.09617	0.0000985	0.557793	0.558365
6.5104	0.556334	0.09617	0.0001236	0.557718	0.558369
6.6540	0.556316	0.09704	0.0000985	0.557726	0.558391
7.4404	0.556236	0.09704	0.0001238	0.557651	0.558395
7.5839	0.556221	0.09792	0.0000984	0.557659	0.558417
8.3705	0.556143	0.09792	0.0001239	0.557584	0.558421
8.5139	0.556126	0.09879	0.0000984	0.557592	0.558443
9.3006	0.556046	0.09879	0.0001241	0.557517	0.558447
9.4439	0.556030	0.09966	0.0000984	0.557525	0.558470
10.2306	0.555953	0.09966	0.0001242	0.557450	0.558473
10.3738	0.555935	0.10052	0.0000984	0.557459	0.558496
11.1607	0.555858	0.10052	0.0001244	0.557384	0.558500
11.3038	0.555840	0.10139	0.0000983	0.557393	0.558523
12.0907	0.555763	0.10139	0.0001246	0.557318	0.558527
12.2337	0.555745	0.10225	0.0000983	0.557327	0.558550
13.0208	0.555667	0.10225	0.0001247	0.557252	0.558554
13.1637	0.555650	0.10311	0.0000983	0.557261	0.558578
13.9508	0.555572	0.10311	0.0001249	0.557187	0.558582
14.0937	0.555554	0.10397	0.0000982	0.557196	0.558605
14.8809	0.555477	0.10397	0.0001250	0.557121	0.558609
15.0236	0.555459	0.10483	0.0000982	0.557131	0.558633
15.8109	0.555382	0.10483	0.0001252	0.557056	0.558637
15.9536	0.555364	0.10569	0.0000982	0.557066	0.558661
16.7410	0.555287	0.10569	0.0001253	0.556991	0.558665
16.8836	0.555269	0.10654	0.0000981	0.557001	0.558689
17.6711	0.555192	0.10654	0.0001255	0.556927	0.558694

17 8135	0. 555174	0. 10739	0. 0000981	0. 556937	0 558719
18. 6011	0. 555096	0. 10739	0. 0001256	0. 556862	0. 558722
18. 7435	0. 555079	0. 10824	0. 0000981	0 556872	0. 558747
19 5312	0. 555001	0. 10824	0. 0001258	0. 556798	0. 558751
19. 6734	0. 554983	0. 10909	0. 0000980	0 556808	0. 558776
20 4612	0. 554906	0. 10909	0. 0001259	0 556734	0. 558780
20. 6034	0. 554888	0 10994	0. 0000980	0. 556745	0. 558805
21. 3913	0. 554811	0. 10994	0. 0001261	0. 556671	0. 558810
21. 5334	0. 554793	0. 11078	0. 0000980	0 556681	0. 558835
22 3213	0. 554716	0. 11078	0. 0001262	0. 556607	0. 558839
22. 4633	0 554698	0. 11162	0. 0000979	0. 556618	0. 558864
23 2514	0. 554621	0. 11162	0. 0001263	0. 556544	0. 558869
23 3933	0. 554603	0. 11247	0. 0000979	0. 556555	0. 558894
24 1814	0. 554526	0. 11247	0. 0001265	0. 556481	0. 558899
24 3232	0. 554508	0. 11331	0. 0000979	0. 556492	0. 558924
25. 1115	0. 554431	0. 11331	0. 0001266	0. 556418	0. 558929
25. 2532	0. 554413	0. 11414	0. 0000978	0. 556430	0. 558955
26. 0416	0. 554336	0. 11414	0. 0001268	0. 556356	0. 558960
26. 1832	0. 554318	0. 11498	0. 0000978	0. 556367	0 558986
26 9716	0. 554241	0. 11498	0. 0001269	0 556294	0. 558991
27. 1131	0. 554223	0. 11581	0. 0000978	0. 556305	0. 559017
27 9017	0. 554146	0. 11581	0. 0001270	0. 556232	0. 559022
28. 0431	0. 554128	0. 11665	0 0000977	0. 556243	0. 559048
28. 8317	0. 554051	0. 11665	0. 0001272	0. 556170	0. 559053
28. 9731	0. 554033	0. 11748	0. 0000977	0. 556182	0. 559079
29. 7618	0. 553955	0. 11748	0. 0001273	0. 556108	0. 559084
29. 9030	0. 553937	0. 11830	0. 0000977	0. 556120	0. 559111
30. 6918	0. 553860	0. 11830	0. 0001274	0. 556047	0. 559116
30 8330	0. 553842	0. 11913	0. 0000976	0. 556059	0. 559142
31. 6219	0. 553765	0. 11913	0. 0001276	0. 555986	0. 559146
31. 7629	0. 553747	0. 11996	0. 0000976	0. 555998	0. 559175
32. 5519	0. 553670	0. 11996	0. 0001277	0. 555925	0. 559180
32. 6929	0. 553652	0. 12078	0. 0000976	0. 555938	0. 559207
33. 4820	0. 553575	0. 12078	0. 0001278	0. 555864	0. 559213
33. 6229	0. 553557	0. 12160	0. 0000975	0. 555877	0. 559239
34. 4121	0. 553480	0. 12160	0. 0001279	0. 555804	0. 559245
34 5528	0. 553462	0. 12242	0. 0000975	0. 555817	0. 559272
35. 3421	0. 553385	0. 12242	0. 0001281	0. 555744	0. 559276
35. 4828	0. 553367	0. 12324	0. 0000975	0. 555757	0. 559305
36. 2722	0. 553290	0. 12324	0. 0001282	0. 555684	0. 559311
36. 4128	0. 553272	0. 12405	0. 0000974	0. 555697	0. 559338
37. 2022	0. 553196	0. 12405	0. 0001283	0. 555624	0. 559345
37. 3427	0. 553178	0. 12487	0. 0000974	0. 555638	0. 559372
38. 1323	0. 553101	0. 12487	0. 0001284	0. 555565	0. 559376
36. 2727	0. 553083	0. 12568	0. 0000974	0. 555578	0. 559406
39. 0623	0. 553006	0. 12568	0. 0001285	0. 555506	0. 559412
39. 2026	0. 552988	0. 12649	0. 0000973	0. 555519	0. 559440
39. 9924	0. 552911	0. 12649	0. 0001287	0. 555447	0. 559446
40. 1326	0. 552893	0. 12730	0. 0000973	0. 555460	0. 559474
40. 9224	0. 552816	0. 12730	0. 0001288	0. 555388	0. 559480
41. 0626	0. 552798	0. 12811	0. 0000972	0. 555402	0. 559508
41. 8525	0. 552721	0. 12811	0. 0001289	0. 555329	0. 559515
41. 9925	0. 552703	0. 12891	0. 0000972	0. 555343	0. 559543
42. 7826	0. 552626	0. 12891	0 0001290	0. 555271	0. 559549
42. 9225	0. 552608	0. 12972	0. 0000972	0. 555285	0. 559576
43. 7126	0. 552531	0. 12972	0. 0001291	0. 555213	0. 559584
43. 8525	0. 552513	0. 13052	0. 0000971	0. 555227	0. 559613
44. 6427	0. 552437	0. 13052	0. 0001292	0. 555155	0. 559619
44. 7824	0. 552419	0. 13132	0. 0000971	0. 555170	0. 559648
45. 5727	0. 552342	0. 13132	0. 0001293	0. 555098	0. 559655
45 7124	0. 552324	0. 13212	0. 0000971	0. 555112	0. 559684
46 5028	0. 552247	0. 13212	0. 0001295	0. 555040	0. 559690
46. 6423	0. 552229	0. 13292	0. 0000970	0. 555055	0 559719

