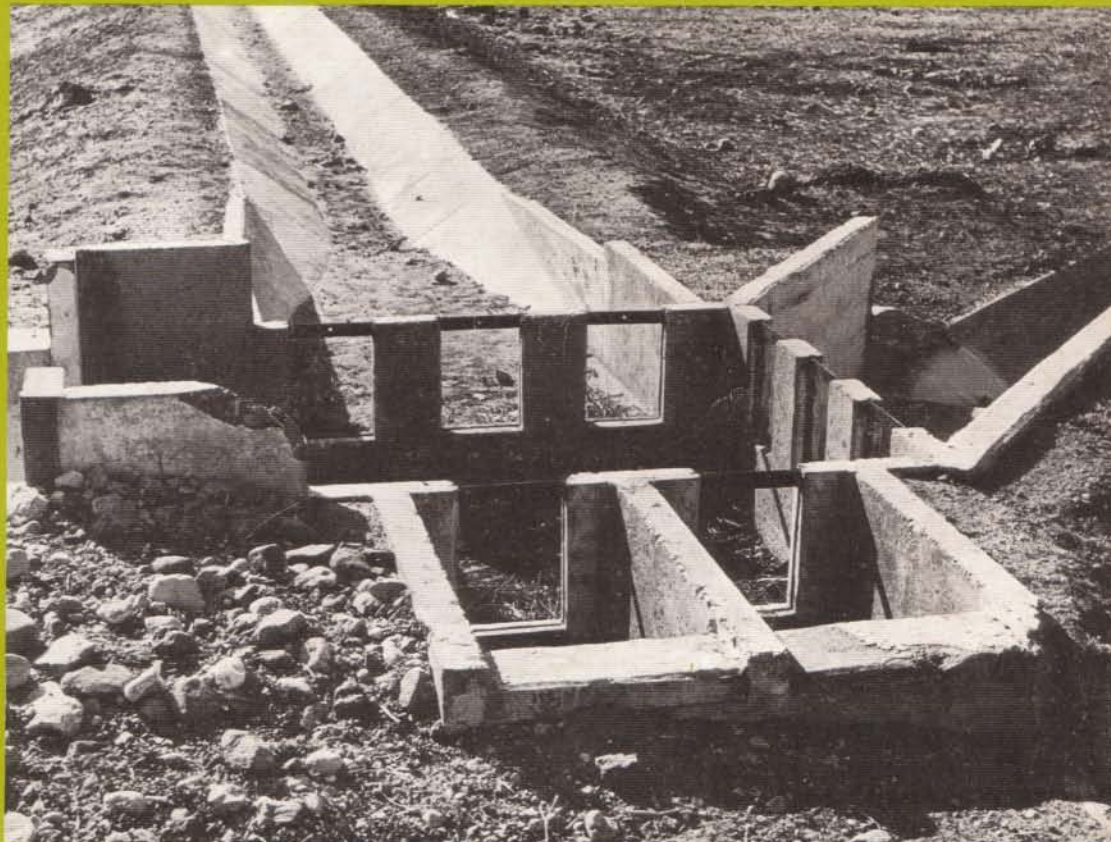**26** /1

small hydraulic structures

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small hydraulic structures

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PREFACE

This publication is the result of a joint effort by the Food and Agriculture Organization of the United Nations (FAO) and the International Commission on Irrigation and Drainage (ICID) in producing a Handbook on small hydraulic structures and devices used in open-channel irrigation distribution systems.   There has been general recognition of a need to review the abundant information and experience available on the subject and to condense and dovetail them into a comprehensive and practical Handbook.   Much basic material for the Handbook has been generously provided by National Committees of the ICID and by FAO projects and contacts in Member Countries, while complementary data and information have been assembled from the extensive survey of the literature.

The scope of the Handbook is confined as the title suggests to <u>small structures</u> used at the farm level in fields, and in networks with small discharges at the intakes, such as from small surface or ground water resources.   Such structures, having capacities of less than 1 cubic metre per second, and, indeed, many of them having capacities of less than 300 litres per second, account for more than 70 per cent of all the hydraulic structures installed in many irrigation networks.

In the past these small structures have not always received the attention they deserve from planners and designers.   It should be recognized that irrigation head works, and other irrigation engineering works, however spectacular, would have little value without an efficient distribution system (requiring small structures) extending right down to the farmers' fields.   The heavy investments normally involved in an irrigation system can be justified, through conversion into cash benefits and the social welfare of the rural population, only by paying full attention to the function and place of each of the small structures described in this Handbook.

The Handbook is published in three volumes. Volume 1 comprises Chapters 1 to 5. The types of small hydraulic structures available, and their importance for efficient distribution of irrigation water supplies are discussed in Chapter 1. Chapter 2 discusses the operation of irrigation systems and how this governs the choice of the type of small hydraulic structure best suited to the purpose. Chapters 3 to 5 deal with small intake structures, small flow-dividing structures, and outlets or farm and field turnouts. Volume 11 comprises Chapters 6 and 7. Chapter 6 deals with small water-level and velocity control structures (i. e. checks or cross regulators, falls or drops, and chutes) and Chapter 7 with small hydraulic structures and devices useful for measuring flow in irrigation networks. Volume 111, which will be issued at a later date, will cover small cross-drainage works, escapes and miscellaneous structures and will include a chapter on the detailed design of gates.

For definitions of terms, reference should be made to the ICID Muitilingual Technical Dictionary on Irrigation and Drainage. Units of measurement are generally expressed in the units from which the formulae, designs, tables and graphs have been derived (and are thus best known in that system) but in certain cases it has been considered advantageous to convert English to Metric units for application in countries using only the Metric system.

Since the Handbook attempts to assemble and describe many types of small hydraulic structures which have proved successful in certain countries, and which may be used elsewhere under similar conditions, it is hoped that it will prove useful to young engineers, technicians and extension workers involved in the remodelling of existing irrigation systems or in the design of new projects. It is also hoped that the Handbook will stimulate exchanges of ideas and information on techniques and designs which have often been evolved in isolation.

The present edition is a provisional version; it is intended that an updated version covering Volumes 1 to 111 will be printed in final form at a later date. Any comments or further contributions which readers might like to offer will be gratefully received and will be considered for incorporation in the next edition.

Acknowledgments are due to the many who have assisted in the production of this Handbook, some with systematic contributions, such as the ICID National Committees of:

Arab Republic of Egypt	Ecuador
Australia	Federal Republic of Germany
Bulgaria	Guyana
Canada	Hungary
Columbia	India
Czechoslovakia	Japan
Republic of Korea	Republic of China
Malaysia	Sri Lanka
Mexico	Turkey
Philippines	U.S.A.
	U.S.S.R.

and personnel of FAO and individual contacts who have rendered valuable information and advice, and to Mr. I. Constantinesco for his lucid editing of the manuscript.

Dated

Edouard Saouma
Director
Land and Water Development
Division
Food & Agriculture Organization
of the United Nations

K. K. Framji
Secretary General
International Commission
on Irrigation & Drainage

1. INTRODUCTION

Irrigation, with an adequate water supply, suitable soil, and good management, should ensure sustained high yields of crops per unit area of land. When the water supply is less than adequate or costly, the aim must be to obtain the best possible yields per unit of water in combination with carefully selected agronomic and managerial practices. The success of an irrigation project in meeting these requirements depends, to a large extent, on the proper functioning of its water conveyance and distribution system. Proper functioning is essentially identified with proper operation of the system so that equitable and reliable apportionment of water among users and the conveyance of water with minimum losses can be ensured. While operation is dependent on good organizational and institutional backing, its effectiveness is basically dependent on a well planned, designed and constructed network from the source of the water supply down to the farmer's field.

Engineering planning, design and construction of dams, barrages, diversion weirs, main intake works, pumping stations and main canals are usually carried out at a high degree of efficiency. Generally the operation of such headworks is also efficient and well organized and thus the amount of water lost from the total supply is usually small. Sometimes, however, secondary and tertiary canals and control structures are less carefully made, while smaller canals and those at the farm level and their structures are more often badly made or omitted entirely from engineering plans. It must not be overlooked that besides headworks and larger canals, irrigation works involve the building of many small structures and small earthworks of unsophisticated design spread over extensive areas of land. Engineers have often neglected these "minor" works, particularly those required at the farm level; to contractors they do not mean much profit and they are dispersed and difficult to supervise; and last but not least, authorities have sometimes appeared less willing to invest in tens of thousands of such small scattered works than in large works having greater prestige value. This results in many omissions of essential small structures, and failures or unnecessary deficiencies in some irrigation systems.

The great impact of small structures on satisfactory operation and overall performance of gravity irrigation systems is, however, apparent from their large number. In gravity flow systems, 90 out of 100 structures usually have capacities of less than 1,000 litres per second. The total number per unit area depends largely on the size of holdings and fields, on the delivery pattern and on the topography, but ranges from a few hundred to several thousand per thousand ha. The total irrigated area of the world at present exceeds 200 million ha and potentials exist for doubling this area. The number of small hydraulic structures already in existence exceeds 100 million, and the number that will need to be modified, replaced or newly built every year is likely to run into millions.

In view of their great impact on the saving, equitable delivery and reliable supply of water, small hydraulic structures must be designed, built and operated with much the same completeness, efficiency and accuracy as large ones.

The factors governing the design and subsequent construction and operation of irrigation works are the water resources available, the methods of water delivery to farmers, and the methods of water application practised by them. Successful operation requires adequate facilities for the control and measurement of flow at all strategic points along the whole network, including the farm and field levels. Each small hydraulic structure must be efficient, simple in design, construction and operation, and must be durable.

The "largest" structures discussed in this publication are the intakes from tertiary canals or intakes from small rivers into complete irrigation systems, the head discharges of which do not exceed one cubic metre per second. Intakes are required to control flow into a subsequent canal or canal system; often they are combined with silt-excluding devices. Intakes should be designed to control and regulate water with minimum entrance losses and as little disturbance as possible. Flow-dividing structures or proportional distributors are closely related control structures, but are discussed separately in this handbook.

For the purpose of this manual an intake structure is one for regulation of flow into an offtaking canal with a full supply capacity of not more than 25 per cent of that of the parent canal. When the offtaking canal or canals leave less than 75 per cent of the discharge in the parent canal downstream, the structure effecting the division of the

flow into two, three or more portions is considered a "divisor" or "proportional distributor". The proportions may be fixed, or adjusted by varying the control opening. A divisor on a farm channel is usually called a 'division box'.

An outlet (or farm turnout), the purpose of which is to control water delivery to the farm, is a structure at the head of a watercourse, farm supply ditch or field channel which connects with a distribution canal (e.g. a tertiary or a quaternary). The distribution canal is under the control of an operator of the authority in charge of the irrigation system. Thus, the farm outlet is the connecting link between the authority and the farmer. The fact that there are approximately as many farm outlets as there are farm holdings, or even as many field outlets as there are fields in an irrigation system, underlines the importance of this type of structure.

A check or cross regulator is a structure designed to raise the water level in a channel. This would be necessary, for example, during periods of low discharge in the parent channel when the check would raise the water level sufficiently to feed an offtake canal, or several in rotation. A check may also serve to close temporarily and divert the supply to the downstream end of a parent channel to allow time for repairs or maintenance. Checks also help in temporarily absorbing fluctuations of water supply in various sections of the canal system, in controlling flow velocities and in preventing breaches in the tail reaches. Falls, measuring structures, bridges and other structures can also be combined with checks or cross regulators. At the farm level, checks are required to divert water from one to another canal or to serve as a control for water delivery to the field.

Whenever the slope of the land is greater than the grade required of the irrigation canal, the difference is adjusted by constructing drops (or falls) or chutes at suitable intervals. Generally, a water level control structure will be called a drop or fall when the reduction of the slope is accomplished over a short distance. When water is conveyed over longer distances and along slopes that are more gentle, but still steep enough to maintain high velocities, the structures used are known as chutes.

The measurement of irrigation water is an essential element for its fair distribution and economical use. Measurement serves to ensure the maintenance of proper delivery schedules, to determine the amounts of water delivered and to single out anomalies in distribution. Knowledge of the amount of water delivered facilitates proper

application to the field and, where applicable, provides a basis for water charges. It is also useful for estimating conveyance losses and detecting their origin. A variety of measuring methods, devices and structures has been developed to suit various conditions. Measuring structures and devices are commonly combined with other structures, such as farm outlets, checks or falls. Some types have been standardised on a national scale in several countries; others are available commercially.

Whenever an irrigation canal intercepts natural streams or drainage channels in its passage, cross drainage works have to be constructed. Cross drainage works may be either syphons, aqueducts or level crossings, which pass the drainage water either under or above or at the same level as the irrigation canal. Sometimes drainage and irrigation water is deliberately intermixed.

Escapes act as safety valves for disposing of surplus water resulting from conditions such as the following: slack or no demand for irrigation water due to excessive rainfall on the commanded area; sudden closure of a canal due to a breach or breaches; admittance of drain water into the canal through inlet and level crossings; temporary closure of a canal for urgent repairs; errors in regulation; and failure of pumps.

The regulation and distribution of flow through a hydraulic structure on an irrigation system is, to a large extent, controlled by gates of various types. The type of gate required for a particular purpose depends on the dimensions of the gate, the water pressure, the mode of operation and on the availability and cost of local materials.

The wide range of types of gates available has merited a special chapter in this handbook, to deal in detail with the design of gates which have proved successful in practice.

In addition to the structures mentioned above, distribution systems may require bridges, road culverts, cart and cattle crossings; and other complementary structures.

Automatic mechanical gate control has become a common asset in headworks and large conveyance canals, while in the terminal distribution system and on the farm such means of control are still quite rare.

An ideal completely automatic irrigation system is one in which the water require-

ments of the plants are met by means of devices which measure soil moisture and initiate a series of operations to convey the necessary water through the network at the right moment and at the required rate and duration. Technically, automation is much more difficult to introduce in open flow networks than in pressure pipe systems. Therefore, in new projects where labour and other conditions favour automatic delivery on demand, the trend is towards piped systems, rather than open-channel systems. However, existing open-channel systems cannot be readily converted into closed pipe systems. Yet social changes and changes in cultivation practices (multiple cropping, use of high yielding varieties, etc.) may call for modification of traditional gravity flow networks.

The problem here is how to provide economically for the large capacities required, whilst minimising modification of existing engineering works. This requires reconsidering existing safe limits with a view to increasing water levels and velocities, i.e. flow capacity and reduction of spill and other losses, especially in the terminal portion of a network.

Auto-mechanisation of the control of irrigation water supply is being tried at the farm level in some countries with a view to economising on water deliveries to the field and to save labour.

In new farm distribution networks it is sometimes difficult to plan in advance the layout of the entire distribution system so that it meets actual operational demands in respect of locations, dimensions and types of outlets and control structures. Therefore, it is sometimes advantageous to install temporary structures at first and then to replace them with permanent ones after gaining experience of operating the system.

Farm irrigation structures that remain in place for more than one irrigation season are considered permanent; those that are moved from place to place during each irrigation are considered portable or temporary. Provisional and temporary structures may be made of wood, metal or plastic, while permanent ones are usually built of concrete or masonry. Water levels and discharges are regulated by wooden flashboards or wooden or steel gates.

In many gravity irrigation systems maintenance is poor, resulting in reduced efficiency and the deterioration of the structures. Where the reasons for poor mainten-

ance cannot be easily overcome, the only remedy is the construction of simple, robust and "over-dimensioned" structures. Where technically feasible, multifunctional structures such as drops, outlets, flow-dividing structures and measuring equipment, should be employed.

The large number of structures of the same types and sizes used in large projects lends itself to standardization and centralized production. In fact many small irrigation structures have been standardized and are mass produced or precast in quantity in central workshops. The advantages are savings in costs; better co-ordination of supply and transport; reduction in time of construction; better quality and uniformity of the final products; and quicker repairs or replacements. (It is noteworthy also that the off-demand periods of irrigation in many areas coincide with unsuitable weather for in situ construction.) Furthermore, stock-piled precast components of structures can be installed relatively quickly at any convenient time. Savings of 20 to 30 per cent in precast as compared with in situ construction have been cited from experience in the U.S.S.R.

There is also a trend to be noted in the production of prefabricated parts or components which can be combined and assembled to form different structures, thus reducing both initial cost and repair or replacement expenses.

Despite its advantages prefabrication is unrealistic in countries and areas where labour-intensive construction would help to diminish under-employment and at the same time result in a saving in costs.

Successful irrigation ultimately depends to a large extent on the response of the farmer, and thus the irrigation facilities made available to him must be designed and maintained with a view to meeting his requirements and aptitudes. Wherever possible, farmers and the staff operating the irrigation network should be encouraged to participate together in the selection of suitable structures, and in their operation and maintenance. Such co-operation can prevent the design and installation of structures which may prove too complicated or difficult to be mastered by people lacking the necessary technical training, and which would in practice be found to be functionally inadequate.

2. IRRIGATION NETWORK OPERATION

2.1 GENERAL CONSIDERATIONS

An irrigation system comprises all the physical and organizational facilities and services required to convey water for the irrigation of crops from a source of supply to the farmers' fields. Systems may vary in size, scope and design from one merely aiming at spreading the flood water of a river over adjacent areas, or conveying small flows from a surface or ground water source over limited distances and areas to large networks ramifying over the land like the branches of a tree (Figure 2-1)
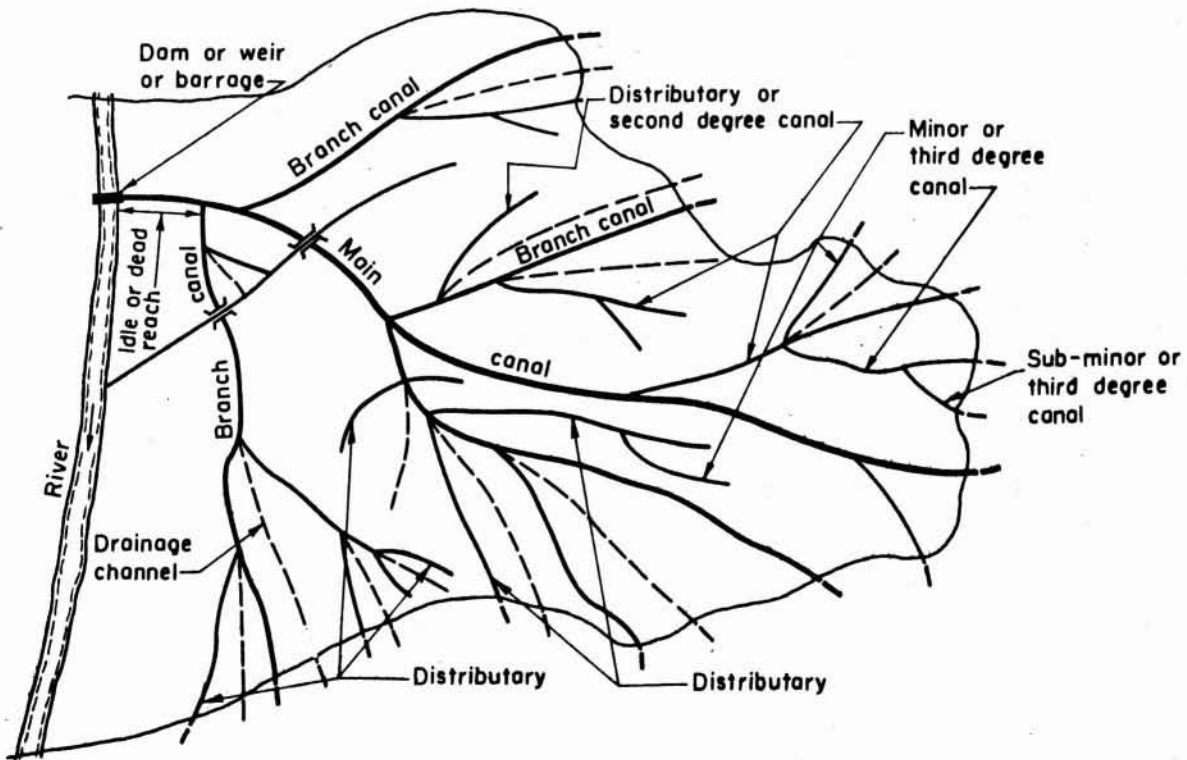
FIGURE 2-1. - Sketch of a typical irrigation system (Punjab).

Networks for gravity flow irrigation may consist of open canals and ditches or of low pressure buried pipes, or both, in the tail portion of the system. Buried pipes are not usually economical in gravity flow irrigation systems where the pipe diameter has to be large enough for a flow exceeding 1000 l/sec.

2.1.1   Need for Flow Control

Irrigation is in effect synonymous with artificial flow control from the water source to the plant. Artificial flow control is achieved by the interaction of physical facilities (e.g. hydraulic structures) and organized operation (e.g. an irrigation programme). Planning and design of a flow system should extend from the farmers' fields towards the source of water supply. The basic data for the entire system are those related to the soil, the climate, the plant, the farmer and his fields. The adequacy of these data and their correct application will ultimately determine the success of the project. The distribution system must allow the delivery of a definite quantity of water to each field and it must be possible to vary the quantity with time. In addition to irrigating efficiently, or more precisely, and thus satisfying the crops' needs for water, the quantities delivered to each point must be so determined that they also satisfy certain economic and practical requirements. This is usually achieved by establishing in advance a "programme" for the system as a whole and for each branch canal as well[1]. The establishment of a programme should be the joint concern of the agronomist, the irrigation practices specialist, the farmer and the engineer. The latter then has to take all the steps necessary to ensure that the operations can be smoothly and accurately carried out. If the water cannot be distributed strictly to programme the crops will suffer and a danger of exhausting the water resources or of waterlogging, salinity, etc. could develop. The need to save water and to provide accurate metering facilities from which realistic unit costs of water can be computed must always be the primary consideration. The amount of water to be supplied to each farm plot varies from year to year, and even during the same year, depending on the crops grown and the needs of the plants during the various stages of their growth. And, lastly, the effects of the weather during the growing period have to be taken into account.

[1] Guidance as to the synthesis of the different factors involved is provided in references (77) and (23)

Since the very aim of irrigation is to supply crops with the water they need for their growth at the right time, ideal distribution implies the ability to control the discharge from each canal at any instant so that the canal or its branches can satisfy the net demand of the area they serve. In other words, the response to demand must be as accurate and immediate as possible.

At the same time as the <u>discharges</u> to the water consumers (irrigators) are met the <u>water level</u> must be controlled, for two main purposes, depending on circumstances: to raise the water level as high as economically possible with a view to increasing the area under irrigation; and to control variations in the water level:

- to a minimum level, to prevent canal deterioration or to store water in the network

- to a maximum level, if there is a danger of overflowing

- to a predetermined level for crops that have to be completely submerged (such as rice), the depth depending on the stage of growth of the plants

- to a restricted range, as an aid to discharge control at intakes, farm outlets, etc.

The conveyance and distribution of water can be controlled by two principal methods - upstream control and downstream control, and by combinations of both these methods.

2.1.2 Upstream Control

In an upstream controlled irrigation system water is discharged from the supply in a predetermined amount at the head of the main irrigation canal. The amount of water to be discharged is specified in a previously drawn up irrigation programme. The subsequent distribution in the branch and various distributary canals occurs according to recognized demands or "water duties".

Hydraulic structures for this system are designed to maintain a constant and predetermined upstream water level, the discharge capacity of the structure being set by a fixed crest or by manual or automatic means using such devices as stop-logs, slide gates and radial gates. The volume of water in upstream controlled canals remains practically constant during operation, whatever the flows

through the various sections may be.

Upstream control of an irrigation system is used where water resources are limited, at least during part of the year. Where water is delivered in rotation, it may be applied in this way to entire sub-networks or may be practised among single lateral canals such as distributaries, minors, and water-courses or among individual farms along a lateral, depending on the size of the whole system, the supply-demand situation and other factors (see also Chapter 5).

2.1.3   Downstream Control

If the supply available at the head is always greater than the demand, dis-charge control considerations become less imperative, and each user can be allowed to regulate his own supply, providing that the effect of the amount he draws is transmitted step by step to the head of the system and causes the overall supply to the network to be adjusted to suit the cumulative demand. In the practical application of downstream control some form of automatic control equipment is necessary. (See 2.3 and 2.4 below.)

2.2   ENTIRELY MANUALLY OPERATED SYSTEMS

Under entirely manually operated systems water supplies are released from upstream (upstream control) on a cropped area basis, or on a volumetric basis, or according to special agreements. The intake to the main canal and all subsequent regulating and control structures are manually controlled (or are of fixed crest or fixed orifice type in which case, except for periodical inspection, no human interference is required). Large gates in these systems are often operated by motors, but the motors have to be manually started and stopped. Manual operation is the most wide-spread and traditional form of conveyance management and continues to be dominant in all major irrigation areas of the world. Manual operation differs little from one country to another except in organizational aspects. Within the scope of this Handbook manual control as practised in India serves as an example.

Example from India

In the case of reservoir storage ensuring adequate water supplies the water

is manually released (from upstream) at the time and to the extent required by the recognized demand. In the case of run-of-the-river supplies there are often seasonal shortages of water and under these conditions the principle employed is equitable distribution of the supply available. Either each distributary system is run in rotation with full supply or with a supply proportional to the total available supply in the system at the time. (Usually there is an acceptable period every year during which the whole network is closed down for maintenance work; in Northern India this period is three weeks).

The staff responsible for the manual operation of a canal system is headed by a "Divisional Engineer", in charge of the headworks; under him is a "Sub-Divisional Officer (Headworks)", who issues orders for the regulation of supplies in the canals under the instructions of a "Regulating Officer". These orders are based on the recorded knowledge of the water levels throughout the system of canals down to the head of the distributaries and of the "indents" (requests) for water supply received from other officers responsible for the detailed distribution of water in individual canals or parts of the system.

Early every morning gauges installed at various control points on a canal system are read by appointed gauge-readers and the data are communicated to a staff of signallers who transmit them by telegraph to the Regulating Office at the headworks where they are tabulated on a printed form. This tabulated form is generally delivered to the Regulating Officer by 8.00 a.m. each day. By this time he will have received the indents for a reduction or an increase of discharges of water at different control points of the system. He is thus in a position, (by correlating gauge readings and discharge indents) to issue the orders necessary to bring about a redistribution of the water supply in accordance with the indents. Twenty-four hours' notice is usually required to meet an indent.

For the safety of the canal system, it is important that the amount of water in each canal be within its safe capacity. In order to allow for mis-regulation and for errors and omission by gauge readers and gate operators, for unforeseen rainfall and for preventing canal breaches, safety devices (such as escapes) are provided in the system for the disposal of surplus water.

In the summer, when supplies from the run-of-the-river or the storage

reservoir are plentiful, all canals of a canal system are run at full capacity throughout the season, except when there is little or no demand for irrigation water due to heavy rainfall, or when there are major floods in the river from which supplies are being taken, (in which case the canal has to be partially or fully closed at the head to prevent excess silt entering into it).

During the winter, river supplies are limited and it is not possible to run all the canals at the same time, and thus partial supplies are run in the main canal or the branch canals but distributaries are run full in rotation. For efficient distribution it is highly desirable that distributaries and/or minors run either full or remain closed; in Northern India it is common practice to open them for 20 days and to close them for 10 days. (This practice has been developed in India to suit local conditions; there are of course alternative ways of dealing with seasonal water shortages.)

Manual control by the Irrigation Authority does not usually reach further down the system than the intake gate to a minor canal. From this canal a number of farm outlets release the water "automatically" into farm water courses. A water course may serve an average of 15 farms. The water is shared according to an agreed roster on the basis of equitable distribution. It may be necessary for the Irrigation Authority to intervene in the internal rotation system when disputes arise among users. In India there are two systems on which the roster distribution of water supplies is planned: the "fixed-turn" system and the "follow-on" system.

Under the "fixed-turn" system, individual holdings take water for a definite period according to the size of the fields, so that the turn of each holding occurs once a week on a fixed day at a fixed hour for a fixed time. The turns are so computed that the total of their periods is equal to one week. The advantage of this system is that every cultivator knows exactly when he is responsible for handling the discharge from the watercourse. On the other hand, should it be necessary to employ rotational running and different times of delivery because of a water shortage one or more of the cultivators may completely miss a turn, or even two, and the crops may suffer in consequence.

Under the "follow-on" system, the turn time for each cultivator is calculated on the basis of so many hours use of discharge per 100 acres. The

rate varies in different localities according to the crops grown and other relevant factors. When one cultivator has finished his turn, he hands over the supply to the next on the roster. The obvious advantage of this system is that every cultivator gets his turn in due course. The cultivator who irrigates at the end of one period of the channel rotation, resumes irrigation at the next running and thus finishes his allotted turn. This requires careful assignment of hours to each area.

Under both these systems the roster lists have to be prepared very carefully. The water should be turned into the fields nearest to the outlet, first to the right and then to the left, but always proceeding down the watercourse. On reaching a branch watercourse irrigation should proceed down that branch and be completed on all areas covered by it. Then irrigation should start again from the junction and proceed down the opposite branch, if any, in a similar manner. Finally, irrigation should then proceed down the main watercourse again. This method ensures that water is utilized as soon as there is water in the supply line. When a cultivator's turn at the end of a main watercourse or branch watercourse is over, the branch watercourse is closed off and the water left in the channel may run into his field. This 'balance' of water compensates the cultivator for any shortfall in supply because of the distance of his field from the water source. When the watercourse is long the volume of water left in it is substantial. Even so, disputes can arise among the cultivators at the extreme end of a watercourse as to their share in this 'balance' water. These disputes can be settled by letting the cultivators take turns at being last to receive water and by making due allowance for the amount of balance water received by adjusting the time. The problem of irrigating high areas, too, may be settled by experimenting on the site, in the presence of all the cultivators involved. The exact extra time of flow required for the cultivator concerned in such areas during a turn would be accepted by all once they had seen the justice of the arrangements made.

In order to avoid hardship to any cultivator whose turn falls during the night, two sets of rosters are drawn up, each differing from the other by twelve hours. Each of these sets is operated by rotation in alternate years.

Systems varying slightly from these practices, in accordance with local

needs, are followed in different parts of India.

2.3   HYDRO-MECHANICALLY AUTOMATED FLOW CONTROL SYSTEMS[1]/

Hydro-mechanically automated flow control systems are those in which various types of gates equipped with floats and weirs with long crests are used for automatic regulation of water levels in irrigation systems.   The most important of these are the NEYRPIC constant level upstream gate (AMIL) and the constant level downstream gates (AVIS) and (AVIO).   Technical descriptions of these gates and the weirs are given in Chapter 6.1.   Functional aspects are discussed below.

2.3.1   Hydro-mechanical Devices for Automatic Control of Constant Upstream Water Level

Hydraulically-automated constant upstream water level gates (AMIL), or weirs of long crest (such as duckbill weirs or diagonal weirs) divide the canals into successive reaches.   The constant upstream level gate, as its name implies, automatically maintains a constant water level upstream with only insignificant variations.   Similarly, a long crest weir, (because of its crest length in relation to discharge), automatically controls the upstream water level within narrow limits, and more or less independently of variations in discharge.

The intervals between these automatic level control structures depend upon the grade of the canal and the number and size of offtakes along it.   Regulation at the main intake is achieved by manual means and follows a previously established programme.   Subsequent distribution in the branch or distributary canals is carried out in accordance with recognized demands.   The demands may be based on the area served or the actual demands of the farmers based on their crop water requirements.

Figure 2-2 illustrates the function of this system; in a given canal reach the smallest level range between full and zero discharge occurs immediately upstream of the gates or weirs, while the highest level range occurs downstream

[1]/ This section is largely based on reference (69)
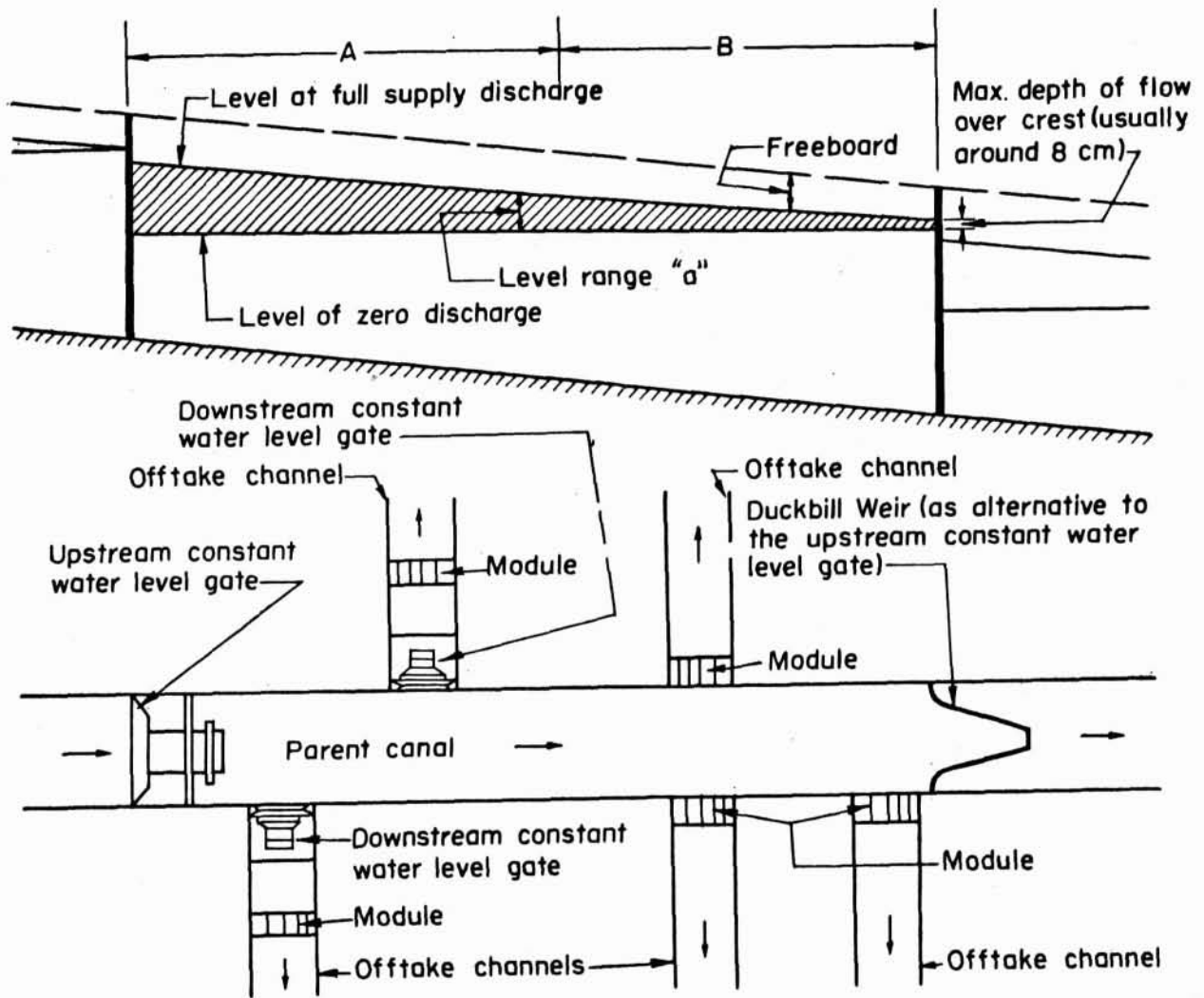
FIGURE 2-2. - Diagrammatic layout of an upstream
controlled network.

of these control structures.   Consequently it is desirable to group offtakes in the vicinity upstream of the control structures (Zone B).   Offtakes which for other reasons have to be located near the downstream side of a control structure (Zone A) have to be equipped with a device for automatic constant downstream level regulation (e. g. an AVIO or AVIS gate) in order to ensure that the discharge in the offtaking canal is independent of variations of water level in the parent canal. Alternatively, the distances between the constant upstream control gates in the parent canal may be reduced.   The optimum design to employ has to be arrived at by comparing the results, and construction costs, of using a given number of small level gates in the offtaking canals with the addition of one large gate in the parent canal.

In this system, the shutters of the offtakes or outlets are usually of semi-module type and manually operated.   Once set they give the desired discharge. The most commonly used devices are the NEYRPIC distributors (described in Chapter 3).

2.3.2   Hydro-mechanical Devices for Automatic Control of Constant Downstream Water Level

A downstream-controlled distribution system is one in which the discharges through it are controlled by the users along the canal.   Each individual demand is automatically transmitted back to the head of the system and causes the overall supply to the network to be adjusted to suit the cumulative demand.   The step-by-step transmission of the demand is ensured by constant downstream level gates, regularly spaced along the entire length of the network.   At a call for water in a given canal section, the gate at the upstream end of the canal opens to compensate for the falling level; this causes the section upstream of it to start emptying, whereupon the next gate lifts - a process repeated all along the line. Decreasing demand has the opposite effect; all the gates close one after the other, downstream to upstream.   Therefore the mere act of setting a water intake to a plot of land to the required flow is enough to ensure that the corresponding amount is supplied right through the entire network.

Figure 2-3 is a diagram showing the layout of a downstream controlled system.   As can be seen, the offtakes equipped with shutters only are grouped immediately downstream of the constant level gate (I in Zone A) while offtakes

located further down the reach and particularly those near the upstream side of the control gate (II in Zone B) are equipped with additional constant downstream level gates (III), in order to ensure that the discharge is independent of level variations in the parent canal (level range "a"). Similar to the upstream control system there is an economical optimum between the length of the control reaches in the parent canal and the number of offtakes to be equipped with constant downstream level gates. Shutters are of the same type as in the upstream controlled system, (see also Chapter 3).
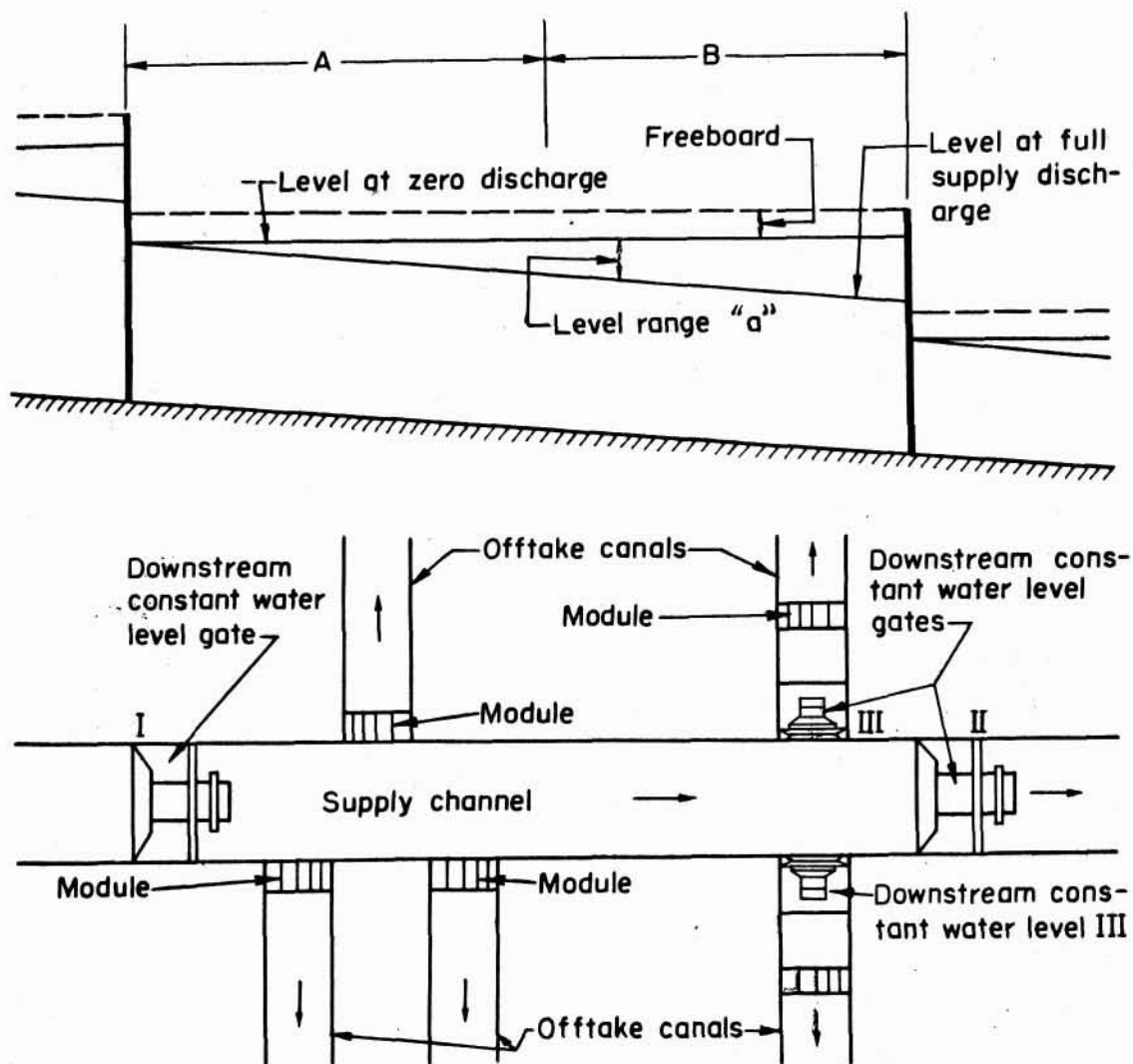
FIGURE 2-3. - Diagrammatic layout of a downstream controlled network.

2.3.3   Comparison between Upstream and Downstream Control Systems

Wherever the consumers cannot be provided with unlimited water because
of inadequate supplies or for other reasons, upstream control is the only possible
method - at least as far as the terminal portion of a network is concerned, since
it enables each user's consumption to be fixed fairly in accordance with the water
supplies available.

However, the supply of water to the head of the system and the sharing out
of the water available among the users requires a large and highly experienced
operating staff.   It is practically impossible to set all the individual discharges
in a network exactly equal to the sum of the discharge drawn off from the supply
canal plus leakage and evaporation.   Thus deficits in distribution, or some
wastage due to over-distribution, can hardly be avoided.   Since the lower
reaches are only supplied when the upper reaches are full, a certain amount of
excess water has to be kept in reserve upstream as a safety margin.   This is
obviously wasteful, causing a loss of as much as 10% of the total water supplied,
even assuming a carefully drawn up programme and painstaking distribution (69).

A further drawback is the unavoidable time lag in the transmission of
orders.   This arises from the fact that the amount of water stored in each canal
section, and therefore the storage capacity of the network, increases with canal
discharge, (see hatched section in Figure 2-2).   If the amount of water supplied
at the head increases, the first section has to fill up to the level associated with
the higher discharge before the first gate lifts;   similarly, the second constant
upstream level gate does not open until the second reach has found its new level,
and so on all the way down the line.   The aforementioned applies, with only
negligible variations, to long crested weirs for upstream control.   To increase
the discharge at the tail of such a system, a certain amount of water (which is
stored in the successive, upstream, reaches) has first to be sent through the
system;   conversely, closure of the head gate does not have any effect on the flow
at the downstream end until some of the water in all the reaches has first run off.
Thus, several hours may sometimes be required for a given discharge set at the
headwork, to reach the farmers, particularly where canals are long and con-
veyance velocity is low.

The main feature of downstream control is that it provides a completely

automatic distribution system that responds immediately to every demand. Water is saved because the quantities supplied are exactly the same as those drawn off, even allowing for leakage and exceptional demands. Another outstanding feature is that no water is wasted if consumption ceases completely; the discharge supplied to the head of the system, during the interval of time required for the closing order to be transmitted, is merely added to the amount already stored in the system, where it remains available until a fresh demand is made.

However, downstream control also has serious drawbacks. It is seldom possible to get irrigators to limit their demand, and during a water shortage, therefore, the canal will gradually empty section by section from its upstream end downwards, and deprive the consumers depending on the upper reaches of water, whilst those at the lower end are still drawing their full supply. The same situation would occur in the case of a breakdown in the system, such as blockage of a gate, breakdown of a pump, or a canal breach. In such cases all automatic gates situated downstream of the point of accident would open, in order to meet the current demand. Consequently the network below would be completely emptied. Apart from inconvenience to the consumers, serious damage to the canal (e.g. uplift of linings) might occur.

Downstream control generally requires fairly large canals, since their capacities have to be large enough to contain the volume of water corresponding to the levels at zero discharge which are of course above those for full discharge. Therefore, extensive earth and canal lining works are required. A comparison between the costs of establishing downstream and upstream controlled systems shows that the cost of the former rapidly becomes prohibitive as the slope of the ground increases. Slopes of more than 30 centimetres per kilometre may be considered as a practical upper limit for downstream control (69).

The specific limitations of either the upstream or downstream control methods can be alleviated or eliminated by combining them with each other or by integrating them with other control measures discussed in the following Sections of this chapter.

2.3.4   Combined Hydro-Mechanical Control Systems

There are two systems in use where upstream and downstream control are

combined; one may be described as the "longitudinally combined system" and the other the "composite gate system".

The longitudinally combined system

In many instances it is advantageous to use different control systems in the lower and upper part of an irrigation network.

Upstream control is usually resorted to for the terminal portions of a network in view of the general necessity of keeping the offtakes under the supervision of the operating staff, to avoid exhausting the water resources and to enable the consumption to be checked. The main supply system, however, (that is to say the main conveyance canals), may be equipped for downstream control, thus making it very much easier to carry out the irrigation programme by doing away with the necessity of going all the way to the head intake (sometimes situated a considerable distance away) whenever the discharge supplied to the network has to be changed. Moreover, the system can be brought into operation much more quickly and no water will be wasted when setting the controls. With such a layout, the programme can be made very comprehensive and even allow for unexpected demand. Finally, a system of this kind can usually be kept within economic bounds since the main canals are often laid out along a contour line and the secondary canals run more or less down the steepest slope.

The composite gate system

The composite gate system is confined to the use of NEYRPIC hydro-mechanical gates of composite type in the main network and constant downstream level control gates, followed by shutters (distributors), in the offtakes or outlets.

When conditions are normal, (i.e. when the supply is equal to the consumption) composite control gates behave in exactly the same way as constant downstream level control gates and therefore have all their characteristics. When the water supply exceeds the consumption, the gates, which then control a constant level upstream, open to prevent overflowing. Here the composite gate serves exactly the same purpose as emergency siphons or escape structures in downstream controlled systems, without however wasting water. When the supply to the canal is below the overall downstream demand, the gates close before the reaches upstream of them are completely exhausted so that some water is always

kept in reserve throughout the system. This means that the canals are no longer in danger of running dry, nor do the upstream users suffer, while those down-stream are still drawing their full supply. Thus the total reserve available can be shared out fairly among all the users until fresh supplies are made available.

A canal network thus equipped with composite control gates can be made to function as a compensation reservoir, which can absorb supply-demand differences and these differences may be caused by:

varying head supplies resulting from upstream hydro-power generation;

the need to make use of a constant flow supply while demand varies during the course of an irrigation day;

sudden rises due to storm water entering into the system;

sudden drops due to a canal breach or other breakdown.

2.4   ELECTRICALLY-ELECTRONICALLY AUTOMATED FLOW CONTROL

2.4.1   Local Electric Control, Telemetering and Remote Electric Control

Electrically operated gates can be used for any of the previously described control methods - upstream, downstream and combined. They can be controlled either locally or remotely. Control is usually based on electrical sensing of the water level using floats or electric probes. Variations in water level in the control section of the canal are conveyed to the gate by means of a transmitter, an amplifier and an integrator in accordance with the operational scheme adopted. Thus individual control gates can act automatically in much the same way as hydro-mechanically operated gates. Electrically controlled gates are usually chosen for intakes and other structures on large canals, where a supply of electricity is readily available and where gate dimensions are too large for hand operated or hydraulically automated devices. Usually the controls are set to operate one gate while the other gates or stop-logs have to be adjusted manually as demand changes during the season.

Telemetering and remote control

Electric sensing of water levels or flows and electric operation of gates lend themselves to telemetering and remote control, which provide the advantage of integrating single gate operations into the overall operation of the network at a central point. Telemetering or remote metering is a means by which water level, flow, pressure and other data can be measured remotely and communicated to a control point for display or recording or both.

Remote control or supervisory control in this context is a means whereby an operator at one location can control the function of devices at a location remote from himself with electrical or electronic equipment, or both. The operator simply controls the operation of the remote devices by activating push-buttons or other kinds of switches on a console. Gates or valves can be opened or closed, with indicator lights showing their position. Pumps can be started or stopped, with indicator lights showing whether they are working or not. Alarms may be activated to alert operators to a high or a low water level, a power failure, or the loss of a communication or control channel.

An example of a remote or supervisory control system is the one installed on the Salt River Project in the U.S.A., which serves about 100,000 ha of irrigated land. The operating agency is installing a supervisory electronic control system by means of which it will monitor and operate 174 canal gates and 54 of its 250 deep well pumps from a central operating station (109). The system comprises and provides remote control and information facilities as listed below:

- stilling wells with water level transducers (to permit sensing of upstream and downstream water levels)

- gate position transducers (to sense and indicate gate positions)

- gate controls (to open or close a gate or to set the gate at the required opening)

- pump controls (to turn the pumps on or off)

- valve controls (to adjust a valve from open to closed)

- alarm system to sense and alert the operator to high or low water, local power failures, and communication link failure

- device for information retrieval

- data logging for special records

- graphic display panel of the canal and lateral system

- operations console with all means for control and warning

- all central control equipment for the whole system.

2.4.2   Comparison of Hydraulically and Electrically Controlled Systems

Among the advantages of hydraulic control are:

- simplicity of installation (prefabricated devices available)

- robustness of the devices (float equipped gates)

- automatic functioning not requiring skilled operators

- little maintenance

- no electric energy supply needed.

Unfavourable points are:

- a greater amount of earthwork and lining is required
  (concerns only downstream control)

- the working parameters of a hydraulic control gate cannot
  easily be modified once the gate has been installed, while
  a change in the working parameters of an electrically
  automated gate simply means modification in the transmitters

- the hydraulically automated device cannot be operated by
  remote control, thus the many advantages of centralized
  operation (with or without computer aid) cannot be obtained

- susceptibility to blockage by debris, algae, etc.

As a consequence of the factors listed above it is preferable in most cases
not to introduce either of the systems alone throughout an irrigation network but
to control the supply canals electrically and the distribution canals hydraulically,
or manually.

2.4.3   Dynamic Flow Control

With the increasing complexity of water distribution systems (multiple water sources, multiple uses, compound networks) and with the growing water shortage and rising labour costs, the trend in several countries is towards computer aided automatic operation; this offers control possibilities far beyond any previous control system, in that discharges and water levels throughout the system can be continuously and automatically kept at the optimum.

Dynamic regulation may be defined as a means of seeking and implementing the regulation optimum in relation to a set of conditions existing at a given moment and in line with a given number of criteria. In this context the set of conditions refers to water level, flows, gate positions, valve openings, etc, and the criteria may be consumption forecasts, physical and economical constraints, and safety margins.

The main feature of dynamic regulation is that all reaches of a canal and even different canals (like components of a big reservoir) take part in meeting demands or in absorbing scheduled or unscheduled differences between supply and demand. The operation of a particular gate thus depends not only on one or two measurements of the water level (downstream or upstream of a gate, or both) but on a group of water level measurements, on a series of checks on consumption by users, on the position of other gates and on water levels of reserves, etc. Thus, while a constant downstream controlled gate is blind to what happens beyond its "parent" reach, a gate in the dynamic regulation system is sensitive to conditions in a number of reaches both up and downstream of the gate.

Example of dynamic flow control in France

One of the most recent and most sophisticated of flow control systems is the one developed by the Société du Canal de Provence for the "Verdon Canal Extension" of the Canal de Provence, France. The Verdon canal network, consisting of approximately 60 km of main and branch canals, conveys water for multiple uses (irrigation, industry and urban water supply) in the Region of Aix-en-Provence. The system is also of considerable complexity with regard to varying types of canal (size, grade, lining), type and age of structures, tunnels, pressure pipes, pumping stations, reservoirs, etc. The nominal head

discharge is 6.5 m^3/s; water is delivered on demand by means of pressure pipe systems. Because of this complexity and a number of other considerations special attention was paid to the operational system to be adopted for the project. Efforts were made to find out how best to make use of the network and electrical installations which already existed. The studies resulted in the development of the control system known as dynamic regulation.

Dynamic regulation is based on the simultaneous sensing of water levels, by means of float devices or electric probes, at a large number of points within the network. The system cited includes a communication network for the transmission of data (on water levels, flows, gate positions etc) to and from a general control centre (situated in the town of Tholonet). In order to collect, check and interpret these data and process the orders to be given to the various gates, pumps, etc. (by application of a mathematical model) and so as to give these orders at the right time and check their proper execution, the installation of a computer, (working on a real-time basis) was found to be indispensable. The choice fell on an IBM 1800. It was necessary, furthermore, to develop computer programmes for the implementation of regulation principles and to adapt them to the characteristics of the equipment to be operated.

The system has been subjected to various tests to determine the hydrodynamic properties of the network while maintaining canal banks and other engineering structures in their original form as far as possible. The tests have also helped in appraising the overall dependability of the regulation system by simulating breakdowns and faulting of demand schedules, etc. This dynamic control system has been operating successfully since 1971.

Advantages

Apart from operational advantages previously discussed (which result in considerable reductions in conveyance losses and savings in labour) dynamic regulation brings about considerable savings in civil engineering works by:

- lowering of canal banks, which no longer need to be kept horizontal (as required for downstream controlled systems

- reduction in volume of balancing reservoirs because of more

accurate regulation (estimates of consumption, time of transit, etc.)

- greater flexibility as to the choice of reservoir sites

- reduction in overall size of canal reaches and structures as compared to conventional downstream controlled systems (in which each reach is calculated to cope with any demand or non-acceptance, even the most improbable).

Dynamic regulation also affords a greater safety margin in the event of random accidents. Any conveyance system is more or less susceptible to accidents or malfunctions at the gates or faulty operation of recording devices or other failures, which in conventional systems may result in wrong operation. A dynamic regulation system through a programme for comparing data received can recognize that the data from a level recording device is wrong or that the device is out of action and can order a temporary remedy.

Even in case of failure during transmission the computer is warned of the abnormal condition because of a built-in self-control system. The computer itself is provided with an automatic protective system against power supply breakdowns, etc., and signals an alarm to the operator in case of improper working of the control programme.

With dynamic regulation, the flow at the head of each main branch of a canal is adjusted, for example, in line with the state of the water reserves at a given moment and in accordance with estimated consumption. If the probability of this estimate is known as a result of a preliminary statistical study, the amount of regulation can be determined in conformity with the standards chosen by the operator. Fine adjustments to distribute the volume of water in line with demand among the various reaches is achieved by the automatic hydraulic devices on the basis of water levels, as mentioned earlier. Thus the average water level in the canal will often be low. In case of an accident this leaves a much greater margin of manoeuvre before the accident causes over-spill.

2.5 MANUAL VERSUS AUTOMATED CONTROL

Manual control essentially refers to visual level measurement, verbal

transmission of data and orders, and hand operation of gates, stop-logs or valves in regulating structures along an irrigation network. The amount of water passed, for example, through a gate depends both on the extent to which the gate is open and on the upstream and downstream water levels, so that, even if stable upstream and downstream levels are assumed to exist after a setting has been made, the discharge is difficult to assess, since it has to be calculated, or determined, from tables. Furthermore, the water levels never remain constant because of the settings being carried out on neighbouring offtakes and changes in canal conditions. The fact that the amount of water supplied to an individual parcel of land thus depends on such an extensive group of regulating works and a large number of control operations, all of which are likely to occur anywhere along many kilometres of canals, may create a fairly complicated system. Finally, any change in the operation of a network requires a certain time lag, and settings are bound to be inaccurate because the control operations often have to be carried out before the new conditions have been stabilized. Control can be made easier, and distribution improved, by installing regulators or checks at suitable points along the main canal, which divide it into successive reaches and reduce the amplitude of the water level variations above the offtakes; this also partly solves the problem of controlling the levels. Nevertheless the results obtained in this way are still incomplete, and not only the head losses remain high, but the positions of the gates or heights of the stop-logs frequently have to be adjusted, requiring a considerable amount of handling.

In the terminal portions of manually operated systems distribution is often carried out in the "automatic" fashion described under Section 2.2, i.e. proportional distribution through ungated outlets designed to provide equitable delivery of water over each area. This system functions reasonably well, as long as operational conditions remain close to those for which the system was designed. In practice, however, it often fails because of poor supervision, inadequate supply level, tampering (see Figure 2-4) and lack of farmers' confidence that they are receiving their fair share of the supply. The system also lacks flexibility, which may become desirable at a later date, to enable, for example, changes in water delivery or cropping pattern.

FIGURE 2-4. - Example of tampering. Brush and stone dam built by farmers across a minor irrigation canal to increase flow through a pipe outlet.

When considering improvement of existing systems or the design of new ones there are several possibilities to be examined before introducing automated control. Such possibilities include organizational and supervisory improvements (for example, extending control of water delivery by an irrigation authority to the farm level). Further improvements which could be introduced are: regular water measurement, record keeping and evaluation of records for improved irrigation programmes, equipping of farm outlets with gates, road access along canals, etc.

The introduction of automation into the conveyance and distribution of irrigation water is not merely a replacement for human action but is primarily a means of making water transport more efficient, more reliable and more flexible. It is clear that, when water conveyance efficiency remains at a low level, while demands increase steadily, or when labour costs exceed a certain limit, it is advisable to consider whether and how to make use of automation in one form or another.

The change from manual to automated control can be made gradually,

starting perhaps with the electro-mechanization of the head gate and proceeding, as know-how in handling the equipment increases, to downstream regulating structures. Because of cost considerations, the uses of supervisory control or dynamic regulation systems are restricted to large complex water transport systems having a number of functions. However, remote control and tele-metering equipment may be considered for use in smaller systems.

Questions of considerable current interest with regard to automation are the modification of rigid rotational systems to ones which are more demand oriented, and conversions of non-stop delivery systems into ones of daytime delivery only. The main problem to be solved here is how to provide for the larger capacities required without, for economic reasons, having to make major modifications of existing engineering works. This requires reconsideration of existing safeguards with a view to increasing water levels and velocities, i.e. capacity and reduction of spill and other losses, especially in the terminal portions of the systems. In other words, an elaboration of the entire physical as well as the organizational systems is required, and this can only be accomplished with some form of automation or remote control.

2.6 WATER DISTRIBUTION ON THE FARM

2.6.1 Conventional Surface Irrigation

After water has entered the farm unit through the farm turnout, it is con-veyed in a system of primary and secondary channels (or ditches) to the head of the field to be irrigated. The system has to be laid out so that water can be made available to each part of the farm at the correct rate and elevation for the selected method of water application. The capacities of channels in small farms are usually designed to accept the total rate at which water is delivered. In large farms the flow may be split into several streams, especially during peak demand periods, to irrigate several areas simultaneously. Operation of the field channel is characterized by a period of flow of a few hours only, whereas the main channel may be in operation for one or two days during each irrigation period. The dimensions selected for the channels which deliver the water should be such that one man can handle the flow or that two men can handle it in the case of long channels or borders. The system should be sufficiently flexible to meet monthly

variations in demand and to allow for possible future changes, e.g. in cropping patterns and intensity of cropping. The actual field application requires a depth of water which depends on the soil moisture content and on the depth of wetting required which for efficient irrigation must be much the same as the depth of the plant root zones at the time of application. The water is applied from the farm channel(or ditch) to the fields by free surface outlets, pipe outlets, siphons or spiles, depending mainly on the method of irrigation. These structures are discussed in Chapter 5.