47.4328	0.552152	0.13292	0.0001296	0.554983	0.559726
47.5723	0.552134	0.13371	0.0000970	0.554998	0.559755
48.3629	0.552058	0.13371	0.0001297	0.554926	0.559762
48.5023	0.552040	0.13450	0.0000969	0.554941	0.559791
49.2929	0.551963	0.13450	0.0001298	0.554869	0.559759
49.4322	0.551945	0.13529	0.0000969	0.554885	0.559828
50.2230	0.551868	0.13529	0.0001299	0.554813	0.559835
50.3622	0.551850	0.13608	0.0000969	0.554828	0.559865
51.1531	0.551774	0.13608	0.0001300	0.554757	0.559872
51.2922	0.551755	0.13687	0.0000968	0.554772	0.559902
52.0831	0.551679	0.13687	0.0001301	0.554701	0.559909
52.2221	0.551661	0.13766	0.0000968	0.554717	0.559939
53.0132	0.551584	0.13766	0.0001302	0.554645	0.559946
53.1521	0.551566	0.13844	0.0000967	0.554661	0.559976
53.9432	0.551490	0.13844	0.0001303	0.554589	0.559984
54.0820	0.551472	0.13923	0.0000967	0.554606	0.560014
54 8733	0.551395	0.13923	0.0001304	0.554534	0.560021
55.0120	0.551377	0.14001	0.0000967	0.554550	0.560052
55 8033	0.551301	0.14001	0.0001305	0.554479	0.560059
55.9420	0.551283	0.14079	0.0000966	0.554495	0.560090

The channel specifications are as follows

Channel bottom width = 0.45700
Channel side inclination to horizontal = 45.00
Channel roughness coeff = .0150
Channel length = 55.960
Channel bed slope = 0.000000

The notch dimensions are as follows

Notch crest width = 0.0635
Notch side inclination to horizontal = 66.80
Spacing between notch centers = 0.9300
Vallue of C in the notch discharge eqtn = 0.080
Value of power x in the notch discharge eqtn = 1.8000

The water surface profile in the channel is as follows

Distance upstream (M)	Depth of flow (M)	Channel flow rate (CuM/S)	Profile slope (M/M)	Specific Energy (M)	Total energy (M)
0.0000	0.557000	0.09000	0.0000217	0.558196	0.558196
0.1442	0.556997	0.09089	-.0000020	0.558217	0.558217
0.9300	0.556998	0.09089	0.0000219	0.558220	0.558220
1.0742	0.556995	0.09177	-.0000021	0.558241	0.558241
1.8600	0.556997	0.09177	0.0000221	0.558244	0.558244
2.0042	0.556994	0.09266	-.0000021	0.558265	0.558265
2.7900	0.556995	0.09266	0.0000223	0.558269	0.558269
2.9342	0.556992	0.09355	-.0000021	0.558290	0.558290
3.7200	0.556994	0.09355	0.0000225	0.558293	0.558293
3.8642	0.556991	0.09444	-.0000022	0.558315	0.558315
4.6500	0.556992	0.09444	0.0000227	0.558319	0.558319
4.7942	0.556969	0.09532	-.0000022	0.558340	0.558340
5.5800	0.556991	0.09532	0.0000229	0.558344	0.558344
5.7242	0.556987	0.09621	-.0000023	0.558366	0.558366
6.5100	0.556989	0.09621	0.0000231	0.558370	0.558370
6.6542	0.556986	0.09710	-.0000023	0.558392	0.558392
7.4400	0.556988	0.09710	0.0000233	0.558396	0.558396
7.5842	0.556984	0.09798	-.0000024	0.558416	0.558416
8.3700	0.556986	0.09798	0.0000235	0.558422	0.558422
8.5142	0.556983	0.09887	-.0000024	0.558444	0.558444
9.3000	0.556985	0.09887	0.0000237	0.558448	0.558448
9.4442	0.556981	0.09976	-.0000024	0.558471	0.558471
10.2300	0.556983	0.09976	0.0000239	0.558475	0.558475
10.3742	0.556980	0.10064	-.0000025	0.558498	0.558498
11.1600	0.556982	0.10064	0.0000241	0.558502	0.558502
11.3042	0.556978	0.10153	-.0000025	0.558526	0.558526
12.0900	0.556980	0.10153	0.0000243	0.558529	0.558529
12.2343	0.556977	0.10242	-.0000026	0.558553	0.558553
13.0200	0.556979	0.10242	0.0000245	0.558557	0.558557
13.1643	0.556975	0.10330	-.0000026	0.558581	0.558581
13.9500	0.556977	0.10330	0.0000247	0.558585	0.558585
14.0943	0.556974	0.10419	-.0000027	0.558609	0.558609
14.8800	0.556976	0.10419	0.0000249	0.558613	0.558613
15.0243	0.556972	0.10508	-.0000027	0.558637	0.558637
15.8100	0.556974	0.10508	0.0000250	0.558642	0.558642
15.9543	0.556971	0.10596	-.0000028	0.558666	0.558666
16.7400	0.556973	0.10596	0.0000252	0.558670	0.558670
16.8843	0.556969	0.10685	-.0000028	0.558695	0.558695
17.6700	0.556971	0.10685	0.0000254	0.558699	0.558699

17.8143	0.556968	0.10774	-.0000029	0.558724	0.558724
18.6001	0.556970	0.10774	0.0000256	0.558729	0.558729
18.7443	0.556966	0.10862	-.0000029	0.558754	0.558754
19.5301	0.556969	0.10862	0.0000258	0.558758	0.558758
19.6743	0.556965	0.10951	-.0000030	0.558784	0.558784
20.4601	0.556967	0.10951	0.0000260	0.558788	0.558788
20.6043	0.556964	0.11040	-.0000030	0.558814	0.558814
21.3901	0.556966	0.11040	0.0000262	0.558818	0.558818
21.5343	0.556962	0.11128	-.0000030	0.558844	0.558844
22.3201	0.556965	0.11128	0.0000264	0.558849	0.558849
22.4643	0.556961	0.11217	-.0000031	0.558875	0.558875
23.2501	0.556963	0.11217	0.0000266	0.558879	0.558879
23.3943	0.556959	0.11305	-.0000031	0.558906	0.558906
24.1801	0.556962	0.11305	0.0000268	0.558910	0.558910
24.3243	0.556958	0.11394	-.0000032	0.558937	0.558937
25.1101	0.556960	0.11394	0.0000270	0.558942	0.558942
25.2543	0.556957	0.11483	-.0000032	0.558968	0.558968
26.0401	0.556959	0.11483	0.0000271	0.558973	0.558973
26.1843	0.556955	0.11571	-.0000033	0.559000	0.559000
26.9701	0.556958	0.11571	0.0000273	0.559005	0.559005
27.1143	0.556954	0.11660	-.0000034	0.559032	0.559032
27.9001	0.556956	0.11660	0.0000275	0.559037	0.559037
28.0443	0.556952	0.11749	-.0000034	0.559065	0.559065
28.8301	0.556955	0.11749	0.0000277	0.559070	0.559070
28.9743	0.556951	0.11837	-.0000035	0.559097	0.559097
29.7601	0.556954	0.11837	0.0000279	0.559103	0.559103
29.9043	0.556950	0.11926	-.0000035	0.559130	0.559130
30.6901	0.556953	0.11926	0.0000281	0.559136	0.559136
30.8343	0.556949	0.12014	-.0000036	0.559164	0.559164
31.6201	0.556951	0.12014	0.0000283	0.559169	0.559169
31.7643	0.556947	0.12103	-.0000036	0.559197	0.559197
32.5501	0.556950	0.12103	0.0000285	0.559203	0.559203
32.6943	0.556946	0.12192	-.0000037	0.559231	0.559231
33.4801	0.556949	0.12192	0.0000286	0.559237	0.559237
33.6243	0.556945	0.12280	-.0000037	0.559265	0.559265
34.4101	0.556946	0.12280	0.0000288	0.559271	0.559271
34.5543	0.556944	0.12369	-.0000038	0.559300	0.559300
35.3401	0.556947	0.12369	0.0000290	0.559305	0.559305
35.4843	0.556942	0.12457	-.0000038	0.559334	0.559334
36.2701	0.556945	0.12457	0.0000292	0.559340	0.559340
36.4143	0.556941	0.12546	-.0000039	0.559369	0.559369
37.2001	0.556944	0.12546	0.0000294	0.559376	0.559376
37.3443	0.556940	0.12635	-.0000039	0.559405	0.559405
38.1301	0.556943	0.12635	0.0000296	0.559411	0.559411
38.2743	0.556939	0.12723	-.0000040	0.559440	0.559440
39.0601	0.556942	0.12723	0.0000297	0.559447	0.559447
39.2043	0.556938	0.12812	-.0000041	0.559476	0.559476
39.9901	0.556941	0.12812	0.0000299	0.559483	0.559483
40.1343	0.556937	0.12900	-.0000041	0.559513	0.559513
40.9201	0.556940	0.12900	0.0000301	0.559519	0.559519
41.0643	0.556935	0.12989	-.0000042	0.559549	0.559549
41.8501	0.556939	0.12989	0.0000303	0.559556	0.559556
41.9943	0.556934	0.13078	-.0000042	0.559586	0.559586
42.7801	0.556938	0.13078	0.0000305	0.559593	0.559593
42.9243	0.556933	0.13166	-.0000043	0.559623	0.559623
43.7101	0.556937	0.13166	0.0000307	0.559630	0.559630
43.8543	0.556932	0.13255	-.0000043	0.559661	0.559661
44.6401	0.556936	0.13255	0.0000308	0.559667	0.559667
44.7843	0.556931	0.13343	-.0000044	0.559698	0.559698
45.5701	0.556935	0.13343	0.0000310	0.559705	0.559705
45.7143	0.556930	0.13432	-.0000045	0.559736	0.559736
46.5001	0.556934	0.13432	0.0000312	0.559743	0.559743
46.6443	0.556929	0.13520	-.0000045	0.559775	0.559775

47.4301	0.556933	0.13520	0.0000314	0.559782	0.559762
47.5743	0.556928	0.13609	-.0000046	0.559814	0.559814
48.3601	0.556932	0.13609	0.0000316	0.559821	0.559821
48.5043	0.556927	0.13698	-.0000046	0.559853	0.559853
49.2901	0.556931	0.13698	0.0000318	0.559860	0.559860
49.4343	0.556926	0.13786	-.0000047	0.559892	0.559892
50.2201	0.556930	0.13786	0.0000319	0.559899	0.559899
50.3643	0.556925	0.13875	-.0000048	0.559932	0.559932
51.1502	0.556929	0.13875	0.0000321	0.559939	0.559939
51.2943	0.556925	0.13963	-.0000048	0.559971	0.559971
52.0802	0.556928	0.13963	0.0000323	0.559979	0.559979
52.2243	0.556924	0.14052	-.0000049	0.560012	0.560012
53.0102	0.556928	0.14052	0.0000325	0.560019	0.560019
53.1543	0.556923	0.14140	-.0000050	0.560052	0.560052
53.9402	0.556927	0.14140	0.0000327	0.560060	0.560060
54.0843	0.556922	0.14229	-.0000050	0.560093	0.560093
54.8702	0.556926	0.14229	0.0000328	0.560101	0.560101
55.0143	0.556921	0.14318	-.0000051	0.560134	0.560134
55.8002	0.556925	0.14318	0.0000330	0.560142	0.560142
55.9443	0.556920	0.14406	-.0000051	0.560176	0.560176

The channel specifications are as follows

Channel bottom width = 0.45700
Channel side inclination to horizontal = 45.00
Channel roughness coeff = .0150
Channel length = 55.960
Channel bed slope = -.000050

The notch dimensions are as follows

Notch crest width = 0.0635
Notch side inclination to horizontal = 66.80
Spacing between notch centers = 0.9300
Vallue of C in the notch discharge eqtn = 0.080
Value of power x in the notch discharge eqtn = 1.8000