In order to apply water efficiently to the fields the water level in the head channel should be 15 to 30 cm above the level of the ground to be irrigated. If possible the channels should be nearly level (less than 10 cm per 100 m fall) so that the water can be backed up for a maximum distance, thus requiring a minimum of checks and labour to control the flow. It should be kept in mind, however, that the higher the water has to be dammed up in the head channel, the larger will be the wetted perimeter, and the water losses in the channel, if it be unlined. Accurate distribution of water to the various fields is controlled by various types of structures. For efficient operation it is important that the correct type of structure be selected and properly located. Farm irrigation structures may be permanent, semi-permanent or portable. Although it is desirable to use permanent structures from the very beginning of the construction of farm irrigation systems, it is sometimes hardly possible to avoid errors in location or elevation, with the result that the structures work ineffectively and have to be removed later on; thus it is easier to use semi-permanent structures such as wooden division boxes, checks and outlets or portable checks or siphons. When it is known that no changes will be made in the elevation or location of a structure, both the original and replacement structures are usually made of concrete or masonry.

Operation of farm distribution systems is to a large extent conditioned by the skills of farmers, their experience, habits and traditional outlooks. Thus in the design of new irrigation systems or in the remodelling of existing systems both social and technical aspects must be considered. During the planning and designing of a new irrigation project it is difficult to foresee all future farm operations with accuracy, but the establishment of field trials or pilot farms at the earliest possible moment can do much towards providing the data necessary

for forward planning. The various physical data required on irrigation practices can be accumulated from these trials but the exact behaviour of the farmers will still be difficult to predict. Therefore allowances for unpredictable behaviour or events should be incorporated in the designs of the supply and distribution systems without, however, supplying water in excess of realistic field requirements.

2.6.2   Auto-Mechanization of Surface Irrigation

Auto-mechanization of surface irrigation on the farm refers to the use of mechanical gates, structures, or other devices and systems that automatically divert water onto a farm field in the right amount and at the right time to meet crop demands; it enables the farmer to apply water more efficiently and with a minimum of labour.

Border and basin irrigation systems are particularly well suited for automation and have received the most attention. Furrow and corrugation systems are much more difficult to automate; obtaining uniform water distribution for all furrows is a problem. Automated structures operate as either water level control or as discharge control devices. In either case they automatically terminate irrigation on one portion of the field or farm and direct water to the other sections in sequence. They can be portable or permanent and are used in both lined and unlined channels or ditches.

Mechanical irrigation structures, devices and systems are normally classified as semi-automatic or automatic, depending upon their method of operation. Some portions of a given system may be automatic while others are semi-automatic or manual. Semi-automatic systems and equipment require manual attention for each irrigation. These normally use mechanical timers, such as alarm clocks, or electric or hydraulic devices to trip the structures at a preset time. The irrigator usually determines the need for irrigation and its duration, and also manually resets or returns the devices to their initial position or moves them from one location to another prior to an irrigation. Automatic structures, on the other hand, normally operate without attention from the operator other than for periodic inspections. The irrigator frequently determines when and for how long to irrigate, turns water into the system, and/or starts programmed controllers before the automated portions of the system function. Fully automatic systems sense the need for irrigation, introduce water

to the farm distribution channels, and complete the irrigation without operator intervention. The need for irrigation is customarily determined by soil moisture sensors, such as electrical resistance blocks or tensiometers; these activate electrical control apparatus when the soil moisture has fallen to a predetermined level. The duration of irrigation is usually controlled by programmed timers, or soil or surface water sensors. Technical design of automatic control gates will be discussed in Volume III of this Handbook.

Auto-mechanization enables more efficient water conveyance and application to be achieved in addition to the saving of labour. The higher equipment costs and the greater emphasis that needs to be put on maintenance and operational skills compared to conventional surface irrigation suggests that the use of auto-mechanized equipment is feasible only where it is important to save labour as well as water.

3. INTAKE STRUCTURES

3.1 INTRODUCTION

Intake structures or head regulators are hydraulic devices built at the head of an irrigation canal. (Irrigation canals in this context include main canals and branch canals or distributaries, minors and sub-divisions of them.) The purpose of these devices is to admit and regulate water from a parent canal or original source of supply, such as a dam or a river. These structures may also serve to meter the amount of water flowing through them.

The scope of this chapter is based largely on practice and experience in India and is limited to the intakes of small to medium sized distribution canals, whose source of supply may be a river or a stream, the main canal, a branch canal or a minor, and to the intakes of minors (and sub-divisions of them) whose head discharges are one cubic metre per second or less. In cases where the discharges of the offtake canals are more than 25% of the capacity of the parent canal, the control structures regulating the flow into two or more sub-canals are called "flow dividing structures" (see Chapter 4); on the farm they are usually called "division boxes".

The flowing water in the parent canal may be silt-free or charged with sediment, and the discharge may be constant, at almost constant water level, or it may vary.

For constant head discharge, the intake structure may be a module of one kind or another, and be operated manually or automatically (built-in or remotely controlled). In this case there is no need for a check structure on the parent canal. But, when the parent canal runs periodically with low discharge, it is essential to build a check or cross-regulator in the parent canal to raise the water level sufficiently to feed the offtake canal up to its full demand, or to raise the water level for installing automatic hydraulically operated upstream or downstream constant level gates. The cross regulator or check also helps to absorb
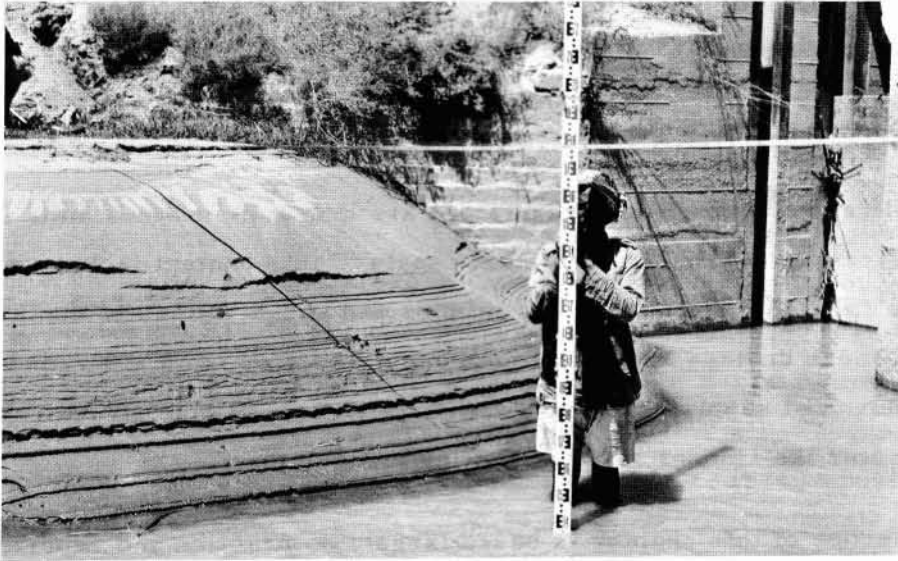
FIGURE 3-1 (a) and (b). - Silt deposition at the
intake to a secondary canal.

fluctuations in the various sections of the canal system, to facilitate the provision
of a bridge across the canal at little additional cost, and to shut off or reduce flow
temporarily for repairs to breaches in the lower sections of the canal.

When the water in the parent canal is silt-free, the centreline of the offtake
canal may be at any angle with the centreline of the parent canal. When the water
is laden with silt an important function of the intake is to control the entry of silt
into the offtake canal so that it draws its fair share of the sediment charge, which
should as far as possible, be carried in suspension to the fields. This may
require either having a suitable offtake from the outside of a curve in the parent
canal, or a suitable alignment of the offtake to the centreline of the parent canal,
or silt-excluding devices in the parent canal at the head of the intakes. Figure
3-1 (a and b) illustrates the magnitude of the silt problem when an intake is not
properly located or designed. In this case the silt content is very high, while
the intake - a gated pipe - is taking off laterally instead of being inclined towards
the direction of flow.

This chapter provides descriptions of 15 types of intake structures and 7
types of devices for controlling the entry of silt into offtake canals as listed below.
The reference numbers correspond to the section numbers in the text and the
Table of Contents.

A. INTAKES

3.2 Intakes of small canals (Punjab type).

3.3 Silt selective head intake.

3.4 Constant-head orifice intake/turnout.

3.5 Neyrpic orifice module intake.

3.6 Double orifice module intake.

3.7 Pre-cast RCC open regulator.

3.8 Intake discharging into a flume (U.S.S.R.).

3.9 Pipe regulator (with pre-cast RCC crossing, U.S.S.R.).

3.10 Intake for secondary canals, combined with fall (Colombia).

3.11 Gate valve intake (Czechoslovakia).

3.12 Venturi head intake.

3.13 Square head intake.

3.14 Dupuis canal intake.

3.15 Intake with stone-mesh weir.

3.16 Groyne intake and ancillary works (Cyprus).

B. SILT CONTROL DEVICES

3.17 King's silt vanes.

3.18 Gibb's groyne.

3.19 Curved wing with silt vanes

3.20 Silt platforms:
> (a) simple platform;
> (b) silt platform with a guide wall

3.21 Reverse vanes.

3.22 Vortex tube sand trap.

3.23 Sloping-sill sand screen.

Designers should study the designs particularly as regards relative costs, simplicity and ease of construction, and availability of labour and local materials, with a view to selecting the structure best suited to their conditions.   General guidelines as to which structures may be suitable under given conditions of water supply are summarized below.

(a) Under constant water supply in the parent canal (at designed discharge) and free from silt.

Use one of the intake structures 3.1 or 3.3 to 3.8, or 3.10 to 3.13.   All these structures may be used in combination with falls or bridges if required.

(b) Water supply in the parent canal lower than the designed discharge and free from silt.

Use one of the intake structures 3.1 or 3.3 to 3.8, or 3.10 to 3.13 with a check or cross regulator in the parent canal, or constant upstream or downstream water level gates.

(c) Water supply in the parent canal constant (at designed discharge) and charged with silt.

Use one of the intake structures 3.1 to 3.3 or 3.6, 3.8, 3.12 or 3.13.   Suitable silt-excluding devices may be used in conjunction with 3.1, 3.3 and 3.12, if necessary.

(d) <u>Water supply in the parent canal below the designed discharge and charged with silt.</u>

Use of the intake structures 3.3 or 3.12 or 3.13.

Structures 3.1 to 3.10 and 3.13 have features which enable them to meter the water let into the offtaking canals.

Structure 3.14 is particularly suitable when there is gravel sub-stratum at the site, when the use of sheet pile protection of the weir is precluded, and when, due to the porous nature of the strata, the conventional type of weir is either impractical or too expensive.

Structure 3.15 is used on large gravel bed rivers with a bed width up to 600 ft.

Among the silt-excluding devices, 3.17 is not suitable where a small offtaking canal is situated between two large canal branches and when its bed is at a high level, and/or where the water level is likely to surge over a considerable range. When the offtaking canal has its bed at a high level, the device 3.20 is preferable.

3.18 is used when both the offtaking and the supply canals have the same sediment carrying capacity. When the effect of this device is not sufficient to control the entry of silt into the offtaking canal, the device 3.17 may be used in addition to enhance the performance of 3.18.

The silt platforms (3.20) are suitable only where the parent canal is deep. The device 3.20 (b) has the advantage that (on account of the slight heading caused by the curved wing) a small head of 3 to 4.5 cm is created at the offtake which increases the velocity of the water and prevents silt being deposited in the head reach of the offtaking canal.

When a canal divides into two canals, one of which silts up very badly, and there is not enough room to accommodate vanes, the device 3.21 may be built to pass more silt into the canal which does not silt.

The device 3.22 is suitable for small canals whose bed widths are less than 3 metres. It requires that some extra discharge be let into the offtaking canal

for the operation of the tube.

The device 3.23 has been used in Egypt to correct local asymmetry in the flow pattern at the intake of distributary canals.

A.   INTAKES

3.2   INTAKES OF SMALL CANALS (PUNJAB TYPE)

3.2.1   General

The intakes for minor and sub-minor canals developed in Punjab and Haryana are designed for proportional distribution of supplies.   The types in common use are the "Open Flume" and the "Adjustable Proportional Flume (APF)".   Whichever of the two types are adopted the following conditions should be satisfied.

(a)   For open flumes the setting of the crest should be at 0.9 of the full supply depth of the parent canal (y_1); the crest must be above the level of the downstream canal bed; the width of the crest across the flow must be at least 6 cm.

(b)   If the above conditions cannot be satisfied, an adjustable proportional flume should be used, where the crest will be at 0.75 of the full supply depth of the offtake canal (y_2), and so that the depth of the underside of the roof block below the full supply level in the parent canal $(H_{(sof)})$ ranges between $0.35 y_1$ to $0.48 y_1$.   The setting of the crest must also be above the level of the downstream canal bed.

In both cases, the drowning ratio should be 0.8 or less.   These types of intakes, designed and set according to the conditions indicated above, will ensure proportional distribution over a wide range of flows.

No control work is required in the parent canal.   Even paving of the bed section of the parent canal opposite the intake is not needed.   However, when a bridge is required over the parent canal at the site of the intake, a common wing wall is provided between the intake and the bridge.   The intake structure itself may have a bridge over it, if required.

These intake structures are automatic in action and need no manual control.

Very little maintenance is required except for routine inspections. The capacity of the APF can be increased or decreased, when needed, by simply adjusting the roof block.

3.2.2 Structural Design

The Open Flume intake structure consists of upstream approach walls to the throat walls, upstream curved glacis joining the bed of the parent canal with the crest and upstream curtain wall, crest, downstream glacis and downstream expansion, cistern, curtain wall and downstream protection.

The Adjustable Proportional Flume (APF) has the same structural parts and details except that it is fitted with a roof block having its face set 5 cm from the starting point of the parallel throat.

The entire structure (except the roof block, which is made partly of reinforced concrete) is in brick masonry.

3.2.2.1 Upstream approaches to the throat

When there is no bridge on the parent canal, the radius for the curve joining the upstream side walls of the throat of the intake to the toe of the 0.5 : 1 or 1 : 1 side slopes of the parent canal should be equal to 7.5_{y_1}. The downstream side walls of the throat should be carried straight into the parent canal to meet its side slopes. The depth of the upstream curtain wall (including concrete grouting) should be equal to $\frac{y_1}{3}$.

3.2.2.2 Crest

The throat should start from the point where the upstream curve approach wing wall meets the side walls of the intake tangentially. The length of the throat or length of the crest will be equal to $2H_{(crt)}$ (where $H_{(crt)}$ is the head over the crest).

In the case of the APF a roof block of reinforced concrete (Figure 3-2) is set with its face 5 cm from the start of the throat. It has a laminated curve at the bottom with a tilt of 1 : 7.5 in order to converge the water, (instead of a horizontal base which would diverge it). The reinforced roof block should be about 30 cm thick.
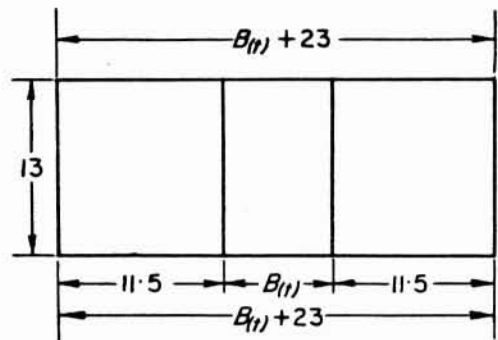
Front elevation
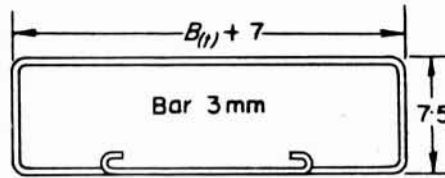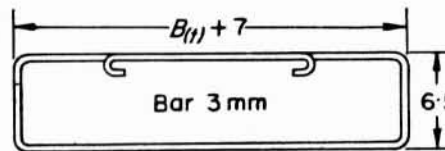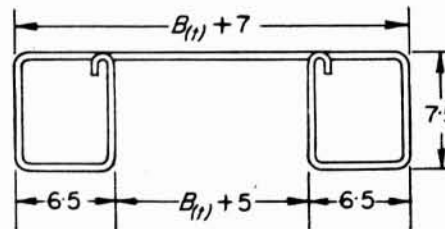
Plan

Bar No. 1

Bar No. 2

Bar No. 3

Details of precast R.C. Roof
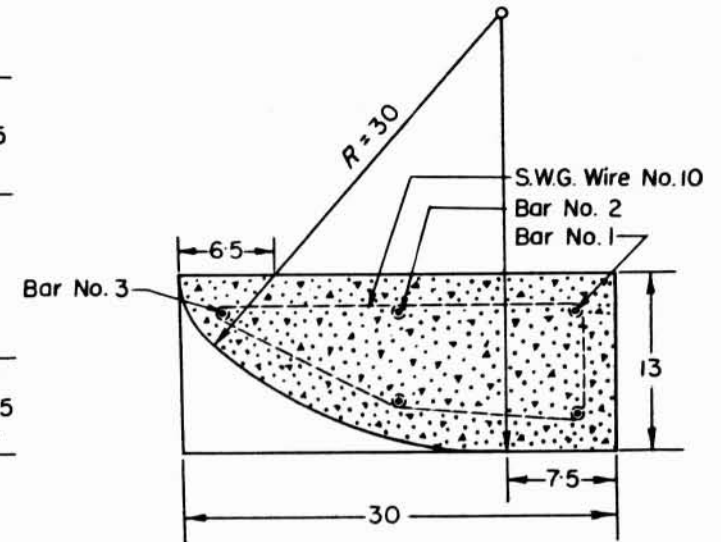
Cross section on B-B

S.W.G. (Standard wire gauge)
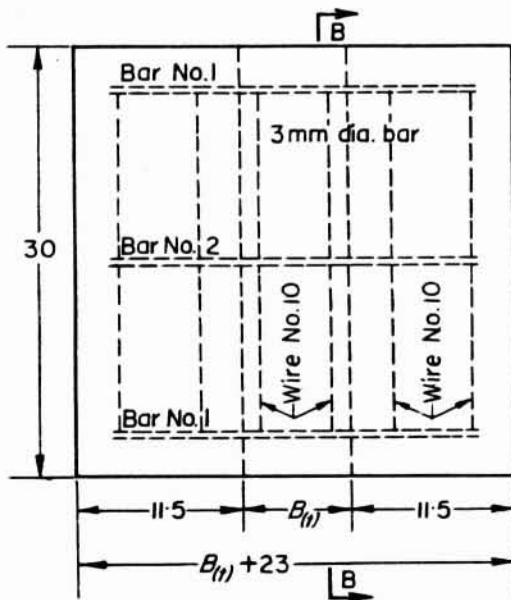
Note: All dimensions are in centimetres.

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INTAKE TO SMALL CANALS (PUNJAB TYPE)
DETAILS OF PRECAST R.C. ROOF BLOCK

Project, Region, Country
India and Pakistan

Figure No. 3 - 2

The crest should be joined with the upstream bed level of the parent canal with a radius $R_{(b-c)} = 2.5 H_{(crt)}$.

Let $L_{(app)}$ be the length of the upstream glacis

Then $L_{(app)} = \sqrt{\left\{ 5 H_{(crt)} - H_{(b-c)} \right\} H_{(b-c)}}$

where $H_{(b-c)}$ is the height of the crest above the upstream bed level.

3.2.2.3 Downstream glacis

The downstream glacis should have a 2.5 : 1 slope and will join the crest with a curve of radius 0.60 m.

Let $L_{(gl)}$ be the length of the downstream glacis.

Then $L_{(gl)} = 2.5 H_{(c-SB)}$

where $H_{(c-SB)}$ is the depth of the cistern floor below the crest.

3.2.2.4 Cistern

The depth of the cistern should be calculated thus:

$q = \dfrac{Q}{B_{(t)}} \cdot 0.67$ (approximate discharge intensity at point of standing wave)

Find H_c by the formula $H_c = \left(\dfrac{q^2}{g} \right)^{\frac{1}{3}}$

For a given working head, $h_{(wk)}$ (difference between the full supply levels in the parent canal and the offtake canal), calculate the value

$\dfrac{h_{(wk)}}{H_c}$

From Table 3-1 find the value of $\dfrac{E_2}{H_c}$ for the calculated value

of $\dfrac{h_{(wk)}}{H_c}$

Then E_2 can be worked out.

TABLE 3-1

$h_{(wk)}/H_c$	E_2/H_c	$h_{(wk)}/H_c$	E_2/H_c	$h_{(wk)}/H_c$	E_2/H_c
6.8587	2.9048	4.5091	2.6697	1.4836	2.1937
6.7836	2.8982	4.4642	2.6645	1.3604	2.1646
6.7095	2.8917	4.4198	2.6594	1.2479	2.1365
6.6364	2.8852	4.3760	2.6542	1.1450	2.1095
6.5643	2.8787	4.3327	2.6491	1.0506	2.0835
6.4932	2.8722	4.2899	2.6440	0.96415	2.0585
6.4230	2.8658	4.2476	2.6389	0.88477	2.0344
6.3538	2.8595	4.2059	2.6339	0.81184	2.0111
6.2855	2.8592	4.1646	2.6289	0.74479	1.9887
6.2182	2.8469	4.1237	2.6239	0.68311	1.9670
6.1517	2.8406	4.0834	2.6189	0.62633	1.9461
6.0861	2.8344	4.0435	2.6140	0.57404	1.9260
6.0214	2.8283	4.0041	2.6091	0.52586	1.9065
5.9575	2.8222	3.9652	2.6042	0.48145	1.8877
5.8954	2.8161	3.9267	2.5994	0.44051	1.8685
5.8323	2.8100	3.8886	2.5945	0.40276	1.8519
5.7709	2.8040	3.8510	2.5897	0.36795	1.8349
5.7103	2.7980	3.8738	2.5850	0.33585	1.8185
5.6504	2.7921	3.7770	2.5802	0.30625	1.8027
5.5914	2.7862	3.7407	2.5755	0.27895	1.7874
5.5331	2.7803	3.7047	2.5708	0.25379	1.7726
5.4755	2.7745	3.6692	2.5661	0.23061	1.7583
5.4187	2.7687	3.6341	2.5614	0.20925	1.7445
5.3626	2.7629	3.5993	2.5568	0.18959	1.7311
5.3072	2.7572	3.5650	2.5522	0.17149	1.7183
5.2526	2.7515	3.5310	2.5476	0.15485	1.7058
5.1986	2.7458	3.4974	2.5430	0.13957	1.6939
5.1452	2.7402	3.4642	2.5385	0.12553	1.6823
5.0926	2.7345	3.4314	2.5340	0.11266	1.6712
5.0406	2.7290	3.3989	2.5295	0.10087	1.6604
4.9892	2.7234	3.3668	2.5250	0.090083	1.6501
4.9385	2.7179	3.3350	2.5206	0.080289	1.6402
4.8884	2.7125	3.0356	2.4773	0.042760	1.5961
4.8389	2.7070	2.7668	2.4360	0.020319	1.5609
4.7901	2.7016	2.5247	2.3967	0.0080090	1.5340
4.7418	2.6962	2.3062	2.3591	0.0022304	1.5151
4.6941	2.6909	2.1085	3.3231	0.00026350	1.5038
4.6470	2.6855	1.9293	2.8887	0.00060	1.5000
4.6005	2.6802	1.7666	2.2557		
4.5545	2.6750	1.6185	2.2241		

The depth of the cistern below the bed level of the offtake canal, H_{SB}, will be $E_2 - y_2$, and in no case should be less than 7.5 cm.

The length of the cistern L_{SB} should be equal to $y_2 + h_{(wk)}$.

3.2.2.5 Downstream expansion

The length of the downstream expansion, $L_{(exp)}$,

$$= \frac{3(B_2 - B_{(t)})}{2} \qquad \text{or}$$

$$= L_{(gl)} + L_{(bas)}$$

The radius of expansion, $R_{(exp)}$,

$$= \frac{L^2_{(exp)}}{B_2 - B_{(t)}} + \frac{B_2 - B_{(t)}}{4}$$

3.2.2.6 · Curtain wall and downstream protection

The depth of the downstream curtain wall

$$= \frac{y_2}{2} \qquad \text{subject to a minimum of 0.45 m.}$$

The length of the bed protection of brickbats

$$= y_2 + h_{(wk)}$$

The thickness of the bed protection

$$= 0.15 \text{ m.}$$

The thickness of the walls of the throat, upstream approach, and downstream expansion should be as given in Figure 3-3.

3.2.3 Design

3.2.3.1 Open Flume

$$Q = CB_{(t)} H^{\frac{3}{2}}_{(crt)}$$

Value of 'C' to be as follows:

Masonry wing walls

Note: Coping to be of cement concrete (1:3:6) with
brick ballast. Thickness to be 15 cm.

When bed is unprotected

When bed is protected
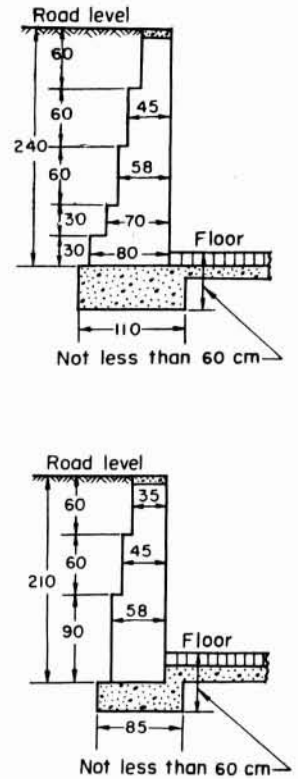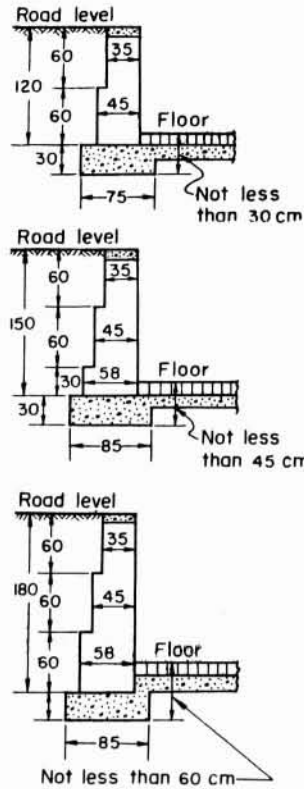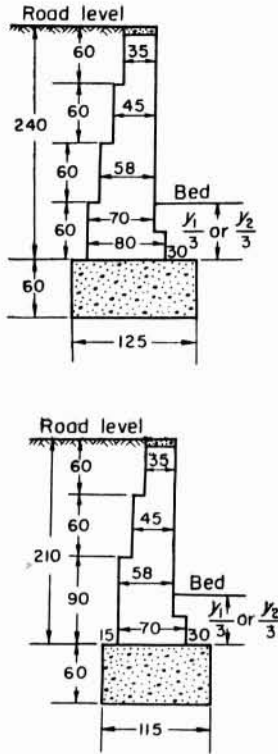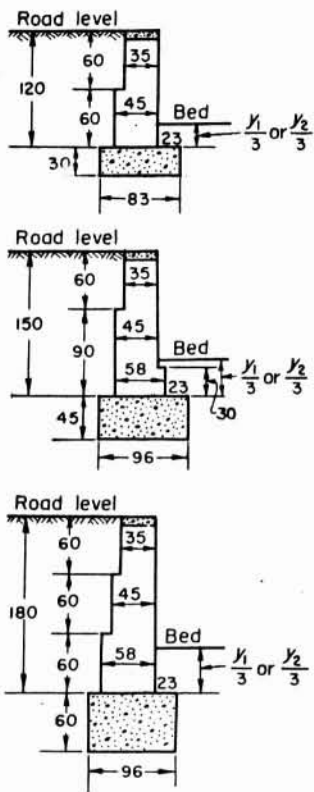
Masonry abutments with protected floor

Note: Bearing slabs for the decking to be of 15 cm thickness,
cement concrete 1:3:6 with brick ballast.
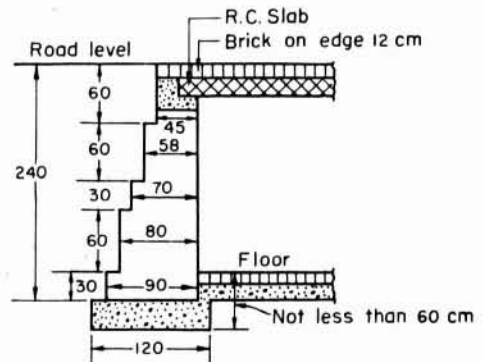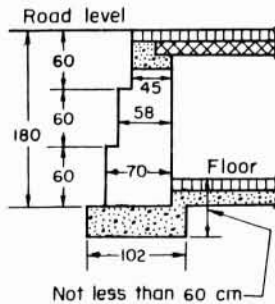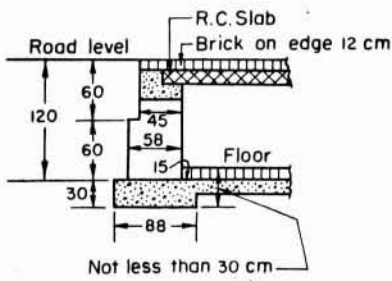
(All dimensions are in centimetres)
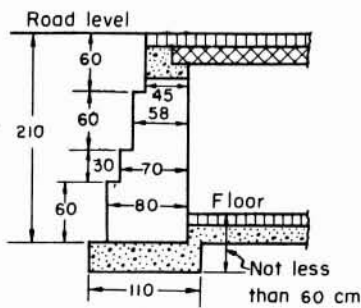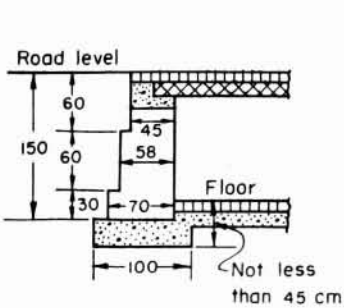
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INTAKE TO SMALL CANALS (PUNJAB TYPE) STANDARD SECTIONS FOR WING WALLS AND ABUTMENTS OF C.D.O. TYPE FALL (PUNJAB)
Country, Region, Project Punjab and Haryana (India)
Figure No. 3-3

Q	Value of 'C'	
	Intake angle 60°	Intake angle 45°
up to 0.56 m³/s	1.60	1.61
0.57 to 1.4 m³/s	1.61	1.63

3.2.3.2 <u>Adjustable Proportional Flume</u> (APF)

$$Q \;=\; 0.0403 \, B_{(t)} \; H_{(orf)} \; \sqrt{H_{(sof)}}$$

Where $H_{(orf)}$ = the height of the opening or orifice above the crest,

$H_{(sof)}$ = the depth of the underside of the roof block below the full supply level in the parent canal,

$B_{(t)}$ = width of throat.

The value of $H_{(sof)}$ should fall within the range of $0.375 \, y_1$ to $0.48 \, y_1$.

3.2.4 <u>Numerical Example</u>

3.2.4.1 <u>Design an intake structure for a minor canal with the following data:</u>

	Parent Canal		Minor Offtake
	Upstream	Downstream	
Discharge	4.2 m³/s	3.7 m³/s	0.50 m³/s
Bed level (above sea level)	100.00	99.94	99.86
Full supply depth	0.96 m	0.90 m	9.50 m
Full supply level	100.96	100.84	100.36
Bed width	7.8 m	6.6 m	2.18 m
Angle of offtake			60°
Working head (100.96 - 100.36)			0.60 m

(i)   Crest level and throat width

$$H_{(crt)} = 0.9 y_1$$
$$= 0.9 \cdot 0.96$$
$$= 0.864 \text{ m.}$$

Crest level $= 100.960 - 0.864$
$$= 100.096$$

which is higher than the bed level of the offtake channel.

$$C = 1.60 \quad \text{(from section 3.2.3.1).}$$

$$B_{(t)} = \frac{Q}{C \cdot H_{(crt)}^{\frac{3}{2}}}$$

$$= \frac{0.50}{1.60} \cdot 0.864^{\frac{3}{2}}$$

$$= 0.39 \text{ m.}$$

Fluming ratio $= \frac{0.39}{2.18}$

$$= 0.18$$

Length of crest,
$$L_{(crt)} = 2 H_{(crt)}$$

$$= 2 \cdot 0.864$$

$$= 1.728 \text{ m.}$$

(ii)   Glacis and cistern

q, after allowing 67% for splay

$$= \frac{Q}{B_{(t)}} \cdot \frac{67}{100}$$

$$= \frac{0.50}{0.39} \cdot 0.67$$

$$= 0.859 \text{ m}^3/\text{s}$$

working head, $h_{(wi)}$
$$h_{(wk)} = 0.60$$

$$H_c = \left(\frac{q^2}{g} \right)^{\frac{1}{3}}$$

$$= \left(\frac{0.859^2}{9.81} \right)^{\frac{1}{3}}$$

$$= (0.0752)^{\frac{1}{3}}$$

$$= 0.42 \text{ m.}$$

Now $\dfrac{h_{(wk)}}{H_c}$

$$= \dfrac{0.60}{0.42}$$

$$= 1.43$$

From Table 3-1, for $\dfrac{h_{(wk)}}{H_c}$ = 1.43

$$\dfrac{E_2}{H_c} = 2.1811 \text{ (by interpolation)}$$

$$\therefore \quad E_2 = 2.1811 \;.\; 0.42$$

$$= 0.916 \text{m.}$$

Depth of cistern below bed level of the offtake canal

$$= 0.916 - 0.50$$

$$= 0.416 \text{m.}$$

Floor level of cistern $= 100.36 - 0.916$

$$= 99.44$$

$H_{(c-SB)}$ $= 100.096 - 99.444$

$$= 0.652 \text{m.}$$

Length of downstream
glacis $= 2.5 \;.\; 0.652$

$$= 1.63$$

Length of cistern $= y_2 + h_{(wk)}$

$$= 0.50 + 0.6$$

$$= 1.1 \text{m.}$$

(iii) Downstream expansion

$$L_{(exp)} = \dfrac{3(B_2 - B_{(t)})}{2}$$

$$= \dfrac{3(2.18 - 0.39)}{2}$$

$$= 2.685 \text{m}$$

or, length of Glacis + cistern length

$$= 1.63 + 1.1$$

$$= 2.73\,m.$$

$$\text{Adopt} \quad L_{(exp)} \quad = \quad 2.73\,m$$

$$\text{Radius of expansion} \quad = \quad \frac{L^2_{(exp)}}{B_2 - B_{(t)}} \quad + \quad \frac{B_2 - B_{(t)}}{4}$$

$$= \quad \frac{2.73^2}{2.18 - 0.39} \quad + \quad \frac{2.18 - 0.39}{4}$$

$$= \quad 4.15 + 0.45$$

$$= \quad 4.60\,m.$$

(iv) **Downstream curtain wall below floor level of cistern**

Let d_{wc_2} be depth of downstream curtain wall.

$$\text{Then} \quad d_{wc_2} \quad = \quad \frac{y_2}{2}$$

$$= \quad \frac{0.50}{2}$$

$$= \quad 0.25\,m.$$

$$\text{Minimum required} \quad = \quad 0.5\,m.$$

(v) **Upstream curtain wall**

Let d_{wc_1} be depth of upstream curtain wall.

$$\text{Then} \quad d_{wc_1} \quad = \quad \frac{y_1}{3}$$

$$= \quad \frac{0.96}{3}$$

$$= \quad 0.32\,m.$$

Adopt 0.45 m minimum, i.e. 0.30 m masonry over 0.15m concrete.

(vi) **Downstream bed protection**

Let $L_{(prot)_2}$ be length of bed protection.

$$\text{Then} \quad L_{(prot)_2} \quad = \quad y_2 + h_{(wk)}$$

$$= \quad 0.50 + 0.60$$

$$= \quad 1.1\,m.$$

Plan

Bed width, $B_1 = 7.8$

Bank width $B_{(cbk)}$

Berm

$\frac{y_1}{2}$

0.35
0.45

$R_1 = 2H_{(crt)}$
$R = 2H_{(crt)} - \frac{y_1}{2}$

Length of crest $2H_{(crt)} = 1.728$ m

$60°$

0.58

0.64

Length of glacis = 1.63 m
Length of cistern = 1.1 m

$(ss) = 0.5 : 1$

$B_{(1)} = 0.39$ m

Supply channel

$B_2 = 2.18$

Longitudinal section

R.L. 101.86
R.L. 101.38
Precast R.C. Slab 8 cm thick

R.L. 101.86
R.L. 100.096

Berm
Bank width
Working head 0.60

W.S.L. 100.96
$y_1 = 0.96$
Bed level 100.00
0.40
Water surface level 100.36

Floor 12 cm thick over cement concrete 15 cm
Bed level 99.86
$y_2 = 0.5$

$R_{(b-c)} = 2.5 H_{(crt)}$
Floor level of cistern 99.44

Section on A-A

Road level 101.38
R.C.C. Slab 8 cm (4 inch) thick

0.35
0.80
0.11
0.45
0.60
0.13
0.58
2.04
Bed level 99.86
0.84
0.30
$\frac{y_1}{3}$ or $\frac{y_2}{3}$ 99.34
0.12
0.60
0.70
98.74
1.15

Section on B-B

100.96
101.08
0.35
0.35
0.60
0.45
0.39
Crest
0.75 100.09
99.73
0.30 99.43
1.59

Note: All dimensions are in metres unless otherwise specified.

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INTAKE STRUCTURE FOR

A MINOR CANAL AT 60°

Project, Region, Country
India and Pakistan

Figure No. 3 - 4

Thickness of bed protection of brick-bats

$$= \quad 0.15\,\text{m}.$$

(vii)   Upstream approach

Setting of crest above the upstream bed level,

$$H_{(b-c)} \quad = \quad 100.096 - 100.00$$
$$= \quad 0.096$$

$$L_{(app)} \quad = \quad \sqrt{\left\{5\,H_{(crt)} - H_{(b-c)}\right\}\,H_{(b-c)}}$$
$$= \quad \sqrt{(5 \cdot 0.864 - 0.096)\,0.096}$$
$$= \quad 0.637\,\text{m}$$

Radius joining the crest with the upstream bed,

$$R_{(b-c)} \quad = \quad 2.5\,H_{(crt)}$$
$$= \quad 2.5 \cdot 0.864$$
$$= \quad 2.160\,\text{m}.$$

The design is shown on Figure 3-4.

3.3   SILT SELECTIVE HEAD INTAKE

3.3.1   General

The design of a silt selective intake was evolved by the late K. R. Sharma of the Punjab Irrigation Department in 1936 on the assumption that the concentration of silt in a stream in the lower layers is greater than that in the upper ones, and if the lower layers were allowed to escape without interfering with the silt distribution, the remaining water would have less silt per unit volume than the water upstream of the intake. Full-sized glazed models were made to trace the silt laden stream-lines and the following conclusions were arrived at:

(a)   Under ideal conditions, the ability of the intake to conduct silt does not vary with the discharge of an offtaking canal so long as the depth in the approach chamber is not changed;

(b)   The ability to conduct silt varies according to some power (here 1/3) of the ratio of the depth in the approach channel to the depth in the
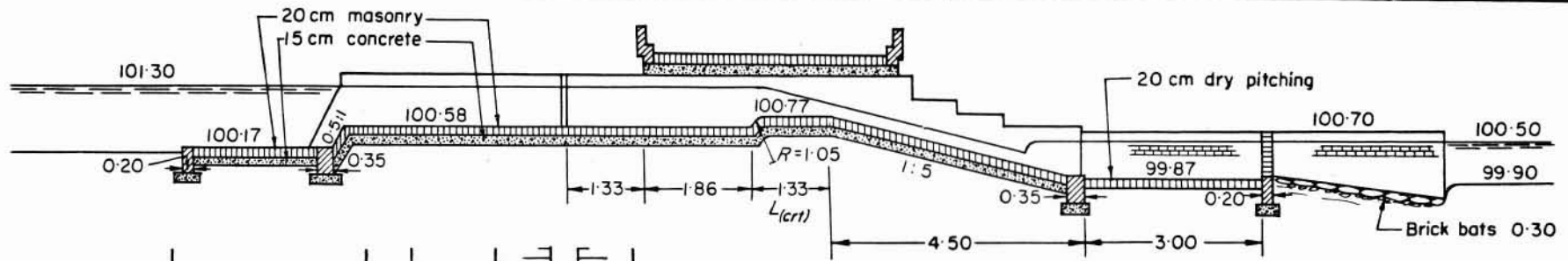
20 cm masonry
15 cm concrete
101·30
100·17
0·20
0·5:1
0·35
100·58
100·77
$R=1·05$
1·33 1·86 1·33
$L_{(crt)}$
20 cm dry pitching
100·70
100·50
99·90
1:5
0·35
99·87
0·20
Brick bats 0·30
4·50
3·00

Section on A-B-C-D-E-F

Y_1
1:10 pitching
1:1
3·39
3·39
0·5:1 pitching
$R=12$
0·5:1
0·96
$B_{(sp)}$
Approach chamber
Platform
A
B
C
$R=15$
0·35
2·26
Y_1
1:10 pitching
1:1
0·5:1 pitching
90°
$B_{(ac)}=2·34$
Gauge well 1·5 x 1
D
E
1·2
F
3·00
3·00

Plan

Note: All dimensions are in metres unless otherwise stated.

56.
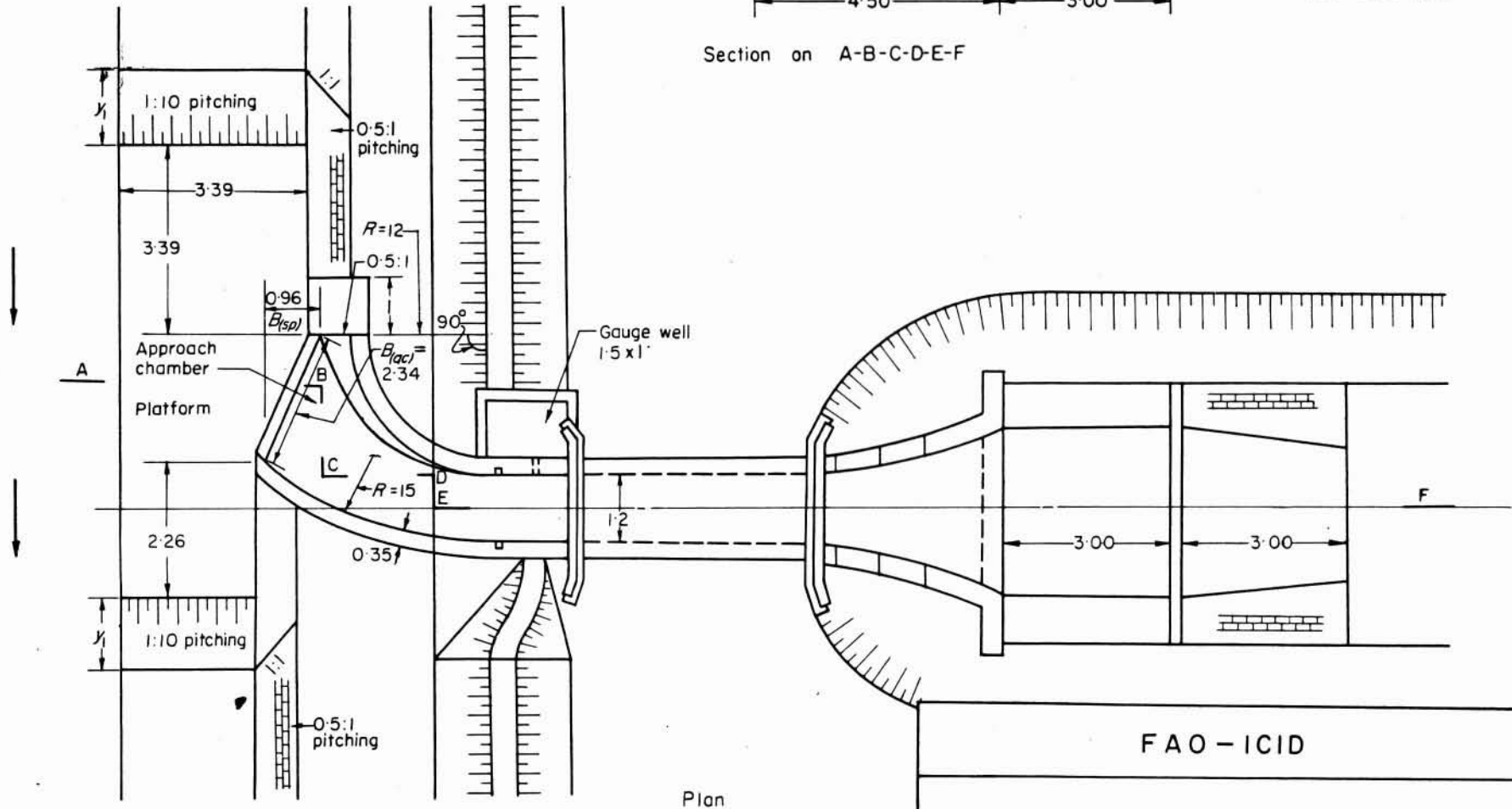
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SILT SELECTIVE HEAD INTAKE

Project, Region, Country
Punjab, India

Figure No. 3 - 5

parent canal.

The structure serves to regulate, to reduce silt in the offtaking canal and to meter the water flowing through it.

3.3.2 Structural Design

The structure consists of three parts : the approach chamber; the regulator; and the flume. Silt selection is carried out in the approach chamber. The discharge is regulated upstream of the weir flume which meters the supply. In addition, side and bed pitching is provided in the parent as well as in the off-taking canals.

A platform is provided in front of the structure in the parent canal and the bed of the platform is 10% higher than the average depth of the parent canal. The length of the platform is equal to three times the depth of the platform upstream and two times the depth of it downstream.

The floor of the approach chamber must be higher than the canal bed. The slope from the platform to the approach chamber is set at 0.5 : 1 because a vertical wall would cause disturbance. The width of the approach is determined by the formula -

$$B_{(ac)} = C \frac{Q_2}{Q_1} \left\{ B_1 + \frac{y_1}{2} \right\} \frac{0.9y}{y_{(ac)}}$$

where $B_{(ac)}$ = width of approach chamber

C = a constant varying between 1.5 and 2.0

Q = discharge of offtake canal

Q_1 = discharge of parent canal

B_1 = width of parent canal

y_1 = depth of parent canal

$y_{(ac)}$ = depth in approach chamber.

The radius of the upstream wing wall of the approach chamber is 3 times the depth in the approach chamber and the wing wall is flared as shown in Figure 3-5.

The projection of the downstream wing wall is given by $\frac{Q_2}{Q_1}\left\{ B_1 + \frac{y_1}{2} \right\}$ in metres and its radius is two times the depth in the approach chamber.

Grooves are provided for regulation by stop-logs or vertical needles. A gauge chamber at a distance of 2.5 $H_{(crt)}$ downstream of the grooves is provided up to the crest of the flume, where $H_{(crt)}$ = head over crest of the flume.

The width of the flume, $B_{(t)}$, should not be less than $\frac{B_{(ac)}}{2}$.

The length of the crest is 2.5 $H_{(crt)}$, and the vertical approach curve from floor to crest has a radius of $2\,H_{(crt)}$.

The crest is followed by a glacis at 1:5 which rests on a toe wall.

The floor length is calculated according to Bligh's theory. (Refer to "square-head intake" for computation procedure.)

Bed and side pitching of 3 m beyond the glacis is provided, which is followed by a 3 m brick-bat protection in the bed and side brick pitching in the same length. The bed pitching rests on a toe wall.

3.3.3   Hydraulic Design

(a) The required silt selective ability, $r_{(sch)_1}$, is determined from the formula,

$$r_{(cvr)} = r_{(sch)_1}^{\frac{1}{3}} \quad r_{(sch)_2}^{\frac{1}{3}} \quad r_{(sch)_3}^{\frac{1}{6}}$$

where $r_{(cvr)}$ = ratio of critical velocity ratios downstream of the intake of an offtake to that in the parent canal. The method of determining critical velocity ratio, (cvr), is given below.

$r_{(sch)_1}$ =   ratio of silt charge downstream of the intake to that in the parent canal

$r_{(sch)_2}$ =   ratio of silt grade downstream of the intake to that in the parent canal

$r_{(sch)_3}$ =   ratio of depth in parent canal to depth in offtake

$(cvr) = \dfrac{v}{v_c}$, where $v = \dfrac{Q}{A}$, Q = discharge, in m^3/s;

A = area in m^2. $v_c = C_y^{0.64}$, where y = depth in metres,

and C is a constant determined as tabulated below in the Kennedy formula.

Type of silt	Value of C
For soils of Punjab and Uttar Pradesh (UP)	0.55
Coarse silt and sand	0.70 (as in South India)
Sandy loam	0.65 (as in Tamil Nadu)
Light sandy silt of coarser variety	0.59 (as in Burma)
Fine sandy silt	0.55 (as in Punjab and UP)
Very fine silt	0.41 (as in Sind)
Pea gravel	0.85) in rivers or head
Gravel and Pea gravel	1.83) reaches of canals
Boulders, etc.	2.1)

usually $r_{(sch)_1} = r_{(sch)_2}$; $r_{(cvr)} = r_{(sch)}^{0.63} \, r_{(sch)_3}^{\frac{1}{6}}$

(b) Determine $y_{(ac)}$ from the formula

$$r_{(sch)} = \left(\dfrac{y_{(ac)}}{0.9\,y} \right)^{\frac{1}{3}}$$
to determine floor level of the approach chamber

(c) Work out projection of the downstream wing wall by the formula given in 3.3.2, i.e.

$$\dfrac{Q_2}{Q_1} \left\{ B_1 + \dfrac{y_1}{2} \right\} = B_{(sp)}$$

(d) Work out $B_{(ac)} = C \, \dfrac{B_{(sp)} \cdot 0.9\,y_1}{y_{(ac)}}$, where

C may vary from 1.5 to 2.

(e)   Work out the crest $H_{(crt)}$ from the formula

$$Q = 1.71 \, B_{(t)} \, H_{(crt)}^{\frac{3}{2}} \qquad \text{where } B_{(t)} = \frac{B_{(ac)}}{2}$$

(f)   Dimensions of wings, crest, gauge placing and protection are determined according to the formulae given under 3.3.2 Structural Design.

3.3.4   Numerical Example

See Figure 3-5.

Design a silt selective head intake in accordance with the following data :

Parent canal	Offtake canal
Q_1 = 8.5 m³/s	Q_2 = 0.8 m³/s
B_1 = 1.30 m	B_2 = 0.60 m
y_1 = 9.5 m	y_2 = 3.00
(ss) = 0.5 : 1	(ss) = 0.5 : 1
Full supply level = 101.30	Full supply level = 100.50

Adopt a value of C in the Kennedy formula equal to 0.55.

3.3.4.1   Velocities and velocity ratios

$$v_1 = \frac{8.5}{1.3 \cdot 9.5 + \dfrac{1.3 \cdot 1.3}{2}} = 0.644 \text{ m/s}$$

$$v_c = 0.55 \cdot 1.3^{0.64} = 0.65 \text{ m/s}$$

$$(cvr) = \frac{0.644}{0.65} = 0.99$$

$$v_2 = \frac{0.85}{0.21}$$

$$= 0.404 \text{ m/s}$$

$$v_c \quad = \quad 0.55\,(0.6^{0.64}) \quad = \quad 0.396 \text{ m/s}$$

$$(cvr) \quad = \quad \frac{0.404}{0.396} \quad = \quad 1.02$$

$$r(cvr) \quad = \quad \frac{1.02}{0.99} \quad = \quad 1.03$$

$$r(sch)_3 \quad = \quad \frac{\text{depth in parent canal}}{\text{depth in offtake canal}} \quad = \quad \frac{1.3}{0.6} \quad = \quad 2.166$$

3.3.4.2 Silt selective ability of intake, assuming

$$r(sch)_1 \quad = \quad r(sch)_2 \quad = \quad r(sch)$$

$$r(cvr) \quad = \quad r(sch)^{0.63} \quad r(sch)_3^{\frac{1}{6}}$$

$$1.03 \quad = \quad r(sch)^{0.63} \;(2.166)^{\frac{1}{6}}$$

$$r(sch) \quad = \quad \left\{ \frac{1.03}{(2.166)^{\frac{1}{6}}} \right\}^{\frac{1}{0.63}}$$

$$\qquad = \quad \frac{1.03^{\frac{1}{0.63}}}{2.166^{\frac{1}{0.378}}} \qquad = \quad 0.85$$

3.3.4.3 Depth in approach chamber for 85% selection of silt in the parent canal

$$r(sch) \quad = \quad \left(\frac{y(ac)}{0.9\,y}\right)^{\frac{1}{3}} \quad = \quad \left(\frac{y(ac)}{0.9 \cdot 1.3}\right)^{\frac{1}{3}} \quad = \quad \left(\frac{y(ac)}{1.17}\right)^{\frac{1}{3}}$$

$$r(sch) \quad = \quad 0.85$$

$$y(ac) \quad = \quad 0.85^3 \cdot 1.17 \quad = \quad 0.719 \text{ m}$$
$$\text{say } 0.72$$

Radius of upstream wing $\quad = \quad 3y(ac) \quad = \quad 3 \cdot 0.72 \quad = \quad 2.16 \text{ m}$
$$\text{say } 2.2$$

3.3.4.4 Projection of downstream wing $B(sp)$

$$B(sp) \quad = \quad \frac{Q_2}{Q_1}\left\{ B_1 + \frac{y_1}{2} \right\} \quad = \quad \frac{0.8}{8.5}\left(9.5 + \frac{1.3}{2}\right)$$

$$\qquad = \quad 0.955 \text{ m} \qquad \text{say } 0.96 \text{ m}$$

3.3.4.5   <u>Radius of downstream wing</u> $R_{(dsw)} = \dfrac{1.5 y_1 + 6}{2 s_1 r^2 \frac{\alpha}{2}} + B_{(t)}$

where α = angle of offtake = $60°$

$R_{(dsw)} = \left(\dfrac{1.5 \cdot 0.6 + 6}{2 \cdot 0.25} \right) + 1.2 = 15.0 \text{ m}$

3.3.4.6   <u>Width of approach</u> $B_{(ac)} = K \dfrac{B_{(sp)} \quad 0.9y}{y_{(ac)}}$;

where K = 1.5

$B_{(ac)} = 1.5 \dfrac{0.96 \cdot 0.9 \cdot 1.3}{0.72} = 2.34 \text{ m}$

3.3.4.7   <u>Width of flume,</u> $B_{(t)} = \dfrac{B_{(ac)}}{2} = \dfrac{2.34}{2}$

$= 1.17 \text{ m, say } 1.20 \text{ m}$

3.3.4.8   <u>Discharge in offtaking canal,</u> $Q_2 = 1.71 \, B_{(t)} \, H_{(crt)}^{\frac{3}{2}}$

$= 1.71 \cdot 1.20 \, H_{(crt)}^{\frac{3}{2}}$

$0.8 = 1.71 \cdot 1.20 \, H_{(crt)}^{\frac{3}{2}}$

$H_{(crt)} = \left(\dfrac{0.8}{1.71 \cdot 1.20} \right)^{\frac{2}{3}}$

$= 0.534 \text{ m, say } 0.53 \text{ m}$

3.3.4.9   <u>Length of crest</u> $L_{(crt)} = 2.5 \cdot H_{(crt)} = 2.5 \cdot 0.53$

$= 1.33 \text{ m}$

3.3.4.10   <u>Distance of gauge hole from the beginning of crest</u>

$= 3.5 \, H_{(crt)} = 3.5 \cdot 0.53$

$= 1.86 \text{ m};$

straight portion upstream $= 2.5 \, H_{(crt)} = 1.33 \text{ m}$

3.3.4.11   <u>Masonry and concrete</u>

Use 20 cm of masonry over 20 cm of concrete for flooring.

3.4 CONSTANT-HEAD ORIFICE (CHO) INTAKE[1]

3.4.1. General

The Constant-Head Orifice is a combination of a regulating and measuring structure that uses an adjustable submerged orifice for the measurement of the discharge. It has been developed and widely adopted as a delivery device to small canals or as a farm turnout, by the United States Bureau of Reclamation. It is also in use in some other countries.

The calibration tests for the turnout have been conducted in the laboratories of the Bureau of Reclamation, Denver, Colorado, and tests have been made at the Colorado Agricultural Experiment Station to investigate effects of changes in upstream and downstream water levels, sediment deposits, plugging of the orifice gate with weeds and debris, and approach flow conditions.

3.4.2 Structural Characteristics and Design

The Constant-Head Orifice Intake or Turnout (Figures 3-6, 3-7 and 3-8) consists of a short entrance channel leading to a head wall containing one or more gate-controlled openings, a stilling basin, and a downstream head wall with one or more gate-controlled openings that release the flow into a delivery conduit. The conduit is a pre-cast concrete pressure pipe (horizontal or inclined as required), the length of which depends on the width of the canal bank and whether there is a road crossing or not.

Originally, the head differential across the orifice, or upstream gate, was determined by reading staff gauges just upstream and downstream from the head wall. Fluctuations of the water levels at these gauges, particularly during large flows, caused significant reading errors. The staff gauges are now placed in external stilling wells (Figure 3-9(b)) upstream and downstream from

[1] Essentially based on the information given in the USBR Water Measurement Manual, second edition.
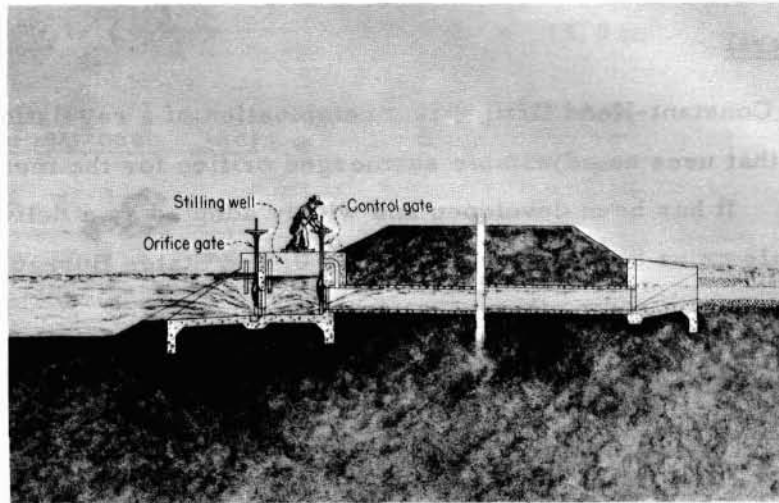
FIGURE 3-6. - Diagram of a constant-head orifice intake or turnout.

the orifice gate to increase the accuracy of head readings, and hence of the discharge measurement. For existing structures, small wooden or metal baffle-type stilling devices across the entrance passage and across the stilling basin passage near the staff gauges help to reduce reading errors.

The whole structure is of reinforced concrete and thus strong, but initial cost is relatively high because two gates are necessary. Some savings have been effected by using a cheaper downstream gate. This gate need not be watertight since it is used for regulation only, the shut-off being accomplished by the upstream gate. The structure containing the gates is designed so that it is essentially self-cleaning except when operating at very low flows. If backwater is excessive, flow through the structure will be low.

3.4.2.1   Dimensions

For accurate measurement, there must be a level floor in front of the orifice gate, of a length equal to or greater than the height of the orifice gate opening at full capacity. The minimum inside length of the measuring box

FIGURE 3-7. - Constant-head orifice farm turnouts with check-gate in foreground - East Ghor Canal Project, Jordan.
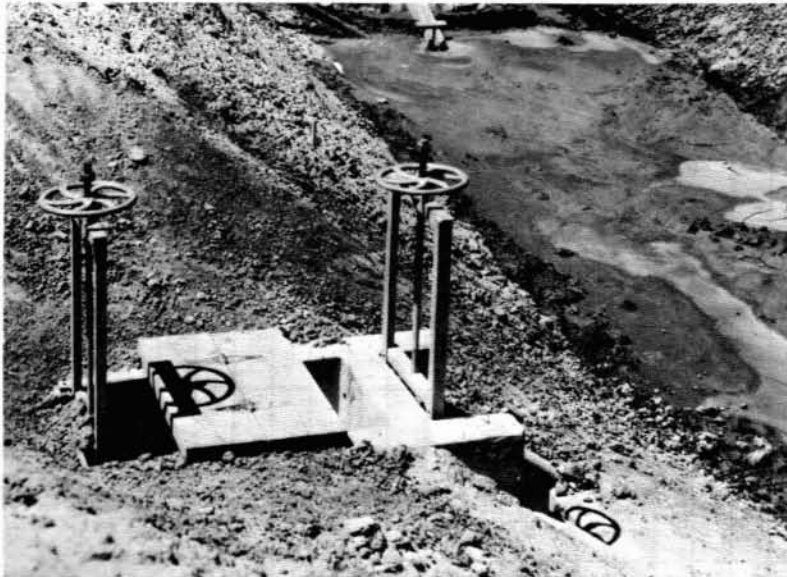
FIGURE 3-8. - A single-barrel constant-head orifice turnout.

TABLE   3-2

O	Type	$H_{(c-cws)}$	$H_{(b-nws)}$	L_{o1}	$H_{(b-Hw)2}$	$Y_{(b-bk)2}$	$H_{(p-b)2}$	$L_{(prot)2}$	J	L_1	L_2	L_3	$L_{(p)2}$	L_4	y_2	Min.Loss	$h_{v(p)}$	Sc	Pipe Dia.	Orifice gate Size		Turnout gate Dia.
2	4	2.00	3.50	-	-	1.5	0.50	-	-	5.25	10.55	10.25	-	-	0.50	0.67	0.1011	0.00315	12"	18 . 12	5.0	12"
	5	2.00	3.50	-	-	1.5	0.50	-	-	5.25	10.55	10.25	6.00	4.51	0.50	-	0.1011	0.00315	12"	18 . 12	5.0	12"
4	1	2.20	4.25	5.75	1.50	-	0.50	-	15"	6.00	7.25	11.75	-	-	0.75	0.76	0.1652	0.00383	15"	18 . 18	6.0	15"
	2	2.20	4.25	5.75	1.50	1.5	0.50	5.0	15"	6.00	7.25	11.75	6.25	4.70	0.75	-	0.1652	0.00383	15"	18 . 18	6.0	15"
	3	2.20	4.25	8.33	3.33	1.5	-	-	BO-1	6.00	7.08	14.33	3.75	7.40	0.75	-	0.1652	0.00383	15"	18 . 18	6.0	15"
	4	2.20	4.25	-	-	1.5	0.50	-	-	6.00	11.30	11.00	-	-	0.75	0.81	0.1652	0.00383	15"	18 . 18	6.0	15"
	5	2.20	4.25	-	-	1.5	0.50	-	-	6.00	11.30	11.00	6.25	4.70	0.75	-	0.1652	0.00383	15"	18 . 18	6.0	15"
6	1	2.00	4.25	6.25	1.75	-	0.50	-	18"	6.00	7.37	12.25	-	-	1.00	0.76	0.1797	0.00327	18"	24 . 18	6.0	18"
	2	2.00	4.25	6.25	1.75	1.75	0.50	5.0	18"	6.00	7.37	12.25	7.50	6.45	1.00	-	0.1797	0.00327	18"	24 . 18	6.0	18"
	3	2.00	4.25	8.33	3.33	2.00	-	-	BO-1	6.00	7.08	14.33	4.50	8.15	1.00	-	0.1797	0.00327	18"	24 . 18	6.0	18"
	4	2.00	4.25	-	-	1.80	0.50	-	-	6.00	11.30	11.00	-	-	1.00	0.82	0.1797	0.00327	18"	24 . 18	6.0	18"
	5	2.00	4.25	-	-	1.80	0.50	-	-	6.00	11.30	11.00	7.50	5.95	1.00	-	0.1797	0.00327	18"	24 . 18	6.0	18"
9	1	1.75	4.25	6.75	2.25	-	0.75	-	24"	6.75	7.87	13.50	-	-	1.25	0.81	0.2175	0.00322	21"	30 . 24	6.0	21"
	2	1.75	4.25	6.75	2.25	2.25	0.75	6.0	24"	6.75	7.87	13.50	8.75	8.02	1.25	-	0.2175	0.00322	21"	30 . 24	6.0	21"
	3	1.75	4.25	8.33	3.33	2.10	-	-	BO-2	6.75	7.83	15.08	4.50	8.90	1.25	-	0.4028	0.00732	18"	30 . 24	6.0	18"
12	1	0.70	4.25	6.75	2.25	-	0.75	-	24"	7.50	8.62	14.25	-	-	1.50	0.82	0.2269	0.00281	24"	30 . 30	7.0	24"
	2	0.70	4.25	6.75	2.25	2.25	0.75	6.0	24"	7.50	8.62	14.25	10.00	10.02	1.50	-	0.2269	0.00281	24"	30 . 30	7.0	24"
	3	0.70	4.25	8.33	3.33	2.10	-	-	BO-2	7.50	8.58	15.83	6.00	11.15	1.50	-	0.2269	0.00281	24"	30 . 30	7.0	24"
15	1	0.50	4.25	6.75	2.25	-	0.75	-	24"	7.75	8.87	14.50	-	-	1.50	1.05	0.3537	0.00438	24"	36 . 30	7.0	24"
	2	0.50	4.25	6.75	2.25	2.25	0.75	6.0	24"	7.75	8.87	14.50	10.00	10.27	1.50	-	0.3537	0.00438	24"	36 . 30	7.0	24"
	3	0.50	4.25	8.33	3.33	2.10	-	-	BO-3	7.75	8.83	16.08	6.00	11.40	1.50	-	0.3537	0.00438	24"	36 . 30	7.0	24"
18	1	0.65	4.42	8.50	3.50	-	0.75	-	30"	8.00	9.00	16.50	-	-	1.75	0.97	0.3190	0.00338	27"	42 . 30	7.0	30"
	2	0.65	4.42	8.50	3.50	3.00	0.75	7.0	30"	8.00	9.00	16.50	11.25	13.36	1.75	-	0.3190	0.00338	27"	42 . 30	7.0	30"
	3	0.65	4.42	8.33	3.33	2.10	-	-	BO-3	8.00	9.08	16.33	6.00	11.24	1.75	-	0.5105	0.00632	24"	42 . 30	7.0	24"
24	1	0.45	4.92	8.50	3.50	-	0.75	-	30"	9.00	10.00	17.50	-	-	2.00	1.05	0.3718	0.00342	30"	48 . 36	7.0	30"
	2	0.45	4.92	8.50	3.50	3.00	0.75	7.0	30"	9.00	10.00	17.50	12.50	14.40	2.00	-	0.3718	0.00342	30"	48 . 36	7.0	30"
	3	0.45	4.92	10.00	4.08	2.00	-	-	BO-4	9.00	10.63	19.00	6.75	13.45	2.00	-	0.5672	0.00600	27"	48 . 36	7.0	30"

should be: at least 2.25 times the orifice gate opening at maximum capacity or 1.75 times the wall opening, whichever is the larger, for turnouts with maximum capacities up to approximately 300 l/s (10 ft^3/s); and 2.75 times the height of the orifice gate opening at maximum capacity for structures with maximum capacities above approximately 300 l/s (10 ft^3/s) (see Figure 3-11). The inlet walls should be parallel unless extra width is needed at the inlet cut-off to prevent it acting as a control, in which case extra width may be obtained by flaring the walls, usually at 8 : 1.

The distance between the inlet cut-off and orifice gate should be a minimum of 1.5 times the difference in elevation between the invert at the cut-off and at the orifice gate. In an earth canal, the top of the sloping inlet walls should intersect the canal side slope at or a few inches above normal water level. The inlet walls are usually sloped steeper than the canal side slope and set back into the bank so that they will not be out in the canal if it is widened at the bottom during cleaning or reshaping. The following tabulation shows recommended inlet wall slopes corresponding to various canal side slopes.

Canal side slope	Inlet wall slope
2.5 : 1	2 : 1
2 : 1	1.5 : 1
1.5 : 1	1.5 : 1 set into the bank 12 to 24 inches depending on size of canal and local conditions.

The dimensions of other parts are given in Table 3-2.

3.4.2.2   Pipe velocities

Full pipe velocity is limited to about 1.07 m/s (3.5 ft/s) if the structure has no concrete outlet transition, but, if it has, the full pipe velocity should be about 1.5 m/s (5 ft/s).

The top of the pipe at the outlet should have a minimum submergence of $h_{v(p)}$.

3.4.3   Hydraulic Properties

The rate of flow is measured by using the principle that a submerged orifice of a given size operating under a specific differential head will always pass a known quantity of water.   The upstream gate or gates constitute the orifice, the size of which can be increased or decreased by opening or closing the gates.   The head across the orifice is usually about 6 cm (0.2 ft) but may be more than 6 cm if additional head is available.   This differential head is maintained by adjusting the downstream gate or gates, and is measured by staff gauges mounted inside the stilling walls, upstream and downstream from the orifice gate head wall (Figure 3-9 (b)).   To set a given flow, a typical orifice would be operated as follows.

The opening of the orifice for the desired discharge is obtained from discharge Tables 3-3, 3-4, or 3-5.   With the upstream gate or gates set at the opening, the downstream gate or gates are adjusted until the differential head across the orifice, as measured by the staff gauges in the stilling wells, is at 6 cm (0.2 ft).   The discharge will then be at the desired value.

3.4.3.1   Effects of upstream water depths

see section 3.4.4.

3.4.3.2   Flow through the structure

Flow through the structure is controlled by the size of the orifice and the head across the orifice.   The loss through the structure is computed and this loss will determine the maximum elevation to which delivery can be made.   If the canal below a turnout is dry and the turnout gate is opened some manipulation of the gate may be required until the canal fills.   (This would be true of any gated structure.)   Outside of this filling operation, tail-water does not control the flow through a turnout.

3.4.3.3   Effects of sediment and weeds

Sediment particles of the sizes normally found in field installations are swept through the orifice gate and the downstream gate during the normal course

Lateral turnout type 1
$(L_2 = L_1 + L_{OL} - 1.5 H_{(b-HW)_2} - 2.25), (L_3 = L_1 + L_{OL})$

Lateral turnout type 2
$(L_2 = L_1 + L_{OL} - 1.5 H_{(b-HW)_2} - 2.25), (L_3 = L_1 + L_{OL}), (L_4 = L_1 + L_{OL} + L_{(p)_2} - 1.17* - 2.4142 H_{(b-nws)} - 3.7320 H_{(p-b)_2}), * = -0.92$ for 18 ft^3/s and greater

Farm or lateral turnout type 3
$(L_2 = L_1 + L_{OL} - 1.5 H_{(b-HW)_2} - 2.25), (L_3 = L_1 + L_{OL}), (L_4 = L_1 + L_{OL} + L_{(p)_2} - 0.42 - 2.4142 H_{(b-nws)})$

Farm turnout type 4
$(L_2 = L_1 + 1.5 H_{(p-b)_2} + 4.55), (L_3 = L_1 + 5')$
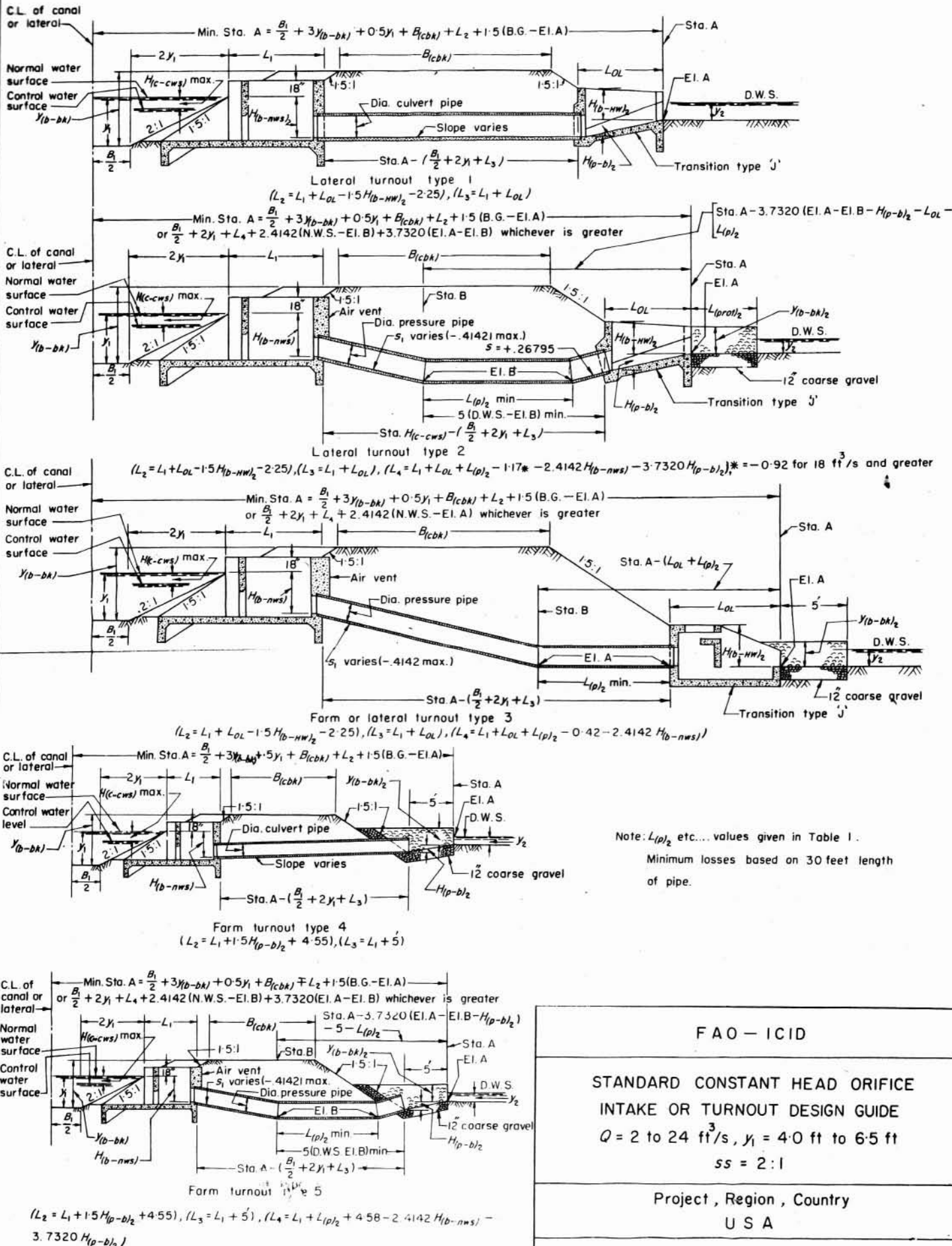
Note: $L_{(p)_2}$ etc.... values given in Table 1.
Minimum losses based on 30 feet length of pipe.

Farm turnout type 5
$(L_2 = L_1 + 1.5 H_{(p-b)_2} + 4.55), (L_3 = L_1 + 5'), (L_4 = L_1 + L_{(p)_2} + 4.58 - 2.4142 H_{(b-nws)} - 3.7320 H_{(p-b)_2})$

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STANDARD CONSTANT HEAD ORIFICE
INTAKE OR TURNOUT DESIGN GUIDE
$Q = 2$ to 24 ft^3/s, $y_1 = 4.0$ ft to 6.5 ft
$ss = 2:1$

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U S A

Figure No. 3-9 (a)
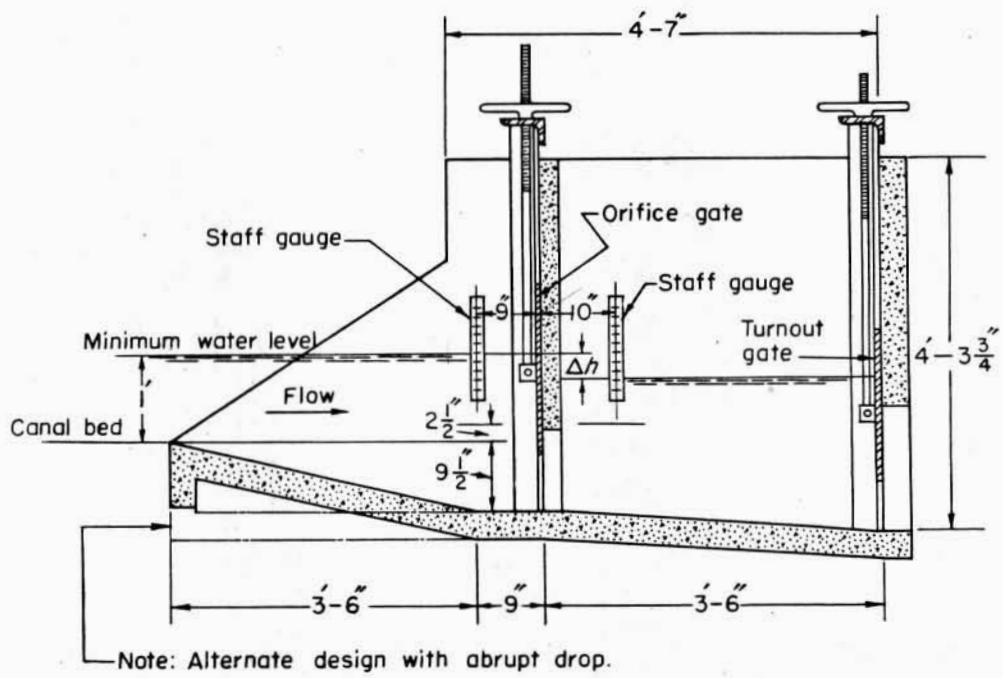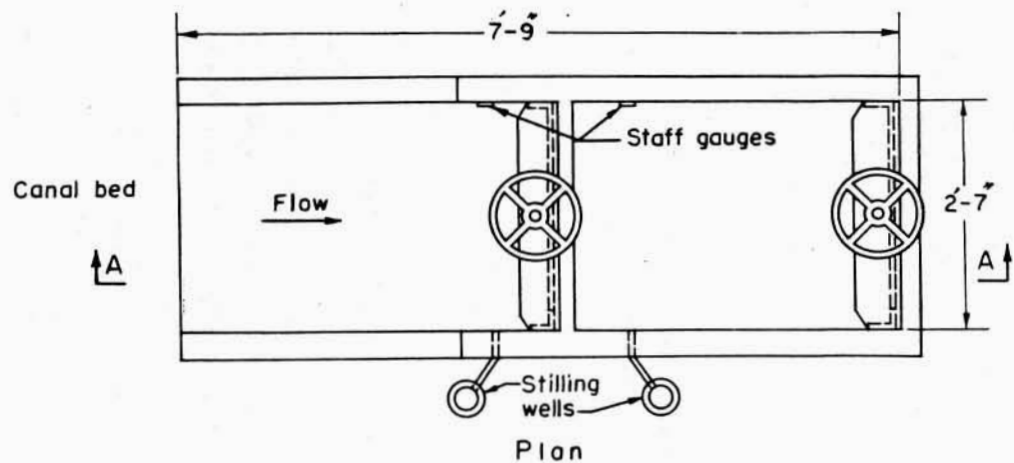
Plan

Section A-A

Schematic view of a constant head orifice turnout with a
sloping inlet channel and with piezometers and stilling wells.

Note: Alternate design with abrupt drop.
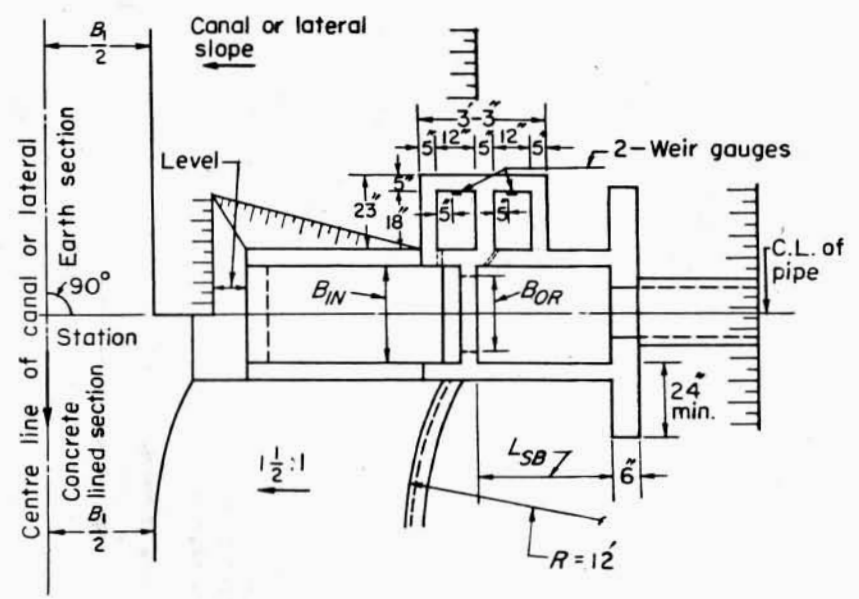
Part plan of Constant Head Orifice
showing inlet portion

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CONSTANT HEAD ORIFICE
INTAKE OR TURNOUT

Project, Region, Country
U S A

Figure No. 3-9 (b)

73.

Orifice gate
Staff gouge
Staff gouge

Minimum water level

Flow

0·2'

6"

0·3'

7"

8"

Baffle for vortex
prevention at upstream
staff gouge

Stilling basin for
downstream staff gouge

3 - 1" diameter holes

Stilling basin and anti-vortex baffles extend across
channel to fit tightly against side walls.
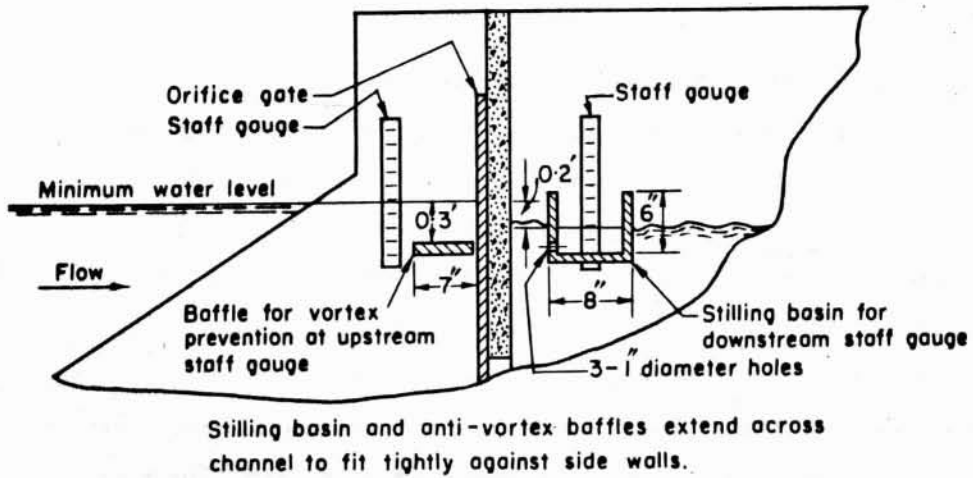
FIGURE 3-10. - Stilling baffles to reduce water surface
fluctuations at staff gauges.

$H_{(go)}$ = Gate opening for max. Q, $\frac{H_{(go)}}{H_{(fgt)}}$ max.

$H_{(fgt)}$ = Full gate leaf travel

$H_{(s)}$ = Submergence

Orifice gate leaf

W.S.

0·2'

W.S.

L_1

$H_{(s)}$

T

H_1

H_{OR}

$H_{(go)}$
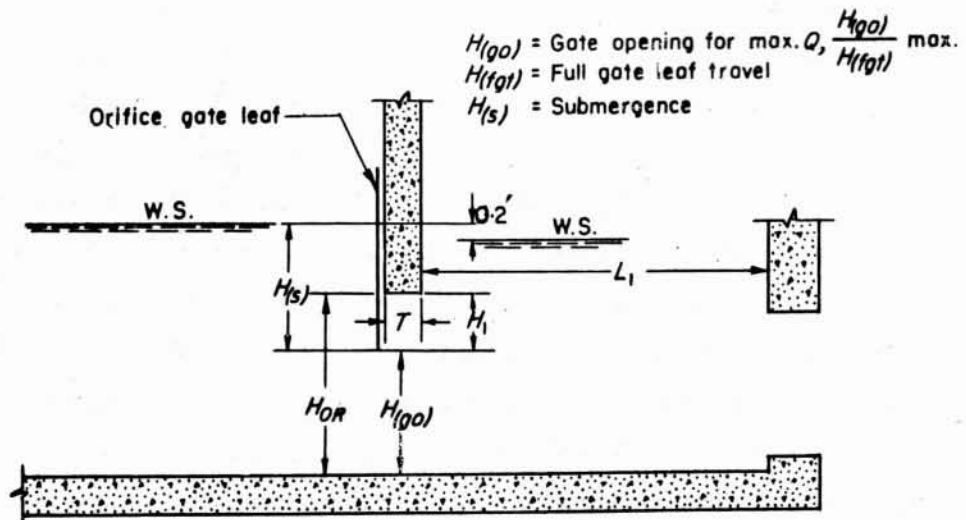
'H_1' must be equal to or greater than '(T)' for max. Q.

'$H_{(s)}$' is equal to or greater than '$H_{(go)}$' for good accuracy.

For Q up to 10 ft^3/s, L_1 must be at least $2\frac{1}{4}$ $H_{(go)}$ or $1\frac{3}{4}$ $H_{(fgt)}$,
whichever is greater. ($3'-6"$ minimum).

For Q above 10 ft^3/s, L_1 = $2\frac{3}{4}$ $H_{(go)}$ minimum.

FIGURE 3-11. - Dimensions for a constant head orifice.

of operation. The small amount of sediment which accumulates in the stilling basin between the gates has little or no effect upon constant-head orifice turnouts.

On the other hand choking by weeds that become lodged within the measuring orifice can be serious. Moreover, choking can be difficult to detect when sediment-laden water is flowing because the orifice cannot be seen. The principal cause of choking is the presence of waterlogged weeds that get caught in the opening. These weeds may trap other particles and eventually plug the turnout. The measuring accuracy of constant-head orifice turnouts is sensitive to the condition of the orifice and is greatly reduced by the presence of even a few weeds. Therefore, during regular operation, care must be taken to ensure that the orifice and the area upstream of it are kept completely clear of weeds and other debris.

3.4.3.4   Effects of approach flow conditions

Usually the turnouts are placed at 90° to the canal centreline. As a result, when the water in the canal flows past the turnout, an eddy and related flow disturbances occur at the turnout entrance. This eddy and other flow disturbances affect the flow into the turnout. The intensity of the disturbances depends largely upon the velocity of the canal flow. For small gate openings, the discharge coefficient, C_Q, for the turnout increases from a value of 0.64 at a canal velocity of about 0.3 m/s (1 foot per second) to a value of 0.69 for a canal velocity of about 0.9 m/s (3 feet per second). On the other hand, with large gate openings, increasing the canal velocity near the turnout decreases the coefficient from high values of about 0.74 at 1.0 ft/s to low values of about 0.63 at 3.0 ft/s. This appreciable, but inconsistent, effect upon the measuring accuracy of constant-head orifice turnouts must be recognized. The detrimental effect is greatest at the larger orifice openings. Therefore, whenever possible, installations should be designed so that relatively low flow velocities prevail at the turnout, especially if larger openings are to be used. Fortunately, the normal velocity distribution in canals provides relatively low velocities near the banks.

3.4.4 Design

The tests carried out show that for the general case, the rate of flow can be computed from the formula

$$Q = C_Q A_{(go)} \sqrt{2g \, \Delta h}$$

where
Q = discharge of the turnout in ft^3/s

Δh = differential head on orifice gate = 0.20 ft

$A_{(go)}$ = area of orifice gate opening in ft^2

C_Q = coefficient of discharge

g = acceleration due to gravity, 32.2 ft/s/s

The discharge coefficient, C_Q, is approximately 0.65 for the normal operating condition where the head upstream from the turnout is 2.5 or more times the maximum gate opening and no reinforcing strip is used at the bottom of the gate. If an angle-iron reinforcement is used, the coefficient will be increased to approximately 0.72. For convenience in use, discharge Tables 3-3 and 3-4 have been prepared from experimental data for the normal operation of single and double-barrel installations using 24 by 18 inch and 30 by 24 inch constant head orifice turnouts. Tables 3-2 and 3-3 show the orifice gate openings necessary to pass the desired discharges.

Studies show that when the depth of water upstream of the orifice gate is four or more times the height of the opening of the orifice, the coefficient of discharge, C_Q, remains essentially constant at 0.65. However, when the depth of water upstream is less than four times the orifice opening, the coefficient increases. The rate of increase is moderate at submergence ratios between 4 and 2.5, but is rapid at submergence ratios below 2.5. It is impractical and inaccurate to attempt to predict the coefficients for different installations having low submergence ratios, and the practice of doing so is not recommended. Instead, and if possible, the structures should be installed so the water depth in front of the orifice gate be 2.5 times, and preferably four or more times, the maximum expected gate opening. In some cases, to place the structure low enough, it may be necessary to slope the inlet channel downward (Figure 3-9(b)). An alternative design in which the inlet floor is stepped abruptly downward is also used.

TABLE 3-3

Discharge of Constant-Head Orifice Turnout in ft^3/s.,
Capacity 20 ft^3/s, Gate Size 30 by 24 Inches
h = 0.20 ft

Discharge ft^3/sec	Gate opening in ft		Discharge ft^3/sec	Gate opening in ft	
	2 gates	1 gate		2 gates	1 gate
0.25	0.02	0.04	10.25	0.81	-
0.50	0.04	0.08	10.50	0.83	-
0.75	0.06	0.12	10.75	0.85	-
1.00	0.08	0.16	11.00	0.87	-
1.25	0.10	0.20	11.25	0.89	-
1.50	0.12	0.24	11.50	0.91	-
1.75	0.14	0.28	11.75	0.93	-
2.00	0.16	0.32	12.00	0.95	-
2.25	0.18	0.36	12.25	0.97	-
2.50	0.20	0.40	12.50	0.99	-
2.75	0.22	0.44	12.75	1.01	-
3.00	0.24	0.48	13.00	1.03	-
3.25	0.26	0.52	13.25	1.05	-
3.50	0.28	0.56	13.50	1.07	-
3.75	0.30	0.60	13.75	1.085	-
4.00	0.32	0.64	14.00	1.10	-
4.25	0.34	0.68	14.25	1.12	-
4.50	0.36	0.72	14.50	1.14	-
4.75	0.38	0.755	14.75	1.16	-
5.00	0.40	0.79	15.00	1.18	-
5.25	0.42	0.83	15.25	1.20	-
5.50	0.44	0.87	15.50	1.22	-
5.75	0.46	0.91	15.75	1.24	-
6.00	0.48	0.95	16.00	1.26	-
6.25	0.50	0.99	16.25	1.28	-
6.50	0.52	1.03	16.50	1.30	-
6.75	0.54	1.065	16.75	1.32	-
7.00	0.56	1.10	17.00	1.34	-
7.25	0.58	1.14	17.25	1.355	-
7.50	0.60	1.18	17.50	1.37	-
7.75	0.62	1.22	17.75	1.39	-
8.00	0.64	1.26	18.00	1.41	-
8.25	0.66	1.30	18.25	1.43	-
8.50	0.68	1.34	18.50	1.45	-
8.75	0.70	1.375	18.75	1.47	-
9.00	0.72	1.41	19.00	1.49	-
9.25	0.74	1.45	19.25	1.51	-
9.50	0.76	1.49	19.50	1.53	-
9.75	0.775	1.525	19.75	1.545	-
10.00	0.79	1.56	20.00	1.56	-

TABLE 3-4

Discharge of Constant-Head Orifice Turnout in ft^3/s,
Capacity 10 ft^3, Gate Size 24 by 18 Inches

h = 0.20 ft

Discharge ft^3/sec	Gate opening in ft 2 gates	1 gate	Discharge ft^3/sec	Gate opening in ft 2 gates	1 gate
0.25	0.025	0.05	5.25	0.525	-
0.50	0.05	0.10	5.50	0.55	-
0.75	0.075	0.15	5.75	0.575	-
1.00	0.10	0.20	6.00	0.60	-
1.25	0.125	0.25	6.25	0.625	-
1.50	0.15	0.30	6.50	0.65	-
1.75	0.175	0.35	6.75	0.675	-
2.00	0.20	0.40	7.00	0.70	-
2.25	0.225	0.45	7.25	0.722	-
2.50	0.25	0.50	7.50	0.74	-
2.75	0.275	0.55	7.75	0.765	-
3.00	0.30	0.60	8.00	0.79	-
3.25	0.325	0.65	8.25	0.815	-
3.50	0.35	0.70	8.50	0.84	-
3.75	0.375	0.745	8.75	0.865	-
4.00	0.40	0.79	9.00	0.89	-
4.25	0.425	0.84	9.25	0.915	-
4.50	0.45	0.89	9.50	0.94	-
4.75	0.475	0.94	9.75	0.965	-
5.00	0.50	0.99	10.00	0.99	-

Current USBR designs provide standard CHO turnouts for 2, 4, 6, 9, 12, 15, 18, 24 and 30 ft^3/s deliveries. On the 2 ft^3/s size, with minimum canal water surface elevation and maximum recommended gate opening, the submergence ratio is about 4.0. As the turnout size increases, the minimum submergence ratio decreases to become about 2.0 for the 15 ft^3/s and larger sizes. Approximate discharges based upon a coefficient of 0.70 are provided in Table 3-5, on the understanding that if accuracies better than about $\pm$ 7% are

required, careful field ratings of the turnout must be made. For discharges larger than about 30 ft^3/s special structures involving multiple gates and barrels must be designed for the particular site and flow requirements.

TABLE 3-5

Discharges for Standard 2, 4, 6, 9, 12, 15, 18, 24 and 30 ft^3/s
Constant-Head Orifice Turnouts (C$_Q$ = 0.70)

2 ft^3/s Turnout (width of orifice 1.5 ft)

Discharge ft^3/s	Orifice gate opening, ft
1	0.27
2	0.53

4 ft^3/s Turnout (width of orifice 1.5 ft)

Discharge ft^3/s	Orifice gate opening, ft
1	0.27
2	0.53
3	0.80
4	1.06

6 ft^3/s Turnout (width of orifice 2.0 ft)

Discharge ft^3/s	Orifice gate opening, ft
1	0.20
2	0.40
3	0.60
4	0.80
5	0.99
6	1.19

9 ft^3/s Turnout (width of orifice 2.5 ft)

Discharge ft^3/s	Orifice gate opening, ft
1	0.16
2	0.32
3	0.48
4	0.64
5	0.80
6	0.96
7	1.12
8	1.27
9	1.43

12 ft^3/s Turnout (width of orifice 2.5 ft)

Discharge ft^3/s	Orifice gate opening, ft
1	0.16
2	0.32
3	0.48
4	0.64
5	0.80
6	0.96
7	1.12
8	1.27
9	1.43
10	1.59
11	1.75
12	1.91

TABLE 3-5 (Cont'd.)

15 ft³/s Turnout (width of orifice 3 ft)		24 ft³/s Turnout (width of orifice 4 ft)	
Discharge ft³/s	Orifice gate opening, ft	Discharge ft³/s	Orifice gate opening, ft
1	0.13	2	0.20
2	0.27	4	0.40
3	0.40	6	0.60
4	0.53	8	0.80
5	0.66	10	1.00
6	0.80	12	1.19
7	0.93	14	1.39
8	1.06	16	1.59
9	1.19	18	1.79
10	1.33	20	1.99
12	1.59	22	2.19
15	1.99	24	2.29

18 ft³/s Turnout (width of orifice 3.5 ft)		30 ft³/s Turnout (width of orifice 4 ft)	
Discharge ft³/s	Orifice gate opening, ft	Discharge ft³/s	Orifice gate opening, ft
1	0.11	1	0.10
2	0.23	2	0.20
3	0.34	3	0.30
4	0.46	4	0.40
5	0.57	6	0.60
6	0.68	9	0.90
7	0.80	12	1.20
8	0.91	15	1.49
9	1.02	18	1.79
12	1.37	21	2.09
15	1.71	24	2.39
18	2.05	30	2.99

80.

Small variation of levels
→ Parent canal
Module
Offtake channel
(a)

Large variation of levels
→ Parent canal
Guard gate
Constant level maintained
Stilling basin
Constant adjustable discharge
Offtake canal
(b)

Constant level maintained
→ Parent canal
Module
Constant adjustable discharge
Offtake canal
(c)

Constant level maintained
Parent canal
Module
Constant adjustable discharge
Or Avio if the chute is important
Offtake canal
(d)

→ Parent channel
Intake
Diagonal weir
(e)

Intake
→ Parent channel
Transversal weir
(f)

Intake
→ Parent channel
Duckbill weir
(g)

→ Parent channel
Intake
Longitudinal weir
(h)

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VARIOUS ARRANGEMENTS OF NEYRPIC ORIFICE MODULE WITH AUXILIARY EQUIPMENT AND STRUCTURES

Project, Region, Country
France and North Africa

Figure No. 3-12

3.4.5   Numerical Example

Designs can be worked out on the basis of the data and tables in the preceding paragraphs.

3.5   NEYRPIC ORIFICE MODULE INTAKE

3.5.1   General

The Neyrpic orifice module, invented by the Neyrpic Laboratories, Grenoble (France), is used as an intake for distribution canals as well as a farm outlet or farm turnout.   It is a metering device and is suitable when water is supplied on a volumetric basis.

In order that the module may draw the amount of water for which it has been designed the water level in the parent canal should be more or less constant.   If there is only a small variation in the water level in the parent canal the intake is installed directly on the bank of the parent canal (Figure 3-12(a)).   If the water level in the parent canal fluctuates beyond tolerable limits for constant flow in the offtaking channel, a constant downstream level gate must be installed at the head of the offtaking channel upstream of the module, (see also section 6.15).

Figure 3-12(b) shows a combination used when the discharge of the intake is small compared to the discharge of the parent canal.   The intake structure has a protection gate and a constant downstream water level gate before the orifice module.

Figure 3-12(c) has a constant upstream water level gate in the parent canal below the intake structure.   This arrangement is systematically used in upstream controlled networks.   Figure 3-12(d) shows an arrangement where the module intake is set immediately below the constant downstream water level gate installed in the parent canal.   This arrangement is systematically used in downstream-controlled systems.

Figures 3-13 (a) to (c) show perspective views of the arrangements of Figures 3-12(b), (c) and (d).

(b)

(c)
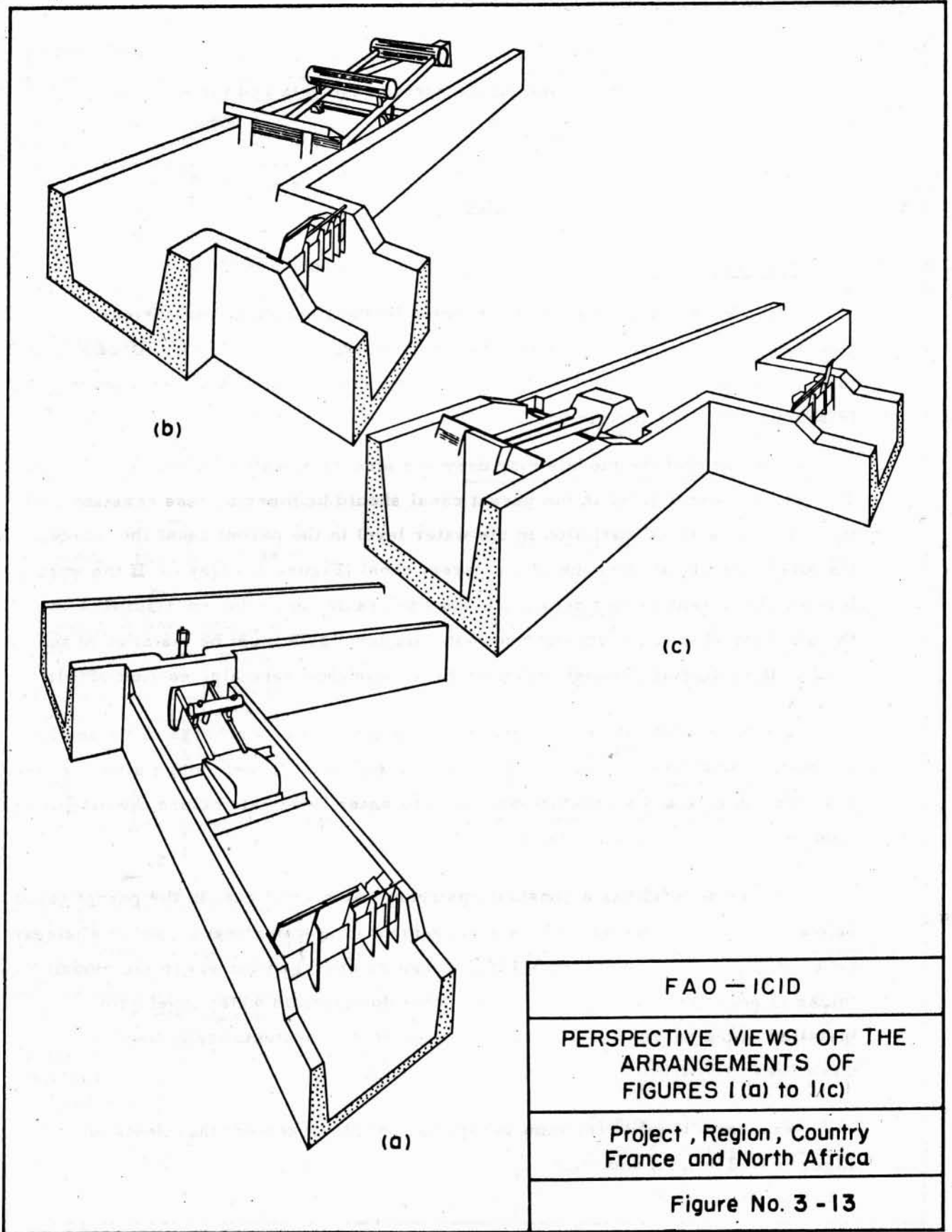
FAO — ICID

PERSPECTIVE VIEWS OF THE
ARRANGEMENTS OF
FIGURES I(a) to I(c)

Project , Region , Country
France and North Africa

Figure No. 3 -13

(a)

Figures 3-12 (e) to 3-12(h) show arrangements wherein the orifice module intake is located just above a diagonal weir, transversal weir, duckbill weir, and longitudinal weir, respectively. The purpose of these regulating structures is to keep the water level in the parent canal on their upstream side nearly constant.

The best arrangement to choose depends on economic and practical considerations such as canal grade, height of canals above the lands to be irrigated, and topographical features.

For this type of intake, only structural and design details of the orifice module intake are given here. Information about the constant upstream and downstream water levels gates and the weirs are given in Chapter 6.

3.5.2   Structural and Design Characteristics

The module consists of a sill, which has an upstream slope of 60° and a downstream glacis slope of 12°, upon which is placed a fixed metallic plate or baffle. The sill and the fixed plate or baffle are enclosed between two vertical, parallel walls, and this arrangement creates an orifice which can be closed by a sliding plate or shutter. When the width of the orifice exceeds 1 m, a footpath, 50 cm wide, is necessary in order to operate the sliding plate. The module functions only when the sliding plate is raised completely.

A distributor usually includes a number of modules connected together, each one of different width and allowing the passage of a pre-determined discharge, the volume of which is indicated on the corresponding sliding plate (see Table 3-7 for various combinations). By combining the raising of different sliding plates, the required discharge can be obtained. Thus a set of three sliding plates (Module type XX/60, Table 3-7), which allows respectively the passage of 10, 20 and 30 1/s, enables one of the following discharges to be diverted:

10, 20, 30, 40, 50 or 60 1/s.

Thus, discharge regulation is very simple. There is no gate opening to be regulated, no regime to establish, no water levels to be checked, no head discharge curve to be plotted.
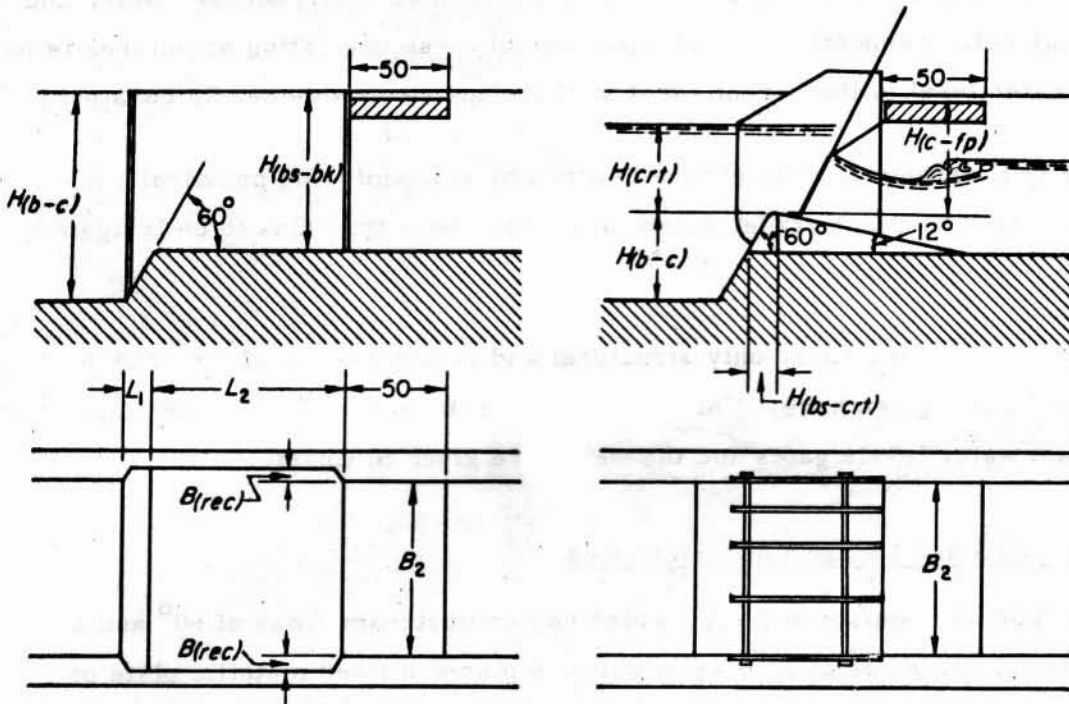
FÍGURE 3-14. - Neyrpic orifice module before and
after installation of the fixed plate.

Figure 3-14 shows the module before and after installation of the fixed
plate, the sliding plate and the sill.   Dimensions for different types of the module
are shown in Table 3-6.

TABLE   3-6

(Dimensions in cm)

Module No	$H_{(b-bk)}$ min.	$H_{(bs-bk)}$ min.	L_1	L_2	$B_{(rec)}$	L_3	$H_{(b-c)}$ min	$H_{(crt)}$	$H_{(c-ft)}$
X	41	30	5	35	5	3	16	17	25
XX	62	48	8	57	5	7	25	27	37
L	110	88	15	85	10	12	55	50	50
C	190	140	25	140	15	20	75	80	
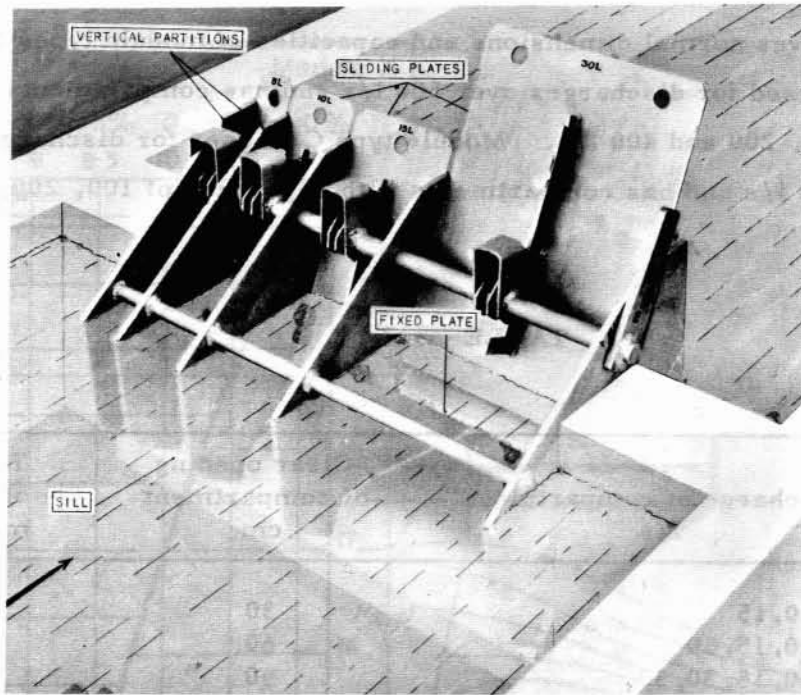
FIGURE 3-15. - Neyrpic distributor with compartments for 5, 10, 15 and 30 1/s.
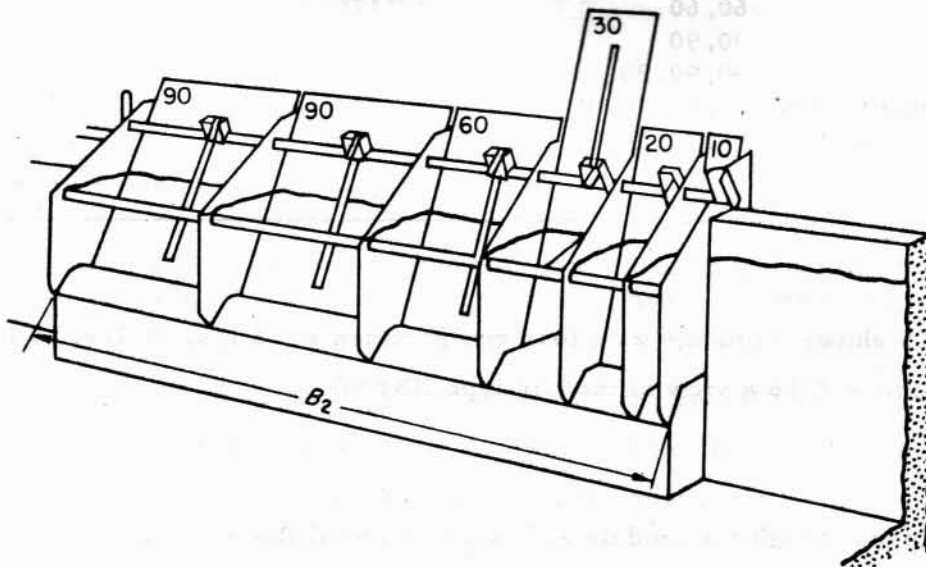
FIGURE 3-16. - Upstream view of Neyrpic distributor type XX/300.

Table 3-7 gives normal dimensions and capacities of module types X and XX. Module type L is used for discharges over 500 1/s and has compartments with capacities 50, 100, 200 and 400 1/s. Module type C is used for discharges greater than 1,000 1/s and has compartments with capacities of 100, 200, 400, 600 and 1,000 1/s.

TABLE 3-7

Type of module	Discharge of compartment 1/s	Clear opening of compartment cm	Total width of compartment, cm
X/30	5, 10, 15	30	32
X/60	5, 10, 15, 30	60	63
X/90	5, 10, 15, 30, 30	90	94
X/120	5, 10, 15, 30, 30, 30	120	125
X/150	5, 10, 15, 30, 30, 30, 30	150	156
XX/30	10, 20	15	16
XX/60	10, 20, 30	30	32
XX/90	10, 20, 30, 30	45	48
XX/120	10, 20, 30, 60	60	63
XX/150	10, 20, 30, 30, 60	75	79
XX/180	10, 20, 30, 60, 60	90	94
XX/210	10, 20, 30, 60, 90	105	109
XX/240	10, 20, 30, 60, 60, 60	120	125
XX/300	10, 20, 30, 60, 90, 90	150	155
XX/360	10, 20, 30, 60, 60, 90, 90	180	186
XX/420	10, 20, 30, 60, 60, 60, 90, 90	210	217
XX/480	10, 20, 30, 60, 90, 90, 90, 90	240	247

Figure 3-15 shows a module with four compartments: 5 1/s, 10 1/s, 15 1/s and 30 1/s and Figure 3-16 a view of module type XX/300.

Advantages

The head loss through the module is low, because of the hydraulic jump being formed on the downstream slope of the sill. The device has a wide range of operation. Wear and tear is almost nil. Any discharge (in multiples of 5) can be obtained by employing the minimum number of compartments. It is not easy to tamper with the outlet. Operation is simple ; all that is required is to
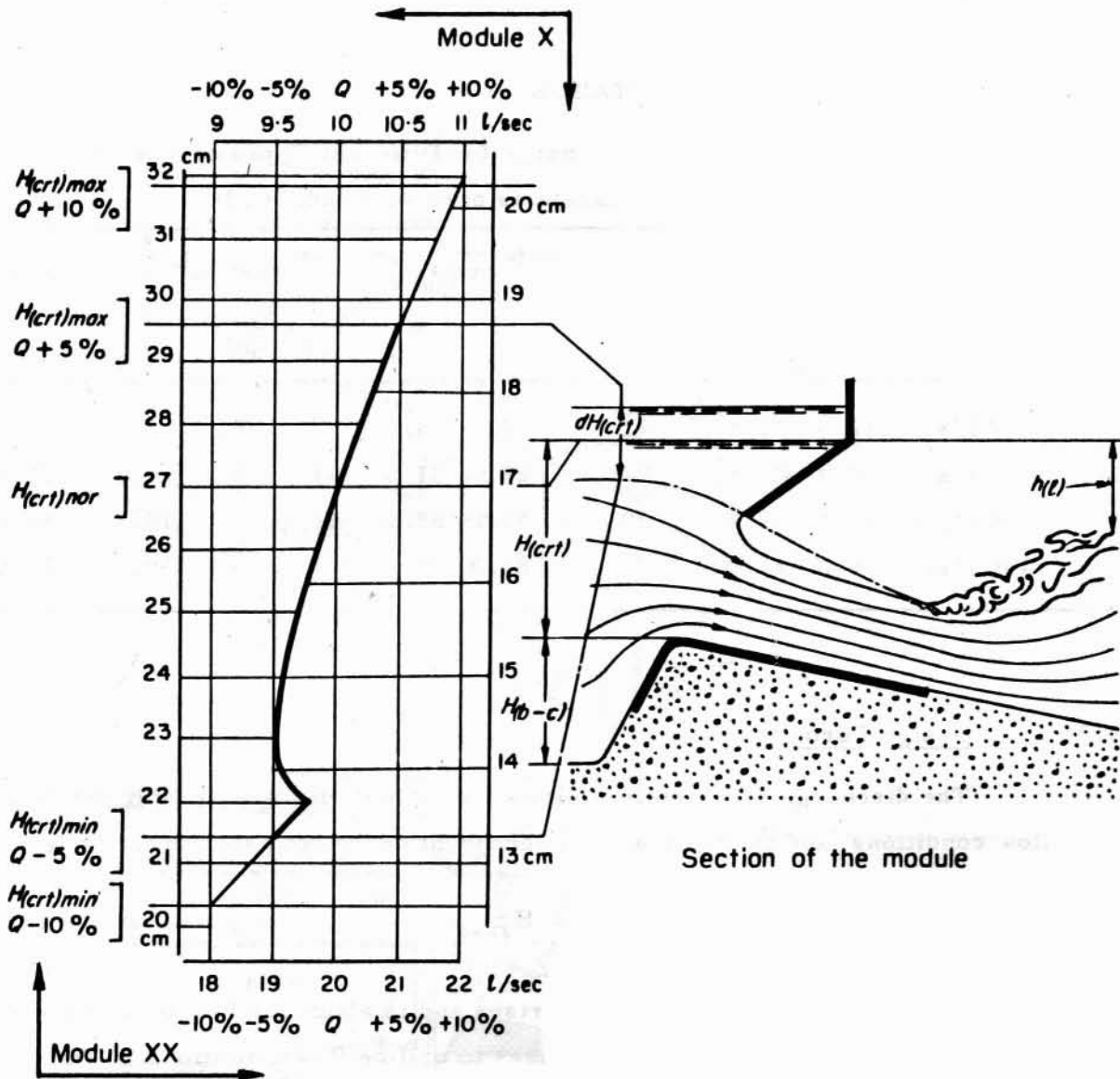
FIGURE 3-17. - Per cent variations in discharges of modules
Types X and XX for variation of $H_{(crt)}$ within pre-determined limits.

open or shut the gates as required; once set, the combination can be locked once
and for all. The amount of water withdrawn from the supply canal or supplied to
users can be determined merely by noting the opening times of the gates.

Disadvantages

The main disadvantage of the module is that it is relatively expensive. It
requires manual labour to open and close the outlet.