The water surface profile in the channel is as follows

Distance upstream (M)	Depth of flow (M)	Channel flow rate (CuM/S)	Profile slope (M/M)	Specific Energy (M)	Total energy (M)
0.0000	0.557000	0.09000	-.0000286	0.558196	0.560994
0.1442	0.557004	0.09089	-.0000524	0.558224	0.561015
0.9300	0.557045	0.09089	-.0000284	0.558266	0.561018
1.0743	0.557049	0.09178	-.0000524	0.558295	0.561039
1.8600	0.557091	0.09178	-.0000282	0.558337	0.561042
2.0043	0.557095	0.09266	-.0000525	0.558366	0.561063
2.7899	0.557136	0.09266	-.0000280	0.558408	0.561067
2.9343	0.557140	0.09355	-.0000525	0.558437	0.561088
3.7199	0.557181	0.09355	-.0000278	0.558480	0.561092
3.8643	0.557185	0.09444	-.0000526	0.558509	0.561113
4.6499	0.557226	0.09444	-.0000275	0.558551	0.561117
4.7943	0.557230	0.09534	-.0000526	0.558580	0.561139
5.5799	0.557272	0.09534	-.0000273	0.558623	0.561142
5.7244	0.557276	0.09623	-.0000527	0.558653	0.561164
6.5098	0.557317	0.09623	-.0000271	0.558695	0.561168
6.6544	0.557321	0.09712	-.0000527	0.558725	0.561190
7.4398	0.557362	0.09712	-.0000269	0.558768	0.561194
7.5844	0.557366	0.09802	-.0000528	0.558798	0.561217
8.3698	0.557408	0.09802	-.0000267	0.558841	0.561220
8.5144	0.557412	0.09891	-.0000528	0.558871	0.561243
9.2998	0.557453	0.09891	-.0000265	0.558914	0.561247
9.4444	0.557457	0.09981	-.0000529	0.558944	0.561270
10.2297	0.557498	0.09981	-.0000263	0.558987	0.561274
10.3745	0.557502	0.10070	-.0000529	0.559018	0.561297
11.1597	0.557544	0.10070	-.0000261	0.559061	0.561301
11.3045	0.557547	0.10160	-.0000530	0.559092	0.561325
12.0897	0.557589	0.10160	-.0000259	0.559135	0.561329
12.2345	0.557593	0.10250	-.0000530	0.559166	0.561353
13.0197	0.557634	0.10250	-.0000256	0.559209	0.561356
13.1645	0.557638	0.10340	-.0000531	0.559241	0.561380
13.9496	0.557680	0.10340	-.0000254	0.559284	0.561385
14.0945	0.557683	0.10430	-.0000531	0.559316	0.561409
14.8796	0.557725	0.10430	-.0000252	0.559359	0.561413
15.0246	0.557729	0.10520	-.0000532	0.559391	0.561438
15.8096	0.557771	0.10520	-.0000250	0.559434	0.561442
15.9546	0.557774	0.10610	-.0000532	0.559466	0.561467
16.7396	0.557816	0.10610	-.0000248	0.559510	0.561471
16.8846	0.557820	0.10701	-.0000533	0.559542	0.561496
17.6695	0.557861	0.10701	-.0000246	0.559586	0.561500

17.8146	0.557865	0.10791	-.0000533	0.559618	0.561525
18.5995	0.557907	0.10791	-.0000244	0.559662	0.561530
18.7446	0.557910	0.10881	-.0000534	0.559694	0.561555
19.5295	0.557952	0.10881	-.0000242	0.559738	0.561560
19.6747	0.557956	0.10972	-.0000534	0.559771	0.561585
20.4595	0.557998	0.10972	-.0000239	0.559815	0.561590
20.6047	0.558001	0.11063	-.0000535	0.559848	0.561616
21.3894	0.558043	0.11063	-.0000237	0.559892	0.561621
21.5347	0.558047	0.11153	-.0000536	0.559925	0.561647
22.3194	0.558089	0.11153	-.0000235	0.559969	0.561651
22.4647	0.558092	0.11244	-.0000536	0.560003	0.561678
23.2494	0.558134	0.11244	-.0000233	0.560047	0.561683
23.3947	0.558137	0.11335	-.0000537	0.560081	0.561709
24.1794	0.558180	0.11335	-.0000231	0.560125	0.561714
24.3248	0.558183	0.11426	-.0000537	0.560159	0.561741
25.1093	0.558225	0.11426	-.0000229	0.560203	0.561746
25.2548	0.558228	0.11517	-.0000538	0.560238	0.561773
26.0393	0.558271	0.11517	-.0000227	0.560282	0.561778
26.1848	0.558274	0.11608	-.0000538	0.560317	0.561805
26.9693	0.558316	0.11608	-.0000224	0.560361	0.561811
27.1148	0.558319	0.11700	-.0000539	0.560396	0.561838
27.8993	0.558362	0.11700	-.0000222	0.560440	0.561843
28.0449	0.558365	0.11791	-.0000540	0.560475	0.561871
28.8293	0.558407	0.11791	-.0000220	0.560520	0.561876
28.9749	0.558410	0.11683	-.0000540	0.560555	0.561905
29.7592	0.558453	0.11883	-.0000218	0.560600	0.561910
29.9049	0.558456	0.11974	-.0000541	0.560635	0.561936
30.6892	0.558498	0.11974	-.0000216	0.560680	0.561944
30.8349	0.558502	0.12066	-.0000542	0.560716	0.561972
31.6192	0.558544	0.12066	-.0000214	0.560761	0.561978
31.7649	0.558547	0.12158	-.0000542	0.560797	0.562007
32.5492	0.558590	0.12158	-.0000211	0.560842	0.562012
32.6950	0.558593	0.12249	-.0000543	0.560878	0.562041
33.4791	0.558635	0.12249	-.0000209	0.560923	0.562047
33.6250	0.558638	0.12341	-.0000543	0.560959	0.562076
34.4091	0.558681	0.12341	-.0000207	0.561004	0.562082
34.5550	0.558684	0.12433	-.0000544	0.561041	0.562111
35.3391	0.558727	0.12433	-.0000205	0.561086	0.562117
35.4850	0.558730	0.12525	-.0000545	0.561123	0.562147
36.2691	0.558772	0.12525	-.0000203	0.561169	0.562153
36.4150	0.558775	0.12618	-.0000545	0.561206	0.562183
37.1990	0.558818	0.12618	-.0000201	0.561251	0.562189
37.3451	0.558821	0.12710	-.0000546	0.561289	0.562219
38.1290	0.558864	0.12710	-.0000198	0.561334	0.562226
38.2751	0.558867	0.12802	-.0000547	0.561372	0.562256
39.0590	0.558910	0.12802	-.0000196	0.561417	0.562262
39.2051	0.558913	0.12895	-.0000547	0.561455	0.562293
39.9890	0.558955	0.12895	-.0000194	0.561501	0.562299
40.1351	0.558958	0.12987	-.0000548	0.561539	0.562330
40.9189	0.559001	0.12987	-.0000192	0.561585	0.562337
41.0652	0.559004	0.13080	-.0000549	0.561623	0.562368
41.8489	0.559047	0.13080	-.0000190	0.561669	0.562375
41.9952	0.559050	0.13173	-.0000549	0.561708	0.562406
42.7789	0.559093	0.13173	-.0000187	0.561754	0.562413
42.9252	0.559096	0.13265	-.0000550	0.561793	0.562444
43.7089	0.559139	0.13265	-.0000185	0.561838	0.562451
43.8552	0.559141	0.13358	-.0000551	0.561878	0.562483
44.6389	0.559185	0.13358	-.0000183	0.561924	0.562490
44.7852	0.559187	0.13451	-.0000552	0.561963	0.562522
45.5688	0.559231	0.13451	-.0000181	0.562009	0.562529
45.7153	0.559233	0.13544	-.0000552	0.562049	0.562561
46.4988	0.559276	0.13544	-.0000179	0.562095	0.562568
46.6453	0.559279	0.13638	-.0000553	0.562135	0.562601

47.4288	0.559322	0.13638	-.0000176	0.562182	0.562608
47.5753	0.559325	0.13731	-.0000554	0.562222	0.562641
48.3588	0.559368	0.13731	-.0000174	0.562268	0.562648
48.5053	0.559371	0.13824	-.0000554	0.562309	0.562682
49.2887	0.559414	0.13824	-.0000172	0.562355	0.562689
49.4353	0.559417	0.13918	-.0000555	0.562396	0.562722
50.2187	0.559460	0.13918	-.0000170	0.562443	0.562730
50.3654	0.559463	0.14011	-.0000556	0.562484	0.562764
51.1487	0.559506	0.14011	-.0000168	0.562530	0.562771
51.2954	0.559509	0.14105	-.0000557	0.562572	0.562805
52.0787	0.559552	0.14105	-.0000165	0.562618	0.562812
52.2254	0.559555	0.14199	-.0000557	0.562660	0.562847
53.0086	0.559599	0.14199	-.0000163	0.562707	0.562854
53.1554	0.559601	0.14293	-.0000558	0.562749	0.562869
53.9386	0.559645	0.14293	-.0000161	0.562795	0.562897
54.0855	0.559647	0.14386	-.0000559	0.562838	0.562931
54.8686	0.559691	0.14386	-.0000159	0.562885	0.562939
55.0155	0.559693	0.14480	-.0000560	0.562927	0.562974
55.7986	0.559737	0.14480	-.0000156	0.562974	0.562982
55.9455	0.559739	0.14575	-.0000560	0.563017	0.563016

The channel specifications are as follows

Channel bottom width = 0.45700
Channel side inclination to horizontal = 45.00
Channel roughness coeff = .0150
Channel length = 55.960
Channel bed slope = -.000100

The notch dimensions are as follows

Notch crest width = 0.0635
Notch side inclination to horizontal = 66.80
Spacing between notch centers = 0.9300
Vallue of C in the notch discharge eqtn = 0.080
Value of power x in the notch discharge eqtn = 1.8000

The water surface profile in the channel is as follows

Distance upstream (M)	Depth of flow (M)	Channel flow rate (CuM/S)	Profile slope (M/M)	Specific Energy (M)	Total energy (M)
0.0000	0.557000	0.09000	-.0000789	0.558196	0.563792
0.1442	0.557011	0.09089	-.0001027	0.558231	0.563813
0.9299	0.557092	0.09089	-.0000787	0.558313	0.563816
1.0743	0.557103	0.09178	-.0001028	0.558348	0.563837
1.8599	0.557184	0.09178	-.0000785	0.558430	0.563840
2.0043	0.557196	0.09267	-.0001028	0.558466	0.563861
2.7898	0.557276	0.09267	-.0000783	0.558548	0.563865
2.9344	0.557288	0.09356	-.0001029	0.558584	0.563886
3.7198	0.557368	0.09356	-.0000780	0.558666	0.563890
3.8644	0.557380	0.09445	-.0001029	0.558702	0.563911
4.6497	0.557461	0.09445	-.0000778	0.558784	0.563915
4.7944	0.557472	0.09535	-.0001030	0.558820	0.563937
5.5797	0.557553	0.09535	-.0000776	0.558902	0.563940
5.7245	0.557564	0.09625	-.0001030	0.558939	0.563963
6.5096	0.557645	0.09625	-.0000774	0.559021	0.563966
6.6545	0.557656	0.09715	-.0001031	0.559058	0.563989
7.4396	0.557737	0.09715	-.0000771	0.559140	0.563992
7.5846	0.557748	0.09805	-.0001031	0.559178	0.564015
8.3695	0.557829	0.09805	-.0000769	0.559260	0.564019
8.5146	0.557840	0.09895	-.0001032	0.559297	0.564042
9.2995	0.557921	0.09895	-.0000767	0.559380	0.564046
9.4446	0.557932	0.09986	-.0001033	0.559417	0.564069
10.2294	0.558013	0.09986	-.0000765	0.559500	0.564073
10.3747	0.558025	0.10076	-.0001033	0.559538	0.564096
11.1594	0.558106	0.10076	-.0000762	0.559620	0.564100
11.3047	0.558117	0.10167	-.0001034	0.559658	0.564124
12.0893	0.558198	0.10167	-.0000760	0.559741	0.564128
12.2348	0.558209	0.10258	-.0001034	0.559779	0.564152
13.0193	0.558290	0.10258	-.0000758	0.559862	0.564156
13.1648	0.558301	0.10350	-.0001035	0.559901	0.564160
13.9492	0.558382	0.10350	-.0000755	0.559983	0.564184
14.0948	0.558393	0.10441	-.0001036	0.560022	0.564209
14.8792	0.558474	0.10441	-.0000753	0.560105	0.564213
15.0249	0.558485	0.10533	-.0001036	0.560144	0.564238
15.8091	0.558567	0.10533	-.0000751	0.560227	0.564242
15.9549	0.558578	0.10624	-.0001037	0.560266	0.564267
16.7391	0.558659	0.10624	-.0000748	0.560349	0.564271
16.8850	0.558670	0.10716	-.0001037	0.560389	0.564297
17.6690	0.558751	0.10716	-.0000746	0.560472	0.564301