TABLE 3-8

Some Data for Variations in Discharge for Different Types of Module

(Dimensions in cm except as otherwise indicated)

Type of module	Discharge per 10 cm width of sill	$H_{(crt)min}$		$H_{(crt)nor}$	$H_{(crt)max}$		$dH_{(crt)}$		$h_{(1)}$	$H_{(b-c)min}$
		Q - 10%	Q - 5%		Q + 10%	Q + 5%	Q + 10%	Q + 5%		
X	10 l/s	12.8	13.5	17	18.5	19.8	7	5	6.5	16.0
XX	20 l/s	20.4	21.5	27	29.5	31.6	11	8	10.5	25.0
L	50 l/s	37.5	39.5	50	54.5	58.5	21	15	19.0	55.0
C	100 l/s	59.0	62.0	80	86.0	90.0	31	24	30.0	75.0

3.5.3   Design Formula

The discharge of the outlet follows the formula for a weir with shooting flow conditions (and is thus always independent of the downstream water level) -

$$Q = C H_{(crt)}^{\frac{3}{2}}$$

When the upstream water level rises and is above the lower tip of the fixed plate or the baffle, the weir flow changes to orifice flow conditions. The discharge falls off slightly at first - due to the vena contracta effect downstream of the baffle and because the reduced section is not quite compensated by the increased flow velocity - before beginning to increase again, thus obeying the characteristics of flow through orifices under pressure, (coefficient C tending to decrease as the height of water increases). Due to its momentary increase in the initial stages of orifice flow, the discharge varies between very narrow limits (5% to 10%) over a comparatively wide range of upstream water levels. Under designed water level upstream, the module gives exactly the discharge indicated on the shutter (Figure 3-17).
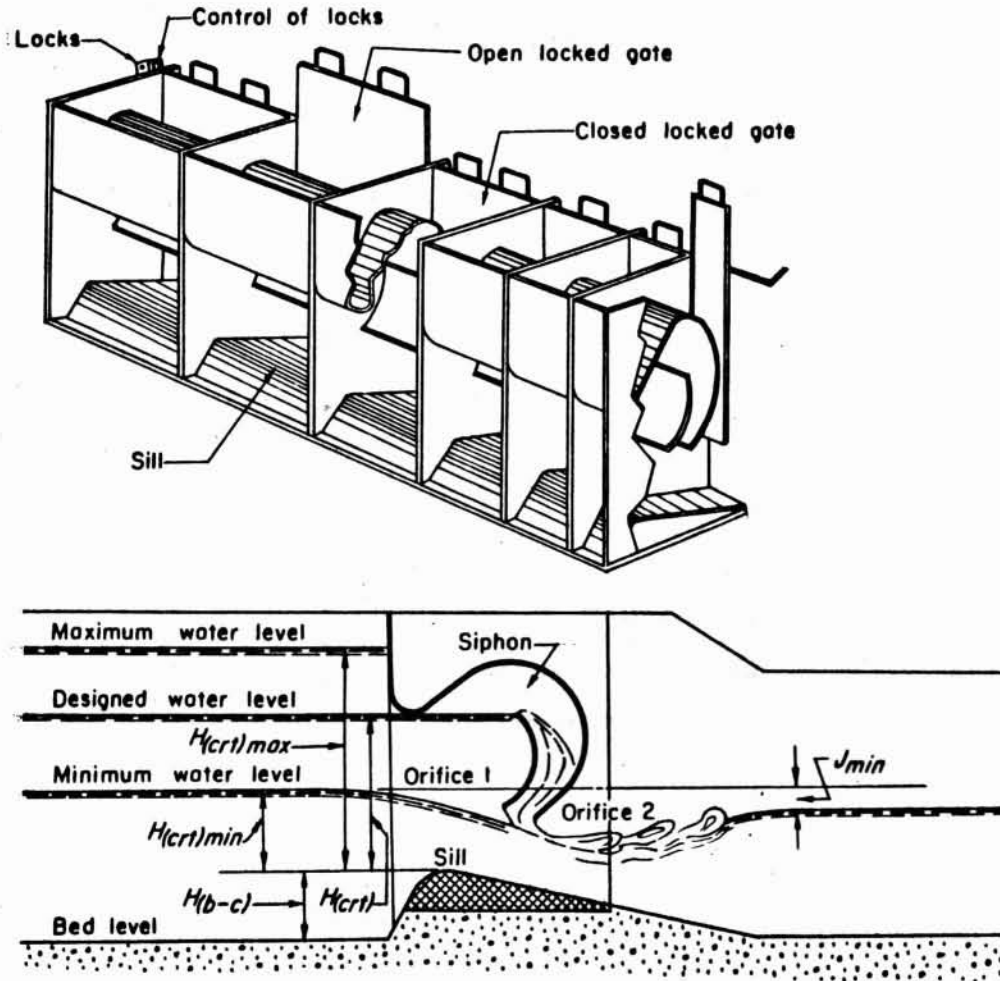
FIGURE 3-18. - Double orifice module
intake.

3.5.4 Numerical Example

Designs can be worked out, with given data, from Tables 3-6, 3-7 and 3-8.

3.6 NEYRPIC DOUBLE ORIFICE MODULE INTAKE

3.6.1 General

The double orifice module is an improvement over the Neyrpic orifice module and caters for greater variations of discharge in the supply canal. Its various arrangements with constant upstream and downstream water level gates and regulating check structures are the same as given under section 3.5.

3.6.2 Structural Characteristics

The double orifice module (Figure 3-18) consists of an inclined sill similar to that of the Neyrpic single orifice module and is provided with two vertical metallic covers forming a siphon. The calibrated openings of different widths discharge, say, 10 1/s, 20 1/s, 30 1/s or more, for a given head.

The other structural properties and dimensions of this module are similar to those of the Neyrpic single orifice module except $H_{(b-c)}$, which is given in Table 3-9, where $H_{(b-c)}$ is the height of the crest above the upstream bed level. This double orifice module has the same disadvantage as the single orifice device, moreover the siphon may become obstructed by weeds.

TABLE 3-9

Characteristics of Double Orifice Module
for $Q = \pm$ 5 per cent

Type No.	Unit discharge 1/s/dm	$H_{(crt)min}$	$H_{(crt)}$	$H_{(max)}$	$dH_{(crt)}$	J_{min}	$H_{(b-c)}$
				Dimensions in centimetres			
1	10	14.5	18.5	21.5	7.0	8	17
2	20	23	29.5	34.2	11.2	13	28
3	30	31	38.5	44.5	13.5	18	37
4	40	36	46	53.5	17.5	20.5	50
5	50	42	54	63	21	23.5	58
6	60	47	61	72	25	26.5	64
7	70	52	67	79	27	28.5	69
8	80	57	74	88	31	32	74
9	90	61	79	94	33	34	77
10	100	66	85	101	35	37	80
15	150	87	112	134	47	48.5	88
20	200	105	134	161	56	59	93

3.6.3   <u>Hydraulic Characteristics</u>

As in the case of the Neyrpic orifice module, there is a shooting flow down-
stream of the sill and the discharge through the module is independent of the
water levels in the delivery canal.

The outlet works as a semi-module as long as the upstream water level does
not reach the bottom edge of the two inner metallic covers. Thereafter, with
further rise in upstream water level, it works as a semi-module orifice till the
siphon primes and flows through, impinging on the jet, further reducing the
discharge.

Thus this improvement allows a larger range of variation in the upstream
levels to give nearly constant discharge. For example, for module Type 2
(Table 3-9) the permissible range for Q = $\pm$ 5% is 11 cm whereas it is 8 cm for
the Neyrpic module Type XX. This permits the passing of 50% more discharge
for the same width, a great advantage over the Neyrpic single orifice module.

3.6.4   <u>Design</u>

The hydraulic characteristics of the module are given in Table 3-9.

3.6.5   <u>Numerical Example</u>

Designs can be worked out, in accordance with given data, from Table 3-9
and the Tables given under section 3.5.

3.7   OPEN INTAKE STRUCTURE MADE OF PRE-CAST REINFORCED CONCRETE
(U.S.S.R.) [1]

3.7.1   <u>General</u>

Early type of intakes in southern U.S.S.R. were built with an inlet sill
(which consisted of an inclined wall), a core wall, and a rectangular flume on the
downstream side. Transition to the trapezoidal cross section of the canal on the
downstream side consisted of inclined walls. These structures were difficult

[1] Based on information supplied by A. T. Koshkina, E.P. Martin, A. V. Shatalova,
D.D. Alev and B.V. Kazarinov (U.S.S.R.)

Cross section AA

Cross section BB

Plan

Cross section CC

(All dimensions are in cm)

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OPEN INTAKE STRUCTURE MADE OF
PRECAST REINFORCED CONCRETE

Project, Region, Country
USSR

Figure No. 3-19

to install and required large quantities of reinforced concrete.

The structure used now is made of pre-cast reinforced concrete and is simple to manufacture. The parts are standardized for assembly and the structure requires low hydraulic drops. The structure does not silt during operation. Sediments deposited at low discharge are removed during high discharges.

3.7.2 Structural Characteristics and Design

The intake (Figure 3-19) consists of a ribbed bulkhead wall (diaphragm) and ribbed stiffening plates made of reinforced concrete. The bulkhead wall is installed in the canal parallel to its axis of flow. A metallic frame with a plate gate is volted to the bulkhead wall. The wall and frame of the gate are suitably sealed. The side slope is 1.25 : 1. The joints between the reinforced concrete plates are filled with cement mortar. At the end of the downstream impermeable apron there is a rock-filled knife-edged support.

Hydraulic structures need to be firm and durable and these requirements have been met by the design, materials used and careful supervision of the construction of this device. Furthermore the construction of this structure allows for modifications if necessary; stiffening plates can be replaced, the basin length increased or other parts may be replaced.

This structure needs only periodical checks of its operation and condition. The jacks need lubrication and the metallic parts need protecting with an anti-corrosive coating.

The structure can be improved by introducing automatic operation and by designing separate parts and stiffening plates with fewer joints in them so that assembly can be speeded up.

The structure is designed for discharges from 0.20 to 0.85 m^3/s. The minimum acceptable canal depth is 40 cm and the maximum depth, 80 cm. Hydraulic drops are adopted at 5 to 20 cm. Embankment height above the surface water level on the upstream side of the structure should be 25-30 cm and on the downstream side 35-50 cm. The discharge capacities of different types of the structure

TABLE   3-10

Discharge, m^3/s

Type of structure	H'(crt)	$\dfrac{h(k), \text{ cm}}{h(go), \text{ cm}}$	5	10	15	20
RO-60 . 40	40	60	0.205	0.244	0.244	-
RO-60 . 60	50	60	0.257	0.316	0.329	-
	60		0.307	0.379	0.415	0.415
RO-80 . 60	50	80	0.359	0.442	0.461	-
	60		0.434	0.538	0.538	0.588
RO-80 . 80	70	80	0.503	0.625	0.704	0.718
	80		0.571	0.712	0.810	0.852

List of Parts

Type of structure	Type of part	Weight of one part kg	Quantity (No)	Total quantity (No)
RO-60 . 40	AO-60	1,170	1	14
	P-60 . 120	110	13	
RO-60 . 60	AO-60	1,170	1	12
	P-60 . 120	110	7	
	P-60 . 180	325	4	
RO-80 . 60	AO-80	2,435	1	12
	P-120 . 180	325	11	
RO-80 . 80	AO-80	2,435	1	14
	P-120 . 180	325	13	

TABLE 3-10 (Cont'd.)

Design Dimensions

Type of construction	$H_{(c-SB)}$ cm	B_{IN} cm	L_{IN} cm	H_{DI} cm	L_{DI} cm	B_{OL} cm	L_{SB} cm
RO-60.40	15	60	120	150	390	60	290
RO-60.60	20	60	120	150	390	60	310
RO-80.60	25	120	180	210	540	120	430
RO-80.80	30	120	180	210	540	120	460

Volume of main works

Name	Material	Unit	Type of structure			
			RO-60.40	RO-60.60	RO-80.60	RO-80.80
Reinforced concrete parts	Concrete	m^3	1.04	1.30	2.40	2.66
	Reinforcement	kg	52.5	57.2	107.9	113.6
Cement	TZRM 100	m^3	0.31	0.43	0.77	0.91
Sand-filling		m^3	1.0	1.4	2.5	2.9
Rock-filling		m^3	0.5	0.9	1.1	2.1
Metal constructions		kg	73.1	72.5	38.4	93.1

are given in Table 3-10.   The intake operates normally at all heads given in that
Table.

3.7.3   Design

Calculating formula

$$Q = C_{(dsp)} \; C_{+(lat)} \; C_Q \; B_{(go)} \; \sqrt{2g} \; H'^{\frac{3}{2}}_{(crt)} \tag{1}$$

where

Q	=	proposed discharge of the regulator (m^3/s);
$C_{(dsp)}$	=	dispersion coefficient;
$C_{+(lat)}$	=	coefficient of lateral contraction;
C_Q	=	discharge coefficient equal to 0.40, obtained from laboratory data;
$B_{(go)}$	=	width of gate opening of the regulator;
$H'_{(crt)}$	=	head over crest with approach velocity
	=	$H_{(crt)} + \dfrac{v^2_{(app)}}{2g}$

where

$v_{(app)}$ =   velocity of approach

The dispersion coefficient is a function of the ratio $\dfrac{H_{(s)}}{H_{(crt)}}$ where $H_{(s)}$ = depth
of submergence (see Figure 3-19).   Values of dispersion coefficient are given in
Table 3-11.

TABLE   3-11

Values of Dispersion Coefficient, $C_{(dsp)}$

$\dfrac{H_{(s)}}{H'_{(crt)}}$	$C_{(dsp)}$	$\dfrac{H_{(s)}}{H'_{(crt)}}$	$C_{(dsp)}$	$\dfrac{H_{(s)}}{H'_{(crt)}}$	$C_{(dsp)}$
Up to 0.7	1.000	0.85	0.855	0.97	0.436
0.75	0.974	0.90	0.739	0.99	0.252
0.80	0.928	0.95	0.552	0.995	0.183

Coefficient of lateral contraction is determined by using the following
formula :

$$C_{+(lat)} = 1 - 0.2 C_{(shp)} \frac{H'_{(crt)}}{B_{(go)}} \qquad (2)$$

where

$C_{(shp)}$ = coefficient of shape of spillway vertical ribs;

$C_{(shp)}$ = 1 for the given structure.

Toe basin depth in these structures is chosen in accordance with the
requirement of hydraulic jump submergence with a submergence coefficient,
$C_{(js)}$ = 1.2.

Total basin length is determined from the formula

$$L_{(SB)} = L_{(SB_1)} + L_{(j)} \qquad (3)$$

where

$L_{(SB_1)}$ = $1.25 \sqrt{ H'_{(crt)} \left\{ H_{(c-SB)} + 0.45 H'_{(crt)} \right\} }$

$L_{(j)}$ = $4 H_{(reci)_2}$ = the length of hydraulic jump;

$H_{(c-SB)}$ = height of the sill from the downstream floor

$H_{(reci)_2}$ = the second reciprocal depth.

The second reciprocal depth $H_{(reci)_2}$ is determined in the following
sequence.

Critical depth is determined by the formula $H_c = \sqrt[3]{\dfrac{q^2}{g}}$

where q = $\dfrac{q}{B_{(go)}}$ = discharge per unit width;

Incident energy   =   total height of upstream energy line over downstream floor on apron

$$= \quad E'_{SB_1} \quad = \quad H'_{(crt)} \quad + \quad H_{(c-SB)}$$

$$\frac{rH_{(go)}}{E'_{SB_1}} \quad = \quad \frac{H_{(go)}}{E'_{SB_1}} \quad \text{is defined by using Table 3-12 in terms of} \quad \frac{q}{E'_{SB_1}}$$

TABLE   3-12

$\dfrac{q}{E'_{SB_1}}$	$\dfrac{rH_{(go)}}{E'_{SB_1}}$	$\dfrac{q}{E'_{SB_1}}$	$\dfrac{rH_{(go)}}{E'_{SB_1}}$	$\dfrac{q}{E'_{SB_1}}$	$\dfrac{rH_{(go)}}{E'_{SB_1}}$
0.1	0.4	0.3	0.120	0.5	0.205
0.2	0.08	0.4	0.162	0.6	0.250

Magnitude of the gate opening is determined from the formula

$$H_{(go)} \quad = \quad E'_{SB_1} \; \frac{rH_{(go)}}{E'_{SB_1}} \tag{4}$$

Depth in the contracted cross section is determined from the following formula

$$H_{(j)} \quad = \quad C_{+(vert)} \; H_{(go)} \tag{5}$$

where $C_{+(vert)}$ =   coefficient of jet vertical contraction   magnitudes of which are given in Table 3-13 in terms of magnitudes

$$\frac{rH_{(go)}}{E'_{SB_1}}$$

TABLE 3-13

$\dfrac{^{r}H_{(go)}}{E'_{SB_1}}$	$C_{+(vert)}$	$\dfrac{^{r}H_{(go)}}{E'_{SB_1}}$	$C_{+(vert)}$	$\dfrac{^{r}H_{(go)}}{E'_{SB_1}}$	$C_{+(vert)}$
0.0	0.612	0.3	0.625	0.5	0.644
0.1	0.615	0.4	0.633	0.6	0.661
0.2	0.620				

The second reciprocal depth is determined by using the following formula:

$$H_{(reci)_2} = C' H_c \qquad (6)$$

where $\quad C' = \dfrac{1}{0.17 + 0.83\, C''}, \qquad C'' = \dfrac{H_{(j)}}{H_c}$

3.7.4 Numerical Examples

3.7.4.1 Data

The total head over sill is $H'_{(crt)} = 0.70$ m, gate opening, $B_{(go)} = 0.8$ m, hydraulic drop, $h_{(wk)} = 0.13$ m. It is required to check discharge of the structure and conditions of the hydraulic jump submergence on the downstream side.

For $\quad h_{(wk)} = 0.13$ m, submergence depth, $H_{(s)} = 0.7 - 0.13$

$\qquad\qquad = 0.57$ m

$\qquad$ For value $\dfrac{H_{(s)}}{H'_{(crt)}} = \dfrac{0.57}{0.70} = 0.814$

$C_{(dsp)} = 0.91$, from Table 3-11

$C_{+(lat)} = 1 - 0.20 \cdot 1 \cdot \dfrac{0.7}{0.8} = 0.825$

from formula (1)

$$q = 0.91 \cdot 0.825 \cdot 0.40 \cdot 0.80 \sqrt{2 \cdot 9.81} \cdot 0.7^{\frac{3}{2}}$$
$$= 0.62 \text{ m}^3/\text{s}$$

The basin length is determined from formula (3).

For structure type RO - 80 . 80 (chosen in accordance with Table 3-10), $H_{(c-SB)} = 0.3$ m. For this value $L_{SB_1} = 1.25 \sqrt{0.7 (0.3 + 0.45 \cdot 0.7)}$
$$= 0.82 \text{ m}.$$

To determine the hydraulic jump length, $L(j)$, various discharge magnitudes are taken, namely, Q, $\frac{Q}{2}$, $\frac{Q}{3}$, $\frac{Q}{4}$, and reciprocal depths are calculated for these discharges as well as the length of the downstream apron. From the data obtained the maximal value is accepted as the length of structural apron downstream.

An example for computing hydraulic jump length for $\frac{Q}{2}$ (0.31 m^3/s) is given below.

Unit discharge, $q = \dfrac{0.31}{0.8} = 0.388 \text{ m}^3/\text{s}$

Critical depth, $H_c = \sqrt[3]{\dfrac{q^2}{g}} = \sqrt[3]{\dfrac{0.388^2}{9.81}}$
$$= 0.249 \text{ m}$$

$E'_{SB_1} = 0.7 + 0.3 = 1.0 \text{ m}$

$\dfrac{q}{E'_{SB_1}} = \dfrac{0.388}{1.0} = 0.388$

The magnitude of the gate opening is determined from formula (4) together with Table 3-12, i.e. $\dfrac{^rH_{(go)}}{E_{SB_1}} = 0.157$

$$H_{(go)} = 1.0 \cdot 0.157 = 0.157 \text{ m}.$$

The depth in contracted cross section is determined from formula (5) together with Table 3-13.

$$H_{(j)} = C_{+(vert)} H_{(go)} = 0.617 \cdot 0.157 = 0.097 \text{ m}.$$

C' is determined from the following formula:

$$C' = \frac{1}{0.17 + 0.83 \; C''} = \frac{1}{0.17 + 0.83 \; \dfrac{0.097}{0.219}} = 2.03$$

The second reciprocal depth is determined from formula (6), i.e.

$$H_{(reci)_2} = 2.03 \; . \; 0.249 = 0.51 \, m$$

The hydraulic jump length is then:

$$L_{(j)} = 4 \; . \; 0.51 = 2.04 \, m$$

The total downstream apron length is determined by using formula (3):

$$L_{(SB)} = L_{(SB_1)} + L_{(j)} = 0.82 + 2.04 = 2.86 \, m$$

The difference in the downstream depth and the second reciprocal depth is then calculated:

$$y_2 - H_{(j)_2} = (0.70 - 0.13) - 0.51 - 0.06 \, m$$

The submergence coefficient of the hydraulic jump,

$$C_{(js)} = \frac{H_{(s)}}{H_{(j)_2}} = \frac{(0.7 - 0.13)}{0.52} = 1.12. \quad \text{Hence, the jump is}$$

submerged with an adequate safety factor.

3.7.4.2   Choosing an open intake structure

Assume that the canal discharge is $0.35 \, m^3/s$, the water depth in the canal is 65 cm, and the canal width is 60 cm.

From Table 3-10 it is seen that a structure of RO-60 . 60 type with $H_{(crt)} = 60$ cm and $B_{(go)} = 60$ cm will convey a discharge of $0.31 \, m^3/s$ for $h_{(wk)} = 5$ cm and a discharge of $0.38 \, m^3/s$ for $h_{(wk)} = 10$ cm. For conveyance of a discharge of $0.35 \, m^3/s$ it is necessary to determine $h_{(wk)}$ by interpolation, resulting in $h_{(wk)} = 8$ cm. With a canal depth of 65 cm, $H_{(crt)} = 60$ cm. Hence the structure should be 5 cm higher than the canal bottom $(65 - 60 = 5 \, cm)$.

Gauging device DRS-60

Gate

716

Detail S

Detail TC

30

H_{TS}

$L_{(p)}$

$h_{(wk)}$

$D_{(p)}$

Pipeline

y_2

26

30

30

200

Detail D

$d_{(toe)}$

100

Cross section A-A

B

C

D

Detail P-I

Flume

A

A

100

$D_{(p)}-120$

Inlet

B

C

D

Plan

Note: All dimensions are in centimetres.

Detail SH

Flume

$H_{(well)}$

B_{WL}

Cross section B-B

Cross section C-C

Cross section D-D
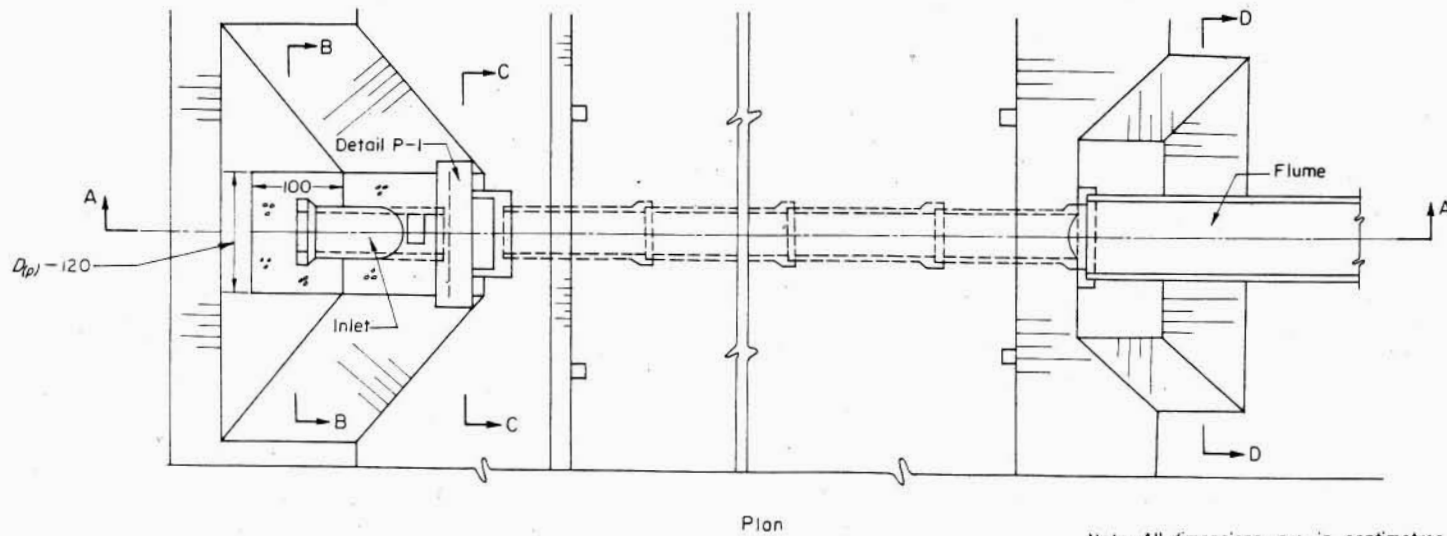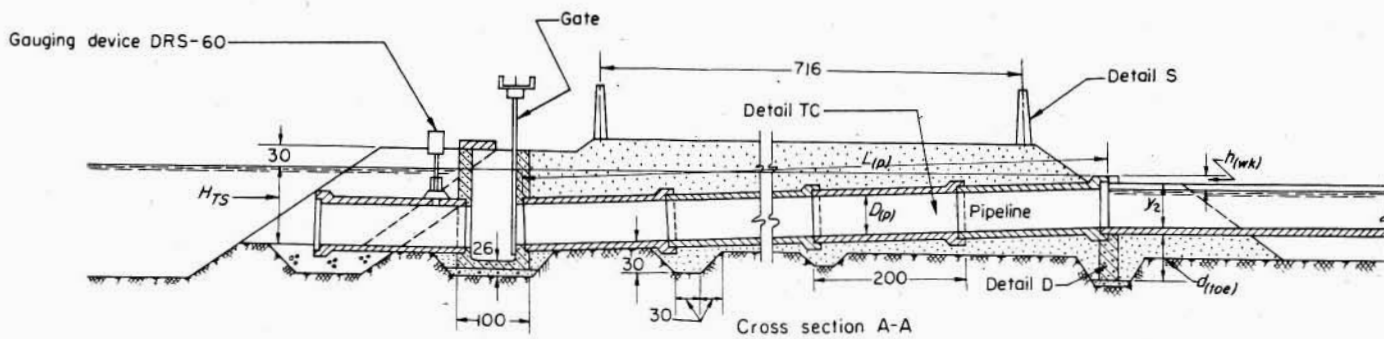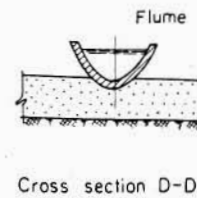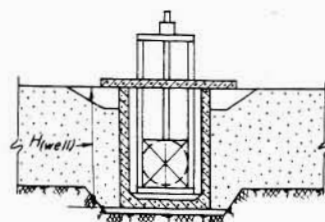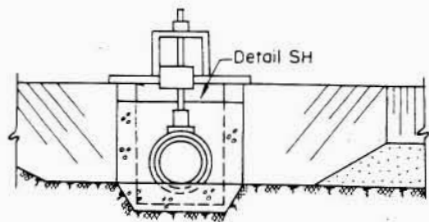
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INTAK STRUCTURE DISCHARGING
INTO A FLUME CHANNEL

Project, Region, Country
USSR

Figure No. 3 - 20

105.

3.8   INTAKE STRUCTURE DISCHARGING INTO A FLUME CHANNEL (U.S.S.R.)

3.8.1   General

The types of intake structures described herein are for delivering water from unlined and lined canals into a flume irrigation canal.   The discharge through the intake into a flume 60 cm in depth is 400 l/s, and into a flume 80 cm in depth, 900 l/s.

3.8.2   Structural Characteristics and Design

The intake structure (Figure 3-20) consists of an entrance structure and a pipeline.   The entrance structure is a well with an adjacent upstream part comprising a reinforced concrete pipe.   There is an opening in the pipe for embedding the lower end of the water gauge device.   The top part of the water gauge device with a DRS - 60 water gauge is secured to a well.   A metal slide gate with a screw jack is mounted in the well.

The pipeline consists of a number of reinforced concrete bell-and-spigot pipes.   A crossing, 7 m wide, is provided above the pipeline.   The pipe and the flume are connected by means of a diaphragm which has cut-outs corresponding to the pipe and the flume outer periphery.

The intake structural parts are prefabricated and mounted and adjusted according to special instructions.

The design, the materials used and careful construction provide for the required strength and life of the device.

The structure requires periodical inspection to see that it is operating correctly and to check on the state of the construction and to carry out running repairs if any defects be found.   It is necessary to lubricate the jacks periodically and to protect the parts by a suitable anti-corrosion compound.

3.8.3   Design

Calculating formula

$$Q = C_Q A_{x(p)} \sqrt{2gh_{(wk)}}$$

where $A_{x(p)}$ =   sectional area of a pipe, m^2

$h_{(wk)}$ =   hydraulic drop or working head, m

Q =   designed discharge, m^3/s

C_Q =   discharge coefficient

$$= \frac{1}{\sqrt{C_{(f-OL)} + C_{(f-IN)} + C_{(f-shaft)} + C_{(f-p)}}}$$

$C_{(f-OL)}$ =   1   =   outlet resistance coefficient;

$C_{(f-IN)}$ =   0.4   =   entrance resistance coefficient, accepted according to laboratory test data;

$C_{(f-shaft)}$ =   0.26   =   well resistance coefficient, accepted according to laboratory test data;

$C_{(f-p)}$ = $\dfrac{2gL_{(p)}}{C^2 R_{(H)}}$ =   pipe length resistance coefficient with coefficient of roughness $N = 0.012$;

$L_{(p)}$ =   pipe length;

C =   72   =   Chezy friction coefficient;

$R_{(H)}$ = $\dfrac{D_{(p)}}{4}$ =   hydraulic radius;

g =   9.81   =   gravity acceleration

The required head at the pipe entrance is calculated in accordance with normal water gauge operation, when the pipe upper edge is 20-25 cm lower than the rated water level.

Tables of discharge capacity, dimensions, and lists of construction materials and parts are given in Table 3-14.

3.8.4   Numerical Example

Data

Design discharge	=	700 l/s	=	0.7 m^3/s
Flume depth	=	y_2	=	80 cm
Pipeline diameter $D_{(p)}$			=	80 cm
Length of pipe			='	12 m

TABLE 3-14

Discharge Capacity

Type of Structure	$h_{(wk)}$, cm	5	10	15	20	25	30
VKLV - 60	Q, m³/s	0.2	0.28	0.34	0.40	-	-
VKLV - 80	Q, m³/s	0.36	0.51	0.63	0.72	0.81	0.89

Design Dimensions

Type of Structure	$D_{(p)}$	y_2 cm	H_TS cm	H_{WL} cm	B_{WL} cm	$d_{(toe)}$ cm	$L_{(p)}$ cm
VKLV - 60	60	60	105	165	120	56	1000
VKLV - 80	80	80	130	185	140	44	1000

Volume of Main Works

Type of Structure	Reinforced concrete details		Cement m³	Gravel Filling m³	Metal Construction kg
	Concrete m³	Reinforcement kg			
Type - 60	3.05	207	0.11	1.33	145
Type - 80	4.38	262	0.14	1.38	167

List of Parts

Type of Structure	Type of part	Weight of part kg	No. of parts	Total number
VKLV-60	TR - 8	660	6	13
	SH - 60	2,200	1	
	P - 1	220	1	
	D - 1	725	1	
	S	125	4	
VKLV-80	TR - 10	1,140	6	13
	SH - 80	2,500	1	
	P - 1	220	1	
	D - 2	875	1	
	S	125	4	

It is necessary to calculate the working head $h_{(wk)}$.

<u>Solution</u>

According to the formula given under 3.8.3

$$C_{(f-p)} = \frac{2 \cdot 9.81 \cdot 12}{72^2 \cdot 0.2} = 0.23$$

$$A_{x(p)} = \frac{\pi D^2_{(p)}}{4} = \frac{3.14 \cdot 0.8^2}{4} = 0.503 \, m^2$$

$$R_{(H)} = \frac{D_{(p)}}{4} = \frac{0.8}{4} = 0.2 \, m$$

$$C = 72$$

$$C^2_Q = \frac{1}{\sqrt{0.4 + 0.26 + 0.23 + 1}} = 0.725$$

$$h_{(wk)} = \frac{Q^2}{C^2_Q \, A^2_{x(p)} \, 2g} = \frac{0.7^2}{0.725^2 \cdot 0.503^2 \cdot 2 \cdot 9.81}$$

$$= 0.187 \, m \text{ or } 19 \, cm.$$

3.9 PIPE REGULATOR WITH CROSSING MADE OF PRE-CAST REINFORCED CONCRETE (U.S.S.R.) [1]

3.9.1 General

Pipe regulators are used for the diversion of water to small canals. As compared with early types in use, the latest structure provides better dissipation of energy on the downstream side and also has only one joint in the pipeline.

3.9.2 Structural Characteristics and Design

The pipe regulator (Figure 3-21) consists of three main units: inlet sill, pipeline and downstream apron.

The inlet sill consists of an inclined wall which is supported by bottom and portal decks.

[1] Based on a note by A.T. Koshkina, E.P. Martin, A.V. Shatalova, D.D. Aliev and B.V. Kazarinov (U.S.S.R.).

Cross section A-A

Cross section B-B

Cross section C-C

Cross section D-D

Cross section E-E

Plan

In the drawing (Plan) alternative applications of pipe regulator are shown at the top — As an impound structure, at the botton— As an outlet.

Note: All dimensions are in centimetres.

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PIPE REGULATOR WITH CROSSING
MADE OF PRECAST R.C.C.

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Figure No. 3-21

The pipeline consists of two pipe lengths with their bells laid on a levelled earth foundation.   The joint is sealed with tow or mineral wool, impregnated with bitumen, and then covered with cement.   The outlet pipe on the downstream side extends from the sloping wall as shown.   This pipe is followed by a damper of semi-cylindrical shape with a ring diaphragm at the end.

Slope stabilization at the inlet sill and the downstream apron is secured by pre-cast reinforced plates; the side slope is 1 : 25 : 1.   In front of the inlet sill there is a well and at the end of the downstream apron there is a rock-filled knife-edged support.   The height of the embankment above water level on the upstream side of the structure is 25–30 cm and on the downstream side 30-35 cm.

The design, materials used and careful supervision of construction ensure that this type of structure has the necessary strength and durability.

The structure requires periodical checking for its correct operation and condition and for carrying our running repairs if required.   Lubrication of the jacks and protection of the metallic parts with an anti-corrosive coating is necessary periodically.

This type of structure conveys discharges from 0. 30 to 0. 90 m^3/s.   The working heads permissible range from 10 to 30 cm, and the water depth in front of the gate is taken at 125 and 150 cm.   See Table 3-15 for discharge capacities, dimensions, construction materials and list of parts.

3. 9. 3   Design

Calculating formula

Discharge capacity (in m^3/s) is determined from the formula

$$Q = C_Q A_{x(p)} \sqrt{2 g h_{(wk)}}$$

TABLE   3-15

Discharge Capacity m³/s

Type of Structure	$\dfrac{h(wk),\ cm}{D(p),\ cm}$	10	15	20	25	30
TR-60-125	60	0.29	0.36	0.42	0.47	0.51
TR-80-150	80	0.53	0.66	0.76	0.85	0.93

Design Dimensions

Type of Structure	$D(p)$ cm	$H_{(crt)}$ cm	L_{IN} cm	$L_{(prot)}$ cm	B_{IN} cm	B_{OL} cm	E'_{SB_1} cm
TR-60-125	60	125	210	320	90	60	115
TR-80-150	80	150	240	580	110	120	150

Volume of main works

Name	Material	Unit	TR-60-125	TR-80-150
Reinforced concrete	Concrete	m³	3.10	5.87
Cast-in-place concrete	Reinforcement	kg	205	359
	Concrete	m³	0.50	0.53
Cement		m³	1.65	2.56
Rock-filling	Rock-filling	m³	1.7	2.9
Metal constructions		kg	130	153

List of Parts

Type of structure	Type of part	Weight of one part kg	Number of parts	Total number
TR-60-125	TR-60	1,350	2	
	ON-60	450	1	
	N - 60	352	2	
	D - 60	418	1	
	GT-60	433	1	20
	P-60 . 120	110	1	
	P-120 . 180	325	4	
	SH - 180	310	4	
	S	100	4	
TR-60-150	TR-80	2,575	2	
	N - 80	598	1	
	N - 80	455	2	
	D - 80	580	1	
	GT-80	928	1	39
	P-120 . 180	325	12	
	SH-120	138	16	
	S	100	4	

where

$$C_Q = \frac{1}{\sqrt{C_{(f-IN)} + C_{(f-OL)} + C_{(f-p)}}}$$

C_Q = discharge coefficient

$C_{(f-IN)}$ = 0.5 = inlet friction coefficient, accepted according to laboratory data;

$C_{(f-OL)}$ = 1 = outlet friction coefficient accepted according to laboratory data;

$C_{(f-p)} = \dfrac{2gL_{(p)}}{C^2 R_{(H)}}$ = friction coefficient along the pipe for roughness coefficient $N = 0.012$;

$h_{(wk)}$ = hydraulic drop or working head, m;

$L_{(p)}$ = pipe length, m;

C = Chezy coefficient;

$R_{(H)} = \dfrac{D_{(p)}}{4}$ hydraulic mean radius, m;

g = 9.81 = gravity acceleration, m/s^2;

$A_{x(p)}$ = pipe cross-sectional area.

Protection length on the downstream side is determined from the following formula :

$$L_{(prot)} = 2.2 \frac{v_{(p)}}{v_{(flu)}} D_{(p)}$$

where

$v_{(p)}$ = velocity in the pipe, m/s;

$v_{(flu)}$ = velocity allowed for wash-out, accepted for medium loam, as equal to 0.8 m/s. In the case fine and sandy loam soils in the foundation of structures $v_{(flu)}$ is assumed to be equal to 0.45 m/s.

3.9.4 Numerical Example

Data

$h_{(wk)}$ = 0.17 m; pipe diameter = 0.8 m, $L_{(p)}$ = 12.5 m
(structure: pipe regulator, TR-80-150 type).

Bed soil: medium loam. C = 72

What is the discharge of the structure, and basin length on the downstream side?

Solution

Determine discharge from the formula:

$$Q = C_Q A_{x(p)} \sqrt{2gh_{(wk)}}$$

where:

$$C_Q = \frac{1}{\sqrt{C_{(f-IN)} + C_{(f-OL)} + C_{(f-p)}}} = \frac{1}{\sqrt{0.5 + 1 + 0.24}} = 0.758$$

$$C_{(f-p)} = \frac{2 \cdot 9.81 \cdot 12.5}{72^2 \cdot 0.2} = 0.24$$

$$A_{x(p)} = \frac{\pi D^2_{(p)}}{4} = \frac{3.14 \cdot 0.8^2}{4} = 0.503 \text{ m}^2$$

$$R_{(H)} = \frac{D_{(p)}}{4} = \frac{0.8}{4} = 0.2 \text{ m}$$

$$C = 72.0$$

hence:

$$Q = 0.758 \cdot 0.503\sqrt{2 \cdot 9.81 \cdot 0.17} = 0.696 \text{ m}^3/\text{s}$$

Assume average velocity, allowed for wash-out, in the canal $v_{(flu)} = 0.8$ m/s (for medium loam), then determine the basin length:

$$v_{(p)} = \frac{Q}{A_{x(p)}} = \frac{0.70}{0.502} = 1.4 \text{ m/s}$$

$$L_{(prot)} = 2.2 \cdot \frac{v_{(p)}}{v_{(flu)}} \cdot D_{(p)} = 2.2 \cdot \frac{1.4}{0.8} \cdot 0.8 = 3.1 \text{ m}$$

Tertiary canal — Limit of structure — D

Maintenance road axis

~10 per cent   ~10 per cent

4

Gate 2

0.20   Concrete block lateral wall
0.20

E   0.20
0.2 | 0.6   0.5
1.5 H(wall)   0.4
1.80   B=1.0   B_ff=1.00 $1.5(H_{(wall)}+0.30)$ $1.5 H_{(wall)}$

Canal axis   1.00   0.75
1.5:1   1.00   1.90   Canal axis
0.90   Gate 1   0.75
0.20   0.4   0.25
                    0.2
A $4.5(H_{(wall)}+0.30)$ $1.5 H_{(wall)}$
2   0.20

0.6m pipe — Limit of the structure
Tertiary canal

Plan

B   C

C.L. Pipeline — Compacted fill of selected material
0.20   2   0.20   0.15
0.9   0.10
Dia. 0.6m   0.25   R=0.35
                    Structure   0.6
                    slab elevation   0.20
0.50   0.50   0.15

Compacted granular material base

Section E-E   Spillway detail

— Variable (1.5 H_{(wall)})   C.L. Canal
C.L. Maintenance road
4   1.8   Dyke
1.5   0.2   0.2   1.5
Compacted embankment   H_{(wall)}
                           Lining
                           Concrete

Section B-B

0.40   0.90   0.40   Compacted fill
1.5   1.5
Mortar fill
0.6 m pipe   Gate   0.10 0.6 m pipe   Tertiary canal
          0.1   0.15 0.10   (This structure will be
Variable   2.60   Variable   dimensioned for every
          C.L. Canal   tertiary canal)

Section C-C

Concrete back or   C.L. Canal
mass concrete wall
0.60   0.15   0.15   0.60
1.50   0.50   0.50   1.00   1.50

Section D-D

Note: All dimensions are in metres.

Elevation A
H_{(wall)}   Elevation A   Maintenance   Elevation D
                    Concrete block wall   ~10 per cent
Elevation B   0.7 0.40   El. B
Cutoff   0.30 0.6 0.15 1.5   H_{(DR)}   0.50   0.25   H_{(wall)}
Lined canal   2   2.20   1.15   0.20 0.7 El.C   El. C
                              $1.5(H_{(DR)}+0.40)$ 0.7 0.40 0.15 0.6 1.25
Weep holes   0.30 Cutoff
see section   Stilling basin length   5
D-D   Lined canal
Section A-A

FAO — ICID

INTAKE STRUCTURE ON SECONDARY CANALS
PLAN-SECTIONS-DIMENSIONS

Project, Region, Country
Colombia

Figure No. 3-22

3.10   INTAKE STRUCTURE ON SECONDARY CANALS (COLOMBIA)

3.10.1   General

The structure described herein meets the requirements for water level control for the diversion of water into tertiary canals, providing at the same time the necessary drop in the secondary canal.

These structures have been designed at the Instituto Colombiano de la Reforma Agraria, INCORA, to meet the requirements of the Bolivar No. 1 Irrigation and Drainage Project, located at the Bolivar Department, in the north of Colombia.   The Project's plan foresees the construction of 120 structures of this type.   Up to now 40 have been built, of which 90% are in operation.

This intake structure (Figure 3-22) is not yet considered sufficiently tested because development of irrigation in Colombia is recent.   The structure is easy to construct from its pre-cast concrete parts.

3.10.2   Structural Characteristics

The structure is designed for flexibility and use under different topographical conditions and at different places, both on the project area and within the canal system.   For example, some dimensions may be varied so that the canal design engineer can fit the structure according to requirements at different sites. The fixed dimensions of the structure depend upon the maximum design flow, and upon the longitudinal fall of the secondary canal.

Based on these parameters, structures have been designed and constructed which allow the diversion of flows between 100 l/s and 750 l/s on earth secondary canals whose capacities are between 500 l/s and 3,000 l/s.   Observations on the prototype under different operating conditions, particularly as to flow and heading up of water, have not indicated important deficiencies in the hydraulic operation of the structure.

The depth of the intake structure on secondary canals is small, between 0.80 m and 1.40 m, and the side walls have been designed to resist both the earth

Note: The broken lines show
the wall perimeter.
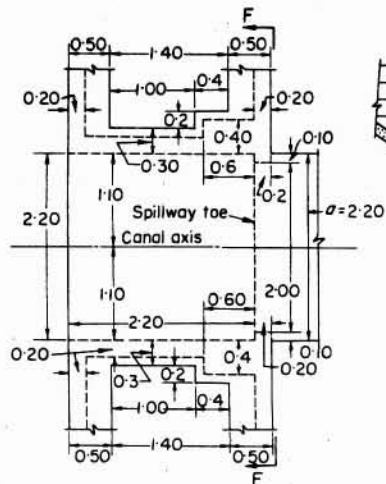
Plan of bottom slab
A and B type structures

Plan of bottom slab

Note: All dimensions are in metres unless
otherwise specified.
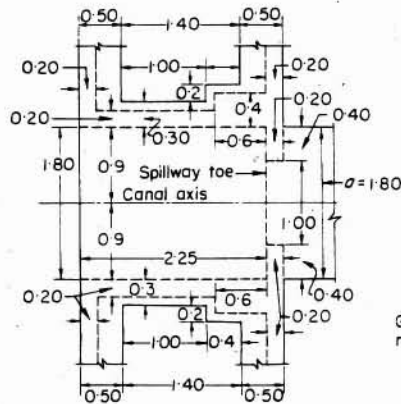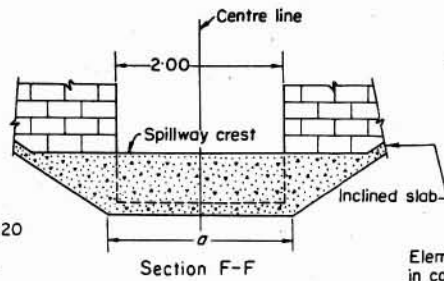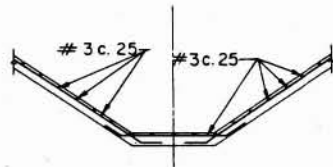
Section F-F

Section D-D
Reinforcement detail
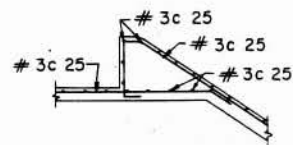
Spillway reinforcement detail

Weep holes detail

Gate I

Element I embedded
in concrete slab

Gate 2

Embedment of the
element

Section I-I

6 mm anchor
plate for
I beam
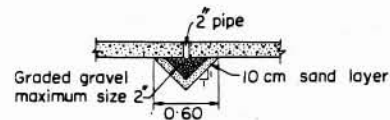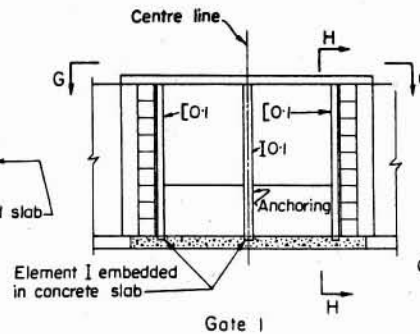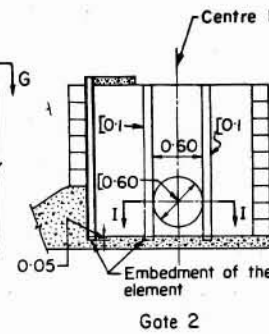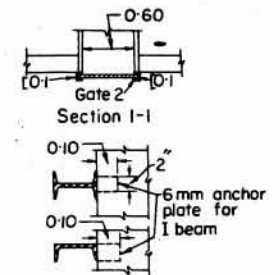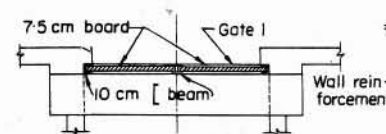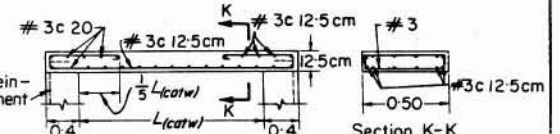
Section G-G

Cotwalk - Longitudinal section

Section K-K

Note:
For structures A and D
$L_{(catw)}$ = 2·20 m
For structures B and E
$L_{(catw)}$ = 1·80 m
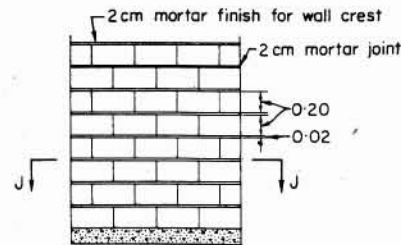
Wall detail
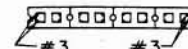
Section J-J

Note: $\frac{3}{8}$" steel bars anchored to the
bottom slab by means of 30 cm
square hooks.

F A O — I C I D

INTAKE STRUCTURE ON SECONDARY
CANALS
CONSTRUCTION DETAILS

Project, Region, Country
Colombia

Figure No. 3-23

load and the maximum water pressure.   To ensure adequate strength of the pre-cast concrete side walls and head walls, structural steel bars are embedded in the joints of the blocks, anchoring them to the slab foundation (Figure 3-23). This reinforced concrete slab has cut-off walls and weep holes to reduce the uplift pressure.   At the inlet and outlet of the structure, there are revetments consisting of 50 cm x 50 cm x 6 cm pre-cast slabs which are placed over a thin layer of concrete.   This lining is provided with a cut-off apron as indicated on Figure 3-22.

For economic reasons the idea of constructing a transition at the inlet of the structure has been discarded, especially as the hydraulic head losses are compensated by the operation of the weir gates.   Wooden gates for the regulation of the diverted discharge include pins on the frame, allowing total or partial opening of the intake (Figure 3-24).
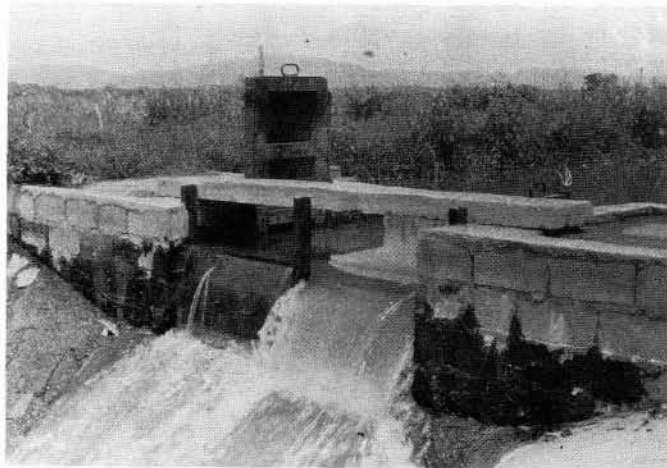
FIGURE 3-24. - Intake structure on secondary canals, spillway and wooden gate.

To calibrate the structure and to measure the diverted flow, a rod and measuring weir are located at the outlet of the intake (Figure 3-25).

FIGURE 3-25. - Intake structure on secondary canals, outlet to tertiary canal.

3.10.3   Design Formulae

3.10.3.1   Design of control weir on secondary canal

The weir length is chosen in accordance with the canal width, and its depth is chosen according to the heading up needed for minimum flow.

3.10.3.2   Wall height

Taking the design maximum flow, $H_{(crt)}$ is calculated by means of the formula:

$$Q = C_Q \; B_{(t)} \; H^{\frac{3}{2}}_{(crt)} \tag{1}$$

where:

Q =   flow in m^3/s;

C_Q =   discharge coefficient

$B_{(t)}$ =   width of weir normal to flow;

$H_{(crt)}$ =   depth of upstream water surface level over the weir.

The wall height of the structure is obtained by adding to $H_{(crt)}$ the weir height over the floor slab and an adequate free-board.

3.10.3.3   Length of the stilling basin

To determine the length of the stilling pool the following formula is used:

$$L_{SB} = 5 \left(H_{(j)_2} - H_{(j)_1} \right) \qquad (2)$$

where:

L_{SB} =   length of the stilling basin

$H_{(j)_2}$ =   theoretical water depth downstream of the hydraulic jump

$H_{(j)_1}$ =   theoretical water depth upstream of the hydraulic jump.

The coefficient '5' is assumed as the stilling basin is of a trapezoidal section; however, there is not yet sufficient experimental evidence to prove this. Nevertheless, dimensions chosen according to this formula have so far proved to be satisfactory on the structures already built.

Water depths, $H_{(j)_1}$ and $H_{(j)_2}$ are calculated by the conventional methods, applying Bernoulli's equation.

3.10.3.4   Dimensions of tertiary intake

The diameter of the concrete pipe is determined assuming that the maximum velocity of the water through the pipe is 1 m/s, for the maximum design flow of the tertiary canal.

Intake losses are determined according to the following formula:

$$J_{(HR)} = h_{(\ell.ent)} + h_{(\ell.f)} + h_{(\ell.OL)} = \left\{ C_{IN} + C_{(f)} \frac{L_{(p)}}{D_{(p)}} + 1 \right\} \frac{v^2_{(p)}}{2g} \qquad (3)$$

where:

$h_{(\ell.ent)}$ =   loss of head at entrance

$h_{(\ell.f)}$ =   loss of head through the pipe

$h_{(\ell.OL)}$ =   loss of head at outlet

$J_{(HR)}$ =   intake headloss

$$C_{(IN)} \quad = \quad \text{inlet coefficient} \quad = \quad 1.5$$

$$C_{(f)} \quad = \quad \text{friction coefficient} \quad = \quad 0.023$$

$$v_{(p)} \quad = \quad \text{velocity through pipe}$$

3.10.4 Numerical Example

A design example for a secondary canal having a maximum capacity of 1,000 l/s with an offtake of 250 l/s is shown below. The bed width of the secondary canal is 1 m. Drop = 1.4 m.

3.10.4.1 Control weir design for the secondary canal

A Creager section is designed for a weir width of 1 m and a height of 0.40 m above the upstream bed level or floor slab.

3.10.4.2 Height of the walls

Applying
$$Q = C_Q \, B_{(t)} \, H_{(crt)}^{\frac{3}{2}}; \quad H_{(crt)} = \left(\frac{Q}{C_Q \, B_{(t)}} \right)^{\frac{2}{3}}$$

Data:
$$Q = 1 \text{ m}^3/\text{s}$$
$$C_Q = 2.00$$
$$B_{(t)} = 1.00 \text{ m}$$

we obtain $H_{(crt)} \quad = \quad 0.63 \text{ m}$

Applying an approximate free-board of 30% of the depth of the water, we obtain:

$$H_{(wall)} \quad = \quad (0.40 + 0.63) \, 1.3 \quad = \quad 1.34 \text{ m}$$

This height of the structure's walls is also applied to the canal design upstream so that heading up does not cause overflow.

3.10.4.3 Length of the stilling basin

For a drop of 1.40 m in the structure and water depth in the canal upstream of 1.03 m, the approximate velocity upstream of the weir will be:

$$v_1 \quad = \quad 0.97 \text{ m/s} ; \quad \frac{v_1^2}{2g} \quad = \quad 0.05 \text{ m}$$

and the total energy will be:

$$H'_{SB} = 1.40 + 1.03 + 0.05 = 2.48 \text{ m}$$

Applying Bernoulli's equation:

$$\frac{v^2}{2g} + 2 = \text{constant}$$

$$H'_{SB} = \frac{v^2_{(j)_1}}{2g} + H_{(j)_1} = 2.48 \tag{4}$$

where $v_{(j)_1}$ and $H_{(j)_1}$ are conditions before the hydraulic jump, introducing a coefficient for the friction losses over the hydraulic jump, equation (4) becomes:

$$v_{(j)_1} = 0.8 \{ 2g (2.48 - H_{(j)_1}) \}^{\frac{1}{2}} \tag{5}$$

On the other hand

$$Q = v_{(j)_1} H_{(j)_1} B_t = 1 \text{ m}^3/\text{s} \tag{6}$$

From (5) and (6) the value of $H_{(j)_1} = 0.186$ is found.

Therefore:

$$v_{(j)_1} = 5.38 \text{ m/s}$$

The depth of water over the hydraulic jump is defined by:

$$\frac{H_{(j)_2}}{H_{(j)_1}} = 0.5 \{ (1 + 8 F_{(j)_1})^{\frac{1}{2}} - 1 \}$$

where:

$$F_{(j)_1} = \frac{v_{(j)_1}}{(gH_{(j)_1})^{1/2}} = 3.98$$

being $F_{(j)_1}$ the Fraude's number.

from where $\quad H_{(j)_2} = 0.96$

and the length of the stilling basin

$$L_{SB} = 5 (0.96 - 0.19) = 3.85 \text{ m}.$$

3.10.4.4   <u>Tertiary canal intake design</u>

Tertiary canal discharge	=	$0.25 \text{ m}^3/\text{s}$
Velocity in the pipe, $v_{(p)}$	=	1.00 m/s
Length of the pipe, $L_{(p)}$	=	10.00 m

$$\text{Hydraulic area} \qquad = \quad \frac{Q}{v} \ = \ \frac{0.25}{1.00} \ = \ 0.25 \ \text{m}^2$$

Diameter of the selected pipe $\qquad = \quad 0.62$ m; (Area $= 0.30$ m^2)

Intake head losses are equal to:

$$J_{(HR)} \qquad = \qquad h_{(\ell.\,ent)} \ + \ h_{(\ell.\,f)} \ + \ h_{(\ell.\,OL)}$$

$$h_{(\ell.\,ent)} \qquad = \qquad 1.5 \ \frac{v^2_{(p)}}{2g} \quad \text{(entrance losses)}$$

$$h_{(\ell.\,f)} \qquad = \qquad 0.023 \ \frac{L_{(p)}}{D_{(p)}} \ \frac{v^2_{(p)}}{2g} \quad \text{(friction losses)}$$

$$h_{(\ell.\,out)} \qquad = \qquad 1.0 \ \frac{v^2_{(p)}}{2g} \quad \text{(outlet losses)}$$

$$J_{(HR)} \qquad = \qquad \left\{ 1.5 \ + \ 0.023 \ \frac{10.0}{0.62} \ + \ 1 \right\} \ \frac{v^2_{(p)}}{2g} \ = \ .146 \ \text{m.}$$

This shows that the water level upstream in the secondary canal should be 0.146 m above the level on the tertiary canal in order to divert 0.25 m^3/s.

The design is shown on Figures 3-22 and 3-23.

3.11   GATE VALVE INTAKE (CZECHOSLOVAKIA)

3.11.1   General

The gate valve intakes described in this section serve to feed water from higher order to lower order canals.   Water level is controlled by a canal seal-gate mounted on a pipe passing through the canal embankment.

3.11.2   Structural Characteristics

3.11.2.1   Parts of the structure

The structure consists of fixed and variable parts as described below.

Fixed parts - (a) an inlet part including sill, side wings, concrete paving of inlet bottom with pitching, inlet walls;

(b) canal seal-gate with lifting mechanism;

(c) outlet wall;

(d) stream bed protection downstream of the outlet including pitching and sill.

Variable parts - (a) earthwork;

(b) length of piping in relation to type and height of structure varying from 125 cm - 2m; 175 cm - 3 m and 250 cm - 5 m.

3.11.2.2   Types of structure

There are five types, according to the inside diameter of the pipe, height of the inlet wall and kind of lift as shown in Table 3-16. For example, Type RS_2 50/250 denotes that the inside diameter of the pipe is 50 cm and the height of the inlet wall is 250 cm.