17.8150	0.558762	0.10808	-.0001038	0.560512	0.564327
18.5990	0.558843	0.10808	-.0000744	0.560595	0.564331
18.7450	0.558854	0.10901	-.0001039	0.560635	0.564357
19.5289	0.558936	0.10901	-.0000741	0.560718	0.564361
19.6751	0.558947	0.10993	-.0001039	0.560759	0.564387
20.4589	0.559026	0.10993	-.0000739	0.560842	0.564392
20.6051	0.559039	0.11086	-.0001040	0.560883	0.564418
21.3888	0.559120	0.11086	-.0000736	0.560966	0.564423
21.5352	0.559131	0.11179	-.0001041	0.561007	0.564450
22.3168	0.559213	0.11179	-.0000734	0.561090	0.564454
22.4652	0.559223	0.11272	-.0001041	0.561132	0.564481
23.2467	0.559305	0.11272	-.0000732	0.561215	0.564486
23.3952	0.559316	0.11365	-.0001042	0.561257	0.564513
24.1787	0.559397	0.11365	-.0000729	0.561340	0.564518
24.3253	0.559408	0.11459	-.0001043	0.561382	0.564545
25.1066	0.559490	0.11459	-.0000727	0.561465	0.564550
25.2553	0.559500	0.11552	-.0001043	0.561508	0.564578
26.0366	0.559582	0.11552	-.0000724	0.561591	0.564583
26.1854	0.559593	0.11646	-.0001044	0.561633	0.564611
26.9685	0.559675	0.11646	-.0000722	0.561717	0.564616
27.1154	0.559685	0.11740	-.0001045	0.561760	0.564644
27.8985	0.559767	0.11740	-.0000720	0.561843	0.564649
28.0454	0.559777	0.11834	-.0001045	0.561886	0.564676
28.8284	0.559859	0.11834	-.0000717	0.561970	0.564682
28.9755	0.559870	0.11928	-.0001046	0.562014	0.564712
29.7584	0.559952	0.11928	-.0000715	0.562097	0.564717
29.9055	0.559962	0.12023	-.0001047	0.562141	0.564746
30.6883	0.560044	0.12023	-.0000712	0.562225	0.564752
30.8356	0.560055	0.12118	-.0001047	0.562269	0.564781
31.6183	0.560137	0.12118	-.0000710	0.562352	0.564786
31.7656	0.560147	0.12213	-.0001048	0.562397	0.564816
32.5482	0.560229	0.12213	-.0000707	0.562480	0.564822
32.6956	0.560240	0.12308	-.0001049	0.562525	0.564852
33.4782	0.560322	0.12308	-.0000705	0.562609	0.564857
33.6257	0.560332	0.12403	-.0001050	0.562654	0.564887
34.4081	0.560414	0.12403	-.0000702	0.562738	0.564893
34.5557	0.560425	0.12498	-.0001050	0.562783	0.564923
35.3381	0.560507	0.12498	-.0000700	0.562867	0.564929
35.4858	0.560517	0.12594	-.0001051	0.562913	0.564960
36.2680	0.560599	0.12594	-.0000697	0.562997	0.564966
36.4158	0.560610	0.12690	-.0001052	0.563042	0.564997
37.1980	0.560692	0.12690	-.0000695	0.563127	0.565003
37.3458	0.560702	0.12786	-.0001053	0.563173	0.565034
38.1279	0.560784	0.12786	-.0000692	0.563257	0.565040
38.2759	0.560795	0.12882	-.0001053	0.563303	0.565072
39.0579	0.560877	0.12882	-.0000690	0.563388	0.565078
39.2059	0.560887	0.12979	-.0001054	0.563434	0.565110
39.9878	0.560970	0.12979	-.0000687	0.563519	0.565116
40.1360	0.560980	0.13075	-.0001055	0.563566	0.565148
40.9178	0.561062	0.13075	-.0000685	0.563650	0.565155
41.0660	0.561072	0.13172	-.0001056	0.563698	0.565167
41.8477	0.561155	0.13172	-.0000682	0.563782	0.565193
41.9960	0.561165	0.13269	-.0001057	0.563830	0.565226
42.7777	0.561248	0.13269	-.0000680	0.563914	0.565233
42.9261	0.561258	0.13366	-.0001057	0.563962	0.565266
43.7076	0.561340	0.13366	-.0000677	0.564047	0.565272
43.8561	0.561351	0.13463	-.0001058	0.564095	0.565306
44.6376	0.561433	0.13463	-.0000674	0.564180	0.565312
44.7862	0.561443	0.13561	-.0001059	0.564228	0.565346
45.5675	0.561526	0.13561	-.0000672	0.564313	0.565353
45.7162	0.561536	0.13659	-.0001060	0.564362	0.565386
46.4975	0.561619	0.13659	-.0000669	0.564447	0.565393
46.6463	0.561629	0.13757	-.0001061	0.564496	0.565428

47.4274	0.561712	0.13757	-.0000667	0.564581	0.565434
47.5763	0.561721	0.13855	-.0001061	0.564631	0.565469
48.3574	0.561804	0.13855	-.0000664	0.564716	0.565476
48.5063	0.561814	0.13953	-.0001062	0.564765	0.565511
49.2873	0.561897	0.13953	-.0000661	0.564851	0.565518
49.4364	0.561907	0.14051	-.0001063	0.564901	0.565553
50.2173	0.561990	0.14051	-.0000659	0.564986	0.565560
50.3664	0.562000	0.14150	-.0001064	0.565036	0.565596
51.1472	0.562083	0.14150	-.0000656	0.565122	0.565603
51.2965	0.562093	0.14249	-.0001065	0.565172	0.565639
52.0772	0.562176	0.14249	-.0000653	0.565258	0.565646
52.2265	0.562186	0.14348	-.0001066	0.565309	0.565682
53.0071	0.562269	0.14348	-.0000651	0.565394	0.565690
53.1565	0.562279	0.14447	-.0001067	0.565446	0.565726
53.9371	0.562362	0.14447	-.0000648	0.565531	0.565734
54.0866	0.562372	0.14547	-.0001068	0.565563	0.565770
54.8670	0.562455	0.14547	-.0000645	0.565669	0.565778
55.0166	0.562465	0.14646	-.0001068	0.565720	0.565815
55.7970	0.562548	0.14646	-.0000643	0.565806	0.565823
55.9467	0.562558	0.14746	-.0001069	0.565859	0.565860