TABLE 3-16

Inside diameter of pipe, cm	Lifting mechanism	Type and height of inlet wall, cm	
30	unshielded	RS_1	30/125
		RS_1	30/175
	shielded	RS_2	30/175
50	unshielded	RS_1	50/175
	shielded	RS_2	50/250

3.11.2.3   Instructions for use

Unshielded lifting mechanisms should not be used in places exposed to the danger of interference or damage by unauthorized persons. In such cases
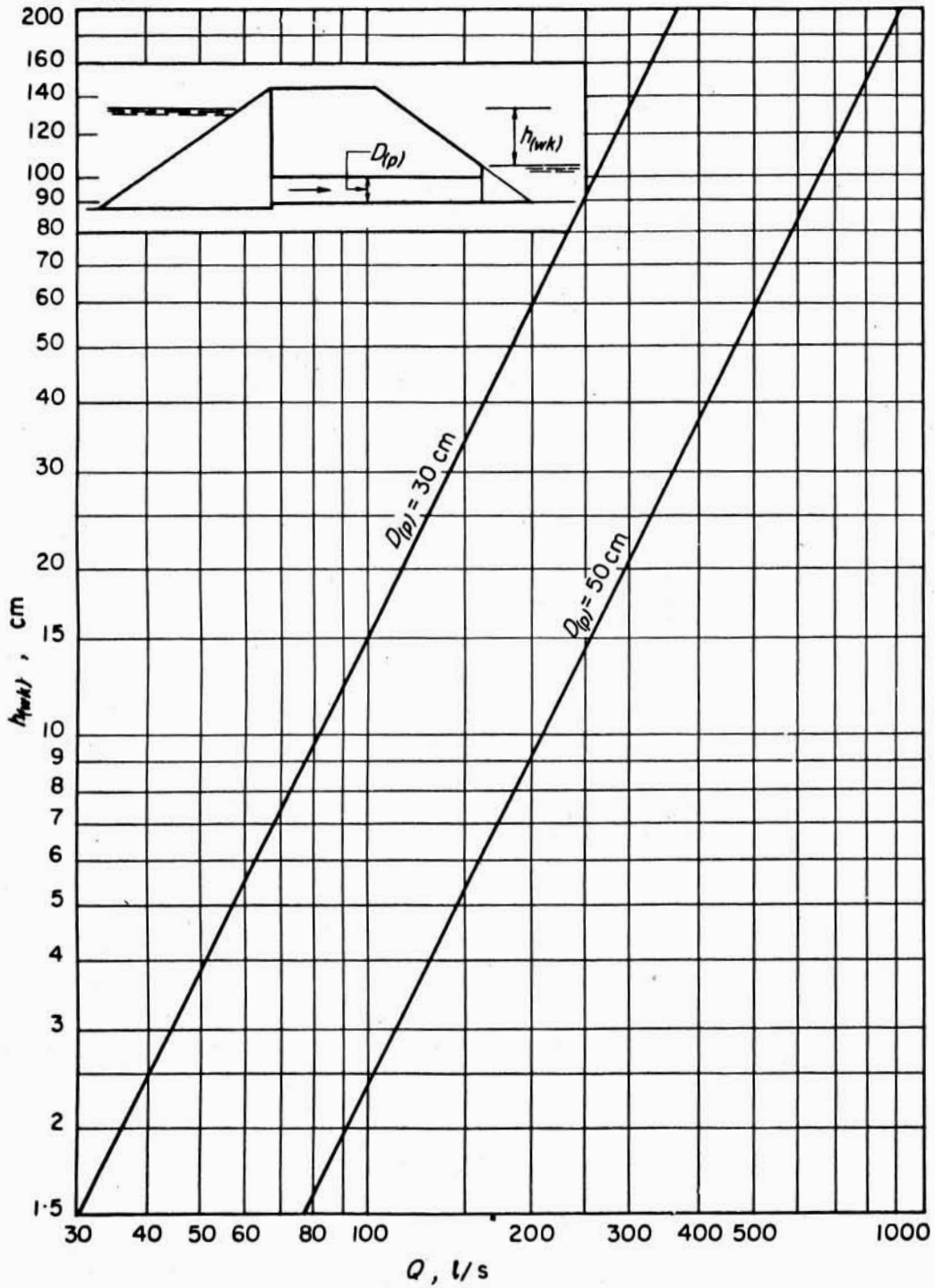
FIGURE 3-26. - Gate valve intake, relationship between
head, discharge and pipe diameter.

shielded lifting mechanisms should be used.   The use of shielded mechanisms is limited by the minimum height of the inlet wall as required by the design.

The height of the structure should be chosen so that the top of the bank of the canal is neither higher than the inlet wall by more than 25 cm nor falls short of it by more than 50 cm; otherwise it will be necessary to arrange access to the lifting mechanism.

3.11.2.4   Building materials required

Table 3-17 gives the type of structure, volume of concrete, paving concrete, and type of seal-gate.

TABLE   3-17

Volume of Main Structures

Type of structure	Concrete m^3	Form work m^2	Paving concrete m^2	Canal Seal-gate		Weight tons
				JS 300 pieces	JS 500 pieces	
RS_1 - 30/125	4.6	16	7.3	1	-	16.7
RS_1 - 30/175	5.8	21	7.8	1	-	20.4
RS_1 - 50/175	6.0	24	14.8	-	1	24.9
RS_2 - 30/175	6.3	25	8.0	1	-	21.4
RS_2 - 50/250	9.0	39	16.2	-	1	33.1

3.11.3   Design

Figure 3-26 gives, for a 4 m pipe length, the relationship between head, discharge and the pipe diameter.

3.11.4   Numerical Example

Design a gate valve structure for a discharge of 500 l/s and a head of 60 cm.   The height of the inlet wall is 175 cm.

RS₁ 30/125: RS₁ 30/175: RS₁ 50/175

RS₂ 30/175: RS₂ 50/250

Sectional view 1-1

Sectional view 2-2 | Sectional view 3-3

RS₁ (unshielded)

RS₂ (Shielded)

Details of canal seal gate

	Legend:	
	1	Concrete HV₄ – 135
	2	Concrete HV₄ –105
Fixed part	3	Concrete pipe
	4	Gate valve
	5	Gravel sand

132.
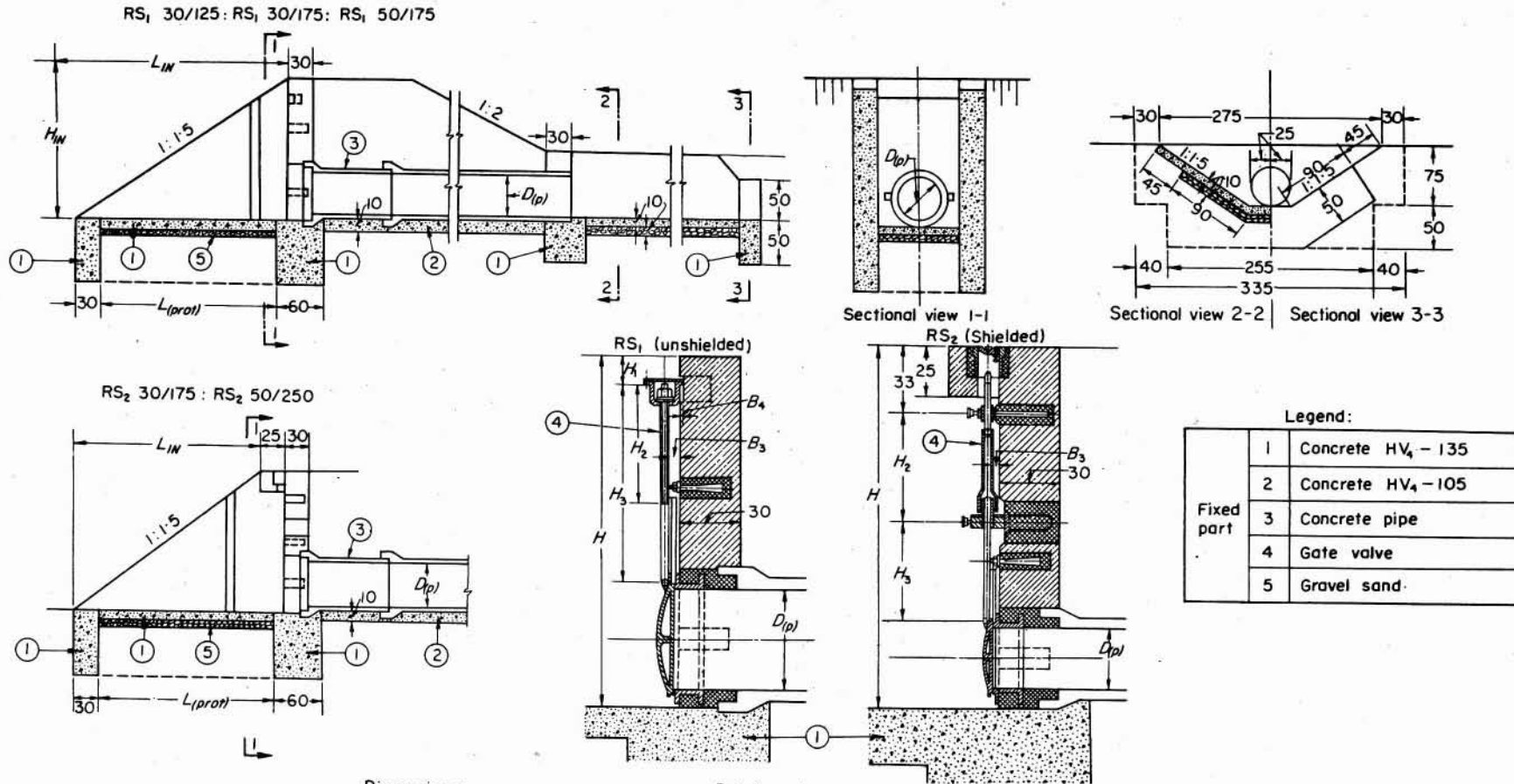
Dimensions

Type	H_1	B_3	B_4	$D_{(p)}$	H_{IN}	H_2	H_3	L_{IN}	$L_{(prot)}$	c
					(in cm)					
RS₁ 30/125	23	5·3	2·7	30	125	60	42	167	142	75
RS₁ 30/175	23	5·3	2·7	30	175	110	42	262	217	75
RS₁ 50/175	23	6·8	1·2	50	175	90	62	262	217	100
RS₂ 30/175	–	–	5·3	30	180	50	55	262	247	75
RS₂ 50/250	–	–	6·8	50	250	75	80	375	355	100

Note: All dimensions are in centimetres.

FAO – ICID

GATE VALVE INTAKE

Project, Region, Country
Czechoslovakia

Figure No. 3 - 27
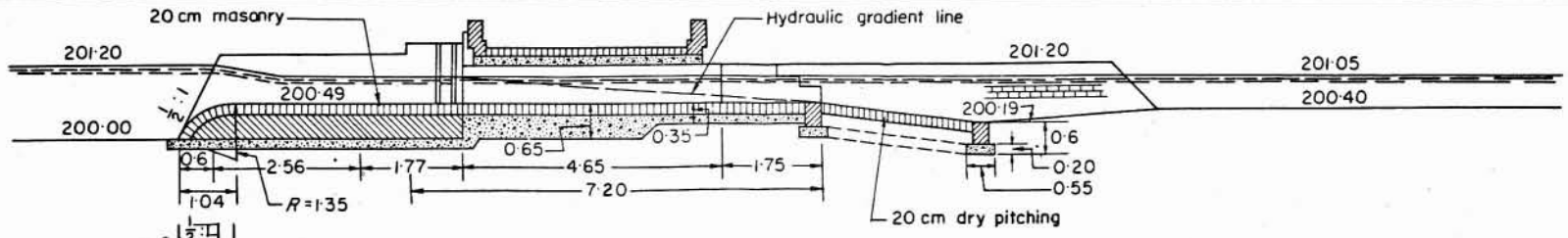
20 cm masonry
Hydraulic gradient line
201·20
201·20
201·05
200·49
201·20
200·40
200·00
200·19
0·6
0·6
2·56
1·77
0·65
4·65
0·35
1·75
0·20
0·55
1·04
R = 1·35
7·20
20 cm dry pitching

Longitudinal section

45°
4·5
0·6
R = 3·35
0·35
1 1/2 : 1
3·16
1·77
4·65
1·75
3·00
3·00
45°
1·04
1·4
0·35
3·5
0·35
45°
1 : 10
1 : 10
A
A
1/2 : 1

Plan

4·5
45°
1/2 : 1

Section A-A

201·05
12 cm masonry
15 cm R.C.C. Slab
0·45
0·60
200·49
0·20
0·58
0·15
0·15
0·65
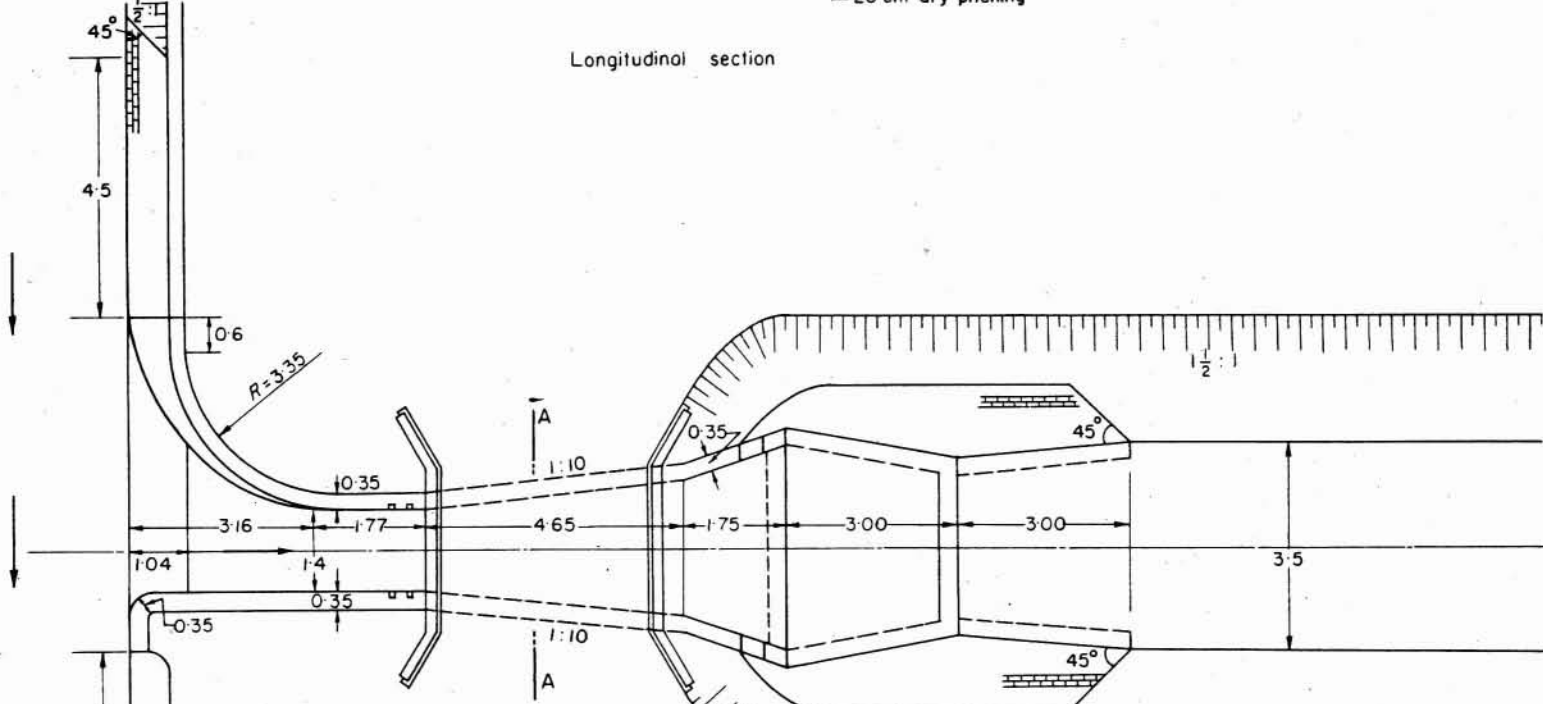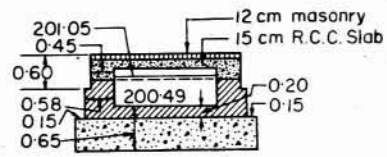
Note: All dimensions are in metres unless otherwise stated.

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VENTURI HEAD INTAKE

Project, Region, Country
U.P., India

Figure No. 3-28

133.

From Figure 3-26, the required diameter of the pipe for 500 l/s discharge and for the head of 60 cm is 50 cm.

See Table 3-16, use type RS_1 50/175.

The dimensions of the various parts are given in Figure 3-27 and other details necessary may be found from this figure for design purposes.

3.12   VENTURI HEAD INTAKE

3.12.1   General

The design of this venturi head intake was evolved in the 1920's during the construction of the Sarda Canal in Uttar Pradesh, India, to effect economy by providing a flumed throat with suitable wing wall connections to restore the full bed width of the offtake canal.

The characteristics of this structure are as follows.

(a)   The headloss is $\dfrac{H_{(crt)}}{9}$ or less and the discharge is a little over the theoretical value due to the streamlined approach.

(b)   The venturi head may be designed for any angle of offtake from 60° to 90° and for any bed width of the offtake canal up to 7.5 m.

(c)   The design is such that the excess energy of the water is dissipated by the formation of a hydraulic jump.

(d)   The structure does not measure discharge correctly and is not successful in controlling the entry of silt into the offtake canal.

A bridge is generally provided over the structure.

3.12.2   Structural Design

The structure (Figure 3-28) consists of upstream wings and side pitching, throat, downstream wings, and downstream bed and side protection.

The upstream face wing is curved and is warped as shown on Figure 3-28. The sill is rounded off at the top.   The radius of the wing wall is generally kept

at $5 H_{(crt)}$. Thereafter it is extended further to meet the 0.5 : 1 slope.
The length of the curved sill is $\sqrt{3} H_{(crt)}^{1.5}$ and its radius is kept at
$\dfrac{L_{(app)}^{2} + H_{(b-c)}^{2}}{2 H_{(b-c)}}$, where $L_{(app)}$ is the length of the curved sill or approach,
and $H_{(b-c)}$ is the difference between the sill level and the bed of the parent canal.
The downstream wing wall in the parent canal is rounded off and turned at right
angles, and it is extended 0.5 to 1.0 m to meet the side pitching.

The width of the throat should not be less than one third the bed width of the
offtake canal nor less than the width determined from the formula
$B_{(t)} = \dfrac{Q}{1.2 H_{(crt)}^{\frac{3}{2}}}$, where $B_{(t)}$ = throat width, and $H_{(crt)}$ is the head over the
sill.

Where there is a cross regulator or a large drop in the parent canal down-
stream of the venturi head, the throat width may be decreased to 0.25 of the bed
width or to $\dfrac{Q}{1.4 H_{(crt)}^{\frac{3}{2}}}$ whichever is greater, subject to a minimum of 0.60 m.
The drop in the water surface should be assumed at at least $\dfrac{H_{(crt)}}{8}$ to be on the
safe side.

The throat should be set back by $(1.4 B_{(t)} + 0.6)$ m from the full supply
line in the parent canal. The side slope of the parent canal should be 0.5 : 1.

The length of the throat should be $2.5 H_{(crt)}$.

The grooves for the stop-logs or gate should be set as far down the throat
as possible to avoid swirling. In narrow flumes the grooves should preferably
be fixed below the flume.

The floor should extend up to the start of the downstream wing walls.

If the bed of the offtake canal is below the throat sill, the drop, if less than
0.15 m, should be negotiated by a glacis at a slope of 1 : 10 and a minimum floor
length of 1.5 m should be provided below the glacis. If the drop exceeds 0.15 m,
a vertical drop immediately below the throat should be provided.

The side walls of the flume should be splayed out from the end of the throat at 1 : 10 for a length of 4.5 m or until the width of the flume becomes two-thirds of the bed width of the offtake channel. Afterwards the splay should be increased to 1 : 3 until the full bed width is attained and should be stepped down at 1 : 1. The side walls should be vertical. The floor end should rest on a toe wall.

Bed and side pitchings should be provided in the offtake canal after the wing walls up to a distance of 3 m, and this should be followed by side pitching only of equal length.

3.12.3 Hydraulic Design

The theoretical maximum discharge of an open venturi flume in which hydraulic jump is obtained, and the velocity of approach is ignored, is given by

$$Q = 1.71 \, B_{(t)} \, H_{(crt)}^{\frac{3}{2}}$$

where $B_{(t)}$ = throat width as determined from the data given above

and $H_{(crt)}$ = the depth of water upstream of the throat measured from the sill.

3.12.4 Numerical Example

Design a venturi head intake for the following data:

Parent canal		Offtake canal	
Q =	$10 \, m^3/s$	Q =	$1 \, m^3/s$
B_1 =	$12.0 \, m$	B_2 =	$3.5 \, m$
y_1 =	$1.2 \, m$	y_2 =	$0.65 \, m$
Bed elevation =	200		

There is no cross regulator or drop in the parent canal downstream of the venturi head.

Design

Full supply level in the parent canal is 201.20. Allow for a drop in head of 0.15 m and keep the full supply in the offtake canal at 201.20 - 0.15 = 201.05.

The bed of offtake canal = 201.05 - 0.65 = 200.4

The width of the throat $B_{(t)}$ should be not less than the lower of the following values -

(a) $\frac{1}{3}$ bed width of offtake canal, i.e. $\frac{3.5}{3}$ = 1.167 m

(b) $B_{(t)} = \dfrac{Q}{1.2\ H^{\frac{3}{2}}_{(crt)}} = \dfrac{1.0}{1.2\ H^{\frac{3}{2}}_{(crt)}}$ or

$$H_{(crt)} = \left(\frac{0.83}{B_{(t)}}\right)^{\frac{2}{3}}$$

The following values of $H_{(crt)}$ corresponding to $B_{(t)}$ are obtained:

$B_{(t)}$, (metres)	1.0	1.2	1.4	1.6
$H_{(crt)}$, (metres)	0.88	0.78	0.71	0.65
Sill level				
$(201.20 - H_{(crt)})$	200.32	200.42	200.49	200.55

The bed level of the offtake canal is at 200.4; therefore the first value is ruled out. Adopt the third combination of $B_{(t)}$ and $H_{(crt)}$, i.e. $B_{(t)}$ = 1.4; $H_{(crt)}$ = 0.71. The sill level will be (200.49 - 200.40) = 0.09 m higher than that of the offtake canal. The drop (0.09 m) is less than 0.15 m; a glacis at 1 : 10 slope will be provided to negotiate the downstream bed level.

Setback of throat = $(1.4\ B_{(t)} + 0.60)$ = 1.4 . 1.4 + 0.60
= 2.56 m
    from the supply level of the parent channel.

Length of throat = $2.5\ H_{(crt)}$ = 2.5 . 0.71 = 1.77 m

Length of curved sill = $\sqrt{3}\ H^{1.5}_{(crt)}$ = $\sqrt{3}\ (0.71)^{1.5}$ = 1.04 m

Radius of sill = $\dfrac{L^{2}_{(app)} + H^{2}_{(b-c)}}{2\ H_{(b-c)}}$ = $\dfrac{1.04^2 + (0.49)^2}{2\ .\ 0.49}$

= 1.35 m

Floor thickness and length

For the worst condition the head will be 201.20 - 200.40 = 0.8 m. The maximum floor thickness will be $\frac{0.80 \cdot 4}{5}$ = 0.64 m, say 0.65 m. The thickness is reduced according to the hydraulic gradient line. A hydraulic gradient of 1:9 may be adopted. The length of floor will be 9 . 0.8 = 7.2 m.

Downstream wing walls

After the throat length (i.e. 1.77 m), the wing walls must be splayed at 1 : 10 to attain two-thirds width of the offtake. The length will be $(\frac{3.5 \cdot 2}{3} \cdot - 1.4) 0.5 . 10$ = 4.65 m. Afterwards the wings must be splayed at 1 : 3. The length of the splayed wings will be (3.5 - 2.33) 0.5 . 3 = 1.755 m. The 1 : 3 splayed wings must be stepped down from the section intersected by a 45° angle from the end of the toe wall of the downstream floor.

Provide a 20 cm thick and 3 m long dry pitching in the bed and the sides and then provide side dry pitching 3 m long.

Figure 3-28 gives the details of the structure. The thickness of the walls of the throat, the upstream approach, and the downstream wings should be as given in Figure 3-3 of section 3.2.

3.13   SQUARE-HEAD INTAKE

3.13.1   General

The square-head regulator is a simple intake structure provided at the heads of secondary and tertiary canals to draw water supplies from a main or branch or secondary canal, the latter being called the parent canal and the former the offtaking canal. The structure is usually sited at right angles to the parent canal. The structure is not a meter and it is not silt-selective. It is primarily meant to regulate water supplies into the offtaking canal. Regulation is effected by means of the insertion of stop-logs or a sliding gate in the grooves provided on the upstream side in the abutments.

A bridge is provided over the structure when the width of the controlling

section is more than 0.60 m or when the canal bank is meant to carry vehicular traffic.

3.13.2   Structural Design

The structure consists of upstream wing walls, upstream bed protection, sill, abutments, downstream wing walls, side and floor protection.

The thickness of abutments for normal loading conditions should be as given in Figure 3-3 of section 3.2.

The wing walls on the upstream and downstream sides are laid out straight to connect the banks. The curves of the wing walls are sharp with little consideration to streamlining.

The floor of the controlling section is designed on Bligh's theory for the worst condition when the parent canal is running at full supply level and the offtake canal is dry.

The length of floor = $C H_{(FS-b)}$ where C is Bligh's coefficient, and $H_{(FS-b)}$ is the difference between the full supply level in the parent canal and the bed level of the offtaking canal. The recommended values of C are as given hereunder.

For fine micaceous sand in North Indian rivers,   C = 15
For coarse grained sand in North Indian rivers,   C = 12
For sand mixed with boulders and gravel and
    loam soil   C = 5 to 9

The floor thickness is designed according to the hydraulic gradient line. The head measured at a point from the hydraulic gradient line is the net head working at that point and a suitable floor thickness is then provided. For concrete floors the thickness is determined by dividing the design head by a factor of 1.25. This factor is the submerged relative density of concrete. At the end of the floor a toe wall is provided.

The downstream bed pitching is laid at a slope of 1 : 10 and 3 m beyond the end of the floor, and side pitching is provided to protect the earth sides from

embayment.   A toe wall is provided at the end of the bed pitching.   The side
pitching also rests on the toe wall.

3.13.3   Hydraulic Design

A depth equal to the full supply depth of the offtake canal is maintained
downstream of the grooves.

The insertion of stop-logs creates over-shot flow conditions.   The
discharge formula, (neglecting velocity of approach), for over-shot flow is

$$Q = 0.65 \, B_{(t)} \sqrt{2g} \left\{ \frac{2}{3} H_{(dr)}^{\frac{3}{2}} + H_{(s)} \sqrt{H_{(dr)}} \right\}$$

where $B_{(t)}$ = width of the sill, $H_{(dr)}$ is the hydraulic drop, $H_{(s)}$ is the
difference between the offtake canal water level and the stop-log crest.

When a gate is operated in the groove, the flow is under-shot, and the
discharge is estimated by the formula

$$Q = 0.6 \, A \sqrt{2g \, H_{(dr)}}$$

where A is the area of the opening.

The velocity in the controlling section should be about 1 m/s.

3.13.4   Numerical Example

Design a square-head intake for an offtake canal, the difference between
the full supply levels of the parent and the offtake canals being 0.2 m.   The
regulation of the water supply in the offtake is effected by a gate.   The other
data are as follows:

Parent canal	Offtake canal
Discharge $Q = 17 \text{ m}^3/\text{s}$	Discharge $Q = 0.90 \text{ m}^3/\text{s}$
$y_1 = 1.8 \text{ m}$	$y_2 = 0.54 \text{ m}$
$B_1 = 120 \text{ m}$	$B_2 = 3.0 \text{ m}$

Design

The width of the opening is determined by the formula

10 cm x 10 cm grooves for gate and stop logs

12 cm brick pitching

15 cm R.C.C. slab

Side brick pitching

(H_{dr})

101·80

$H_{(Fs-b)}$

1·5:1

101·60

100·06

100·06

99·76

20 cm brick pitching

0·30

100·06

1:10

0·30

0·35

0·60

0·35

0·60

0·60

0·55

20 cm brick bed pitching

7·40

3·00

Longitudinal section

Parent channel

$B_{(t)} = 1·4$

45°

$B_2 = 3$

0·70

0·80

0·58

0·58

0·45

0·45

0·35

141.

Half foundation plan

FAO - ICID

SQUARE HEAD INTAKE

Note: Sections of masonry walls are adopted as given in Figure No. 2 of Intakes of Small Canals (Punjab Type).

All dimensions are in metres unless otherwise specified.

Project, Region, Country
India

Figure No. 3 - 29

$$Q \quad = \quad 0.6 \cdot B_{(t)} \cdot 0.54 \sqrt{2 g H_{(dr)}}$$

$$0.9 \quad = \quad 0.6 \cdot B_{(t)} \cdot 0.54 \sqrt{2 \cdot 9.81 \cdot 0.20}$$

$$B_{(t)} \quad = \quad \frac{0.9}{0.6 \cdot 0.54 \sqrt{2 \cdot 9.81 \cdot 0.20}} = 1.40 \text{ m}$$

Velocity in the controlling section will be $\dfrac{0.9}{0.54 \cdot 1.4}$ = 1.2 m/s which is acceptable.

Figure 3-29 shows the structure and hydraulic details of the above example.

When the regulation is done by stop-logs, the flow formula given in 3.13.3 is used to determine the sill level.

Assume the R.L. of the bed of the parent canal to be 100. The full supply level of the parent canal will be 100 + 1.8 = 101.8. The full supply level of the offtaking canal will be 101.8 - 0.2 = 101.6.

The bed level of the offtaking canal will be 101.6 - 0.54 = 101.06.

The sill level will also be 101.06.

The length of the floor, assuming a hydraulic gradient of 1 : 6, will be (101.8 - 101.06) 6 = 0.74 . 6 = 4.44 m, say 4.5 m.

The thickness of the floor just downstream of the gate grooves, which is subject to maximum head, will be $\dfrac{H_{(FS-b)}}{1.25} = \dfrac{0.74}{1.25} = 0.60$ m.

The thickness of the floor beyond half its length should be reduced to 0.30 m.

3.14 DUPUIS CANAL INTAKE

3.14.1 General

The design of the Dupuis canal intake was evolved by C.E. Dupuis in 1903 in Egypt to satisfy the condition that the discharge of an intake on a secondary

Longitudinal section

102·35

Iron gate

0·60 diameter pipe

100·00

102·10

101·10

0·25

0·25

2:1

1·5:1

1:1

2:1

1:1

2·4

2·4

Plan

0·30

0·60

0·30

0·30

0·60

0·30

2·00

Note: All dimensions are in metres.

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| DUPUIS CANAL INTAKE |
| Project, Region, Country
A R E |
| Figure No. 3 - 30 |

canal (distributary) should be proportional to the area it serves. The installation of this structure is now a common practice in Egypt. These intakes are generally spaced at 200 to 300 m apart.

3.14.2   Structural Design

The Dupuis canal intake (Figure 3-30) consists of a metal or earthenware pipe of a given diameter, with masonry heads at entry and exit.

From the two top corners of the canal bank through which the pipe passes, lines of 1.5 : 1 slope are drawn to meet the points on the base on which the pipe rests. From those base points lines of 1 : 1 are projected 0.25 m above the full supply line on the upstream side and about a metre or so above the bed of the offtake canal on the downstream side. The two points so obtained become respectively the top levels of the upstream and downstream head walls. The upstream face of the upstream headwall is obtained by drawing a line at a slope of 0.5 : 1. The downstream face of the downstream headwall is generally kept vertical (Figure 3-30).

The top widths of the headwalls are set at 25 cm.

The side slopes at the upstream face are rounded for smooth entry of water.

3.14.3   Hydraulic Design

The diameter of the pipe, discharge, and the area irrigated is determined from Table 3-18 as given by Dupuis on the basis of $L_{(p)}$ = 10 m, and $H_{(dr)}$ = 0.25 m, where $L_{(p)}$ = length of pipe; $H_{(dr)}$ = hydraulic drop.

Table 3-19 gives the values of correction as a percentage when the $H_{(dr)}$ is smaller or larger than 0.25 m.

Table 3-20 gives values of area irrigated from pipe diameters ranging from 10 cm to 120 cm for a pipe length of 5 to 100 and a head of 0.25 m. The original values of Dupuis are reproduced in the first column of the table.

TABLE 3-18

Diameter of pipe, cm	Area of pipe cm^2	Velocity (approx) m/s	Discharge of pipe m^3/s	Area irrigated in 7 days at 350 m^3 per feddan [1]
30	0.0707	1.36	0.10	166
40	0.1256	1.45	0.18	315
50	0.1963	1.50	0.29	509
60	0.2827	1.55	0.44	759
70	0.3847	1.59	0.61	1,057
80	0.5026	1.61	0.81	1,398
90	0.6359	1.63	1.04	1,791
100	0.7854	1.64	1.29	2,226

[1] one feddan = 4200.83 m^2 = 1.038 acres

TABLE   3-19

Percentage Differences
for Headings-up Below or Above 25 Centimetres

Cm.	0	1	2	3	4	5	6	7	8	9
00	0	20	28	34.6	40	44.7	49	53	56.5	60
10	63.2	66.3	69.2	72	74.8	77.5	80	82.5	84.8	87.1
20	89.4	91.6	93.8	96	98	100	102	104	106	107.8
30	109.6	111.4	113.4	114.8	116.5	118.2	119.9	121.6	123.1	124.7
40	126.3	127.9	129.5	131	132.5	134	135.5	137	138.5	140
50	141.5	143	144.4	145.8	147.2	148.5	149.8	151.1	152.4	153.7
60	155	156.3	157.6	158.9	160.2	161.4	162.6	163.8	165	166.2
70	167.4	168.6	169.8	171	172.2	173.4	174.6	175.7	176.8	177.9
80	179	180.1	181.2	182.3	183.4	184.5	185.6	186.7	187.8	188.9
90	190	191	192	193	194	195	196	197	198	199
100	200									

TABLE

Giving the Diameter and Lengths of

Dupuis' Figures	Diam. in cm	Lengths of pipes in metres and area served in Feddans assuming :								
		5	10	15	20	25	30	35	40	45
Fed.		Fed.	Fed.	Fed.	Fed.	Fed.	Fed.	Fed.	Fed.	Fed.
10	10.0	16	13	11	9	9	8	7	7	7
20	12.5	25	22	19	17	15	14	13	12	12
30	15.0	41	34	30	27	24	23	21	20	19
45	17.5	58	49	43	39	36	33	31	30	28
65	20.0	78	67	59	54	50	46	44	41	39
85	22.5	101	88	78	72	66	62	58	55	53
110	25.0	126	111	100	92	85	80	75	71	68
135	27.5	157	138	125	115	107	101	95	90	86
165	30.0	188	167	152	140	131	123	117	111	106
235	35.0	262	236	217	201	188	177	168	167	154
315	40.0	348	316	292	272	256	242	231	221	212
405	45.0	447	406	380	356	337	320	305	293	281
510	50.0	555	513	479	451	427	407	389	374	360
625	55.0	680	630	590	558	530	506	488	465	450
760	60.0	817	760	715	678	647	618	593	578	551
900	65.0	960	900	847	840	772	740	710	687	663
1,060	70.0	1,120	1,051	998	952	909	873	840	813	785
1,220	75.0	1,290	1,220	1,158	1,105	1,060	1,018	983	950	920
1,400	80.0	1,478	1,398	1,330	1,273	1,222	1,177	1,137	1,100	1,066
1,590	85.0	1,675	1,590	1,515	1,452	1,397	1,347	1,303	1,262	1,223
1,790	90.0	1,880	1,790	1,712	1,644	1,585	1,530	1,480	1,435	1,395
2,000	95.0	2,100	2,000	1,922	1,846	1,785	1,723	1,670	1,618	1,575
2,230	100.0	2,330	2,230	2,140	2,060	1,990	1,925	1,865	1,813	1,760
	105.0	2,575	2,465	2,370	2,285	2,205	2,140	2,080	2,020	1,970
	110.0	2,840	2,720	2,620	2,530	2,455	2,370	2,305	2,240	2,180
	115.0	3,100	2,980	2,870	2,770	2,690	2,610	2,540	2,470	2,410
	120.0	3,390	3,260	3,150	3,050	2,960	2,870	2,790	2,720	2,650

3-20

Irrigation Pipes and Areas Served

Water Duty 50 m^3/Fed. and Heading-up 25 cm.										
50	55	60	65	70	75	80	85	90	95	100
Fed.	Fed.	Fed.	Fed.	Fed.	Fed.	Fed.	Fed.	Fed.	Fed.	Fed.
6	6	6	5	5	5	5	5	5	4	4
11	11	10	10	9	9	9	9	8	8	8
18	17	17	16	15	15	14	14	14	13	13
27	26	25	24	23	22	22	21	21	20	20
37	36	35	33	32	31	30	30	29	28	27
51	49	47	45	44	42	41	40	39	38	37
65	63	61	59	57	55	53	52	51	50	49
83	80	77	74	72	70	68	66	64	63	62
102	99	95	92	89	87	84	82	80	78	76
149	143	139	134	130	126	123	120	117	114	112
204	197	191	185	180	175	171	166	162	159	155
271	262	254	246	240	233	228	222	217	212	208
347	336	326	347	308	300	293	286	280	274	268
435	421	409	398	387	377	368	360	352	345	338
534	528	503	489	477	465	454	444	435	426	417
642	623	606	590	577	562	551	537	527	515	507
763	740	720	701	684	668	652	640	626	614	603
893	869	845	824	805	787	770	754	733	725	711
1,035	1,005	982	955	937	914	895	876	860	842	828
1,190	1,160	1,130	1,103	1,078	1,053	1,034	1,010	992	975	957
1,355	1,320	1,288	1,258	1,233	1,205	1,180	1,157	1,134	1,114	1,095
1,533	1,495	1,460	1,425	1,395	1,365	1,336	1,313	1,295	1,265	1,243
1,717	1,675	1,636	1,600	1,568	1,535	1,503	1,475	1,448	1,424	1,398
1,920	1,875	1,830	1,790	1,750	1,715	1,685	1,655	1,625	1,595	1,565
2,130	2,080	2,035	1,990	1,950	1,910	1,875	1,840	1,810	1,780	1,750
2,350	2,300	2,250	2,200	2,150	2,110	2,070	2,030	2,000	1,970	1,940
2,590	2,530	2,480	2,430	2,380	2,330	2,290	2,250	2,210	2,180	2,150

Covers 4'-3" x 1'-7½" x 2"
Rein.—Arc Ref. No. 610

Top of lifting frame

Spindle

6 x 6" hole for operating spindle

5¾" 6¼" 2"

Ladder rungs

6'-6"

6"

4'   6"

Section A-A

½" dia. bars 6 centres both ways centrally placed

2—½" dia. bars
2' long pipe cast in position but ended
Exposed section to be scabbled

Pipe invert level

1'-9"

Ladder rungs ¾" dia. bars 1' x 9"
1' spacing

Note: Details of sliding gate and lifting frame not shown.

18" diameter pipe

1¾"
6" 2"
5"
A
¼"
6"
A

1'-9"

6"
6"

6" 6"

Screen 1¼" x 1¼" frame arc mesh

45° chamfer

Plan

⅜" dia. anchor bolt cast into box

C.L.

7"

2¼" 6¼"

Part section of gate connection

Notes: 1. Concrete to have a minimum compressive strength of 3500 psi at 28 days. Concrete works shall conform with specification C—S—59.
2. All fillets 2 inches.

F A O – I C I D

PRECAST CONCRETE INLET BOX
FOR INTAKE WITH STONE MESH WEIR

Project, Region, Country
Australia

Figure No. 3-31

149.

3.14.4   Numerical Example

Design a Dupuis intake for a discharge of $0.44 \text{ m}^3/\text{s}$ and a head of 0.25 m. The length of pipe is 10 m.   The data for the parent canal and the offtake canal are:

Parent canal	Offtake canal
$Q = 10 \text{ m}^3/\text{s}$	$Q = 0.44 \text{ m}^3/\text{s}$
	$y_2 = 1.0 \text{ m}$
	$B_2 = 2.0 \text{ m}$

From Table 3-18, the required pipe diameter corresponding to the discharge of $0.44 \text{ m}^3/\text{s}$ is 60 cm.   The area which will be irrigated by this discharge with a water duty of 50 m^3 per day per feddan will be 760 feddans (320 ha)

The design layout is shown in Figure 3-30.

3.15   INTAKE WITH STONE-MESH DIVERSION WEIR (AUSTRALIA)

3.15.1   General

The stone-mesh diversion weir diverts flows in small, rapidly flowing streams.   It is particularly suitable where the sub-stratum of the site is of gravel and precludes the use of sheet pile weirs and where a conventional type of weir would be impractical or too expensive due to the porous nature of the ground.

3.15.2   Structural Design

Figure 3-32 shows a general view of the weir and the intake when almost all the flow is being diverted.

The intake of the stone-mesh diversion weir consists of a pre-cast concrete inlet box with a screen frame (mesh size $1\frac{1}{4}'' \times 1\frac{1}{4}''$) and a concrete pipe 2 ft long with a sliding gate upstream in the inlet box (Figures 3-31 to

FIGURE 3-32. - General view of the stone mesh weir and inlet box. (Almost all the flow - 12 ft^3/s - is being diverted.)

FIGURE 3-33. - Another view showing stone mesh basket construction and inlet box.

3-33). An outlet pipe 4 ft long joins the 2 ft pipe and opens into the weir structure. The weir structure contains a recorder well of reinforced concrete 30 inches in diameter, with its centre 3 ft from the crest. There is a stilling basin 8 ft long on the downstream side and bed protection of stone, and a transition section converging to the designed bed width of the offtake canal. For details see Figure 3-37. Figure 3-35 shows the measuring weir and the dissipation structure at the end of the pipeline. The weir apron is of stone encased in wire baskets (Figures 3-33 and 3-34).

The structure is of low cost, utilizes local materials and does not require skilled labour for its construction. The wires of the baskets are welded at every point of contact and hence if one section breaks the whole structure is not threatened. The weir of the inlet structure cannot be easily damaged and the control gate on the inlet box is padlocked.

The weir can be easily raised or widened by adding more baskets.

The total cost of the structure is about A$ 3,000. The supervisor should inspect the weirs once a week. Maintenance involves removing leaves and small branches from the entrance to the inlet box.

3.15.3 Design

Figure 3-36 is a graph of a rating curve for the measuring weir of the intake structure used for design purposes. It has been developed from model tests for a 6 ft crest suppressed weir.

3.15.4 Numerical Example

Design an intake structure with a stone-mesh weir in the parent canal (or stream) for a discharge of 30 ft^3/s. The upstream water level above the measuring weir is 1380.25.

Design

Refer to the rating curve, Figure 3-36. For a discharge of 30 ft^3/s the head over the crest of the weir will be 1.05 ft. Therefore, the level of the crest is 1380.25 - 1.05 = 1379.20. Other details and dimensions are shown in Figure 3-37.

FIGURE 3-34. - Stone mesh basket construction, stone mesh apron, and location and general construction of inlet box with screen and screw-type gate. (About 3 ft^3/s passing over the weir.)

FIGURE 3-35. - Dissipation structure and measuring weir at the pipe outlet. (Discharge 10 ft^3/s.)
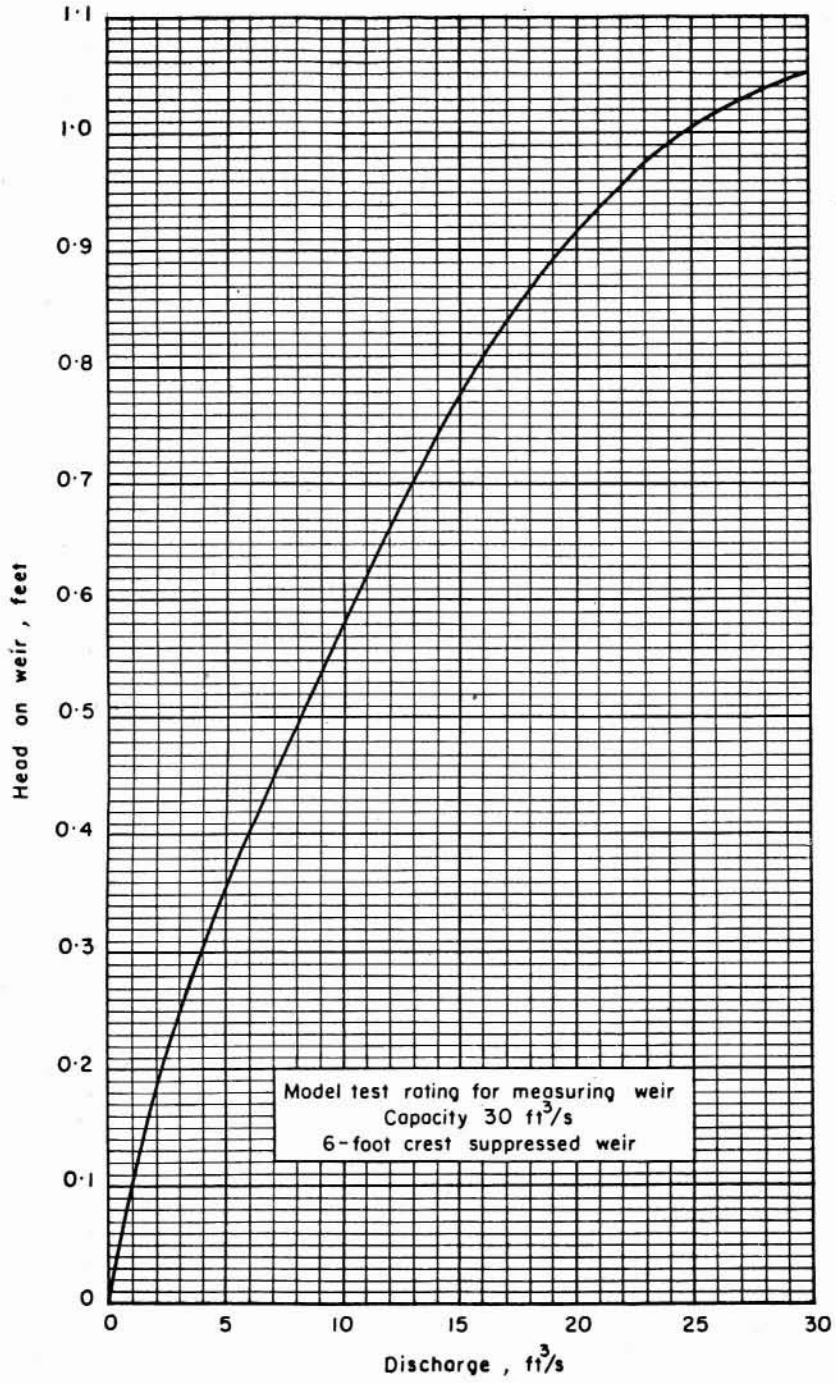
FIGURE 3-36. - Model test rating for measuring weir
capacity 30 ft^3/s   6 ft crest suppressed weir.

Suitable ringlocked mesh to
be attached to rails

R.L.1382·50

1½" ventilation pipe

R.L.1381·50

R.L.1379·50

Crest of bank R.L.1380·00 (approx.)

Beaching R.L.1379·00

N.S.1379·10

1380·25
30 ft³/s

R.L.1378·90

R.L.1379·2
6"

R.L.1378·00

1378·00

Dissipator

R.L.1375·75

R.L.1376·00

R.L.1375·0

12'

12'

4' max.

3'

3"

4'-6"

6"

12'

30" dia.
Direction
of flow

3'

6"

6"

1'-6"

1'-6"

R.L.1373·0

⅜" dia. bars at 6" centres
bothways, centrally plac-
ed. Floor reinf. bonded
into walls

6"

8'

6"

8'

6'

12'

15'

Beaching

1 No. ⅜" dia. bar around pipe

Longitudinal section

Detail 'B'

1/16"

Crest R.L.

45°

Crest 6"x 4"x ⅜" angle.
Top chamfered 45°x 1/16"
and galvanised.
Fixed to concrete with ½"
dia. galvanised M.S. bolts

R.L.1382·50

R.L.1381·50

2'-6"

Crest of bank R.L.1380·00

R.L.1379·20

R.L.1378·00

2:1

8'

R.L.1376·00

Bottom of well may be raised
up to R.L.1375·75 (top of dissi-
pator). If damaged pipe is used,
ends of pipe shall be cut off

R.L.1375·00

Section A-A

30" dia. reinforced concrete pipe
recorder well. Top R.L.1382·50
connected to pit with 1½" galvanized
iron pipe

A

Transition to 6' bed width

30" dia. reinforced
concrete pipe

Dissipator

1½" dia. galvanized
iron pipe

1½" dia. ventilation pipe with breather cap.
Pipe perforated with 4 No. ⅜" dia. holes
at 3" centres

1½"

6"

Slope 2:1

4'

4'

4'

3'

A

Half plan

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INTAKE — STRUCTURE

Detail of measuring weir and stilling
basin

Project, Region, Country
Australia

Figure No. 3 - 37

3.16   GROYNE INTAKE AND ANCILLARY WORKS (CYPRUS)

3.16.1   General

The groyne type of intake is in use in Cyprus on rivers with gravel beds and widths up to about 183 m (600 ft).   In such cases it is uneconomical to build a weir across the full width of the river.   These intake structures have been developed to overcome this difficulty.   They have only a masonry groyne extending out into the river bed.

Such intakes are not, of course, intended to draw off the flood discharge of the river, which may amount to as much as 10,000 ft^3/s or more, and the intake discharge is not more than 10 to 12 ft^3/s.

3.16.2   Structural Characteristics and Design

The structure (Figure 3-38) consists of a groyne or weir about 38 ft long near the head of the intake, and an intake with side-walls of masonry and a 6'' bed of reinforced concrete, a diaphragm and a screw-wheel gate at the intake head.

The groyne across the river bed is a weir type with a crest width of 2 ft, a downstream glacis with a slope of 1 : 3 and a downstream apron.   Near the intake head there is a shutter gate in the groyne for washing out any gravel, soil or sediment that may accumulate on the upstream side of the weir.   The channel below the intake gate is of rectangular cross section with side walls in masonry and coping of concrete on the top.   The side wall facing the river is 2 ft higher than the other side wall.   This is to prevent the discharge passing over the channel when the river is in flood.

An overflow spillway is usually placed in the channel as near to the intake as possible but far enough downstream to be above high flood level.   This is essential under Cyprus conditions where heavy floods would frequently submerge the intake works and overflow the upper part of the channel.   The spillway itself is simply a weir with its crest at the full supply level of the irrigation canal.   The spill channel has to be protected against erosion from both the overflow water and

Section C-D

Concrete lip ¼″ dia. steel ties at 5 feet centres

4″ cement concrete

Rubble masonry wall: Stones as supplied from quarry, bedded in lime mortar 1:2. Slope faced with stones roughly squared with joints in cement mortar 1:3

River bed

Lime concrete

10′   7′-6″   2′   1′-6″   2′-6″   6′   3′   3′   14′   4′   21′   5′

Section E-F

River bed   18″   12″   4′   6″   6″   2′   2′-6″   2′   18″   3′   5′   3′-6″   3′-6″
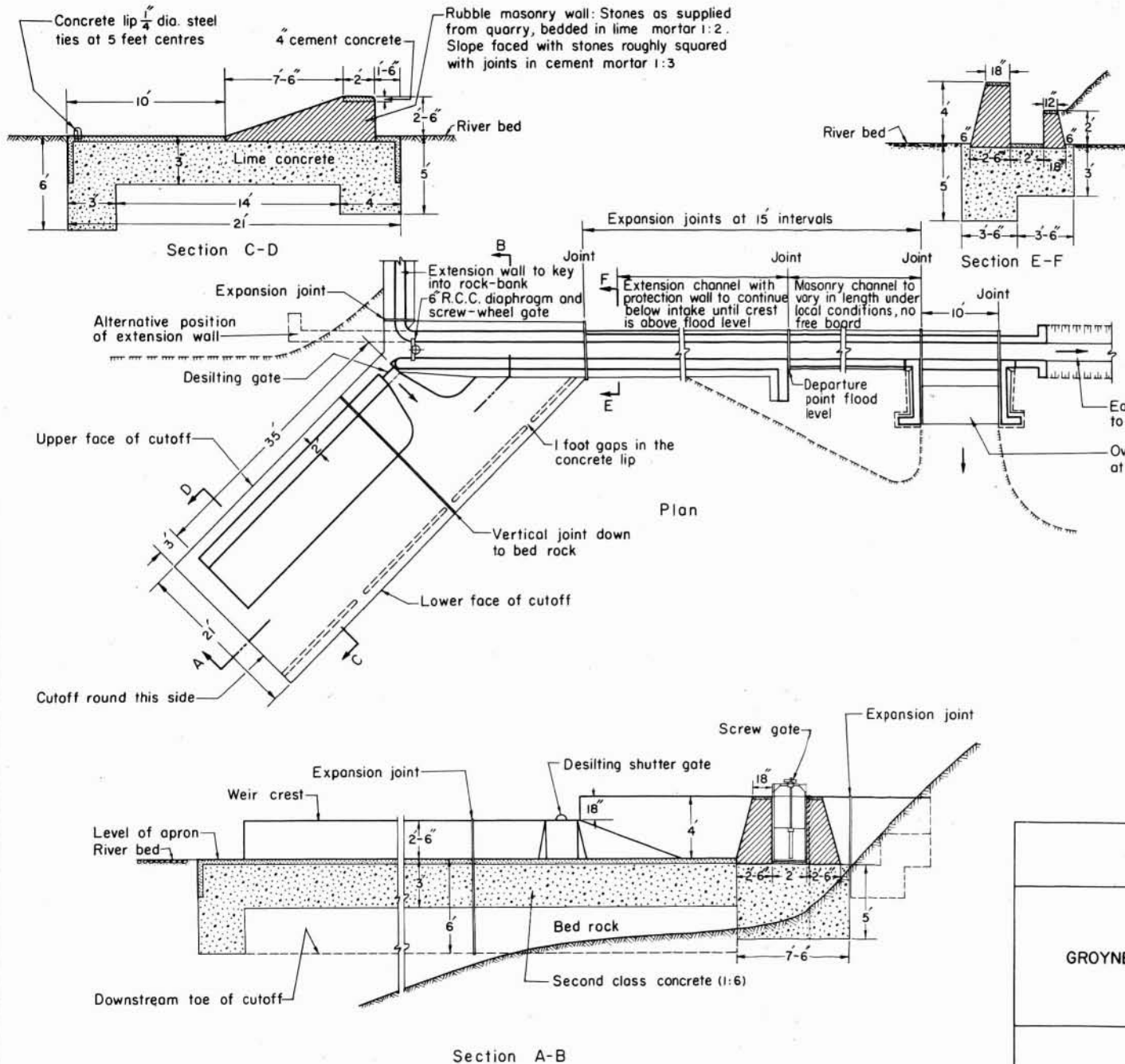
Notes:
Sections between expansion joints to be completed alternately.
Foundation in lime concrete to be cast in lifts 12″ deep; lime concrete usually prop. 1:4, bottom lifts of weir foundations and cutoffs may in some cases be cast in cement concrete 1:3:6.
Facing of walls with "block-in-course" masonry, all interior work in rubble; masonry courses 12″ deep, each separate section to be worked up in complete courses.
Allow one week for settlement of lime concrete foundations; one week before cement plastering and rendering.

Plan

Expansion joints at 15′ intervals

Expansion joint

Extension wall to key into rock-bank
6″ R.C.C. diaphragm and screw-wheel gate

Extension channel with protection wall to continue below intake until crest is above flood level

Masonry channel to vary in length under local conditions, no free board

Alternative position of extension wall

Desilting gate

Departure point flood level

Earth or lined offtake channel to continue beyond the spillway

Overflow spillway to be constructed at suitable site above flood water level

Upper face of cutoff

1 foot gaps in the concrete lip

35′   2′   D   3′   2′

Vertical joint down to bed rock

Lower face of cutoff

Cutoff round this side

Joint

Section A-B

Expansion joint

Screw gate

Desilting shutter gate

Weir crest

Level of apron River bed

18″   18″   2′-6″   4′   3′   6′

Bed rock

2′-6″ 2′ 2′-6″   5′   7′-6″

Downstream toe of cutoff

Second class concrete (1:6)

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GROYNE INTAKE AND ANCILLARY WORKS

Project, Region, Country
Cyprus

Figure No. 3-38

from high floods in the river. The irrigation canal between the intake and the overflow spillway is built to withstand submersion during heavy floods. (The details are shown in Figure 3-38.)

Two points deserve special attention in the construction of intakes of this type. The first is to found the abutment of the groyne on solid rock or at least on very firm ground so that there be no possibility of erosion in, or about, the head of the channel and that the structure be securely anchored. The other point is the depth of the foundation and the width of the base. The depth should be well below the region of normal scour in a shifting channel, particularly at the outer end. The width of the base should be sufficient to prevent tilting of the structure. It is customary in Cyprus to take the foundation down to about 6 ft below the level of the lowest part of the river channel.

FIGURE 3-39. - Photograph of groyne intake structure (Cyprus).

Figure 3-39 shows a slightly modified design with a smaller discharge capacity. The intake gate is replaced by an orifice and a double shut-off gate at a distance of about 3 m downstream. This serves the dual purpose of sediment flushing and discharge control.

B. SILT CONTROL DEVICES

3.17 KING'S SILT VANES

3.17.1 General

In 1933, H.W. King designed a device with curved vanes on the channel bed which would prevent heavier silt entering an offtake. This works on the principle that the water near the bed of the parent canal or channel contains a relatively high silt charge which should, therefore, be deflected away without disturbance, at an angle of about 30° from the direction of the flow.

3.17.2 Structural Design

The layout plan of the vanes is shown in Figure 3-40.

The dimensions of the silt vanes are given in Table 3-21. See Figure 3-40 for X_1, X_2 and R. The length and position of the longest vane and the vane spacing are thus determined.

TABLE 3-21

Width of offtake channel = B_2 in metres		0.6	1.2	1.8	2.4	3.0	3.6	4.8
For strong effect	Value of X_1	1.2	1.5	1.8	1.8	2.1	2.4	2.4
	Value of X_2	0.6	0.9	1.2	1.5	1.8	2.4	2.7
	Value of R	9.0	10.5	12.0	12.0	13.5	13.5	15.0
Cheaper design (less effect)	Value of X_1	0.9	1.2	1.2	1.5	1.5	1.8	1.8
	Value of X_2	0.6	0.6	1.2	1.2	1.5	1.5	1.8
	Value of R	7.5	9.0	9.0	9.0	9.0	9.0	10.5
Minimum dimensions recommended	Value of X_1	0.9	0.9	1.2	1.2	1.5	1.5	1.8
	Value of X_2	0.6	0.6	0.9	1.2	1.5	1.5	1.8
	Value of R	7.5	7.5	9.0	9.0	9.0	9.0	10.5
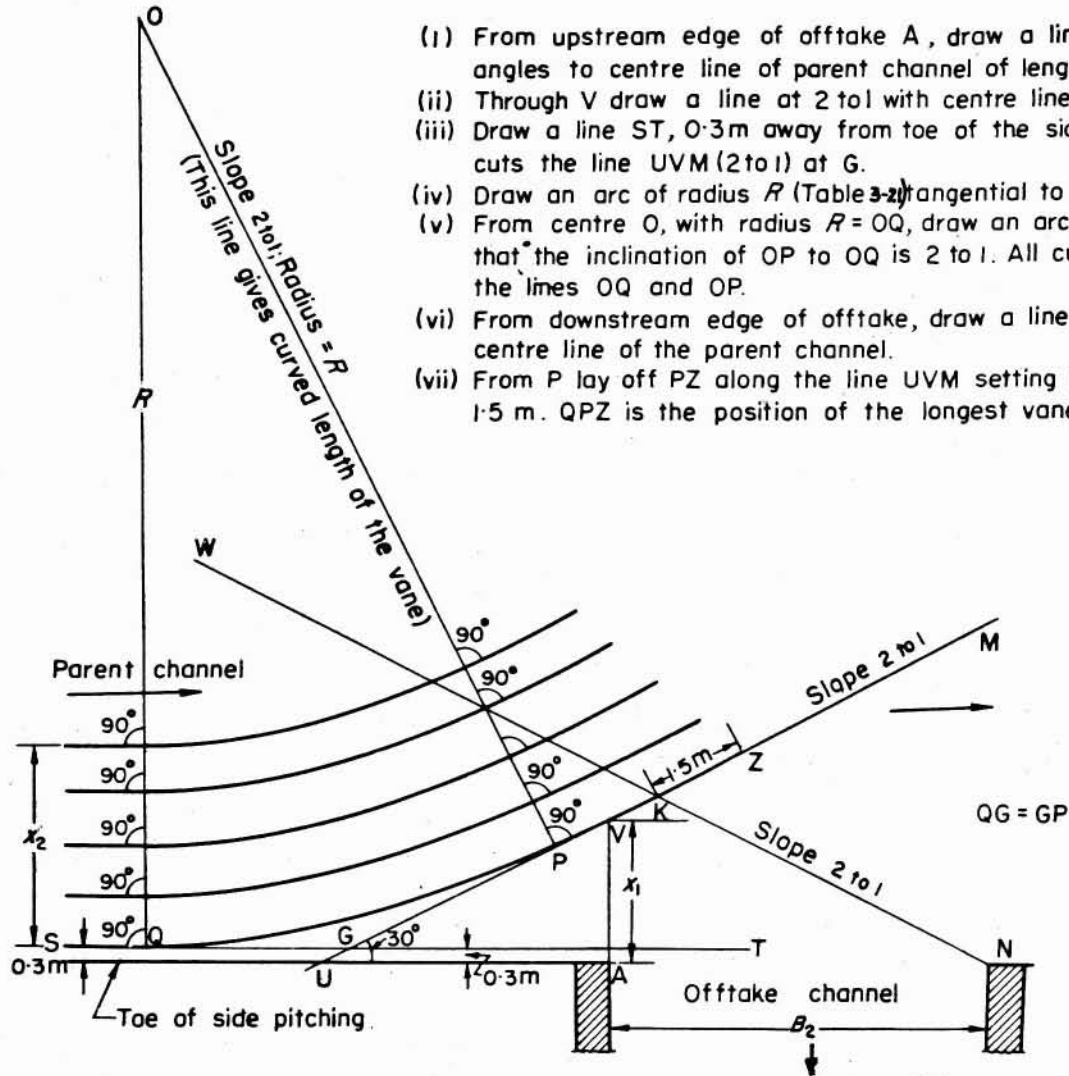
(i) From upstream edge of offtake A, draw a line AV at right angles to centre line of parent channel of length x (Table I).

(ii) Through V draw a line at 2 to1 with centre line of parent channel.

(iii) Draw a line ST, 0.3m away from toe of the side pitching which cuts the line UVM (2 to1) at G.

(iv) Draw an arc of radius R (Table 3-2) tangential to lines ST and UVM.

(v) From centre O, with radius R = OQ, draw an arc QP of such length that the inclination of OP to OQ is 2 to 1. All curves must end at the lines OQ and OP.

(vi) From downstream edge of offtake, draw a line NKW at 2 to1 with centre line of the parent channel.

(vii) From P lay off PZ along the line UVM setting off KZ equal to 1.5 m. QPZ is the position of the longest vane.

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Slope 2 to1; Radius = R
(This line gives curved length of the vane)

O

R

W

Parent channel

90°

90°

90°

90°

x_2

90°

90°

90°

90°

Slope 2 to1   M

Slope 2 to1

1.5m   Z

K

V

x_1

QG = GP

S

90°

Q

G   30°

T   N

0.3m

Toe of side pitching

U   0.3m

A   Offtake channel B_2

FIGURE 3-40. - King's silt vanes, layout plan of the vanes

Parent channel →

0·35 m thick toe wall

Dry pitched floor

1:10

0·35 m thick toe wall

A

B

C

90°

90°

$x_2 = 1·56$

D

E

F

$x_1 = 1·50$

R

2:1

0·35 m thick toe wall

0·30 m centre of vane to bank toe

4·5

15 to 30 m side pitching

3

Offtake channel

Plan

0·90

Straight — Curved

1:3

1:3

1:3

Cut water
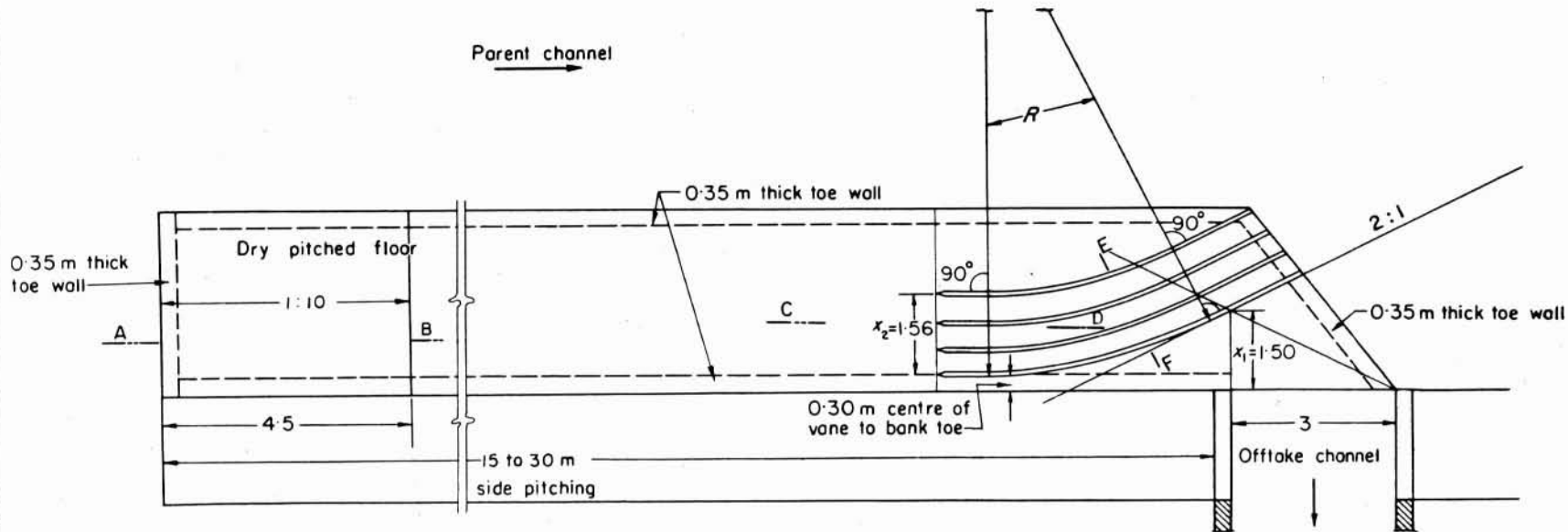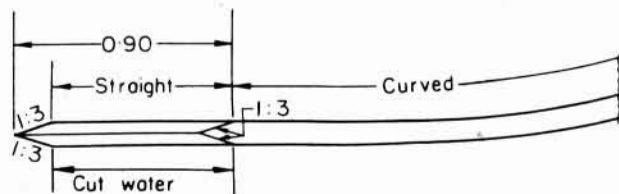
Plan of upstream end of vane
(four times scale)

0·12   0·12   1·56   0·12   0·12
0·4   0·4   0·4
0·30
0·20   0·20 m thick masonry
0·10
0·20 m thick dry pitching

Section E-F
(double scale)

Normal silted bed
0·15
0·35
0·4
0·2
1:10
0·20 m thick dry pitching

Section A-B

1:3   0·3
0·20 m thick dry pitching   0·20 m thick masonry floor
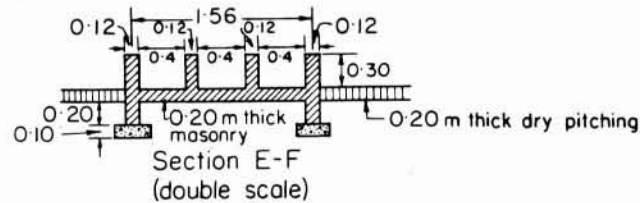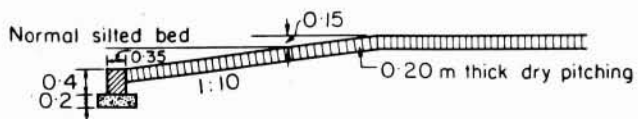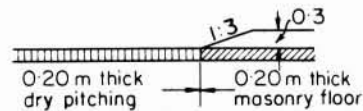
Section C-D

Note: All dimensions are in metres.

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KING'S SILT VANES
DETAILS OF LAYOUT

Project, Region, Country
Punjab, India

Figure No. 3 - 41

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The height of the vanes is one-third to one quarter of the depth of the parent canal.

The thickness of the masonry vanes is 12 cm for a height up to 0.36 m and for greater heights the thickness is 24 cm. However for efficiency the thinner the better.

The width of channels between the vanes is normally $1\frac{1}{2}$ times the height of the vanes.

The upstream ends of the vanes beyond the line OQ (Figure 3-40) must be finished off to a slope of 1 vertical to 3 horizontal in a V-shape to act as cut-waters (Figure 3-41).

The downstream ends of the vanes should be vertical.

The channels between the vanes and the vanes themselves should be plastered.

The bed of the parent canal covered by the vanes, and for a distance of 15 m to 30 m upstream of the vanes, must be smoothly pitched and it should be 15 cm higher than the normal silted bed level. The upstream 4.5 m of the pitching should be built at a slope of 1 in 10.

The side slope of the parent canal on the side of the offtake must be pitched up to the length of the pitched floor.

It should be noted that King's silt vanes are not suitable in the following situations: where the offtake canal discharge is more than one-third of that of the parent canal; where the offtake canal is very small and takes off from a deep parent canal; where the parent canal does not have adequate width; and where violent approach with a strong 'draw' towards the intake head exists.

3.17.3.   Numerical Example

Design King's silt vanes for an intake head with the following data:

	Parent canal	Offtake canal
Q; m^3/s	5	1
B, m	6	3
y, m	1.2	0.6

Design

The values of X_1, X_2, and R from Table 3-21 for the case of recommended minimum values are, $X_1 = 1.5$ m, $X_2 = 1.5$ m, and R = 9.0 m.

Height of vanes $= \frac{1}{3}$ to $\frac{1}{4}$ of $y_1 = 0.4$ to 0.3 m; adopt 0.3 m

Thickness of vanes in masonry $= 12$ cm

Width of channel between vanes $= \frac{3}{2}$. $0.3 = 0.45$ m; number of vanes $\frac{1.5}{0.45} + 1$, say 4.

For other layout details, see Figure 3-41.

3.18   GIBB'S GROYNE

3.18.1   General Features

The Gibb's groyne wall is used in cases where the offtaking canal, on account of its gradient, has the same silt carrying capacity as the parent canal. The Gibb's groyne ensures more or less proportional silt distribution to the offtaking canal.

The device is a curved vertical wall (Figure 3-42) constructed in the parent canal. Its height from the canal bed is equal to the full supply depth plus free board. It starts from the downstream end of the offtake abutment and continues upstream to either the opposite point of the offtake end or three-quarters of the offtake width.

It is not necessary that its upstream end be tangential to the central line of flow of the parent canal nor should the downstream end be tangential to the downstream abutment of the offtake channel.
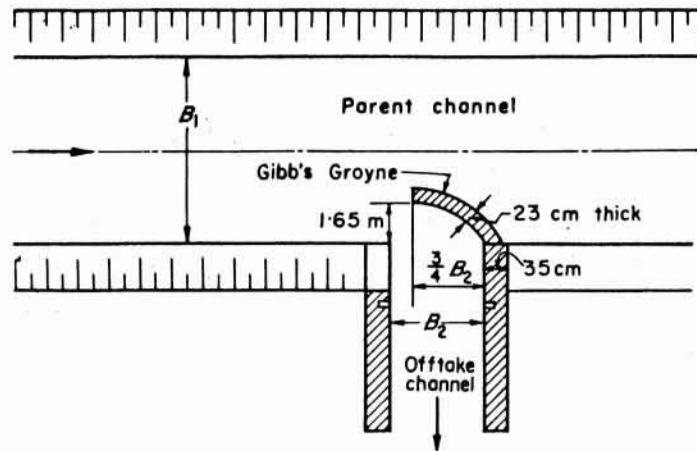
FIGURE 3-42. - Gibb's Groyne, general layout.

It is not necessary to pitch the bed and side of the parent canal.

Normally the Gibb's groyne would project far enough to enclose enough of the discharge of the parent canal to run the offtake canal full when the parent canal is running at its lowest supply level. The amount of projection would also depend upon the actual velocities existing near the offtake head.

3.18.2   Numerical Example

	Parent canal	Offtake canal
Q, m^3/s	5.0	1.0
B, m	6.0	3.0
v, m/s	1.0	-
v, near the offtake	0.80	

Consider 10% extra discharge to fill the offtake approach.

The width to be enclosed in the parent canal for drawing $1 \text{ m}^3/\text{s}$ in the off-take will be $(\frac{1.1}{5})$. 6 . $(\frac{1}{0.8})$ = 1.65 m. Figure 3-42 shows the details.

3.19 CURVED WING WITH SILT VANES

When the effect of a curved wing (Gibb's groyne) alone is not satisfactory to control the entry of silt into the offtake canal, King's vanes may be added to enhance the performance of the curved wing. The curved wing in this case is terminated at the 2 : 1 line of the longest vane.

The design of King's vanes has been explained in section 3.17. Figure 3-43, however, shows the arrangement using King's vanes and a curved wing.
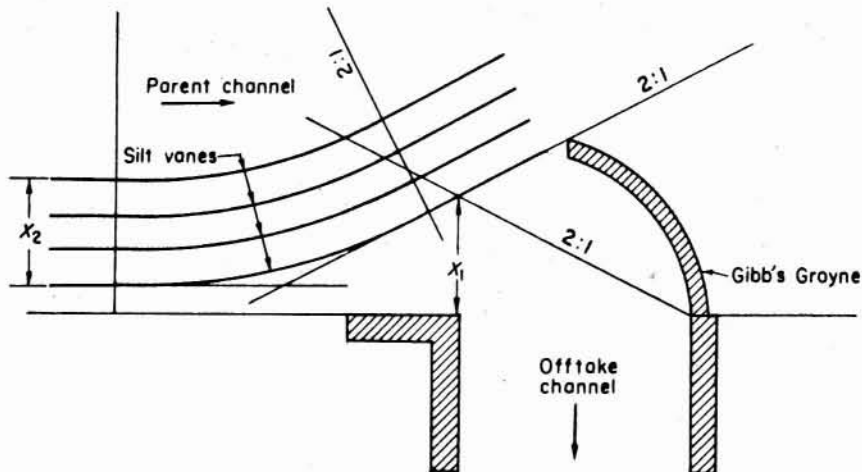
FIGURE 3-43. - Curved wing with silt vanes, general layout.

3.20   SILT PLATFORMS

3.20.1   General

A silt platform consists of a horizontal slab, usually of reinforced concrete supported on piers at a suitable level to exclude bottom water heavily loaded with silt or debris.   The device is suitable only in deep parent canals.

There are two variations of this device :

(a)   a simple platform;

(b)   a platform with a curved extension of the downstream wing wall.

3.20.2   Design of a Simple Platform

3.20.2.1   Level of platform

For practical considerations, the height of the tunnels should be 0.6 m to 0.9 m so that the stray debris may pass through the tunnels without choking them.

Other considerations to be borne in mind are that as much as possible of the bottom water should be excluded and there should be enough top water to fill the offtake.

3.20.2.2   Width of platform

The platform should be wide enough to take enough water over it to fill the offtake with 25% to spare.

The downstream edge of the platform should preferably be at an angle of 60° to the centreline of the parent canal and as well to the upstream edge.

The upstream edge of the platform at the edge of the parent canal may be 1.5 m upstream of the upstream edge of the offtake.

A width equal to the depth of the parent canal will do, provided it is not possible to arrange for a greater width.

Mean side velocity (near the offtake) of the upper water flow in the

Parent channel

0.3

0.23

1.75

15 to 30 m
pitching

1.75

0.23

Central pier

15 to 30 m
pitching

0.23

1.3

1.5

Side pitching

Offtake channel

Plan

Silt tunnel

0.6

1 3

Pitching in bed

Central pier

Section B-B

Section A-A

0.23

0.6

0.6

1.75

0.23

1.75

0.23

170

Note: All dimensions are in metres.

0.3

0.3

A

A

Parent channel

1.5

0.3

0.23 m thick guide wall

15 m to 30 m brick pitching

1.5

0.3

Side pitching

B_2

Plan

Offtake channel

Guide wall

1

1

3

0.6

0.5

Silt tunnel

1

Pitching

Section A-A

Note: All dimensions are in metres.

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SILT PLATFORM CUM-GUIDE WALL

Project, Region, Country India

Figure No. 3-45

parent canal should be equal to that in the offtake canal.

The pier supporting the slab should be sloped at 1 vertical to 3 horizontal at its upstream end and it should be of cut water shape. The sloping part should begin 0.3 m away from the edge of the platform. The floor under the tunnels and the side should be pitched. The length of the pitched floor may range from 15 m to 30 m.

3.20.2.3   Numerical example

Design a silt platform for an offtaking canal from the following data.

(a)   Mean observed velocity in the parent canal when running with minimum discharge   0.6 m/s

(b)   Full supply discharge of the offtaking channel   1 m³/s

(c)   Full supply depth of the offtaking canal   0.6 m.

Solution

The silt platform should allow to pass over it a discharge of 1.0 m³/s + 25 per cent = 1.25 m³/s. Width of the platform placed at 0.6 m below the water level with minimum discharge in the parent canal =

$$\frac{1.25}{0.6 \cdot 0.6} = 3.47 \text{ m},$$ say 3.5 m. The platform to be supported on piers of 23 cm thickness. Provide bed pitching and side pitching each 15 m long.

Figure 3-44 shows the design and details of the device.

3.20.3   Design of a Silt Platform with a Guide Wall

3.20.3.1   General features

A guide wall is sometimes added to improve the flow pattern (see Figure 3-45). The curved wing wall is built over the silt platform. The design principle for arriving at the level of the platform is the same as explained above for the silt platform.

There should be enough water passing over the platform, at the lowest supply in the parent canal to enable the offtake to run full with 10% to

spare.

The downstream edge of the platform is generally curved to the centreline of the parent canal and the upstream edge is terminated at the upstream edge of the offtake.

The design of the piers remains the same as for the silt platform.

The upstream edge of the curved wing wall is sloped at 1 : 1.

3.20.3.2   Numerical example

Design a silt platform with a curved wing for an offtake canal from the following data:

(a)   Observed mean velocity with minimum discharge in the parent canal near the offtake = 0.60 m/s

(b)   Full supply discharge of the offtake canal = 1 m^3/s

(c)   Full supply depth of the offtake channel = 0.6 m.

Solution

The silt platform should pass 1 + 10% = 1.10 m3/s.   The width of the platform, placed at 0.6 m below the full supply of the parent canal will be

$\dfrac{1.10}{0.6 \cdot 0.6}$ = 3.05 m,   say 3.0 m.   Provide a pier to support the platform.

Provide bed and side pitching over a length of 15 m.   See details in Figure 3-45.

3.21   REVERSE VANES

When a canal bifurcates into two sub-canals or channels, one of which silts very badly, reverse vanes may be built to pass more silt into the canal which does not silt up.

The principle of design of the vanes remains the same, as previously described, but they are reversed in direction.   See Figure 3-46 for the general layout.
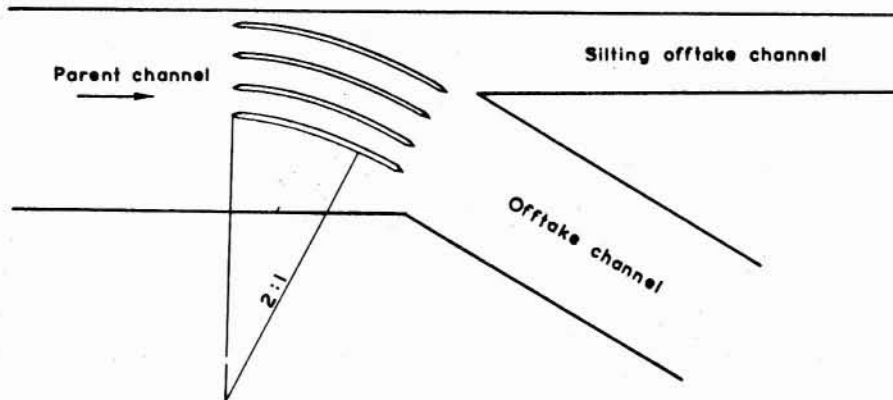
FIGURE 3-46. - General layout of reverse vanes.

3.22   VORTEX TUBE SAND TRAP

3.22.1   General Features

For small canals (under 3 m bottom width) the vortex tube sand trap, developed by the Colorado Agricultural Experiment Station, is essentially used to remove bed load material that is moving at or near the bed and is coarse, i.e. the size is greater than 0.50 mm.

The device consists of a slotted nitch in the floor of a channel placed at an angle with the direction of flow.   The vortex tube is located between the intake head and a measuring structure in the offtake canal.   The extra amount of water necessary for the operation of the tube is returned to the parent canal through a collection chamber.

The velocity of the water over the tube must be great enough to supply the energy necessary to cause rotation of the water in the tube.   Usually a velocity of 1.2 to 1.8 m/s is required.   In operation, sand drops into the tube, the
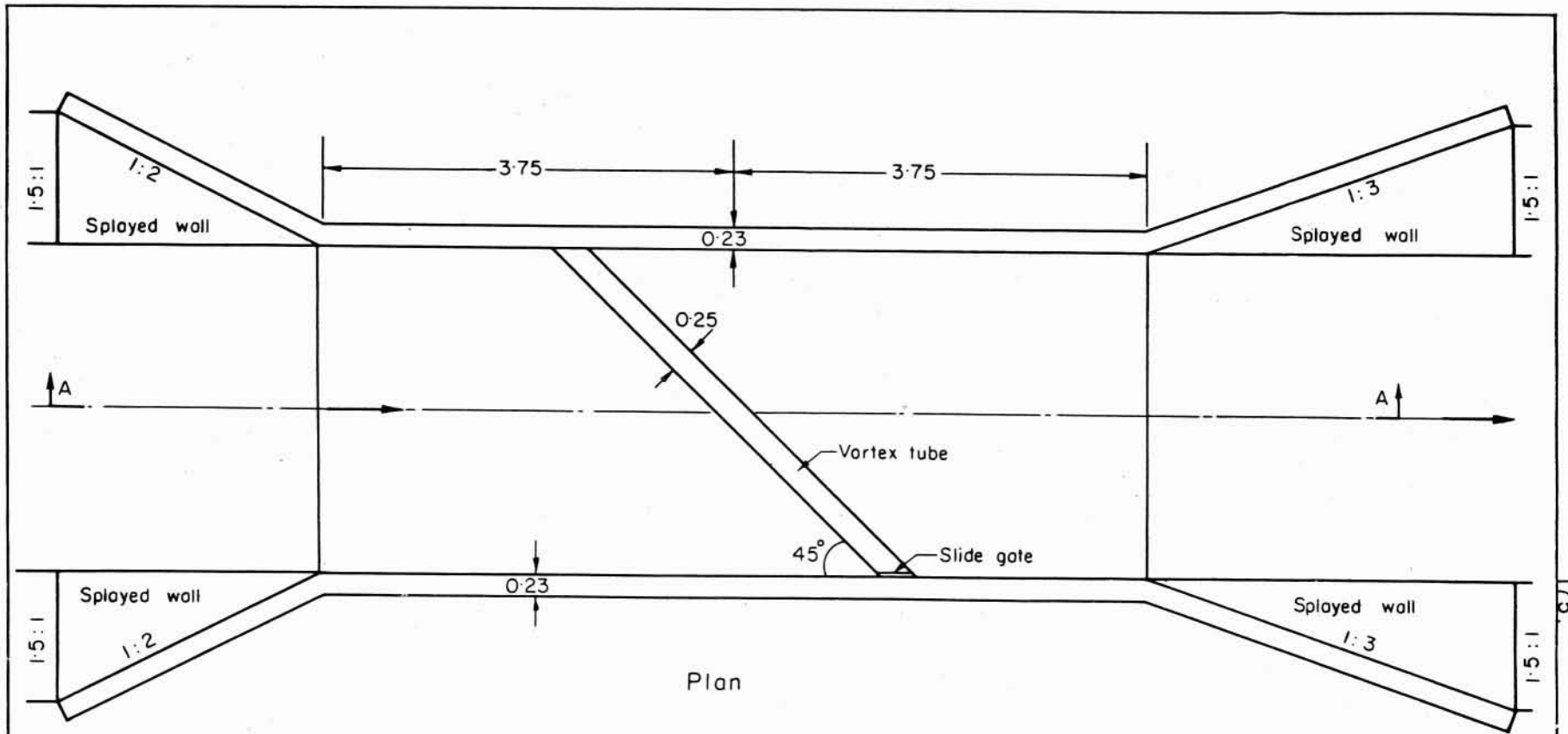
Plan

Note: All dimensions are in metres.
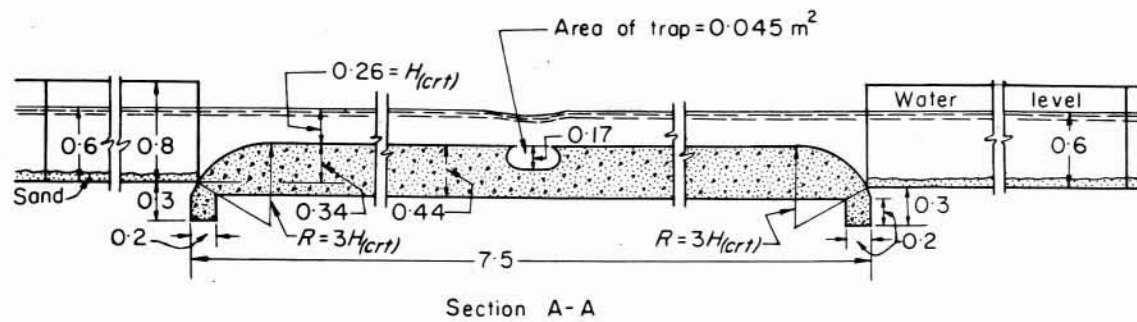
Section A-A

FAO-ICID

VORTEX TUBE

Project, Region, Country
USA

Figure No. 3-47

rotation keeps it agitated and the longitudinal flow in the tube transports it to the outlet. The outlet end should be equipped with a slide gate for control of the flow since it does not need to be fully open to function.

3.22.2 Design Characteristics

The canal section where the tube is installed should be the same as the canal width but with the bottom raised. The Froude number of the flow across the canal section containing the tube should approximate to 0.8. Where the flow varies widely, design flow should be selected that will exist for a greater portion of time. The length of the raised platform is 2.5 times the width of the canal.

The flow removed usually ranges from 5% to 15% of the total discharge.

The width of opening should usually be in the range of 0.15 m to 0.3 m.

The ratio of length of tube to width of opening $L_{(tube)} : B_{(tube)}$ should not exceed 20 with the maximum length of tube being approximately 4.5 m. Several successful field installations have the values 11 to 15 for this ratio.

The tube angle α should be 45°.

Straight tubes operate as well as tapered ones.

The elevation of the lips of the tubes can be the same rather than the downstream lip being lower.

The shape of the tube is not particularly important. A tube made from a commercially fabricated pipe is suitable. Normally the tube is shaped from about 16 gauge galvanized steel.

The required area of the tube can be approximated by the formula

$$A_{(tube)} = \frac{0.06}{\sqrt{2}} \, B_{(tube)} \, L_{(tube)}, \quad \text{where} \quad A_{(tube)} = \text{Area of tube,}$$

$B_{(tube)}$ = Tube opening; $L_{(tube)}$ = Length of the tube.

The depth in the tube, $d_{(tube)}$ is $\frac{2}{3}$ of the tube opening in practice.

When the bed load is excessive, two parallel vortex tubes may be installed.

3.22.3   Numerical Example

Design a vortex tube for an unlined canal with the following data:

Offtake canal

Q, m^3/s	1
B$_2$, m	3.0
y$_2$, m	0.6

for rectangular section, velocity $= \dfrac{1}{3.0 \; . \; 0.6} = 0.56$ m/s

Solution

Length of tube, $L_{(tube)} = 3 \; . \; \sqrt{2} = 4.24$ m

$\dfrac{L_{(tube)}}{B_{(tube)}} \not> 20$, and $0.3 > B_{(tube)} > 0.15$; adopt $B_{(tube)} = 0.25$ m

$\dfrac{L_{(tube)}}{B_{(tube)}} = \dfrac{4.24}{0.25} = 16.96$ say 17

Area of tube $A_{(tube)} = \dfrac{0.06 \; B_{(tube)} \; L_{(tube)}}{\sqrt{2}}$

$= \dfrac{0.06 \; . \; 0.25 \; . \; 4.24}{\sqrt{2}} = 0.045$ m^2

Depth in the tube $= \dfrac{2}{3} \; . \; 0.25 = 0.17$ m

The sides of the tube to be rounded to increase the area to 0.045 m^2 as the rectangular area $= 0.25 \; . \; 0.17 = 0.0425$ m^2.

Length of platform $= 2.5 \; . \; 3 = 7.5$ m

For broad crested weir $Q = 1.7 \; B \; H^{\frac{3}{2}}_{(crt)}$

$1 = 1.7 \; . \; 3 \; . \; H^{\frac{3}{2}}_{(crt)}$

$H_{(crt)} = (\dfrac{1}{5.1})^{\frac{2}{3}} = (0.196)^{\frac{2}{3}} = 0.337$ m

Crest will be 0.60 - 0.337 = 0.263 m above the bed

$$v = \frac{1}{0.337 \cdot 3} = 0.989 \ m/s$$

$$\text{Froude Number} = \frac{v}{\sqrt{g\,H_{(crt)}}} = \frac{0.989}{\sqrt{9.81 \cdot 0.337}} = 0.544$$

The value of Froude Number is less than 0.8.

$$\text{Adopt } H_{(crt)} = 0.26 \qquad v = \frac{1}{0.26 \cdot 3} = 1.28 \ m/s$$

$$\text{Froude Number} = \frac{v}{\sqrt{g\,H_{(crt)}}} = \frac{1.28}{\sqrt{9.81 \cdot 0.26}} = 0.80$$

This value is satisfactory.

The crest will be 0.6 - 0.26 = 0.34 m above the bed of the canal. The upstream and downstream approaches will be rounded as shown in Figure 3-47.

3.23   SLOPING-SILL SAND SCREEN

3.23.1   General Features

The sloping-sill sand screen was evolved in Egypt by engineer Abdel Azim Ismail who observed the local asymmetry in the flow pattern at the intake of distributary canals. The device comprises a curved wall placed in the parent canal opposite the intake of an offtake canal. The screen slopes up from the upstream end to the downstream end.

The length of the screen is determined on the basis that: (i) The depth of water over the screen sill will be $\frac{1}{3}$ to $\frac{1}{4}$ of the depth in the parent canal; (ii) the velocity over the sill will be about the same as in the offtake canal. Once the length has been determined it is possible to ascertain the height.

The difference between the sill elevations (in metres) at each end of the screen is determined from two formulae, one of which is theoretical, and the

other is empirical; the average of the two is adopted for design.

The theoretical formula is as follows :

$$\Delta_3 = \frac{2v^2}{g} \tan \frac{\alpha}{2}$$

where v is the velocity in the offtake canal, α is the angle of offtake measured from the centreline of the parent canal and expressed in radians.

The empirical formula takes the following form:

$$\Delta_3 = \frac{\alpha \, B_2}{35 \, \pi} \; ; \quad \text{where } B_2 = \text{bed width of the offtake}$$

canal in metres.

The sill is made of rectangular section and is 23 cm thick.

3.23.2   Numerical Example

Data

	Parent canal	Offtake canal
Q, m³/s	-	1
B, m	-	3
y, m	1.5	0.6
		90° ($\frac{\pi}{2}$)
v, m/s		0.5

Design

Assume a depth between $\frac{1}{3}$ and $\frac{1}{4}$ of the parent canal over the sill, i.e. between 0.5 and 0.375 m.   Adopt 0.4 m.

Length of screen . depth over screen . velocity = 1 or
length of screen . 0.4 . 0.5 = 1.

$\therefore$ Length of screen $\frac{1}{0.2}$ = 5 m

To find value of Δ_3

From theoretical formula, $\Delta_3 = \frac{2v^2}{g} \tan \frac{\alpha}{2} = \frac{2(0.5)^2}{981} = 0.051$ m
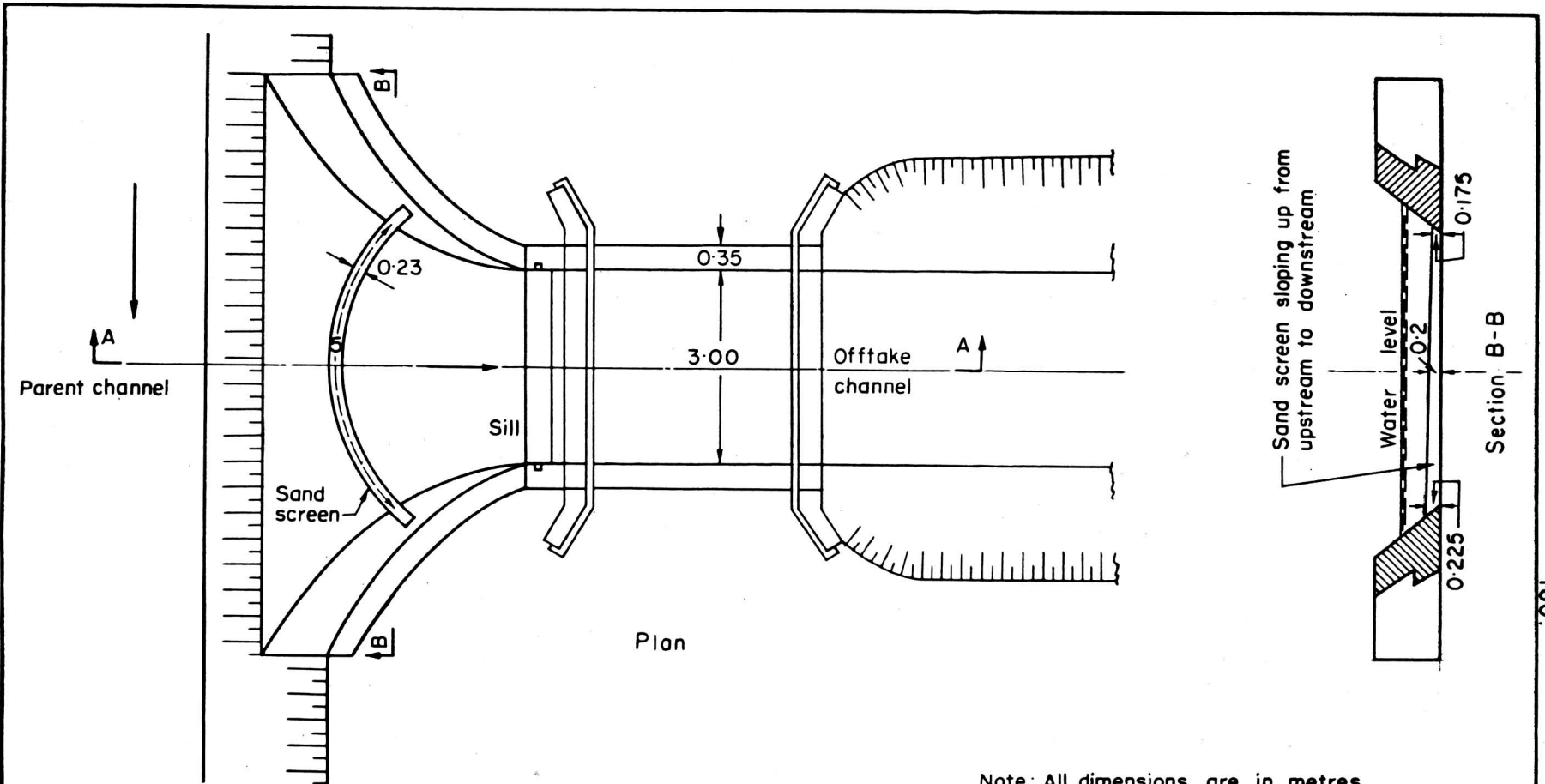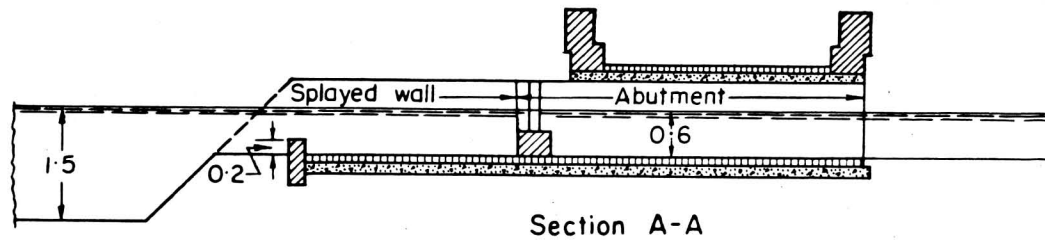
0.23

0.35

3.00

A

A

Parent channel

Offtake channel

Sill

Sand screen

B

B

Plan

Sand screen sloping up from upstream to downstream

0.175

Water level

0.2

0.225

Section B-B

Note: All dimensions are in metres.

Splayed wall

Abutment

1.5

0.2

0.6

Section A-A

180.

From empirical formula $\Delta \zeta = \dfrac{\alpha \cdot B_2}{35 \pi}$

$= \dfrac{3 \pi}{35 \pi \cdot 2}$

$= 0.043$ m.

Average value of $\Delta \zeta = 0.047$ m $= 4.7$ cm, say 5 cm.

Average height of screen $= 0.6 - 0.4 = 0.2$ m $= 20$ cm.

The upstream end will be 17.5 cm above the bottom of the offtake and the downstream end will be 22.5 cm above the bottom of the offtake. For details see Figure 3-48.

4.   FLOW DIVIDING STRUCTURES

4.1   INTRODUCTION

Many structures for flow division have been developed to suit a wide variety
of conditions but all of them are used in irrigation networks to divide the flow of
a channel into two or more parts.   Each part is a defined proportion of the total
flow.   Thus, flow dividing structures differ distinctly from intakes and outlets in
that the latter are designed to draw off a definite fraction of the flow in the parent
channel, but the exact proportion of this fraction to the total flow or to the
remainder in the parent channel is generally immaterial.   A flow dividing
structure requires a control section in both the offtake channel and in the parent
channel.   Nevertheless, not all flow dividing structures are built to give exactly
proportional division.   To distinguish them further from intake structures, a
structure which diverts more than 25% of the flow of the parent channel through it
is also regarded as a flow dividing structure.

Another main distinction can be made between structures which are designed
to provide a permanently fixed proportional division of flow and those which are
equipped with a movable splitter.   The splitter allows the proportion of flow
between the resulting streams to be changed according to requirements.   Such
changes may be required to meet seasonal or temporary variations in demand   in
the areas supplied by the branch canals.

The structures which provide division of flow may be sub-divided into :
those which are designed to give strictly accurate proportions, (using a control
section across the supply channel - causing shooting flow or free fall); and those
which do not provide a strictly proportional division, (at least not over the whole
range from zero flow to full design discharge).   A significant feature of the
former category is that the flow is divided by thin plated (splitting) walls.   The
latter category includes mainly division or diversion boxes which have the
additional function of alternating the flow between different offtake channels, using
slide gates or flash boards.

It should be noted that most flow dividing structures offer excellent con-

ditions for adaptation to water measurement because of their capacity to generate shooting flow or free overfall.

Flow dividing structures can be tested for accuracy by measuring the flows from each side. Such tests should be made frequently and over a wide range of conditions as relationships frequently change with the amount of water being diverted.

4.2   FIXED PROPORTIONAL DIVISORS

The main feature of fixed proportional divisors is that the permanent splitting of the flow into two or more parts takes place in a control section where a state of supercritical flow, i.e. shooting flow, or free fall, is generated. This calls for some head loss in the structure by letting the flow pass a sill or flumed section or by creating a drop, but is not required for the splitting of a flow into two exactly equal proportions provided that: the dimensions are symmetrical; that the flow section in the structure is of uniform roughness; that there is a straight canal alignment of 5 to 10 m upstream of the divisor; and, finally, that no backwater effect is created in either of the offtake channels. By installing a subsequent 1 : 1 divisor, the flow will be split into the proportions of 2 : 1 : 1 or, by rejoining two of the streams, a proportion of 3 : 1 may be obtained.

FIGURE 4-1. - Simple fixed proportional flow divisor on small irrigation canal in Cyprus.
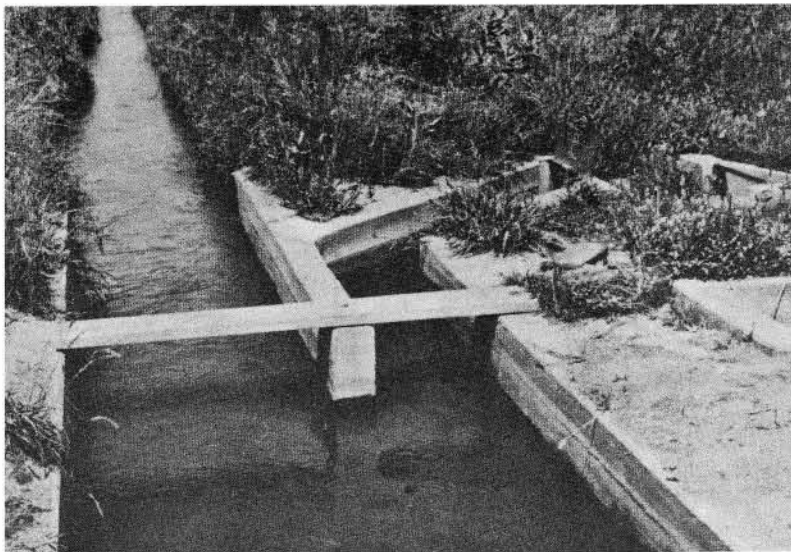
FIGURE 4-2. - Simple fixed proportional flow divisor
of low accuracy.
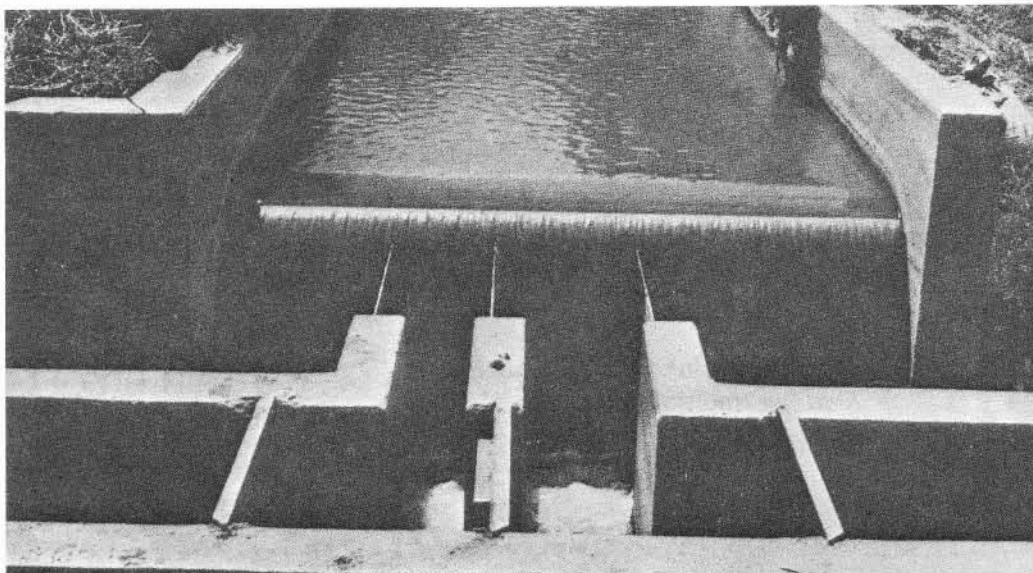
FIGURE 4-3. - Fixed proportional divisor splitting a
given flow into four streams of exact constant proportion.(27)

Plan

Section A-A

Note: All dimensions given above are
to be multiplied by H_c

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DIMENSIONS OF FIXED DIVISOR
SPLITTING FLOW INTO TWO
EQUAL STREAMS

Project , Region , Country
Morocco

Figure No. 4 - 4
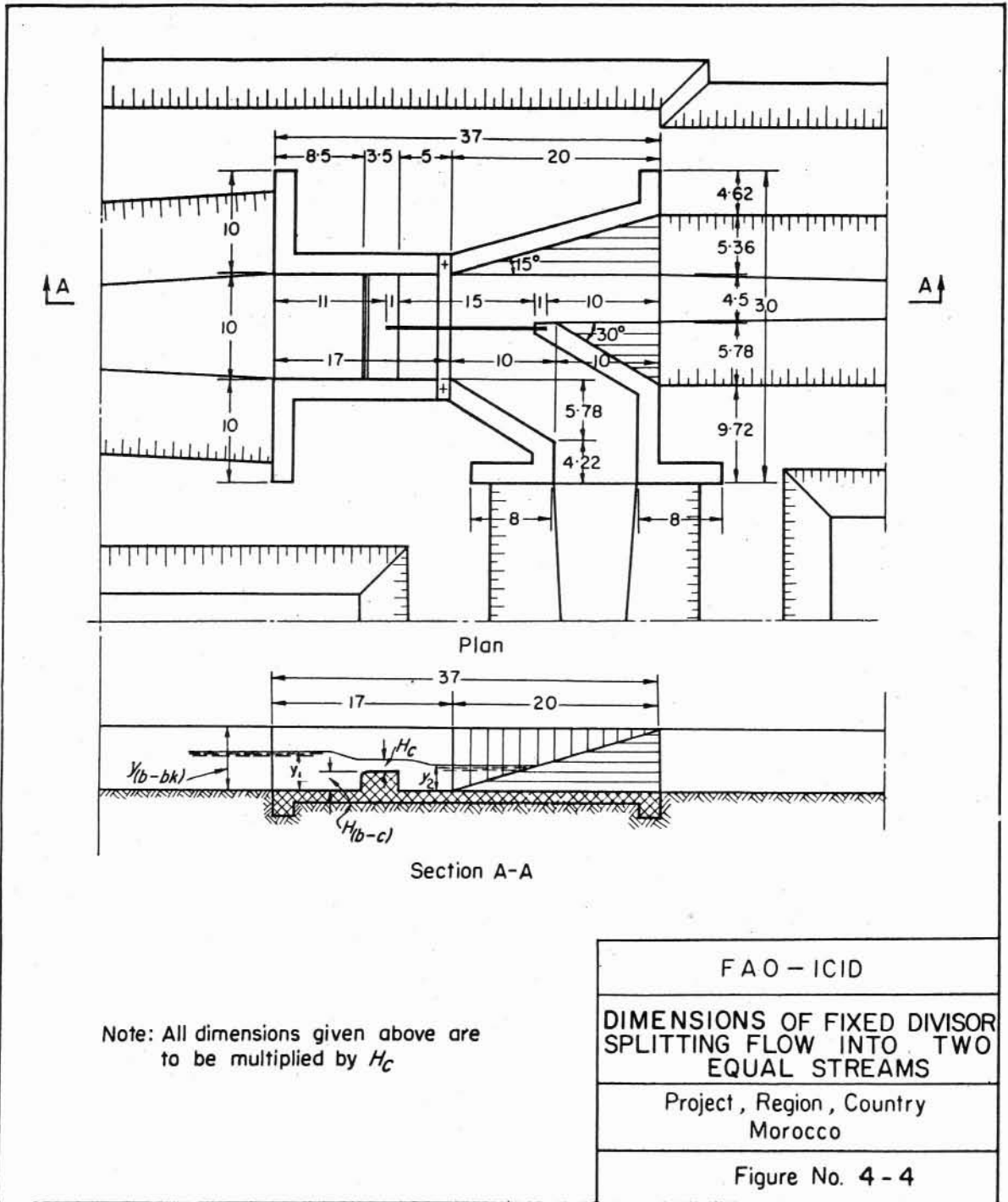
Figure 4-1 illustrates a small fixed divisor correctly designed to provide proportional division, while Figure 4-2 illustrates a solution of poor accuracy because of insufficient velocity (no control section) and unsuitable splitting wall. Figure 4-3 shows a divisor of the free fall type, dividing any given supply up to maximum carrying capacity into four streams of exact constant proportion.

4.2.1   Divisor for Splitting Flow into Two Equal Streams

4.2.1.1   General features

A fixed flow divisor, developed in France and used in Morocco and other countries in North Africa, is shown in Figure 4-4. It is eminently suitable for dividing flow into two equal parts at the tail end of a distributing canal.

This divisor consists of: an upstream approach in an earthen channel (to increase the width of the channel section gradually to the width of the control section of the structure); upstream head walls; side walls of the control section; a raised sill; a thin steel sheet to divide the flow; downstream side walls and wing walls in the two channels; an upstream and a downstream floor with a cut-off at both ends. The structure may be built entirely in concrete or in brick masonry.

This type of flow divisor has two useful features. Firstly, the raised sill generates a state of shooting flow on its downstream side, so that the division of the water into two equal parts is quite independent of the levels in the offtake channels, and thus the discharge passing into each of the offtakes is not affected by control operations in either of them. Secondly, the change back from shooting to tranquil flow in the offtake channels shows up as a hydraulic jump, thanks to which, part of the kinetic energy of the water is regained, and the head losses associated with a flow divisor are therefore kept down to a minimum.

Thickness of walls and floor

The thickness of walls (i.e. the head walls, side walls, downstream side walls and wing walls, except the dividing wall of the two offtakes) should be as given in Table 4-1.

TABLE 4-1

Design of Fixed Flow Divisor - Thickness of Walls

Q in l/s	Thickness in cm	
	Concrete	Masonry
10 to 80	15	30
80 to 200	20	30
> 200	25	30

The minimum thickness of the dividing wall should be 10 cm for concrete and 20 cm for brick masonry constructions. The steel plate should be 6 mm thick. The thickness of the floor should be as given in Table 4-2.

TABLE 4-2

Design of Fixed Flow Divisor - Thickness of Floor

Q in l/s	Thickness in cm	
	Concrete	Masonry
10	10	15
29	10	15
59	12	15
103	12	15
162	15	20
239	15	20
283	15	20
333	15	20

The depth of the cut-offs should be twice the thickness of floors for the corresponding discharges.

The relationship between discharge capacity and quantity of materials is given in Figure 4-5
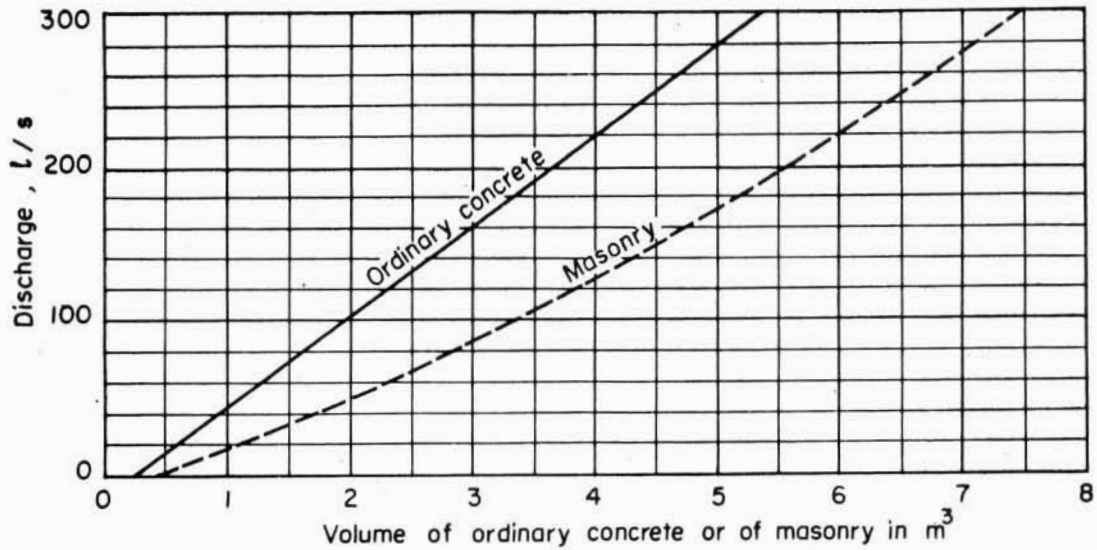
FIGURE 4-5. - Relationship between discharge capacity and materials requirement for fixed flow divisor.

4.2.1.2   <u>Design</u>

The formula generally used for calculating the discharge is:

$$Q = C B_{(t)} \sqrt{2g} \; H_{(crt)}^{1.5}$$

where   Q   =   discharge of the parent channel

        C   =   coefficient

     $B_{(t)}$ =   width of the controlling section (across axis of flow)

  $H_{(crt)}$ =   head over the sill.

C varies from 0.38 to 0.41 (0.38 for a sharp upstream end of the sill and 0.41 for a rounded upstream end (5 to 10 cm radius).

The length of the sill, $L_{(crt)}$, should be equal to 3.5 H_c.

$B_{(t)}$ should be taken as equal to 10 H_c.

$$H_{(crt)} \quad = \quad 1.5\,H_c$$

$$H_c \quad = \quad (\frac{q^2}{g})^{\frac{1}{3}} \quad \text{(critical depth)}$$

For formation of hydraulic jump, the working head, $h_{(wk)}$, should be $\geqslant\ 0.4\,H_{(crt)}$ or $\geqslant\ 0.6\,H_c$.

A working head equal to H_c is advisable.

The dimensions of H_c, $H_{(b-c)}$, y_2, $y_{(b-bk)}$ are given in Table 4-3.

TABLE   4-3

Discharge in l/s	H_c	$H_{(b-c)}$	y_1	$y_{(b-bk)}$
		.... cm		
10	0.04	0.10	0.16	0.35
29	0.06	0.15	0.24	0.45
59	0.08	0.15	0.27	0.50
103	0.10	0.20	0.35	0.60
162	0.12	0.20	0.38	0.65
239	0.14	0.25	0.46	0.70
283	0.15	0.25	0.48	0.70
333	0.16	0.25	0.49	0.75

Figure 4-4 shows dimensions for the various parts which should be multiplied by H_c at the maximum discharge.

4.2.1.3   Numerical example

Design a fixed flow divisor 1:1 in accordance with the following data :

        discharge of supply channel, $Q = 333$ l/s
        depth of water in supply channel, $y = 0.49$ m

        From Table 4-3, H_c for $Q = 333$ l/s is 0.16 m.   Then,
$H_{(b-c)} \quad = \quad 0.25$ m

$$B_{(t)} \quad = \quad 10\, H_c \quad = \quad 1.60\ m$$

$$L_{(crt)} \quad = \quad 3.5\, H_c \quad = \quad 0.56\ m$$

The thickness of the right and left walls = 25 cm for concrete or 30 cm for masonry construction.

The thickness of the floor or apron should be 15 cm for concrete and 20 cm for masonry construction.

The depth of the cut-offs should be 30 cm for concrete and 40 cm for masonry construction.

Other dimensions should be as given in Figure 4-4 but multiplied by 0.16.

4.2.2   Divisor for Splitting Flow into Two Unequal Streams

4.2.2.1   General features

With a broad rectangular sill, as in the structure previously described, the position of the control section is not defined with the precision required for the division of flow into unequal streams of given proportions. In this section a structure is described wherein the sill is triangular, with gentle slopes upstream and downstream. In this divisor, the depth in the control section is just equal to the critical depth, irrespective of the discharge, and is measured exactly from the top of the triangle. The absolute minimum value of the apex of the crest above the bed level of the offtake is equal to that of the hydraulic jump when it forms at a distance of $2\, H_c$ from the apex of the crest or sill.

The masonry floor approach up to the beginning of the upstream glacis is equal to $2.5\, y_1$ where y_1 is the water depth in the parent channel.

The length of the upstream wing walls

$$= \quad y_1 \quad + \quad \left\{ \quad y_1 \quad + \quad (FB) \right\} \quad (ss)$$

where y_1 = depth of water upstream of the structure,

  (FB) = free board,

(ss) = side slope of the canal section.

The depth of the upstream and downstream cut-offs is equal to twice the thickness of the floor.

The upstream glacis has a slope of 1 : 4 to 1 : 5.

The top of the triangular sill is slightly curved in order to avoid contraction of flow. The radius of the curve is equal to $3 H_c$ (H_c is the critical depth).

The length of the rounded crest of the sill is equal to 1.176 H_c. The length of the downstream glacis (slope 1 : 5) from the centre of the triangular portion of the sill is equal to 5 $H_{(c-b)}$ + 0.59 H_c (where $H_{(c-b)}$ is the height of the centre of the triangular portion of the sill above the downstream bed level of the offtaking channel). The beds of the parent channels and offtake channels are at the same levels, and the water depth in the parent channel may differ somewhat from the designed depth, but this may be gradually compensated for in the reach immediately above the structure. Similarly, the bed level of the other offtake immediately below the structure is the same as the parent channel and any modification in water level, if needed, may be effected lower down.

The form and shape of the structure below the downstream glacis is as shown in Figure 4-6.

The thickness of walls (upstream wing walls, side walls of the control section, downstream side walls, wing walls and dividing wall), and the floor, depends on the type of soil at the site and the depth at which the floor is situated. As a rough guide, the thicknesses of these parts for different discharges in the parent channel may be derived from Table 4-4.

TABLE 4-4

Divisor with a Triangular Sill - Thickness of Walls

Q in l/s	Thickness	
	Concrete cm	Masonry cm
10	15	30
29	15	30
59	15	30
103	20	30
162	20	30
239	25	30
283	25	30
333 and above up to 1,000	25	30

Rounded crest 0.21

A ---- A

0.3

1·4

2·5

$2H_c$

1·667

15°

6 mm steel plate

H_c

$10H_c$

1·85

0.833

$10H_c$

0.3

$5·78H_c$

$4·22H_c$

0.3

L_{GL}

1·1

Plan

0.88

0.60

0·20

1:5

1:5

0·40

0.30

R=0.64

0.20

0·20

0·40
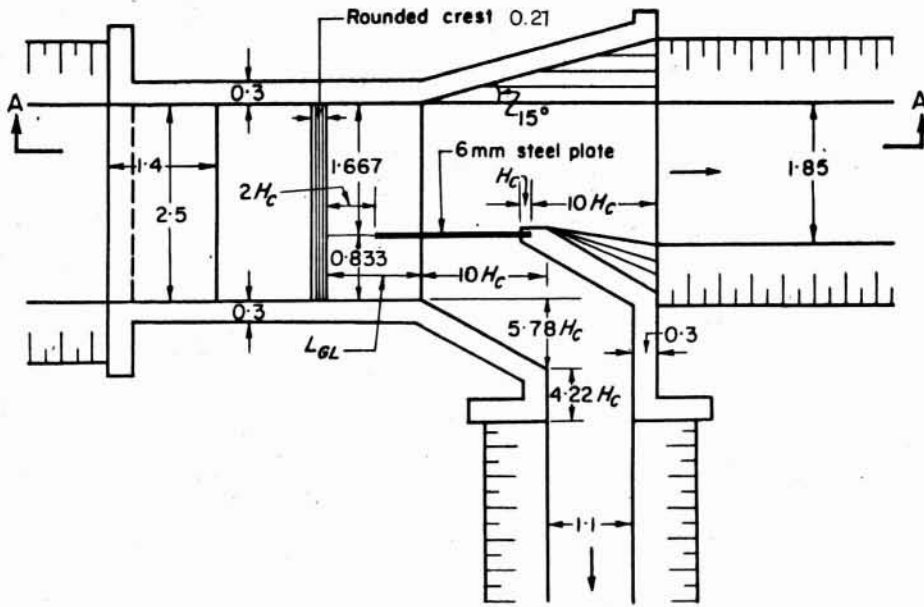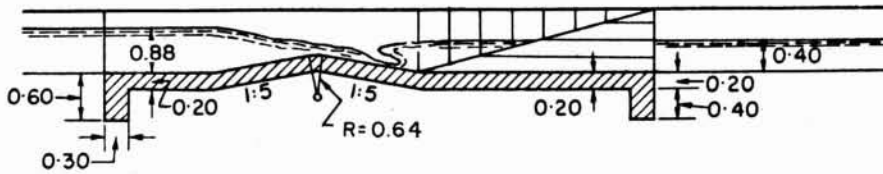
Section A-A

Note: All dimensions are in metres
unless otherwise specified.