APPENDIX C

TRIAL AND ERROR PROCEEDURE FOR CALCULATION

OF THE IDEAL DISCHARGE

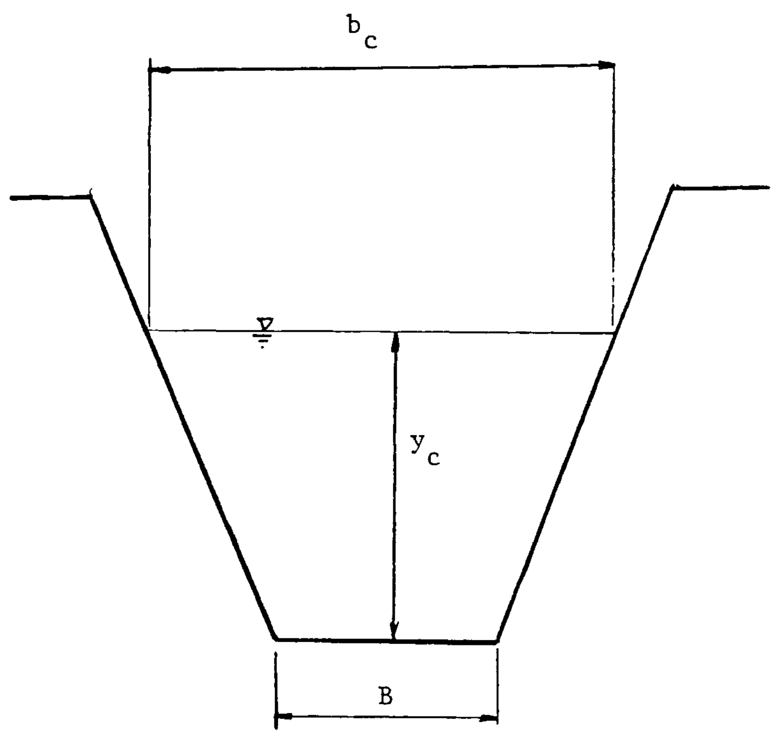


Fig. C.1. Cross Section of the Side Weir at the Location of Critical Depth Occurrence (Control Section).

In order to calculate the ideal discharge for a given head above the weir crest

1. Estimate critical depth as $Y_c = 0.7H$, where H is the hydraulic head on the weir.

2. Calculate top width, b_c, and the critical area, A_c.

3. Calculate the ideal discharge using

$$Q^2 = A_c^2\ Y_c\ g$$

4. Calculate the specific head corresponding to Q

$$H_1 = Y_c + Q^2/2g\ A^2_c$$

5. Check for the accuracy of the estimated Y_c value. If $(H - H_1) / H$ is less than the required accuracy level then Q is correct, if not calculate a new trial value for Y_c

$$Y_c = H - Q^2 / 2g\ A^2_c$$

6. Repeat steps 2 - 5 until the required accuracy level is reached.

The first estimate of Y_c usually converges to the true value of Y_c in two or three iterations.

APPENDIX D

SOME OF THE DATA COLLECTED

Field Data

Table D.1. Discharge measurements on the weirs serving the cotton field at full flow.

Weir No. (-)	Av. Head on the weir (Cm)	Measured Discharge (l/s)
4	-----	3.115
6	9.350	3.718
8	8.450	3.410
11	7.350	2.706
13	6.900	2.257
16	7.900	3.447
19	8.150	3.447
21	7.700	3.302
23	8.100	3.619
25	8.500	3.942
28	8.250	3.644
30	8.100	2.840
33	8.000	3.531
35	8.050	3.061
38	8.050	3.744
40	8.000	3.145
43	8.150	3.233
45	7.950	3.379
48	8.050	3.623
51	8.200	3.440
53	8.300	3.398
55	8.600	3.655
57	8.100	3.669
59	8.100	3.302
62	8.750	3.847
64	8.600	3.795

Table D.2. Discharge measurments on the weirs serving the cotton field at cut-back flow.

Weir No. (-)	Av. Head on the Weir (Cm)	Measured Discharge (l/s)
4	-----	2.424
8	7.100	2.780
13	5.300	1.062
19	6.250	1.790
23	5.950	1.749
28	6.000	1.896
33	5.800	1.780
38	5.880	2.058
43	5.750	1.769
48	5.850	1.983
53	5.880	1.913
57	5.600	1.895
62	6.300	2.095

Table D.3. Crest elevations of the weirs serving the cotton field.

Weir No. (-)	Crest Elevation (m)	Weir No. (-)	Crest Elevation (m)
1	6.140	2	6.150
3	6.145	4	6.140
5	6.143	6	6.153
7	6.142	8	6.145
9	6.142	10	6.138
11	6.135	12	6.140
13	6.140	14	6.143
15	6.140	16	6.137
17	6.135	18	6.137
19	6.149	20	6.133
21	6.135	22	6.142
23	6.140	24	6.143
25	6.150	26	6.133
27	6.145	28	6.148
29	6.155	30	6.143
31	6.154	32	6.143
33	6.143	34	6.140
35	6.138	36	6.153

continued-----

Table D.3. Continued

Weir No. (-)	Crest Elevation (m)	Weir No. (-)	Crest Elevation (m)
37	6.148	38	6.145
39	6.150	40	6.140
41	6.163	42	6.144
43	6.145	44	6.160
45	6.146	46	6.147
47	6.155	48	6.148
49	6.165	50	6.146
51	6.150	52	6.152
53	6.160	54	6.165
55	6.165	56	6.160
59	6.155	60	6.160
61	6.158	62	6.178
63	6.170	64	6.175

Note: Crest elevations are numbered starting from the upstream extremety of the bay.

Table D.4. Discharge measurements on the weirs serving the grass field.

Weir No. (-)	Av. Head on the Weir (Cm)	Measured Discharge (l/s)
7	5.650	1.739
8	5.800	1.854
9	5.600	1.877
10	5.500	1.860
11	5.275	1.773
12	5.275	1.400
13	5.375	1.382
14	5.825	1.739
15	5.900	1.191
16	6.050	2.306
17	6.100	2.164
18	6.175	2.095
19	6.675	2.281
30	5.850	2.325
31	5.975	2.345
32	5.500	1.807

continued-----

Table D.4 Continued

Weir No. (-)	Av. Head on the Weir (Cm)	Measured Discharge (l/s)
33	5.400	1.756
34	5.000	1.722
35	5.250	1.722
46	7.150	3.232
47	7.250	3.070
48	7.800	3.520
49	7.525	3.423
50	7.725	3.350
51	7.575	2.921
52	7.550	3.061
53	7.250	3.061
54	7.000	2.760
55	7.600	3.033
56	7.300	3.061
57	6.900	2.780
58	7.000	2.933

Table D.5. The crest elevations of the weirs serving the grass field.