FAO – ICID
UNFLUMED DIVISOR WITH A TRIANGULAR SILL
Project , Region , Country France
Figure No. 4 - 6

The thickness of the dividing wall is 10 cm for concrete and 20 cm for brick masonry construction.

The thickness of the floor should be as given in Table 4-5.

TABLE   4-5

Divisor with a Triangular Sill - Thickness of Floor

| | Thickness | |
Q in l/s	Concrete cm	Masonry cm
10	10	15
29	10	15
59	12	15
103	12	15
162	15	20
239	15	20
283	15	20
333 and above up to 1,000	15	20

The depth of the downstream cut-offs should be twice the thickness of the floor for the corresponding discharge.

The thickness of the steel plate dividing the flow is 6 mm and it extends up to a distance equal to $2 H_c$ downstream of the apex of the triangular sill.

4.2.2.2   Design

Calculate q (discharge per metre width of the control section) which is equal to $\dfrac{Q}{B_{(t)}}$ where Q is the discharge in m^3/s of the parent channel and $B_{(t)}$ is the width of the control section in metres.

Calculate H_c for q

$H_{(c-b)}$ may be calculated as follows:

The value of $\dfrac{H_{(c-b)}}{H_c}$ may be read from Figure 4-7 for a known value of $\dfrac{y_2}{H_c}$

H_c being known, $H_{(c-b)}$ can then be calculated.

$$q = CH_{(crt)}^{\frac{3}{2}} \sqrt{2g}$$

where $H_{(crt)}$ is height over the crest and C is a coefficient determined from:

$$C = \frac{1 + \dfrac{0.423}{\left\{ 1 + \dfrac{2}{3} \left(\dfrac{H_{(c-b)}}{H_c} \right) \right\}^2}}{\sqrt{2} \left\{ \dfrac{3}{2} + 0.0066 \dfrac{H_{(c-b)}}{H_c} \right\}^{\frac{3}{2}}}$$

and where $H_{(c-b)}$ is the height of the crest above the downstream bed.

Water depth on the upstream side will then be:

$$y_1 = H_{(c-b)} + H_{(crt)}$$

Head loss through the structure $= y_1 - y_2$

4.2.2.3   Numerical example

Design a divisor with a triangular sill in accordance with the following data:

Discharge of the parent channel, Q_1 = 600 l/s = 0.60 m³/s
Discharge of offtake A, Q_2 = 400 l/s = 0.40 m³/s
Discharge of offtake B, Q_3 = 200 l/s = 0.20 m³/s

B_1 = 2.5 m
B_2 = 1.85 m
y_2 = 0.40 m
B_3 = 1.10 m
y_3 = 0.40 m

$B_{(t)_1}$ = 2.5 m
$B_{(t)_2}$ = $2.5 \cdot \dfrac{0.4}{0.60}$ = 1.667 m
$B_{(t)_3}$ = 0.833 m

If the discharge of the parent channel $= 0.60 \, m^3/s$

$$q = \frac{0.60}{2.50} = 0.24 \, m^3/s$$

then H_c (corresponding to $q = 0.24 \, m^3/s$) $= (\frac{0.24 \cdot 0.24}{9.81})^{\frac{1}{3}}$

$$= 0.18 \, m$$

$$y_2 = 0.40 \, m$$

$$\frac{y_2}{0.18} = \frac{0.40}{0.18} = 2.22$$
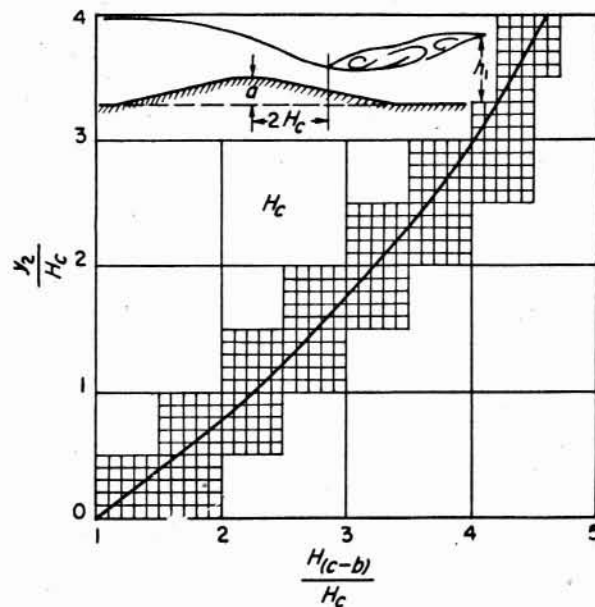
FIGURE 4-7 . - Divisor with triangular sill.
Relationship between $\frac{y_2}{H_c}$ and $\frac{H_{(c-b)}}{H_c}$.

From Figure 4-7, for $\frac{y_2}{H_c} = 2.22$ and $X_{(c-j)} = 2H_c$,

$$\frac{H_{(c-b)}}{H_c} = 3.4$$

$$H_{(c-b)} = 3.4 \cdot 0.18 = 0.612 \, m.$$

$$C = \cfrac{1 + \cfrac{0.423}{\left(1 + \frac{2}{3} \cdot 3.4\right)^2}}{\sqrt{2}\left(\frac{3}{2} + 0.0066 \cdot 3.4\right)^{\frac{3}{2}}} = \frac{1.04}{2.655} = 0.39$$

Now, $\quad q = C H_{(crt)}^{\frac{3}{2}} \sqrt{2g}$

$$0.39 \, H_{(crt)}^{\frac{3}{2}} \cdot 4.429 = 0.24$$

$$H_{(crt)}^{\frac{3}{2}} = \frac{0.24}{1.73} = 0.138$$

$$H_{(crt)} = 0.27$$

Water depth upstream, $\quad y_1 = H_{(c-b)} + H_{(crt)}$

$$= 0.61 + 0.27 = 0.88 \text{ m}$$

Head loss $= y_1 - y_2 \quad = 0.88 - 0.40 = 0.48$

     The design and other dimensions of the structure are shown in Figure 4-6.

4.3   STRUCTURES WITH ADJUSTABLE SPLITTER

4.3.1   General

     Structures with an adjustable splitter usually consist of a hinged gate made of sheet metal which can be moved across the flow section of the parent channel and fixed in any desired position with the help of an arch bar or screw bar. The flow has to be made supercritical for accurate proportioning by the introduction of a control crest or fall in bed level, as already discussed in section 4.2. Figures 4-8, 4-9 and 4-10 show examples of designs from Argentina, Spain and the U.S.A.
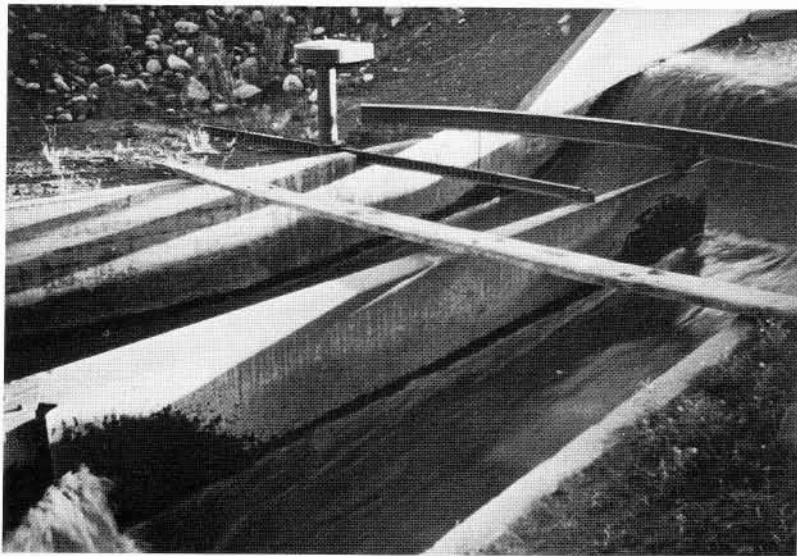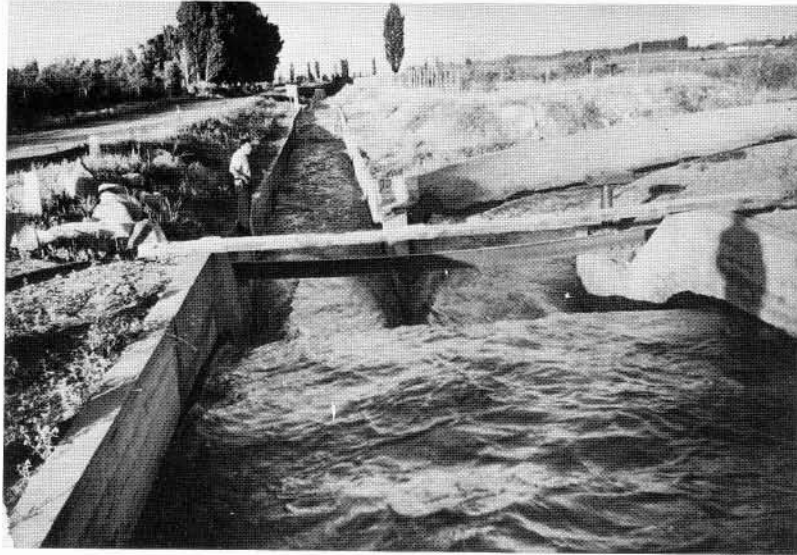
FIGURE 4-8. - Flow divisor with adjustable splitter, Argentina.
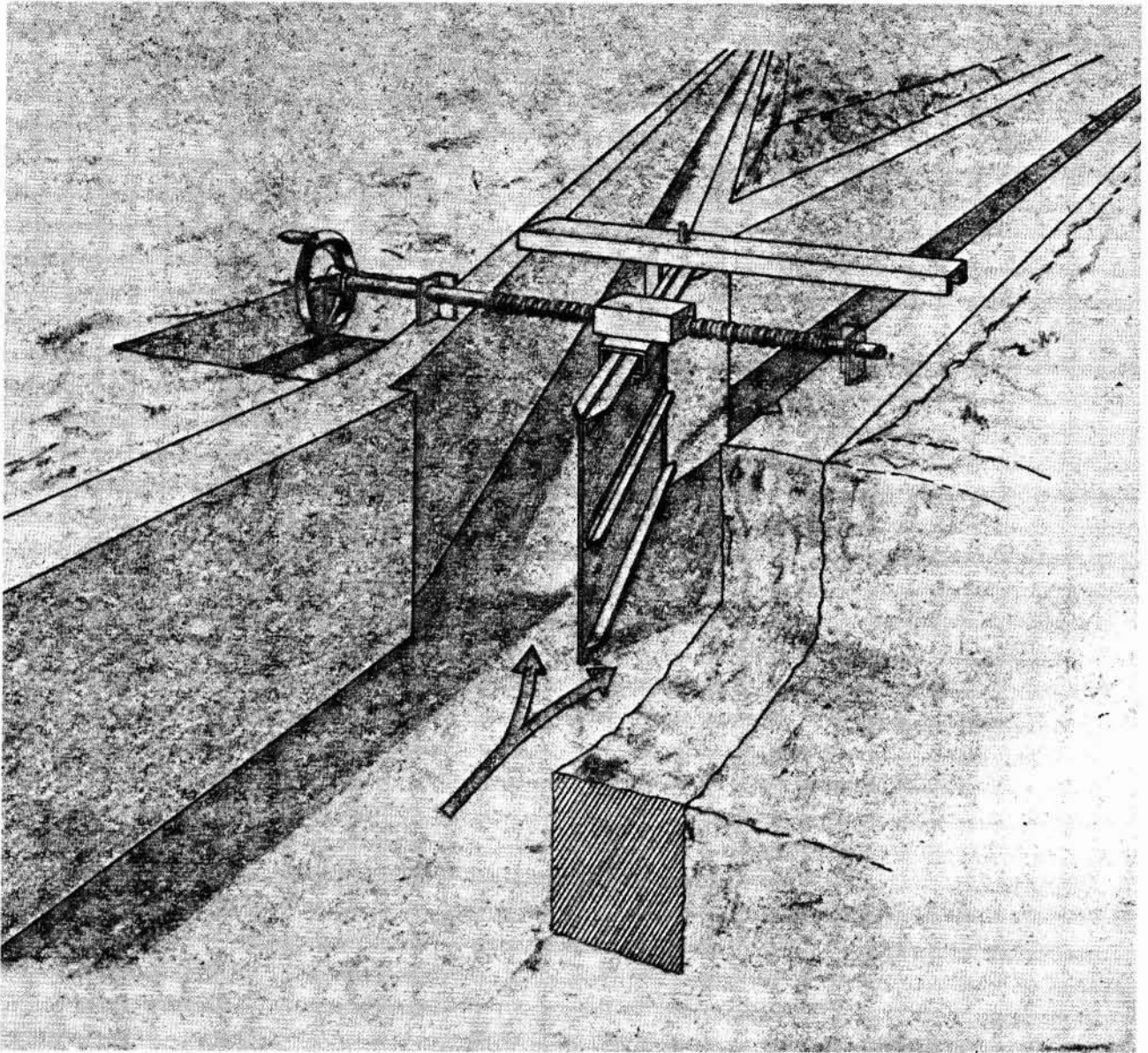
FIGURE 4-9. - Flow divisor with adjustable splitter,
Monegros Canal, Spain (72).
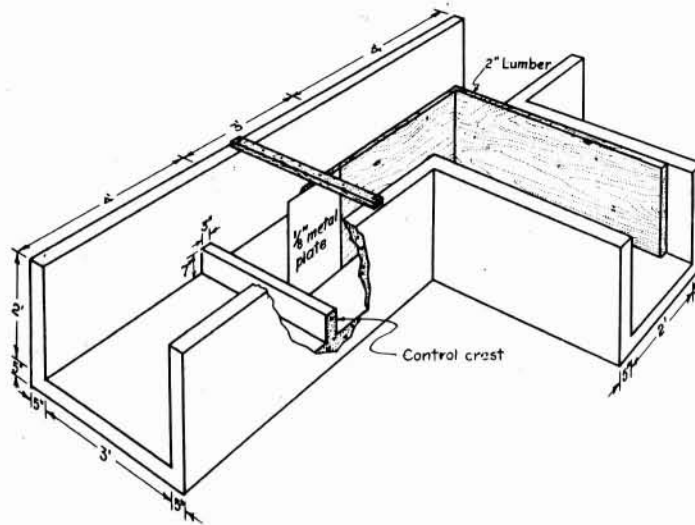
FIGURE 4-10. - Flow divisor with adjustable splitter (U.S.A.)

4.3.2   French Divisor

The French divisor shown in Figures 4-11, 4-12 and 4-13 was developed by
the Neyrpic Laboratories, Grenoble, and is used in Southern Europe and North
Africa.   It is suitable for canals with gentle slopes.

The structure consists of a concrete approach equal to the width of the
parent channel, a flumed section with upstream splayed wall, the crest, the down-
stream sloping glacis, divide wall, channel sections of offtake channels in
concrete with concrete floor and a dividing blade or gate.   The steel dividing
blade is adjustable and calibrations have been made so that the flow may be
varied proportionally between the two channels.   The whole structure is made of
prefabricated parts.

The structure ensures a hypercritical regime and the formation of a
hydraulic jump immediately downstream of the crest.   The division of flow is
therefore independent of the water levels in the offtake channels and any control
operations in them.   Minimum working heads are given in Table 4-6.

Sharp edge

Water surface level in offtake channel

$H_{(crt)}$

y_1

$h_{(wk)}$

y_2

Section

B_2

B_1

Flow

Adjustable

R_{GA}

B_3

Plan

Adjustable blade
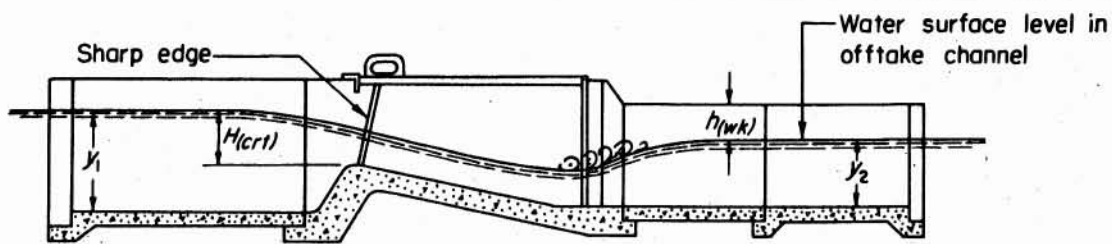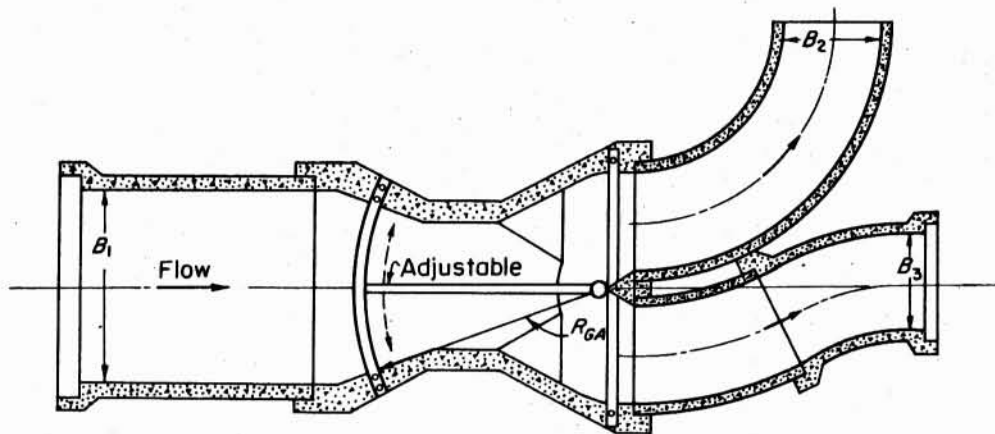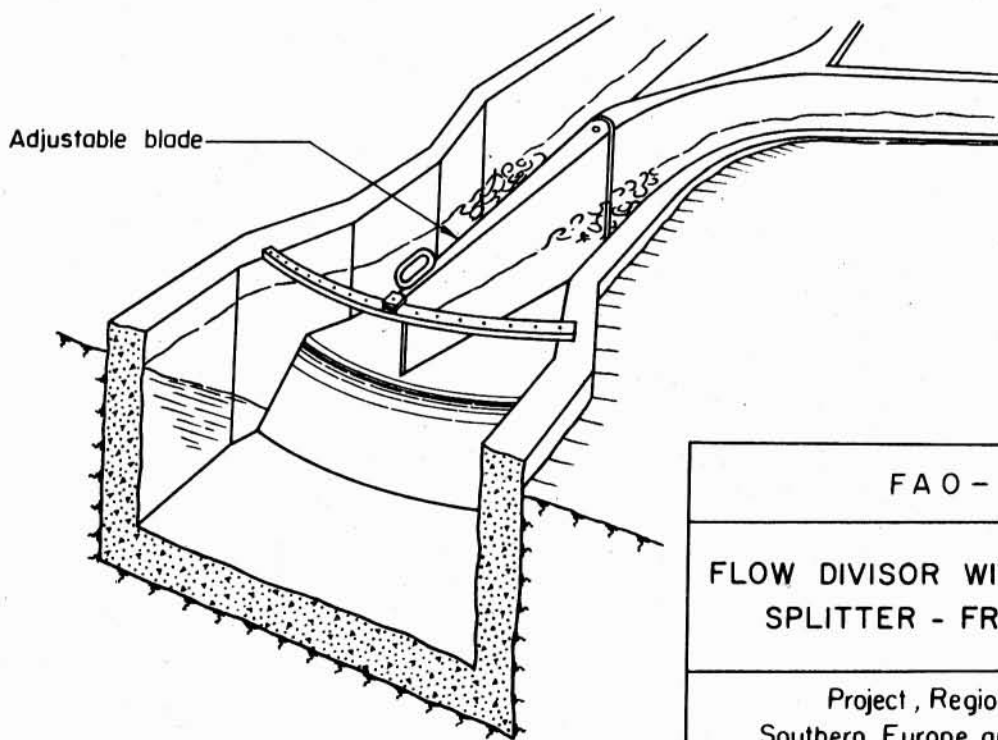
Perspective view

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FLOW DIVISOR WITH ADJUSTABLE SPLITTER - FRENCH TYPE

Project, Region, Country
Southern Europe and North Africa

Figure No. 4 - 11
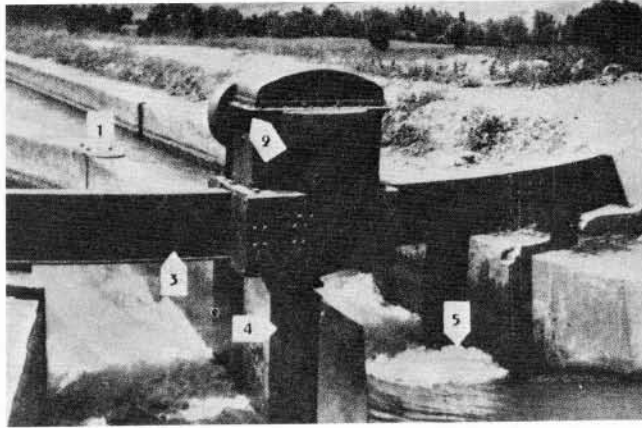
FIGURE 4-12. - Flow divisor (Neyrpic), Oned Fodda network, Algeria.
1. Flap hinge. 2. Control gear. 3. Flap positioning. 4. Movable flap.
5. Hydraulic jump. (69)
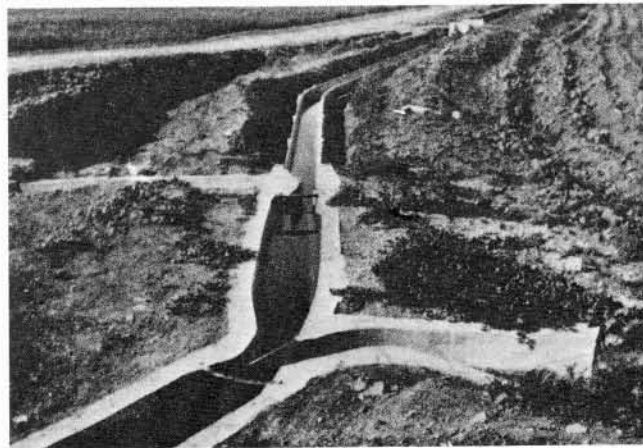
FIGURE 4-13. - Three-way distribution by means of two consecutive
flow divisors. (69)

The design formula used for the structure is as follows:

$$Q = 1.235 \ R_{GA} \ H_{(crt)}^{1.5}$$

where:

Q = discharge of the parent channel in m^3/s

R_{GA} = radius of the blade in metres (see Figure 4-11)

$H_{(crt)}$ = height of upstream water level over crest in m

Minimum loss of head = $0.4 \ H_{(crt)}$.

Table 4-6 gives data for six different types of structure.

TABLE 4-6

Design of Structures with Adjustable Splitter - Minimum Working Heads

Type Number of Structure	Adjustable Blade		Maximum Discharge Q in l/s	Maximum Head Over Crest $H_{(crt)}$ in cm	Head Loss for Maximum Q $h_{(wk)}$ in cm
	Radius in cm	Height in cm			
6	60	35	54	17.5	7.0
8	80	45	110	23.0	9.0
10	100	54	193	29.0	11.5
12	120	65	305	35.0	13.5
16	160	84	625	46.5	18.0
20	200	104	1,093	58.0	23.0

4.4   PROPORTIONAL DISTRIBUTORS

Under this heading structures are described which can be considered a cross between flow divisors, intakes and outlets. A typical feature of proportional distributors is that the flow is not divided into fractions by thin plated divisors but is diverted from the parent channel into the offtakes by means of individual

Parent channel
60°
120°

Parent channel
45°
135°

Parent channel
45°
60°
135°
120°

204.
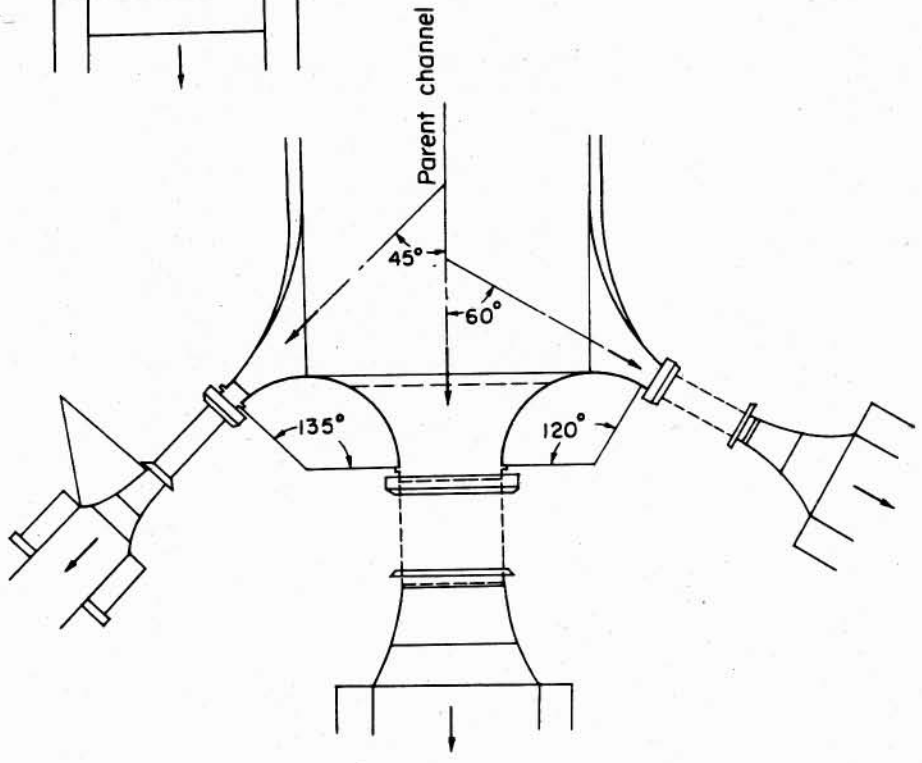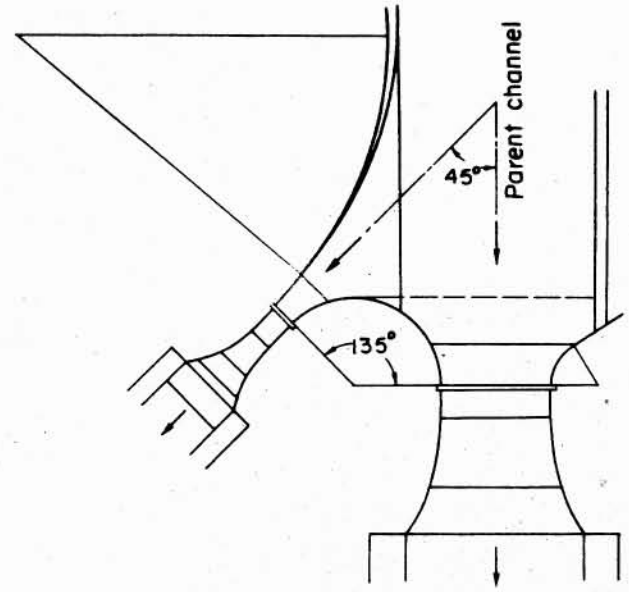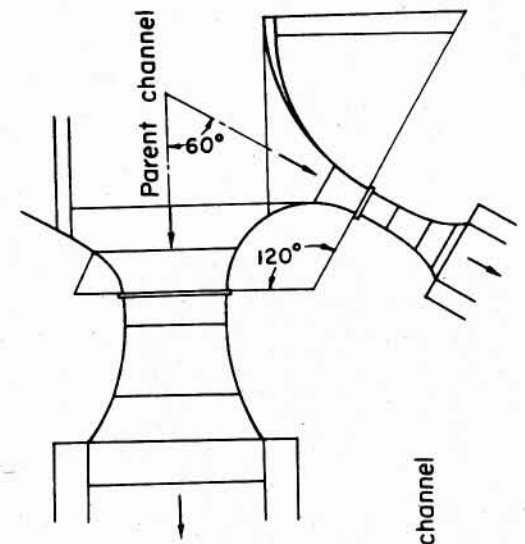
FAO — ICID

THREE EXAMPLES OF
PROPORTIONAL DISTRIBUTORS

Project , Region, Country
India and Pakistan

Figure No. 4 - 14

openings, which, however, are grouped to form a single structure. Each opening or offtake is constructed as a flume or free overfall weir and is dimensioned so as to pass a given fraction of the total flow. In other words, the controlling section (flume section, elevated floor, or weir crest) is not in the supply channel, as in the divisors discussed in section 4.2, but is in the individual offtakes. This arrangement requires accurate calibration by model tests or field rating and great accuracy in construction.

Representative of this category is the proportional distributor developed in the Punjab and Haryana (India) and described in 4.4.1 below. A structure of similar type and function is the "Nasba-Weir" which is in use in Egypt. The latter is described in Chapter 5 as it is considered basically an outlet structure.

As can be seen from the descriptions, these types of structures have comparatively high material and labour requirements, which make them uncompetitive with flow divisors especially for capacities below $1 m^3/s$. It should be noted further that they are also less efficient in drawing proportional silt load than flow divisors. Their distinct advantage is great robustness, which makes them suitable for areas where tampering is a problem.

4.4.1   Punjab-Type Proportional Distributor

Three examples of Punjab-Type proportional distributors are illustrated in Figure 4-14. These structures can distribute water supplies proportionally without regulation, including to a large extent sediment in the water. Example (a) (Figure 4-14) consists of a flume in the parent channel and a left-hand offtake set at 60°; example (b) has a flume in the parent channel and a right-hand offtake set at 45°; the third example (c) has a flume in the parent channel and two offtakes, one on the right set at 45° and the other on the left set at 60°.

Full details of construction, based on example (c), are given in Figure 4-15.

4.4.1.1   Parent channel flume

A point to note regarding the parent channel flume is that a single set of stop-log grooves should be provided on the upstream side for emergency

Data

Particulars	Parent channel		Right off-take at 45°	Left off-take at 60°
	Upstream	Downstream		
Q, m³/s	4·06	2·52	0·56	1·00
B, m	7·50	5·70	2·25	3·30
y, m	0·93	0·84	0·54	0·66
Full supply level	100·40	99·80	100·00	100·10
Bed level	99·47	98·96	99·46	99·44
Working head		0·60	0·40	0·30

Note: 1. All dimensions are in metres.
2. The sections of wing walls and abutments are standard sections as given in Figure 2 under Intakes of small canals (Punjab Type).

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PROPORTIONAL DISTRIBUTOR

(PUNJAB — INDIA)

Project, Region, Country
India

Figure No. 4 - 15

closures. These grooves should have a clearance of 0.15 m from the face of the upstream parapet of the bridge. Another point is that the segment of the arc forming the vertical walls in the included angles should be of radii varying from:

$$3.63\ H_{(crt)}^{1.5} \quad \text{to} \quad 7.26\ H_{(crt)}^{1.5}$$

where $H_{(crt)}$ is the head over the crest, (and this will depend on the throat widths).

When the fall in the supply channel is combined with a bridge, the minimum radii should be 2.55 m and 2.1 m for 60° and 45° offtakes respectively. This will ensure a 6 m radius for the centreline of the road leading from bridge to bridge. Both ends of the arcs of the walls must be tangential to the side walls of both the parent channel and flume and the offtakes.

The length of the approach must correspond to $3.15\ H_{(crt)}^{1.5}$, and the throat should start from the end of the circular walls.

4.4.1.2   Offtake flumes

All offtakes should be designed to take off at 60° or 45°.

The crest of all the offtakes and the flume in the parent channel should be at the same level, and at least 0.15 m above the downstream bed level of the highest channel. This means the fluming of the parent channel may have to vary from 60% to 75%; but such variations should not be made if they can be avoided. To ensure modularity, the submergence ratio should not be greater than 0.8.

The length of throat $(L_{(t)})$ will be $2H_{(crt)}$.

The upstream side approach should be joined with the bed in a curve and flared out to meet the side slopes at 1 : 1 or 0.5 : 1. The radii of these curves will depend on the angle of the offtake and the distance of the throat from the toe of the side slopes in the bed.

The depth of the cistern should be calculated from:

$$q_{(t)} = \frac{Q}{B_{(t)}}$$

where $q_{(t)}$ = discharge per metre of throat width,

Q = total discharge,

$B_{(t)}$ = width of throat.

The value of H_c is given by the formula

$$H_c = \left(\frac{q^2}{g}\right)^{\frac{1}{3}}$$

For a given working head, $h_{(wk)}$ (difference between the full supply level in the parent channel and the offtaking channel) calculate the value $\frac{h_{(wk)}}{H_c}$.

From Table 3-1 in Chapter 3, find out the value of $\frac{E_2}{H_c}$ for the calculated value of $\frac{h_{(wk)}}{H_c}$. Then E_2 can be worked out.

The depth of the cistern below the bed level of the offtake channel, H_{SB}, will be $E_2 - y_2$, and in no case should be less than 7.5 cm.

The length of the cistern should be $y_2 + h_{(wk)}$.

The depth of the curtain walls should be equal to $\left(\frac{y_1}{3} + 0.3\right)$ m and $\left(\frac{y_2}{3} + 0.3\right)$ m at the upstream and downstream ends of the masonry work respectively. The depth of the downstream curtain wall should be tested for maximum permissible safe value of the exit gradient as given below :-

Type of material	Safe value of exit gradient
Shingle	0.25 to 0.20
Coarse sand	0.20 to 0.17
Fine sand	0.17 to 0.14

For Punjab soils the safe value of the exit gradient is taken as 0.30.

To calculate the value of $\dfrac{L^*}{d_{wc_2}}$, where L^* is the length from curtain wall to curtain wall and d_{wc_2} is the depth of the downstream curtain wall, see diagram of Khosla's safe exit gradient curve below.
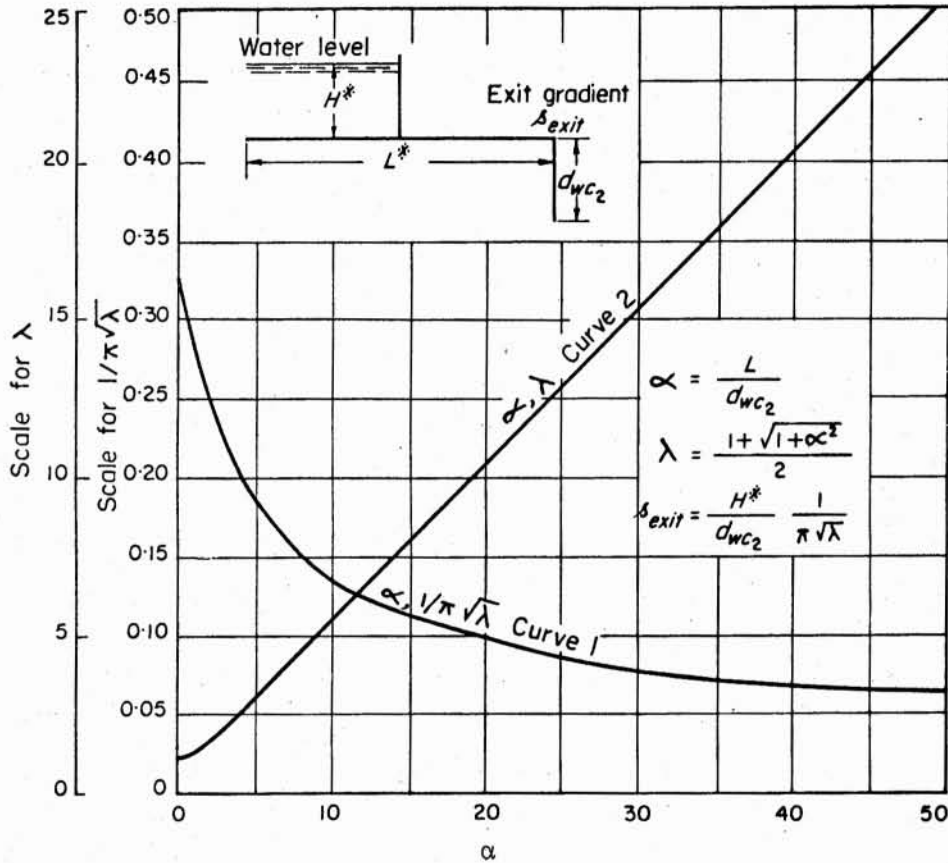
FIGURE 4-16. - Khosla's safe exit gradient curve.

For the calculated value of α, read the value of $\dfrac{1}{\pi\sqrt{\lambda}}$

Then, the exit gradient, $s_{(exit)} = \dfrac{H^*}{d_{wc_2}} \cdot \dfrac{1}{\pi\sqrt{\lambda}}$

The minimum thickness of the floors should be 0.12 m in brick masonry and 0.15 m in concrete.

There should be a single set of stop-log grooves on the upstream side.

A regulating bridge should be provided only when the flume is not combined with a road bridge and the throat width is greater than 0.6 m.

A radius curve of 0.6 m should join the crest with the downstream glacis. The glacis should have a slope of 2.5 : 1.

The length of the downstream expansion, $L_{(exp)}$, will be equal to:

$$\frac{3 (B_2 - B_{(t)})}{2}$$

or the length from the downstream edge of the crest to the end of the cistern, whichever is the greater.

The radius of the expansion walls, $R_{(exp)}$, will be equal to:

$$\frac{L_{(exp)}^2 + \frac{(B_2 - B_{(t)})^2}{2}}{B_2 - B_{(t)}}$$

where B_2 = bed width of the offtake channel,

$B_{(t)}$ = width of throat.

There is no bed protection in the parent channel other than a curtain wall on which the upstream curved glacis rests.

The side protection below the downstream expansion should be equal to $3L_{SB}$ (where L_{SB} = the length of the stilling basin or cistern) and should consist of dry brick pitching, 20 cm thick, supported on a toe wall of depth equal

to 0.5 y_2 (y_2 = water depth in the offtake channel) subject to a minimum 27 cm deep masonry wall over 15 cm thick concrete. It is preferable to lay roughened pitching. The bed protection should consist of brick-bats with thickness as follows:

up to 0.7 m^3/s	15 cm
0.7 to 1.4 m^3/s	23 cm

Thick brick-bat protection in the bed should extend up to a length equal to $y_2 + H_{(dr)}$ beyond the downstream end of the side expansion and should be hand-packed and not simply dumped.

Standard sections of wing walls and abutments should be as given in Figure 3-3 in Chapter 3.

4.4.1.3   <u>Design formula</u>

The design formula used is:

$$Q = CB_{(t)} H^{1.5}_{(crt)}$$

The value of C should be as follows:

Q m^3/s	Value of C	
	Angle of offtake 60°	Angle of offtake 45°
up to 0.56 m^3/s	1.60	1.61
0.57 to 1.4 m^3/s	1.61	1.63

For full details of the design see Figure 4-15.

4.4.2   <u>USBR-Type Proportional Division Structure</u>[1]

The structure described herein is also known as a division box. It has been standardized and is commonly used by the USBR for the division of flows of

[1] Based on information from the U.S.A. National ICID Committee and on the USBR design manual.

Plan

Section A-A

Section B-B

Detail of connection to concrete lining

Section C-C
Detail of precast concrete plank

Details of opening for adjustable weir

Detail A

Estimated quantities
Concrete _____ 4·1 cubic yards
Reinforcement steel _____ 340 lb

Notes
Extend cutoff and wingwalls vertically and horizontally with unreinforced concrete as directed.
Max. Q for 2 adjustable weir is 4 sec. feet
Max. Q for 3 adjustable weir is 6 sec. feet

Typical directions

FAO-ICID

DIVISION BOXES

Project, Region, Country
USBR (USA)

Figure No. 4-17

up to say 350 l/s into two or three take-off streams. Supply and/or take-off channels may be open or piped. A typical design for open channel flow is shown in Figure 4-17. Reference (107) provides standard designs for 9 other types of division boxes, including piped and open channels. The proportional division of a given flow is accomplished by weir structures at each offtake. The weirs may be of the fixed crest type to ensure a permanent proportion between the outlets or they may be fitted with slide gates or stop-logs for variation of flow proportions. It is often desirable to use the weir for water measurement, in which case its design must follow the standard weir conditions described in the USBR Water Measurement Manual, reference (81). In any case, it is recommended that a staff gauge be provided on the wall of the structure to measure the head on the weirs.

The water must be delivered to the structure at an elevation which will provide enough head for the weirs to furnish the required flow at the design delivery water surface. When the water is delivered through a pipe, the velocity in the pipe should not exceed 0.5 m/s to avoid excessive turbulence ahead of the weirs. This division structure is not recommended for systems with heavy silt loads.

4.5   DIVISION OR DIVERSION BOXES

Division structures or boxes regulate the flow from one canal to another, or to several others. They usually consist of a box with vertical walls in which controllable openings are provided. Metal or wooden slide gates or stop-logs are usually installed to regulate the division of flow at all times and to shut off flow in any branch when desired. The walls can be either of concrete (pre-cast or in situ), or of masonry or wood.

The width of each outlet is generally proportional to the division of water to be made. In lined canals, a full gate opening at the intake to the box is made covering approximately the same area as the canal flow section since the canal is designed to carry water at relatively high velocities. In earth canals, gate openings can be dimensioned by assuming a velocity of about 1.5 m/s in the opening section. Reinforced concrete transitions are provided below the gates

Scale

| 0 | 1 | 2 | 3 metres |

Plan

Section A-B
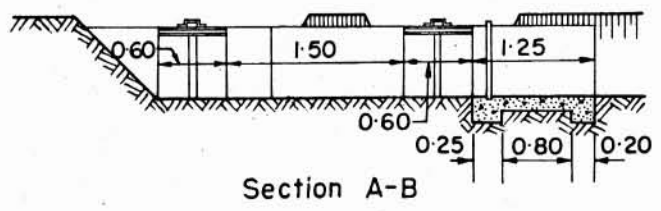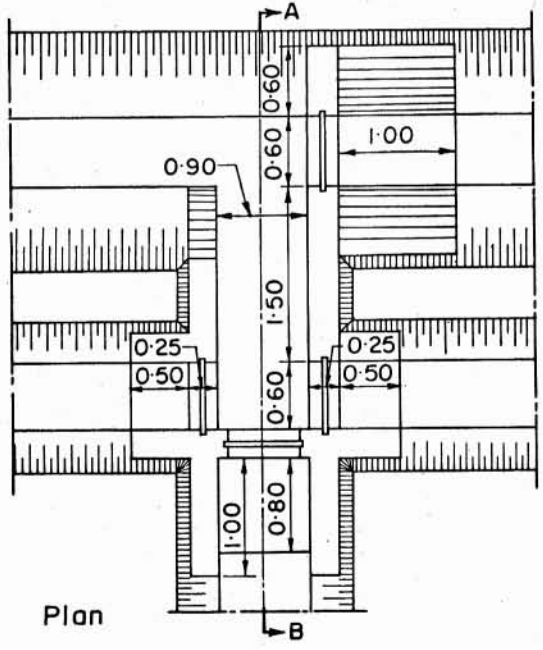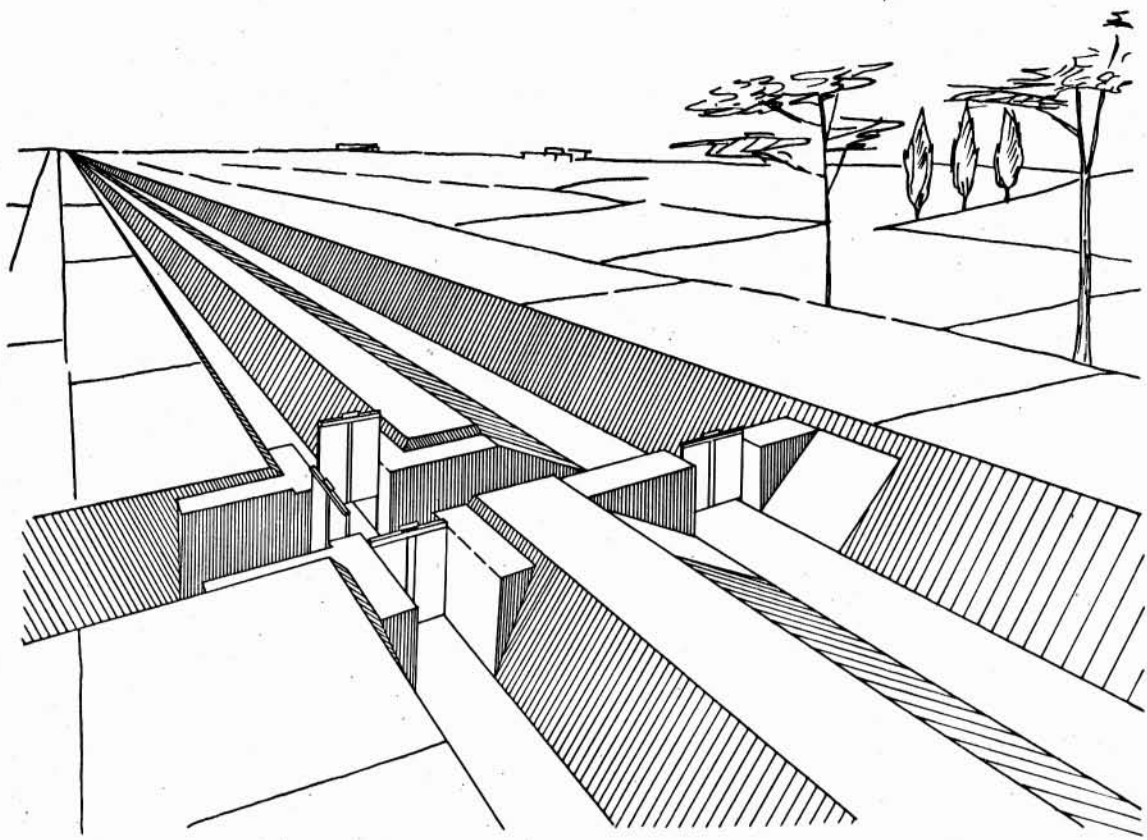
FAO — ICID

STANDARD DIVISION BOX

FOR EARTH CANALS

Project, Region, Country
Italy

Figure No. **4 - 18**

Plan

Isometric view

Sectional elevation A-A

Sectional elevation B-B

Sectional elevation C-C
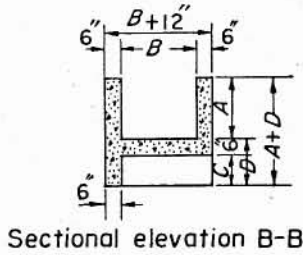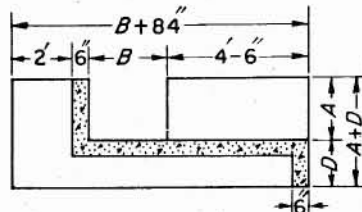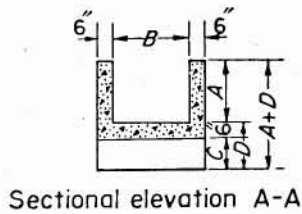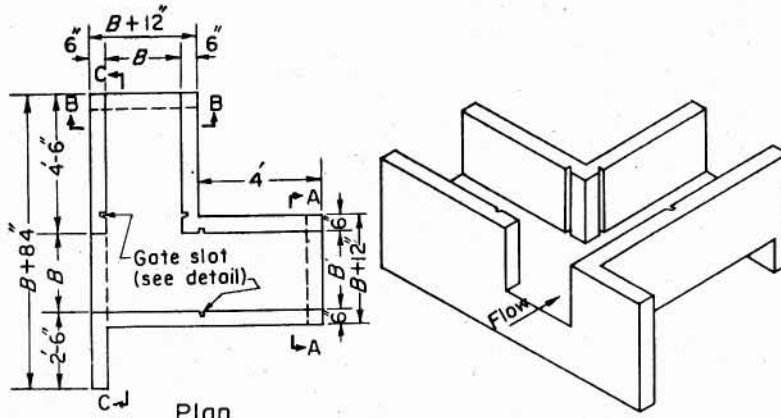
Table of dimensions and quantities

A	B	C	D	Conc. (cyd)	Steel L (feet)
2'-0"	2'-6"	1'-0"	1'-6"	1·87	2 24
2'-6"	3'-6"	1'-6"	2'-0"	2·46	2 76
2'-0"	4'-6"	1'-6"	2'-0"	2·91	3 24
3'-0"	2'-6"	1'-6"	2'-0"	2·45	2 85
3'-0"	3'-6"	1'-6"	2'-0"	2·89	3 32
3'-0"	4'-6"	1'-6"	2'-0"	3·37	3 82
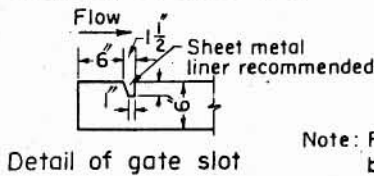
A = Height of side walls
B = Width of flow pattern
C = Depth of wall below slab
D = Depth of wall including slab

Note: Reinforcement to be $\frac{3}{8}$" dia. reinforcing bars on 12" cc placed in centre of concrete slab 6" x 6". No. 10 wire mesh may be used in place of $\frac{3}{8}$" dia. reinforcing bars.

Detail of gate slot

FIGURE 4-19. - Plan for a concrete rectangular division box - U.S.A.

on larger structures. Hydraulic losses through gate openings are seldom controlling factors in designing division boxes. When the gates are operated at full openings, entrance losses are simply transition losses. When operated at partial openings, available heads are not fully utilized so that increased losses due to gate contractions are not important.

4.5.1   In Situ Concrete Division Boxes

In situ concrete is the most commonly used material for permanent division boxes for any capacity from 10 l/s upwards. A standard type of concrete structure used in Italy since 1970 is shown in Figure 4-18. The canals have a capacity of 150 l/s each and are unlined. The structure is cast in place using concrete with 200 kg of cement per m^3 of aggregate. The surfaces are finished with cement plaster. The walls have a thickness of 25 cm while the bottom plate is 10 cm.

An example of another design, developed by the US Soil Conservation Service, is shown in Figure 4-19.

4.5.2   Prefabricated Concrete Division Boxes

The large number of division boxes of identical size required in medium-to-large projects makes them suitable for prefabrication.

Figure 4-20 (a) and (b) shows a standard division box constructed with prefabricated parts. This type is exclusively used in the East Ghor Extension Project in Jordan, where a total of some 225 structures have been installed.

Outlet openings are standardized to a full capacity of 30 l/s each, serving an irrigated area of 30 donums (3 ha). The structure permits quite flexible operation and safety against tampering.

Figure 4-21 shows a standard design adopted for a project in the San Francisco Valley (Brazil). The structure is composed of three types of prefabricated parts. These together with a variable gate opening can be used for a wide range of design flows. The parts are placed in an in situ concrete bed and their vertical sides are held together by steel clamps. The joints can be left unsealed if the backfill is impermeable material or can be sealed by plastic
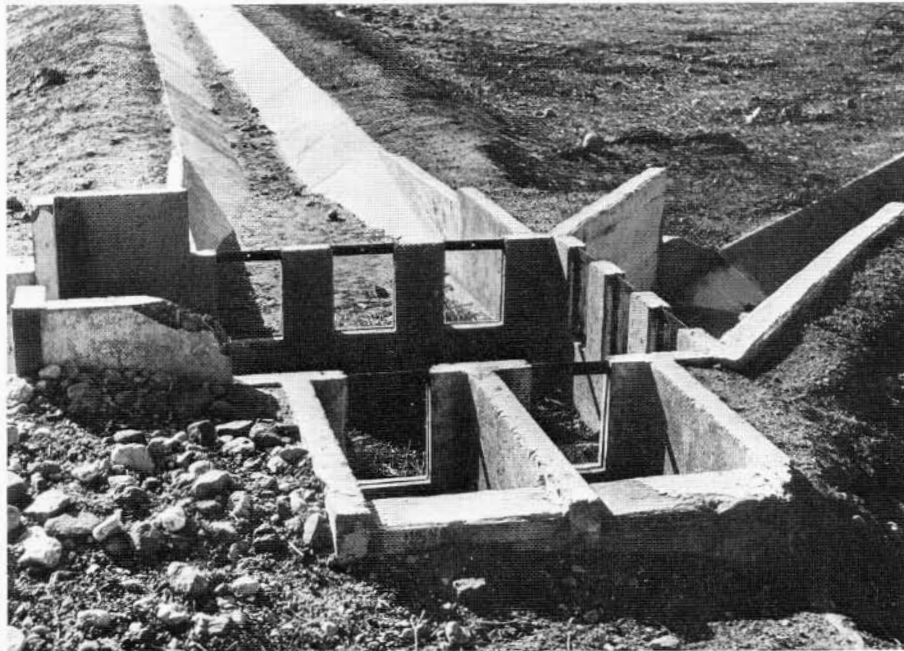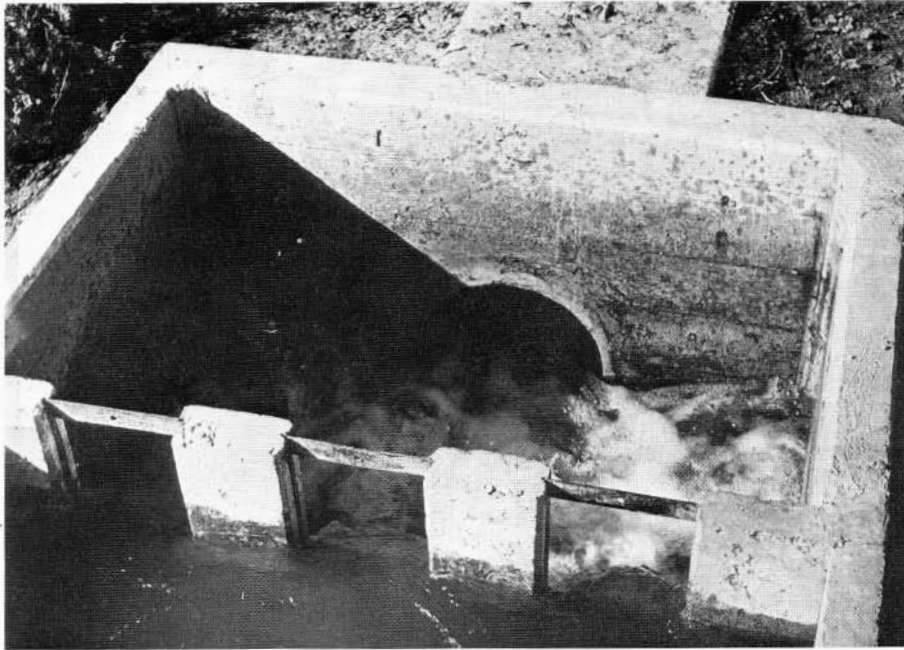
FIGURE 4-20 (a) and (b). - Distribution structure with proportional weirs constructed from prefabricated parts - East Ghor Extension Project, Jordan.

Section A-A

Section I-I

Element M₁

Detail of the junction

Detail L
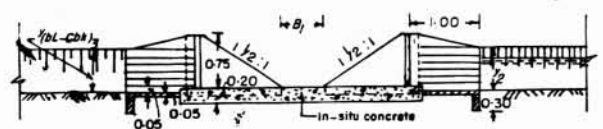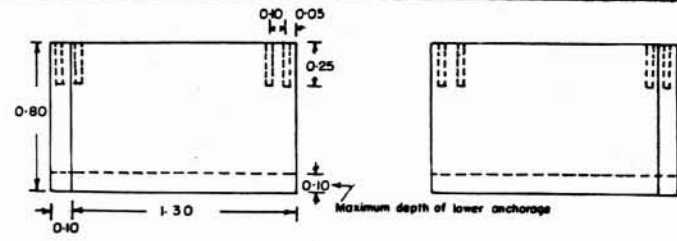
Volume of concrete

M₁ = 0·083 m³
M₂ = 0·051 m³
M₃ = 0·095 m³

Quantity of reinforcement = 2·400 kg/cm²

M₁	∅ 3/16 :	37 m.L =	5·7 kg
	∅ 3/P :	23 m.L =	14·5 kg
	∅ 1/8 :	6 m.L =	1·0 kg
M₂	∅ 3/16 :	23 m.L =	3·6 kg
	∅ 3/8 :	17 m.L =	10·5 kg
M₃	∅ 3/16 :	39 m.L =	6·0 kg
	∅ 3/8 :	23 m.L =	14·5 kg
	∅ 1/8 :	9 m.L =	1·4 kg
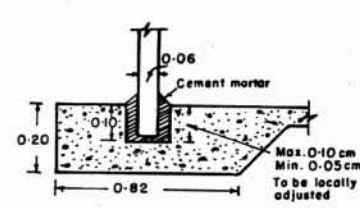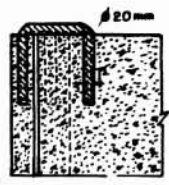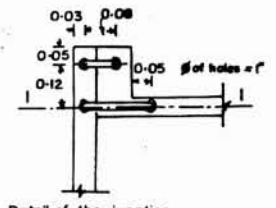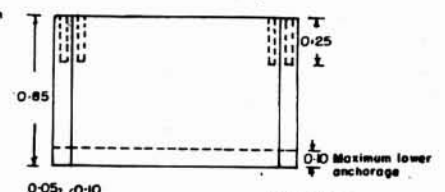
Element M₃
(Connection element)

Note :— horizontal reinforcement II ∅ 3/16"c coda lado
vertical reinforcement I ∅ 3/8" c/12 cm (estribo)

Wings vertical 6 ∅ 3/16
horizontal I ∅ 1/8 c/12 cm

Section C-C

Plan

Section B-B
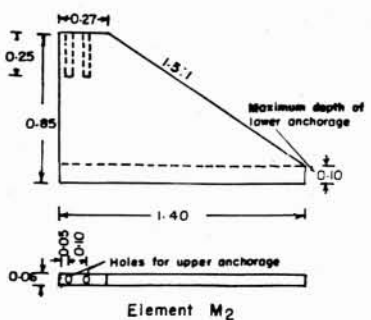
Element M₂

F A O — I C I D

STANDARD   DIVISION   STRUCTURE

USING   PREFABRICATED   ELEMENTS

Project: San Francisco River Basin
Country   Brazil

Figure No. 4 - 21

or rubber strips. For access to the gates, wooden planks are placed over the structure. (The fixing of the gate will be described in Volume III.)

The division box shown in Figures 4-22 and 4-23 serves to rotate the irrigation flow between farm ditches in a rotational irrigation system for paddy rice as practised in Taiwan. A farm ditch usually irrigates some 50 ha of paddy land.

The division box is constructed at the junction point where an irrigation sub-lateral branches into two or three farm ditches. The division box is not used to divide water carried in by the sub-lateral between farm ditches. It is used to turn the whole flow of water alternatively into one of the ditches according to a preset irrigation schedule.

There are three types of design in use. The first is a rigid concrete box; the second is a brick laid box; the third is the design shown, a box made of pre-fabricated concrete parts. In fact, the first type is more extensively used in Taiwan. However, the third type is more economical, is easy to construct and is durable. It is therefore recommended for wider application.