Weir No. (-)	Crest Elevation (m)	Weir No. (-)	Crest Elevation (m)
1	1.3777	2	1.3771
3	1.3789	4	1.3780
5	1.3780	6	1.3783
7	1.3771	8	1.3786
9	1.3771	10	1.3786
11	1.3753	12	1.3756
13	1.3774	14	1.3792
15	1.3807	16	1.3807
17	1.3823	18	------
19	1.3856	20	1.3841
21	1.3868	22	1.3862
23	1.3878	24	1.3853
25	1.3902	26	1.3893
26	1.3905	27	1.3792
28	1.3893	29	1.3868

continued------

Table D.5 Continued

Weir No. (-)	Crest Elevation (m)	Weir No. (-)	Crest Elevation (m)
30	1.3792	31	1.3798
32	1.3753	33	1.3750
34	1.3716	35	1.3750
36	------	37	1.3762
38	1.3780	39	1.3823
40	1.3862	41	1.3887
42	1.3905	43	1.3911
44	1.3917	45	1.3942
46	1.3951	47	1.3954
48	1.3996	49	1.3981
50	1.3969	51	1.3966
52	1.3975	53	1.3945
54	1.3945	55	1.3975
56	1.3960	57	------
58	1.3938	59	1.3905
60	1.3929		

Laboratory Data

Table D.6. Head - Discharge data for the sharp entranced weir with 1/10 bed slope.

V = 0.0 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
20.5	0.3674	20.6	0.3674
20.6	0.3701	30.1	0.7180
30.1	0.7175	30.2	0.7164
30.2	0.7193	30.4	0.7153
40.2	1.1788	40.3	1.1819
40.3	1.1819	50.6	1.7868
50.9	1.8118	61.0	2.5214
61.3	2.5418	72.2	3.4287
72.4	3.4313	72.5	3.4240
94.9	5.6519	94.9	5.6547
105.0	6.8785	105.2	6.9121
116.3	8.4108		

Continued------

Table D.6. Continued

V = 0.1 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
31.8	0.7346	30.95	0.6974
40.7	1.1481	50.15	1.7027
60.73	2.4296	70.0	3.1444
80.8	4.1528	90.0	5.1116
100.7	6.4398	110.7	7.6891
120.0	8.9915	71.7	3.3439
91.45	5.3156	91.50	5.2996
90.8	5.2298	100.75	6.3958
100.80	6.4193	119.70	8.9340

V = 0.2 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
20.2	0.3454	20.9	0.3595
20.3	0.3352	20.3	0.3300
30.0	0.6374	29.7	0.6628
50.2	1.6681	50.2	1.7252
70.6	3.1849	70.7	3.1771
70.85	3.2360	90.3	5.1369
90.5	5.1662	90.5	5.1456
100.4	6.2513	100.4	6.2427
100.4	6.2860	100.4	6.2462
111.1	7.5996	111.1	7.5843

V = 0.3 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
20.00	0.3095	20.10	0.3098
20.10	0.3244	31.7	0.6822
31.6	0.6737	31.65	0.6691
52.2	1.7546	52.2	1.7659
51.7	1.6361	51.7	1.6444
69.0	2.9501	69.0	2.9536
89.1	4.9224	89.1	4.9278
89.1	4.8696	113.0	7.8317
105.1	6.7692	120.2	8.7819

Table D.7. Head-Discharge data for the sharp entranced weir with horizontal bed.

V = 0.0 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
18.9	0.2713	18.8	0.2687
32.4	0.7073	32.4	0.7082
44.1	1.2118	44.1	1.2127
56.8	1.8624	56.8	1.8634
64.1	2.2841	64.1	2.2807
78.1	3.2459	78.15	3.2417
86.4	3.8635	86.4	3.8504
111.0	7.1715	111.0	6.1576
134.5	8.8273	106.0	5.6334
100.5	5.1107		

V = 0.1 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
19.5	0.2805	19.5	0.2840
32.85	0.7120	33.1	0.7089
50.7	1.5310	50.7	1.5327
69.5	2.6274	69.6	2.6353
90.2	4.2092	90.2	4.1991
110.3	6.0639	110.3	6.0671
137.5	9.1619		

V = 0.2 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
18.7	0.2276	31.1	0.5962
31.2	0.5954	40.8	0.9762
52.3	1.5276	52.4	1.5259
62.1	2.0725	73.2	2.8057
73.2	2.8162	81.7	3.4497
90.5	4.1450	101.7	5.1503
101.7	5.1690	111.3	6.0621
111.2	6.0906	122.7	7.3182
122.8	7.3497	31.4	0.6021
31.4	0.6090	101.7	5.1761

Continued-----

Table D.7. Continued

V = 0.2 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
40.8	0.9825	52.4	1.5426
62.1	2.0871	72.6	2.7881
72.55	2.7932	81.7	3.4602
81.7	3.4670	90.6	4.1625

V = 0.3 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
32.45	0.6127	50.8	1.4305
58.6	1.8739	72.2	2.7255
72.2	2.7246	93.4	4.3593
109.7	5.8855	131.1	8.1502
131.1	8.1811	72.2	2.7255
72.2	2.7246	93.4	4.3593
109.7	5.8855	109.7	4.9148
131.1	8.1502	131.3	8.1811
32.45	0.6127	50.8	1.4305
58.6	1.8739		

Table D.8. Head-Discharge Data for the angle entranced weir with horizontal bed.

V = 0.0 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
21.8	0.3793	21.8	0.3808
37.6	0.9772	52.6	1.7301
52.6	1.7334	62.3	2.3295
62.3	2.3266	75.1	3.2398
75.1	3.2438	100.7	5.5903
100.7	5.5903	115.4	7.2895
115.4	7.2817	132.4	9.5377

Continued-----

Table D.8. Continued

V = 0.1 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
20.1	0.3250	20.0	0.3235
34.3	0.8422	34.3	0.8323
34.2	0.8161	48.8	1.5236
48.8	1.5305	70.3	2.9079
70.3	2.9074	92.5	4.7762
92.5	4.7871	111.2	6.7899
111.2	6.7636	131.2	9.4074

V = 0.2 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
29.9	0.6737	29.8	0.6639
50.7	1.6626	81.9	3.8955
99.5	5.5554	99.6	5.5941
120.4	7.9162		
19.5	0.3266	22.5	0.3973
31.2	0.6966	31.2	0.6981
29.0	0.6143	29.0	0.6154
50.3	1.6075	47.4	1.4582
70.7	2.9624	67.8	2.7421
89.9	4.5704	87.2	4.3219
110.4	6.6973	114.7	7.1862

V = 0.3 (m/s)			
Head Above Crest (mm)	Discharge (l/s)	Head Above Crest (mm)	Discharge (l/s)
21.7	0.3737	30.9	0.6948
31.1	0.7022	50.5	1.6175
68.9	2.8307	68.9	2.8278
90.0	4.7182	110.6	7.1424

Note: The data shown are in the order they were taken during the experiment.

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