Viewed on the plan, the division box is a rectangle with four sides open for inserting flashboards (see Figure 4-23). In the two diagonal directions of the rectangle, four baffle walls made of prefabricated concrete are placed with the head enlarged to accommodate flashboard grooves and with the tail portion projected into earth ditch banks for stabilization. The lower portion of the walls, fastened together by connecting plates, is buried underneath the bottom of the ditches as cut-offs.

The earth ditch banks into which the baffle walls of this structure are projected for stabilization may deteriorate after some time. This will endanger the stability of the structure so the earthen banks should be examined from time to time and be repaired if necessary.

Hydraulically, this division box is a very simple structure, since there is no actual division of irrigation flow involved. The whole flow coming into the box goes out of one of the openings into the farm ditch required during rotational irrigation. The head loss for a flow through the box is usually no

more than 3 cm.   It is not recommended that this type of division box be used if
there is a heavy silt load in the irrigation water.

4.5.3   Masonry Division Boxes

An example of a design of division box made of concrete blocks is shown in
Figure 4-24.

The structure should be placed at least 30 cm below the surface of the
ground on a 10 to 15 cm concrete foundation - the floor of the structure.   The
first course of 20 cm x 20 cm x 40 cm block can be laid in a bed of mortar on top
of the foundation or be placed in the fresh concrete of the foundation.   In either
case, care should be taken to level the blocks.

Retaining plate
Stoplog
Groove
Partition wall

Plan of partition wall
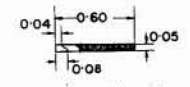
Plan-section of cutoff plate

Plan-section of retaining plate

Perspective drawing of precast division box
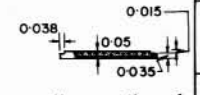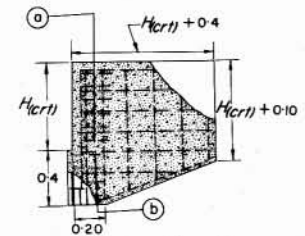
Vertical section of partition wall
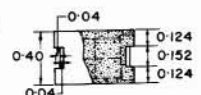
Vertical section of cutoff plate
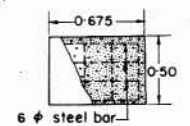
Vertical section of retaining plate

6 φ steel bar

The thickness of washer is 3mm and the outside diameter is 3 centimetres
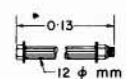
Set-in screw
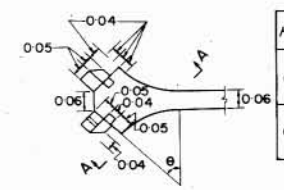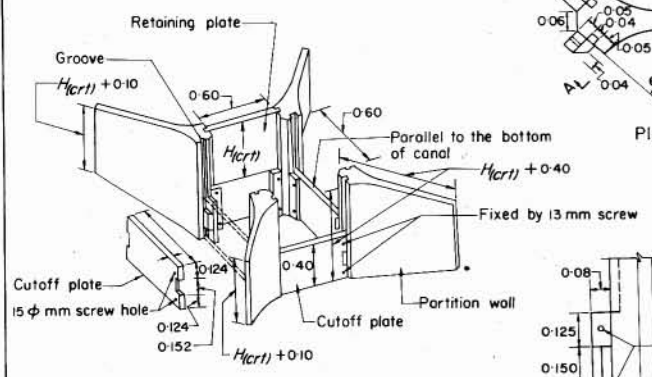
Construction of precast division box

Retaining plate
Groove
Parallel to the bottom of canal
Fixed by 13 mm screw
Cutoff plate
15 φ mm screw hole
Cutoff plate
Partition wall
Partition wall

Plan

A-A
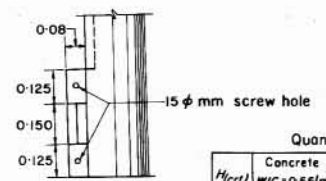
Details of groove connection

15 φ mm screw hole

Quantities required for each partition wall

H(crt)	Concrete W/C=0.66(m³)	Form work (m²)	φ13mm×13cm No. of bolts	Steel bar φ6mm												Total wt.
				Type	I	I	I	I	I	I	—	—	⌐	⌐	I	
0.45	0.056	0.934	4	Length(m)	0.830	0.760	0.690	0.620	0.550	0.430	0.830	0.620	0.870	0.250	0.380	2.823
				Quantities	2	1	1	1	2	4	1	1	8	2		
				Weight(kg)	0.369	0.169	0.153	0.138	0.122	0.191	0.737	0.138	0.193	0.444	0.169	
0.65	0.078	1.113	4	Length(m)	1.030	0.960	0.890	0.820	0.750	0.630	1.030	0.820	1.080	0.250	0.380	3.786
				Quantities	2	1	1	1	1	2	5	1	1	10	2	
				Weight(kg)	0.457	0.213	0.198	0.182	0.167	0.280	1.143	0.182	0.240	0.555	0.169	

Table of division box

Types	Canal B(m)	Canal y(m)	Division box H(crt)(m)	Concrete W/C=0.66 (m²)	Form work (m²)	φ13mm×13cm No. of bolts	Total wt. of steel bar X1-1 (kg)	Cement mortar 1:3 (m³)	Riprap φ5cm thick 10 cm	Cement (pack)
D×1A	0.30	0.40	0.45	0.29	5.41	16	16.63	0.0025	0.06	1.75
	0.30	0.50								
D×2A	0.30	0.60	0.65	0.39	6.30	16	21.21	0.0025	0.06	2.34
	0.40	0.70								
D×1B	0.30	0.40	0.45	0.28	5.00	16	15.57	0.0030	0.06	1.67
	0.30	0.50								
D×2B	0.30	0.60	0.65	0.36	5.72	16	19.80	0.0030	0.06	2.14
	0.40	0.70								

Quantities required for each retaining wall

H(crt)	Concrete W/C=0.66(m²)	Form work (m²)	Steel bar φ6mm			Total weight(kg)
			Types			
0.45	0.015	0.418	Length(m)	0.43	0.63	0.969
			Quantities	4	4	
			Weight (kg)	0.382	0.059	
0.65	0.021	0.572	Length(m)	0.65	0.65	1.281
			Quantities	4	5	
			Weight (kg)	0.577	0.722	

Quantities required for each cutoff wall

H(crt)	Concrete W/C=0.66(m²)	Form work (m²)	Steel bar φ6mm					Total weight(kg)
			Types	⌐	I	—	—	
0.40	0.013	0.318	Length(m)	0.56	0.38	0.58	0.56	0.715
			Quantities	2	1	2	1	
			Weight(kg)	0.249	0.084	0.258	0.124	

Notes:

(1) The sizes on the drawing are all measured by the metre except for extra explanations.

(2) The height of retaining plate is determined by designer according to the hydraulic and topographical conditions.

(3) Stoplog details not shown in the drawing.

(4) Precast division box consists of partition wall, retaining plate and cutoff plate. Every part can be casted in iron model with reinforced concrete and then assembled in place.

(5) The transition is made by earthwork, 200 m long from the out-ridge of cutoff wall.

(6) In the table, A represents two-way division and B represents three-way division.

(7) The thickness of riprap (diameter more than 5 cm) below the bottom of the division box should not less than 10 cm.

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PREFABRICATED DIVISION BOXES
DIMENSIONS

Project, Region, Country

Taiwan (Republic of China)

Figure No. 4 - 23
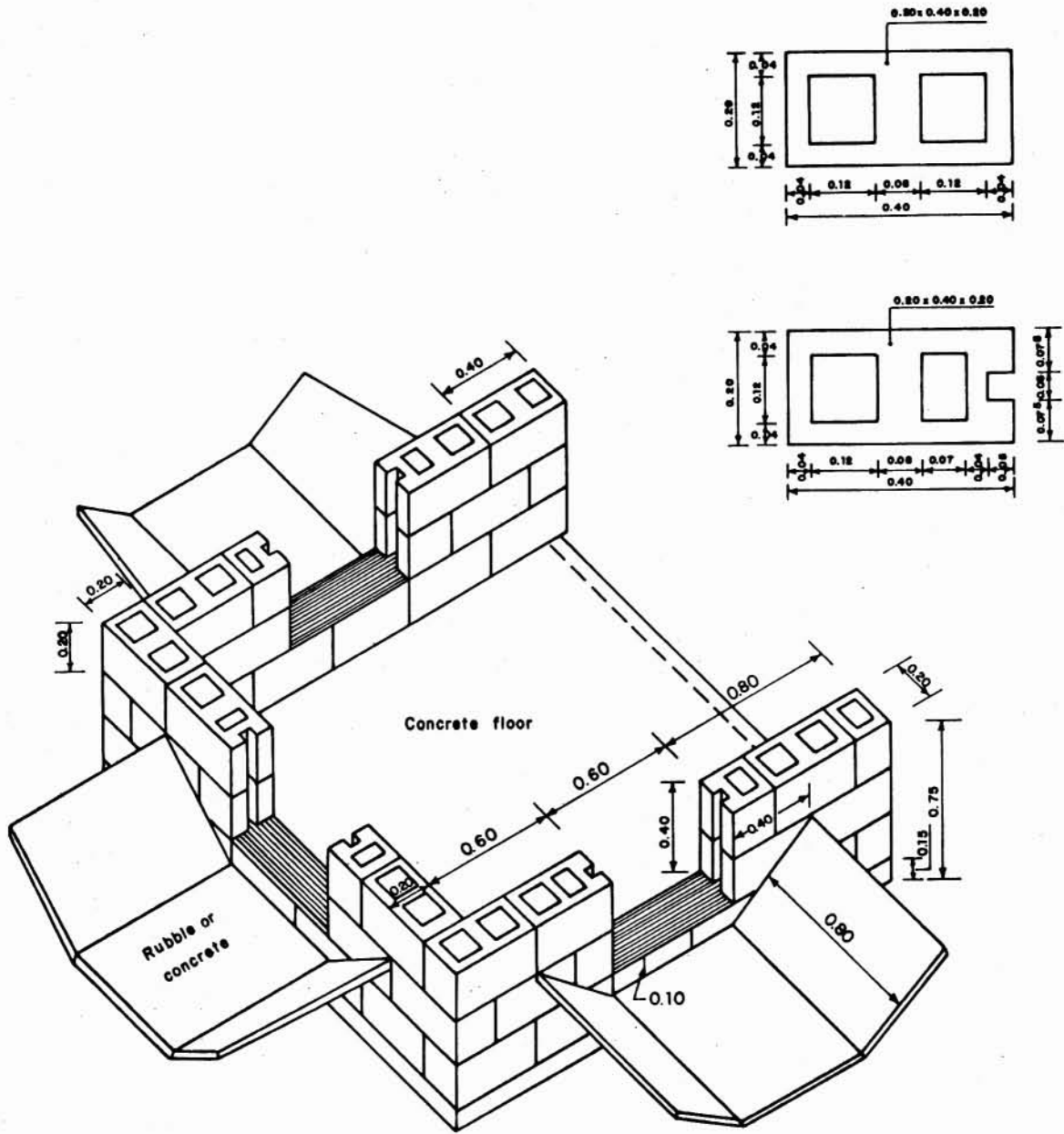
FIGURE 4-24. - Concrete block division box.

The sizes of the openings gauge the amount of water flowing through the structure. Although dimensions of the openings are shown in the drawings, adjustments may be necessary, depending on the grade of the canal or ditch and the amount of water to be handled. The water flow is controlled by removable gates made of 2-inch thick timber.

When the blocks have been laid, a coping should be placed on the top course and on the crest of the openings. This can be done with a coping block or the cores of the block can be filled with concrete.

The ditch invert downstream of the box openings should be protected by rubble or concrete lining for a length of at least 80 cm.

Figure 4-25 shows another example of a design using concrete blocks and wooden gates.
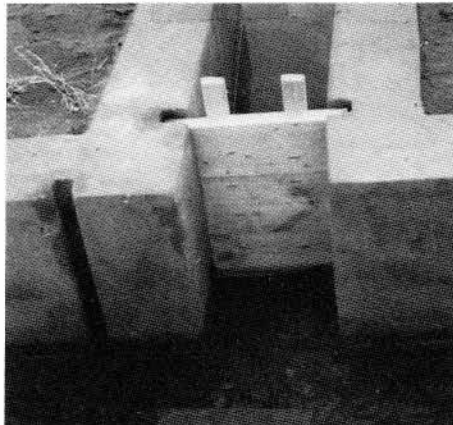
FIGURE 4-25. - Concrete block division box - People's Democratic Republic of Yemen.

4.5.4 Timber Division Boxes

Wooden division boxes are used in farm ditch systems in areas where the construction material, which can be scrap timber, is cheap. Easy and cheap construction and transport has to be weighed against the short life expectancy of the structure. The maximum discharge for wooden structures does normally not exceed 200 l/s. Figures 4-26 to 4-28 show three examples of suitable designs for these timber division boxes.
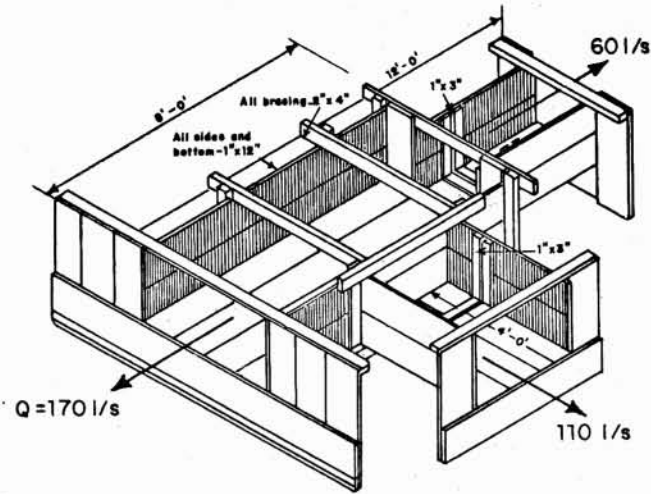
FIGURE 4-26. - Typical timber division box (13)

BILL OF MATERIAL		
SIZE	L-F	B-F
1"x 4"	15'-4"	5.0
1"x 4"	39'-4"	26.2
2"x 6"	34'-0"	34.0
2"x12"	70'-0"	140.0
Total		205.2
Nails	16-d	3.5 lbs
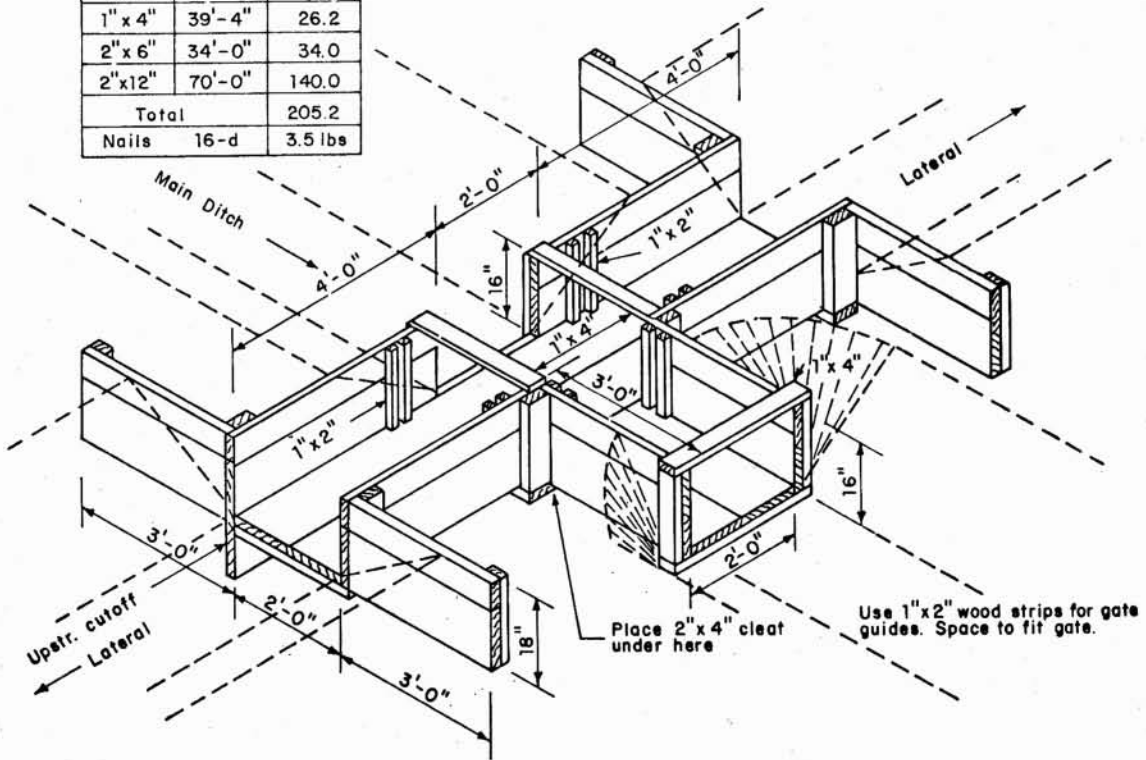
FIGURE 4-27. - Three way timber division box - standard design USDA Soil Conservation Service. (Note: 1' = 30.5 cm; 1" = 2.5 cm)
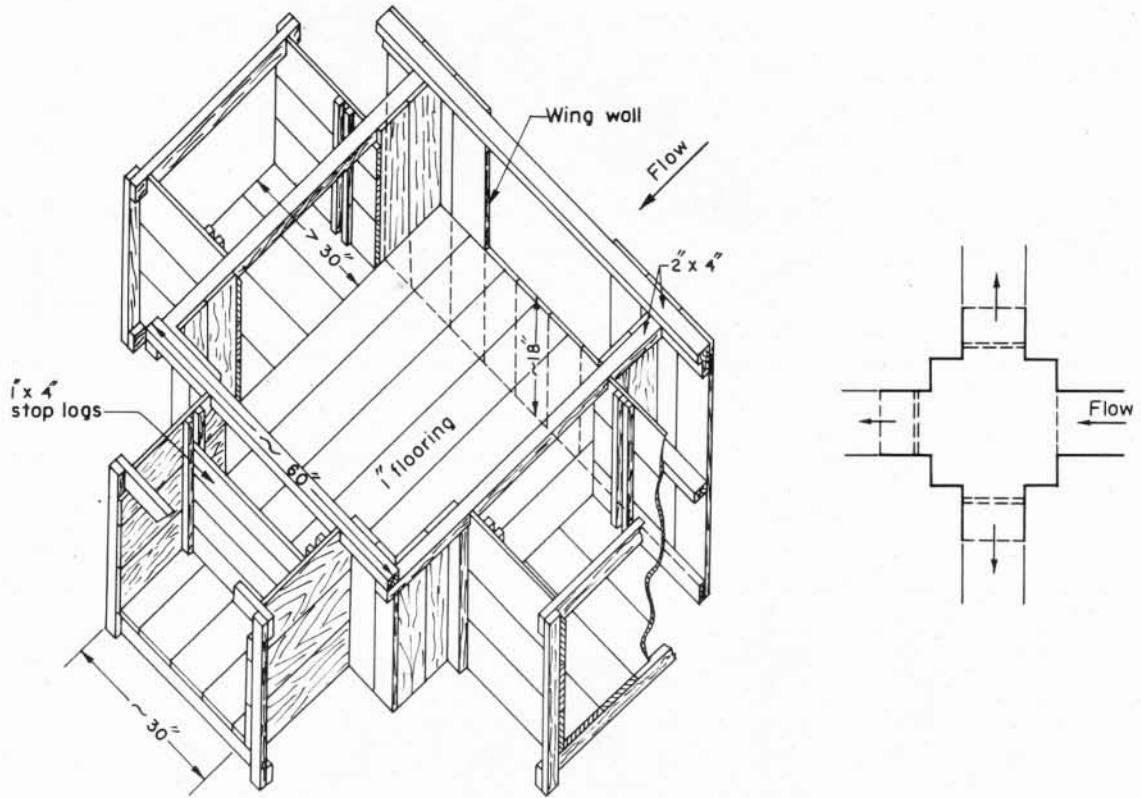
FIGURE 4-28. - Standard timber division box - Alberta Department of Agriculture, Canada.

4.5.5   Irrigation Diverter

The irrigation diverter is an automatic hydraulic device developed by Freeman (37) to divert flows from an irrigation canal to a branch canal or field (see Figure 4-29).
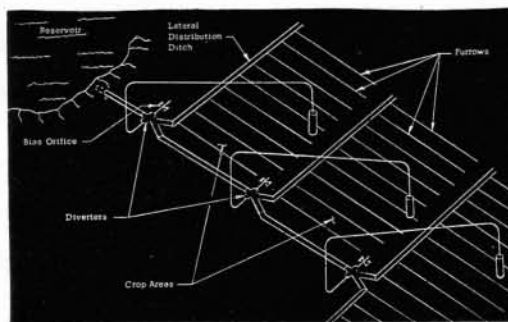
FIGURE 4-29. - Layout of furrow irrigation using an automatic irrigation diverter.

(a)
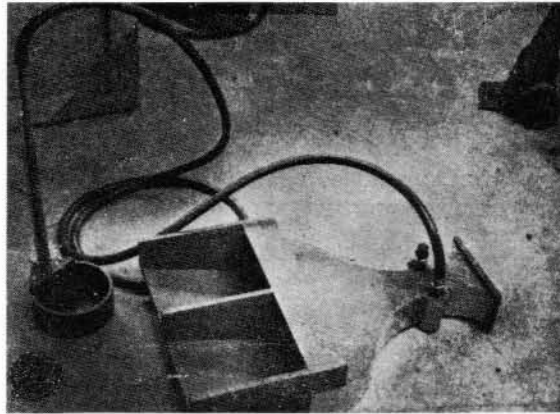
(b)
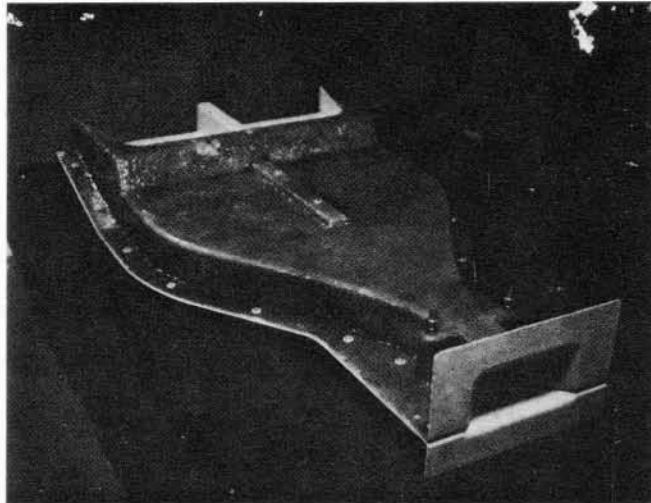
FIGURE 4-30 (a) and (b). - General views of irrigation
diverters (37)

General views of diverters are shown in Figure 4-30 (a) and (b). The unit
resembles a hollow "Y", with water entering into the stem of the Y and discharg-
ing from both of the two branches. One branch supplies the field to be irrigated
or field ditch and the other the downslope supply ditch to the next diverter. The
floor of the device is inclined upwards towards the downstream end. This has
been found necessary in order to divert flow into one outlet without leakage into

Plan

Dividing wall or bank design depending on local conditions

Control port open

Control port closed

$B_1 = 3B_{(t)}$

$B_{(t)}$

$4B_{(t)}$

Top of bank

Vertical slope 1:1

$2B_{(t)}$

Bed of downstream channel

Bed of downstream channel

$2B_{(t)}$

0.5:1

0.5:1

Water flow lines

Section on EF

$3B_{(t)}$

Water surface level

Top of bank

Section on A-B-C

Head wall

Control ports

Top of bank

Water level

Operating head

$H_{(crt)}$

$1.5B_{(t)}$

$2B_{(t)}$

$B_{(t)}$

$1.5B_{(t)}$

$2B_{(t)}$

$1.5B_{(t)}$

$1.5B_{(t)}$

$1.5B_{(t)}$

$B_{(t)}$

$10B_{(t)}$

Irrigation flow into downstream supply ditch

Bank level

Downstream water level

F A O – I C I D

DIVERTER INSTALLATION

Project, Region, Country

U S A

Figure No. 4 - 31

the other outlet.   Two control vents are located on top of the device.   In operation one vent is open and the other is closed.   If the right vent is open, the water flows to the left branch and vice versa.

The functioning of the vents depends on the suction of the stream in the venturi-type inlet.   The device can be manually-operated by plugging one vent at a time with a simple rubber or plastic plug.   The device can also be automated at very little extra cost.   A "dip tube" level sensor is set into the distribution canal so that the bottom of the sensor is at the desired water level in the canal. (See Figures 4-29 and 4-30.)

The dip tube consists of a vertical length of light pipe connected by a light hose to the supply canal control vent.   The vent on the turnout side is restricted by a small orifice.   When the diverter starts flowing much more air enters through the dip tube sensor hose and into the control vent on the supply canal side than through the small orifice.   The water flow is thus directed into the distribution canal until the end of the dip tube sensor is submerged.   This stops the air flow into the supply side control vent, and the air flow through the orifice then switches the discharge into the downstream supply canal to the next diverter in the line, where the sequence is repeated.   In many applications it may be necessary to add a bowl under the dip tube sensor (Figure 4-28 (a)) in order to prevent retriggering of flow into the same turnout before all other turnouts have been serviced.

The device is still in an experimental stage.   Field tests carried out on sugar cane plantations in Hawaii indicate that it provides improvements over the traditional methods of surface irrigation on sloping lands.

5. OUTLETS OR FARM TURNOUTS

5.1 INTRODUCTION

An outlet or a farm turnout is a structure at the head of a watercourse, a farm irrigation canal, or a farm or field lateral, which connects it with a supply canal. The supply (or distribution) canal is usually under the control of an irrigation authority. The authority may be a Government department, a public, or semi-public organization such as a district or an irrigation association. Thus the farm outlet is the connecting link between the canal operator representing the authority and the farmer or user. It follows that the design and operation of the outlet must satisfy the needs of both parties as far as possible. For example, the farmer will want to be satisfied that he receives his due share of the water at the right time and the operator will want to be satisfied that the outlet structures exercise effective control over the distribution of the supplies.

There are more farm outlets than other structures in an irrigation system and therefore they have a decisive influence on the functioning of a system. (In the U.S.A. in 1960 there were some 160,000 farm outlets on irrigation systems. The number of outlets on irrigation systems in a single Province in India (the Punjab) in 1947 was over 41,000. In the U.S.S.R. at present more than half of the total number of structures on irrigation systems are outlets.)

5.1.1 Factors Influencing the Design of Farm Outlets

5.1.1.1 Quantity of water to be delivered

In the design of irrigation distribution systems, the basic factor to be considered is the quantity of water to be delivered at the farm outlet. (This quantity is based on the area of land to be irrigated and a number of other factors, inter alia, the type of soil, water requirements of the crops, effective rainfall, depth to the water table, et cetera.) The design of the outlets will also depend on the method of water delivery adopted (see 5.1.1.3).

5.1.1.2   Sources and nature of water

The main sources of water for irrigation are: rivers, dams, wells, and natural springs.   Water from the three latter sources is generally free of silt and in these cases the farm outlets can be designed for proportional distribution if required (e.g. when the supply in the distributing canal fluctuates). Water from rivers is usually charged with sediment, and this has to be taken into account in the design of the outlets so that the passage of silt is restricted, but at the same time silting up of the downstream reach of the river is avoided.

5.1.1.3   Methods of water delivery

There are three methods for delivering water to farms, known as continuous, rotational and demand respectively.

In the continuous method each farm receives its rightfull share of the supply in relation to the area of each holding in an uninterrupted flow.   That is, water is always available, although it may not always be necessary to use it; and so its actual use may be, and often is, intermittent.   The method and canal system is easy to operate but disadvantages are the tendency to waste time and water, particularly in sandy soils due to percolation, and the possibility of waterlogging.   Where farms or farm units vary greatly in size, the size of the outlets may have to vary from very small to very large structures, thus making it difficult if not impossible to standardize them.

With the rotational method water is in effect moved from one area to another en bloc, and each user receives a fixed amount of water at defined intervals of time.   This kind of rotation can be applied between just two or more farmers, between two or more groups of farmers, or between defined portions of an entire irrigation scheme.   By using large heads water can be moved rapidly over the surface of the ground, thus minimizing losses through percolation and promoting good irrigation efficiencies.

When water supplies are adequate there is no particular problem, but when the supply is insufficient to meet the full demand, the water available has to be distributed according to the principle of equitable sharing (see Chapter 2 - 2.2.1).   A farm outlet may serve just one farmer or a group.   In some countries,

e.g. India and Pakistan, the entire discharge from one outlet is taken by different farmers in turn, the duration of the turn being fixed in proportion to the irrigable area in each case. Under these conditions a more standardized form of outlet can be employed.

The demand delivery method is particularly favoured by farmers because the water is delivered at the farm outlets in the quantity and at the time requested. It permits a farmer to draw any flow of water he may desire, limited only by the capacity of the canal system. At the same time it encourages economical use when water charges are based on the volume delivered. On the other hand it is not economically feasible to construct a canal system of sufficient capacity to satisfy the full demands of all farmers at the same time, and the method is not practicable either in the case of large canals drawing their supplies from uncontrolled river sources. The extent to which farm outlets can be standardized for use in the demand method is only limited.

In many cases, practical considerations call for the use of a combination of two or all three of the delivery methods within the same irrigation project at various times or in various locations. For example, during periods of high seasonal runoff, the main canals can run at full capacity and supply farmers with water (if they require it) by the continuous flow method. At other times, where reserves of water in storage are being drawn, or where the supply in the rivers is insufficient for the demand, the project can be switched to rotational delivery.

5.1.1.4   <u>Methods of assessing water charges</u>

In some countries (e.g. Arab Republic of Egypt, Sri Lanka, Thailand) farmers are not charged for the irrigation water they receive, but in most other countries they have to pay for it. However, where charges are made, the systems of assessing them vary in various parts of the world.

There are four main methods of assessing water charges or water rates for irrigation water based on:

(i)   rate of flow, which entails metering the rate of flow and maintaining the necessary records;

(ii)   volume of water delivered, entailing a volumetric measuring

device or a combined rate of flow and time measuring device;

(iii)   area, and type of, crops irrigated; control and record-keeping is necessary;

(iv)   each irrigation over a given area, e.g. where irrigation is only supplementary, or where the same crop is grown over large areas year after year, such as rice for example; again control and adequate records are necessary.

In the case of methods (i) and (ii) the farm outlets have to be accurate measuring devices. In the case of methods (iii) and (iv) the measurement of discharge is not essential, but the farm outlets should be designed to ensure equitable distribution of water.

5.1.1.5   Operation of the canal system

When water supplies are plentiful throughout the crop season the entire canal system runs at full capacity; under these conditions the design of farm outlets is a simple matter. When the supplies are insufficient to meet the demand their distribution has to be controlled and rationed out. This can be done in the various ways described below.

The various distributing canals can be run continuously carrying their share of the water supplies available. In this case the farm outlets in the canals will have to either (a) draw whatever water is available or (b) run in rotation. In case (a) the outlets should be able to draw a proportion of the discharge. However if the supply is laden with silt, the outlets will not draw their fair share of it and the problem of silting at the head reaches of the distributing canal may arise. In case (b) the outlet needs a check structure in the supply canal to feed it. Furthermore each outlet must be fitted with a manually operated shutting device. There may also be a silting problem in the distributing canal due to heading up of water.

5.1.2   Desiderata for the Design of a Farm Outlet

As far back as 1906, Kennedy set forth desiderata for the efficient design of a farm outlet, in Punjab Irrigation Paper No. 12, quoted below.

"(a)   To keep the discharge automatically constant as adjusted, and indicated, however much (within working limits) the water levels may vary in the distributary channel, or in the watercourse, or in both at once.

(b)   To allow of slight variations in the discharges as adjusted, so as to avoid the need of constantly removing and replacing the outlet, whenever the discharge must be somewhat altered.

(c)   To work with high 'heads' as well as low - down to three inches or so.

(d)   To be free from derangement by silt or weeds.

(e)   To be light, portable, easily removed and replaced elsewhere.

(f)   To be cheap and durable, with no complicated mechanism.

(g)   To be all closed in and immune from outside interference or derangement in working.

(h)   To be capable of being opened or closed off entirely by the cultivators from outside.

(i)   To indicate from outside when the working head is insufficient to give the full discharge, and therefore also the necessity for clearance of the watercourse.

(j)   If so desired and adjusted, to work as a module, only within certain limits of level in the feeder, above and below these limits to give proportionately increased or decreased discharges. (This is with special reference to farmers' canals, where each man is entitled to a proportion of the whole available supply.)

(k)   Floods[1] in the distributary to be passed off by increased discharges through the outlets, so as to avoid damage.

(l)   When the distributary supply is very low and inadequate, it will be more or less proportionally distributed to all outlets, those with very high command not being allowed to draw off all the water there is.

(m)   Discharges to be provided for may be anything between half and four cusecs[2] with possible duplication above the latter figures. "

Of course it is not possible to satisfy all the conditions enumerated by Kennedy in any one type of farm outlet, even in India, for which the desiderata

_____

[1] i.e. excess water.   [2] cubic feet per second.

were propounded.   However, it is highly desirable that every outlet be strong and
so designed that it cannot easily be tampered with.   The cost of construction
should be low, using local materials whenever available, and the aim should be to
standardize as much as possible.   In areas of only little slope the farm outlet
should work efficiently with a small working head, (because the greater the loss of
head the higher the water level required in the supply canal for command and,
consequently, the higher the cost of the entire distribution system).   Where
supplies are delivered on a volumetric basis, the outlet should have metering
facilities and should preferably include a recording device.   Finally, where
supplies are charged with sediment, the farm outlet should draw its fair share of
the silt, without being liable to blockage by silt or weeds.

Throughout the irrigated areas of the world engineers, farmers and others
have invented or designed various kinds of farm outlets for particular conditions.
Some of the ideas never came to fruition but a large number of them have proved
satisfactory and have stood the test of time.   For example the Adjustable Orifice
Semi-Module, developed in the Punjab before 1947 and the Jamrao Type Orifice
Semi-Module, developed in Sind even though developed independently, are inter-
changeable.   Thus, abundant experience has been accumulated on which to base
the choice of the best type of outlet to suit local conditions.   However, this does
not preclude the possibility of improving existing outlets or evolving new types
which may be superior structurally, hydraulically and economically.   In fact a
great deal of research remains to be done on this subject.

5.1.3   Classification and Selection of Farm Outlets

18 types of farm outlets are covered in this chapter as listed below.
In addition, examples of simple outlets for use at the farm level are described
in the last Section.   The reference number allocated to each structure corresponds
to the Section number in the text.   The first three structures listed (5.2, 5.3 and
5.4) are described in Chapter 3 - Intake Structures.

5.2   Constant-Head Orifice Farm Turnout (U.S.A.)

5.3   Orifice Module (France)

5.4   Double Orifice Module or Syphon Module Outlet (France)

5.5   Dethridge Meter (Australia)

5.6	Plastic Syphon Outlet Fitted With an Intake Tube (Turkey)
5.7	Open Flume Farm Outlet (India and Pakistan)
5.8	Adjustable Orifice Semi- Module (India and Pakistan)
5.9	Jamrao Type Orifice Semi-Module (Sind, Pakistan)
5.10	Pipe Semi-Module (India and Pakistan)
5.11	Fayoum Standard Weir Farm Outlet (Arab Republic of Egypt)
5.12	Scratchley Outlet (India and Pakistan)
5.13	Pipe Outlet (India and Pakistan)
5.14	Farm Outlet to a Temporary Feed Ditch (U.S.S.R.)
5.15	Pre-Cast Farm Turnout (Turkey)
5.16	Adjustable Weir Farm Outlet (Malaysia)
5.17	PVC Pipe Turnout (Republic of Korea)
5.18	Pipe Outlet with Standard Inlet (Philippines)
5.19	Gated Pipe Outlet (Ferrara Type)
5.20	Outlet Structures on the Farm

Outlets may be divided broadly into the following three classes:

A   (i)   Modules or modular farm outlets - 5.2, 5.3 and 5.4

    (ii)   Totalizer type meter farm outlets - 5.5

B   Semi-modules - 5.6, 5.7, 5.8, 5.9, 5.10, 5.17 and also

               5.11, 5.12, 5.13, 5.14, 5.16 and 5.18

               when they have a free fall.

C   Non-modular farm outlets - 5.12 and 5.15, 5.13, 5.14, 5.16,

               5.18 and 5.19 under submerged conditions.

The advantages, disadvantages and limitations of each of the outlets listed are discussed in detail under the relevant Section headings. General guidelines on selection of classes and types are given below.

5.1.3.1   Selection of classes of farm outlets

A. Modules

In a module outlet the discharge is, within reasonable working limits, independent of the water level in the supply canal and the watercourse or field lateral. This class of outlet may be regarded as the best type of farm outlet from the farmer's viewpoint. However, modules cannot absorb fluctuations of water

supplies in the parent canal and, therefore, the parent canal could either flood or become dry in the tail reach. Thus, modules should be limited to: branch canals or distributaries and minors in which the supply varies only within predetermined limits; outlets located above control points where water levels can be maintained; canals in which additional water is delivered to certain selected outlets for leaching or for other purposes.

When water is supplied on a volumetric basis modules are ideal. Under any of the cases mentioned above constant-head orifice turnouts may be found adequate, whether the water supply is charged with sediment or not. Other suitable types may be Parshall flume outlets and meter-gates. When the water supply is silt free the constant-head orifice turnout, the Neyrpic orifice module, the double orifice module, and the Dethridge meter may be employed.

In cases where the water is not being supplied on a volumetric basis, but may be at a later date, it may be convenient to resort to pre-cast farm outlets (5.15) which can later be converted into constant-head orifice turnouts.

B. Semi-modules

The discharge of a semi-module outlet is independent of the water levels in the watercourse or field lateral, but dependent on the water levels in the supply canal, so long as a minimum working head is available for the device. These types of modules are not useful for supplying water to farmers on a volumetric basis unless they be accompanied by an auxiliary device, such as a notch weir, a venturi flume, a Parshall flume or an open flow-meter attachment on the downstream side. The usual use of semi-modules is to distribute, more or less equitably, upstream variations in the supply canal within their range of operation. The plastic syphon outlet (5.6) fitted with an intake tube can be used advantageously in small canals. The free-fall outlet to a temporary feed ditch (5.14), the adjustable weir farm outlet (5.16), the PVC pipe turnout (5.17) and the pipe outlet with standard inlets (5.18), may all be used where a shut-off gate is included in the outlet.

When the water supply to the outlets is free of silt and a shut-off gate is not necessary, the following outlets are open to choice.

(i)   Open flume outlets - at tail clusters, and in tail reaches with

setting of the crest at $0.9 y_1$ for proportional discharge.

(ii)   Adjustable orifice semi-module - in head reaches with setting of the crest at $0.6 y_1$ for proportional discharge.

(iii)   Jamrao type orifice semi-module - in head reaches with setting of the crest at $0.96 y_1$ for proportional discharge.

(iv)   Scratchley outlet - if it is not desirable (because of cost) to install any other type of semi-module.

(v)   Pipe semi-module - when the banks of the supply canal are very wide; the setting of the module will be as indicated in (i), (ii), (iii), and (iv) above for the respective conditions.

(vi)   Fayoum standard weir outlet - its setting has been standardized, and it may be used successfully on all distributing canals.

(vii)   Pipe outlet - in view of its low cost, a pipe outlet may be used on all distributing channels with centre of the pipe set at $0.3 y_1$.

When the water supply to the farm outlets is charged with silt, it is essential to use semi-modules which can draw a proportional share of the silt. In this case proportional distribution of the water is neither necessary nor feasible and the following types of outlets may be used.

(i)   Open flume outlet - with setting of the crest at or near the bed of the distributing canal provided the width of the crest is not less that 6 cm and the necessary working head is available.   If the working head available is not sufficient, a combined pipe-open flume outlet may be used which permits a higher setting of the open flume outlet beyond the pipe.   This type is eminently suitable in lower reaches of distributing canals.   It should be used at tail clusters and above control points (within 300 metres upstream).

(ii)   Adjustable orifice semi-module and Jamrao type orifice semi-module - because of their low flexibility, these are eminently suitable for installation in the head reaches of a distributing system, with their settings at or near the bed level of the supply canal, provided the necessary head is available. If sufficient head is not available, an open flume fitted with a roof block having a similar setting may be used.

(iii)   Pipe semi-module - In cases where the crest of the outlet cannot be placed at or near the bed level, a pipe semi-module of the lowest possible flexibility may be used.

(iv)   As far as possible, the same type of farm outlets with the same head over the crests should be used between two control points on a distributing canal.

(v)   Pipe outlets with their upstream end at or near the bed may be used, but their coefficient of discharge is not constant and farmers may be tempted to increase their flow by heading up the water in the watercourse thus partially submerging the outlets.

C.   Non-modular farm outlets

The discharge of non-modular outlets depends on the difference of water levels in the supply canal and the watercourse or farm lateral.   The water level in the watercourse below the outlet varies considerably, depending on: whether high or low areas are being irrigated at any given time;   and where silting occurs, the extent of silt clearance in the farm lateral.   Where silting is a dominant feature, the canals fitted with non-modular outlets are always liable to flooding at the tail of the canal when farmers in the head reach do not clear silt so that they draw their full share of water during periods of slack demand.   On the other hand, water is always in short supply at the tail end during periods of keen demand, when farmers in the upper reaches tend to do the opposite, to clear their watercourses too much.

Non-modular outlets should, therefore, be avoided as far as possible. Their use is justified only when the working head available is so small that a semi-modular outlet cannot be used.

5.1.3.2   Selection of types of farm outlets

As indicated above outlets have been divided into three main classes - namely, modules, semi-modules and non-modular outlets and the general circumstances in which each class may be used is described in 5.1.3.1.   The selection of the particular type of outlet depends on factors such as:

-     cost

-     available working head

- ease of adjustment
- ability to withdraw silt
- immunity from tampering.

It should also be noted that modules with moving parts comprise more or less complicated mechanisms with the resulting possibility of the moving parts becoming jammed.

In conclusion, the information and data on farm outlets presented in this chapter will, it is hoped, be of help to the designer in making the best possible selection of the type or types of farm outlets he should adopt for distributing canals to suit most conditions and requirements. No claim is made as to the completeness of this chapter and it is hoped to make up any deficiency in the revised edition of this handbook.

5.2 CONSTANT-HEAD ORIFICE FARM TURNOUT (U.S.A.)

See Section 3.4 of Chapter 3.

5.3 ORIFICE MODULE (FRANCE)

See Section 3.5 of Chapter 3.

5.4 DOUBLE ORIFICE MODULE OR SIPHON MODULE

See Section 3.6 of Chapter 3.

5.5 DETHRIDGE METER (AUSTRALIA)[1]

5.5.1 Background

The Dethridge Meter is a self-integrating measuring structure used to deliver water to farmers and to check the volume of water supplied for application

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[1] Based on information supplied by the Australian National Committee, ICID.

Wheel
Cylinder and vanes of
14 gauge hot dip galvanised
mild steel.

Old Type Counter
Wire connector
Counter

Pendant
Cyclometer fixed to the wheel

Weight of wheels:
Large meter outlet not galvanised...187
Large meter outlet galvanised......194
Small meter outlet not galvanised.101
Small meter outlet galvanised......107

⅝ (½) diameter M.S. Spokes

Diameter of cylinder 3-4 (2-9)
Depth of vanes radially 10 (7½)
Outside diameter of wheel to tips
of vanes 5-0 (4-0)
Width of wheel 2-6 (1-8)
Clearances between wheel and
emplacement:
    At walls ¾ (¼)
    At floor ¼ (¼)

Length of axle 3-3 (2-5)
(1 internal diameter galvanised
pipe)

Anthony bearing

2-7½
(1-9)

Inlet from supply channel

Cast iron counter housing for
wire connection type counter

4-2
(3-6)

1-6
(1-2)

2-0
(1-9)

S.L.

3-0
(1-

2-6¾
(1-8½)

1-6
(1-3¾)

Natural surface

4-0
(2-6)

Concrete in emplacement is 4 thick
(except where filleted) and reinforced
by a grid of ¼ bars spaced 4 both ways,
for both large and small meters.

*Note: Dimensions shown are for both
Meters (Large and Small). Those
in brackets are for Small Meter
Outlets.*

FAO — ICID
DETHRIDGE METER OUTLET DETAILS
Project, Region, Country Australia
Figure No. 5-1

of water charges.

The device is widely used in the State of Victoria and other States in Australia and to some extent in the U.S.A. and in Asian countries. Approximately 15,500 large meters and 7,000 small meters are in operation in Victoria.

The Dethridge Meter was invented in 1910 by the Commissioner at that time, the late J.S. Dethridge, of the State Rivers and Water Supply Commission, Victoria, and adopted by the Commission after tests for three years under field conditions. The original Dethridge Meter was a simple measuring device giving a positive measurement of volume discharged, and recording it directly in acre-feet, with an accuracy of $\pm$ 3.5 per cent for free outfall conditions over a range of discharges from 42 l/s to 140 l/s $(1.5 \text{ ft}^3/\text{s}$ to $5 \text{ ft}^3/\text{s})$. A small meter for capacities from 14 l/s to 70 l/s $(0.5 \text{ ft}^3/\text{s}$ to $2.5 \text{ ft}^3/\text{s})$ was developed in the 1920's.

The construction of the Dethridge Meter has remained basically the same over the years, i.e. concrete emplacement with minor variations to head wall and transition shape. Laboratory and field tests have resulted in a standard setting of the wheel in relation to channel flow level and mechanical improvements for the wheel and fittings, e.g. wheel with a mild steel plate drum, vanes and water pipe axle, originally with timber spokes and rivetted, now all steel and welded; timber bearing blocks now replaced by a more robust sealed unit; steel gate in a timber frame, now replaced by neoprene guides set in concrete.

5.5.2   Structural Design

The general form and main dimensions of the two standard sizes of meter are shown in Figure 5-1. The wheel is made up of a cylinder of 14 gauge (2.03 mm or 0.08 inch thickness) mild steel sheet, bearing eight external vanes of the same material, and internally braced by three crossed pairs of steel spokes placed at the middle and both ends of the cylinder. The axle is a 2.54 cm (one inch) diameter galvanized pipe welded to the spokes. The cylinder, vane attachments and spokes are fabricated by welding.

The vanes are "V" shaped as shown (Figure 5-1), with the apex of the "V" leading in the direction of rotation. At the base of each vane, and at the apex of

the "V", there is an air vent to facilitate the filling and emptying of adjacent compartments as they enter and leave the stream of water passing under the wheel. The outer corners of the vanes are chamfered to suit the fillets at the junction of the walls and floor of the concrete emplacement.

The complete wheel unit is galvanized for protection against corrosion. The wheel unit is supported accurately in the emplacement with the ends of the axle resting on ball bearings in a Delrin race fixed on the walls of the flume. (Timber bearing blocks were originally used.)

A pendant actuated sealed cyclometer unit is rivetted to the wheel cylinder and rotates with it. The older meters have a cast iron housing attached to the flume wall for the revolution counter, which is connected to the end of the axle by a flexible wire link.

The shape of the flume can best be described by considering it in three parts. Upstream of the wheel it is of simple rectangular section, with level floor in the vicinity of the wheel. While the walls remain plane and parallel, the floor is indented to accommodate an arc of approximately 70° of the wheel's circumference. And immediately downstream of the wheel the walls are splayed outward and the floor is sloped up to a lip at sufficient height to ensure drowning of the passage swept by the vanes under the wheel.

At the entry to the flume a cut-off wall extends to either side into the canal bank, and downward below the natural surface of the ground to prevent seepage around the structure. The galvanized steel sluice gate to control discharge through the meter is fitted in neoprene guide strips set in recesses so that they are flush with the walls and floor at the entry from the channel.

The flume and cut-off wall are constructed of reinforced concrete, although other materials may be employed. In early times of cement shortage, timber was used, but proved unsatisfactory because of leakage between the timbers and distortion.

All concrete is of high quality and 10 cm (4 inch) thickness (except where filleted), and reinforced with a grid of 6 mm (0.25 inch) diameter bars spaced 10 cm (4 inches) apart both ways. The structure may be either cast in-situ or

assembled from pre-cast units.   The pre-cast units provide a more accurate emplacement with better concrete quality.   (See Figure 5-2)
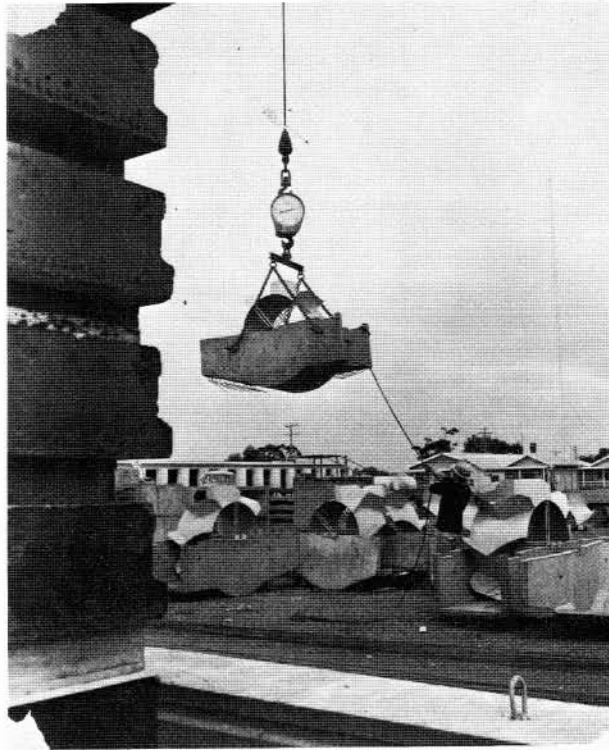
FIGURE 5-2. - Pre-cast large meter emplacement with wheels installed; note reinforcement to tie into cut-offs and pre-cast head wall.

Rip-rap protection is placed on the bed and batters of the farm ditch immediately downstream of the meter to prevent scouring.

For large canals, where continuous access is required along the canal bank, a pipe outlet is installed through the bank and the meter is erected outside the bank at the end of the pipe.

Figures 5-3 (a), 5-3 (b), 5-4 (a) and 5-4 (b) show details of both large and small meters.

Figure 5-5 shows a large Meter Outlet in operation with free outfall.

Groove to be formed in wall 1'-0" above floor level to indicate maximum allowable tailwater level.

C.L. of wheel axle

2'-6¾"

C.L.

Top of gate nominally level with top of headwall. Gates available are 2'-3, 2'-6, 2'-9 or 3'-0 high. Select gate size nearest to required height.

For value of (FB_hw)

(FB_hw)

D.D.L. Channel

1'-6 minimum into banks

Supply level meter

Head wall reinforcement

N.S.

Floor level

Groove to be pointed black with chlorinated rubber point, such as Dunlop C.R.7 on wire brushed surfaces or pointed with coal tar point.

¼" dia. @ 4" centres both ways

Calibration marks. Mark at supply level to be pointed black.

3'-0   3'-0

Half Section A-A   Half Section B-B

C

2'-6¾"

C.L.

Flow

Berm at N.S.

B

2:1   2:1   B

Bank crest

Slots for Neoprene seals formed in emplacement. Precast R.G. Gate frames may be used for remodelling of existing emplacement.

Eye bolt for setting of gate openings

1'-6 minimum

4"

1'-0 x 2½ x 3-2¾ precast R.C. foot plank, set on mortar pad.

Neoprene sealing strips

Earth platform level with top of side wall on both sides

⅜" dia. bolts 7 long, set 5¾ into block-out at 8 centres for Anthony bearings. For timber axle blocks ⅜ dia. bolts are 11 long.

4"   4"

2" deep, 12 long block-outs in side walls for bearings

C.I. Housing with Davies Shephard counter mounted on cover plate

C.L. axle

Anthony bearing

Anthony bearing

Bank of delivery channel

Beaching - average weight of stones 3 lb. (11 lb. min.) on 2'-3 of gravel bedding. Total thickness minimum 7 to be placed on bed and slopes, extending 6 feet downstream of the emplacement.

A   A

4'-0

C

Plan

Notes:
1. If the designed minimum free board to the top of banks is less than 2 feet (FB_hw), if it is equal to or greater than 2 feet, then (FB_hw) = 12".
2. The height of the headwall and gate is fixed from the designed discharge level of the channel.
3. The maximum height of gate suitable for hand lifting is 3'-0. Where larger gate is necessary, type 'C' lifting gear should be used.
4. Anthony bearings are adopted as standard.
5. Concrete for emplacement should have a minimum compressive strength of 3,500 p.s.i. at 28 days for durability.
6. Backfill should be well compacted around the headwall and side walls.
7. Protection of metal work in current use:
    Gates — Galvanised, together with axle.
    All bolts galvanised.
8. Field tests are in hand for cold point applications on gates e.g., Galvafroid with Miconox seal coat, and for aluminium wheels.

Rating of large meter outlet:

The adopted rating is 29·04 ft³/revolution equivalent to 1,500 revolutions = 1 acre-foot based on calibration tests with free outfall over the end sill.
Higher tailwaters increase the volume passed.
The cyclometers read directly in acre ft. 1 r.p.m. = 1 acre-foot/ day approximately.
Clearances between the wheel and emplacement should be accurately maintained and the wheel should not be allowed to turn at less than 3 r.p.m.

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LARGE DETHRIDGE METER OUTLET

Project, Region, Country

Australia

Figure No. 5-3 (a)

Section CC (see Figure 3 (a))

2 x 12 block out for bearings

7" long for Anthony bearings; 11" long for timber blocks

C.L. axle

Wingwall shape when access culvert required or road crossing

Bank crest

Bank of delivery channel

Groove at limit for tailwater level (see section AA, Figure 3(a))

Maximum water level in form delivery channel

2'-8" 1'-8" 1'-6" 2'-0"

(FB hw)

Designed discharge level channel

Supply level meter

12½"
2¾"
2½"
2½ x 12" block out for foot plank

Supply channel

N.S.

Flow

Floor level

Foot plank, bearing and wheel not shown

2'-6¼" radius

6 x 6" fillet

¼" dia @ 4" centres both ways Floor reinforcement continued into walls

3¾ x 3¾" fillet

Floor and side fillet to be given final rendering to required ¼" clearance from vane of wheel

Slope to delivery section and bed over 6 feet transition

2'-0" minimum

4" 7'-10" 4"

1'-3" minimum

Upstream View of Drum

2'-6"

One vane only shown

127'-8'

⅝" diameter spokes

Development of Vane

12'-9" radius
8'-6" radius
2'-9½"
3½"
4" 10" 10"

Side Elevation

Davies Shephard pendant weight counter

14 B.G. Black steel sheet (0.0785)

1⅛" standard washers 3/16" thick welded to spokes

3'-4" diameter

10" 10"

5" diameter

Outer spokes

⅜" lap

Inner spokes

Flow

Note: Use of aluminium wheels of welded or riveted construction is being considered.

Section DD

2'-6"

Vanes not shown

⅝" dia. mild steel spokes welded to drum

3'-3"

4½" 4½" ½"

Setting of large Meter Outlet in relation to supply channel

Upstream check

D.D.L. Channel

Supply level meter

Meter outlet floor level

Downstream check

3/16" dia hole with 3/16" rod inserted and edges of hole peened

1" internal dia standard medium pipe (not galvanised, not oil painted).

Meter Wheel Detail
(See note 7 and 8 on Figure 3(a))

¼ x ¼ x 5" recess for cover plate of C.I. housing (on counter side only)

5"
2¼ 2¼
C.L.
8"
2" 4" 2"
2" 2"
6" 6"

Width of blocks 2¾"

⅞" dia holes for fixing of C.I. housing (on counter side only), with 1½ dia x ⅜ deep recesses

Hole for axle. Diameter to suit 1" internal dia water pipe

3¼"

Timber Axle Block (See note 4 on Figure 3(a))

4" x 2¾ x12"

Block made in halves from seasoned R.G. or other durable timber dressed to dimensions shown. Slot and housing bolt holes required in one set only.

3'-2¾"

E 12" E

¼" dia @ 8" centres in centre of plank

⅜" dia @ 8" centres

2½"

Section EE
Precast R.C. Foot Plank
Timber plank may be used as alternative

Position of Wheel in Emplacement

Flow

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SECTION CC OF FIGURE 3(a) AND
DETAILS OF METER WHEEL
OF LARGE DETHRIDGE METER OUTLET

Project, Region, Country
Australia

Figure No. 5-3 (b)

Groove to be formed in wall 9" above floor level to indicate maximum allowable tailwater level

C.L. of wheel axle

1'-8½"

1'-3¼"

S.L Meter

Groove to be pointed block with chlorinated rubber point, such as Dunlop C.R. 7 or with coat tar point, applied to wire brushed surface

Floor level

12" minimum in solid ground

2¼"

2¾"

Top of gate nominally level, with top of headwall. Gates available are 2-0, 2-6, 2-9 or 3-0 high. Select gate size nearest to required height. See notes 2 and 3.

For value of (FB_hw)

(FB_hw)

C.L.

D.D.L. Channel

12"

2:1

Head wall reinforcement

1'-6" minimum into banks

N.S.

3" minimum

¼" dia. @ 4" centres both ways

Calibration marks. Mark at supply level meter to be pointed black

3'-0"   3'-0"

Half Section A-A   Half Section B-B

Slots for Neoprene seals formed in emplacement. Precast R.C. gate frames may be used for remodelling of existing emplacements

C

1'-8½"

Flow

2:1   2:1   2:1

B

1'-6" minimum

4"

Bank crest

Eye bolt for setting of gate openings, both sides

Neoprene sealing strips

Earth platform level with top of side wall on both sides.

B

⅞" dia. bolts 7" long set 5¾" into block-out at 8" centres, for Anthony bearings. For timber axle blocks ⅝" dia. bolts are 11" long.

4"

2" deep, 12" long block-outs in side walls for bearings

1⅞"

C.I. Housing with Davies Shephard counter mounted on cover plate

Anthony bearing

C.L. axle

Anthony bearing

Bank of delivery channel

Beaching - average weight of stones 3 lb. (1 lb. min.) on 2-3" of gravel bedding. Total thickness minimum 7" to be placed on bed and slopes, extending 6' downstream of the emplacement.

A

4"   4"   A

2-6"

C

Plan

Notes:
1. If the designed minimum free board to the top of banks is less than 2' (FB_hw) = 9", if it is equal to or greater than 2', then (FB_hw) = 12".
2. The height of the headwall and gate is fixed from the D.D.L. of the channel.
3. The maximum height of gate suitable for hand lifting is 3'-0". Where larger gate is necessary, type 'C' lifting gear should be used.
4. Anthony bearings are adopted as standard.
5. Concrete for emplacement should be of minimum compressive strength 3,500 p.s.i. at 28 days for durability.
6. Backfill should be well compacted around the headwall and side walls.
7. Protection of metal work in current use:
   Gates — Galvabond plate.   Wheels — Galvanised, together with axle.
   All bolts galvanised.
8. Field tests are in hand for cold paint applications on gates e.g., Galvafroid with Miconox seal coat and for aluminium wheels.

Rating of small meter outlet:

The adopted rating is 12.45 ft³/revolution equivalent to 3,500 revolutions = 1 acre-foot based on calibration tests with free outfall over the end sill.
Higher tailwaters increase the volume passed.
The cyclometers read directly in acre-feet/r.p.m. = 3 acre-feet/day approximately.
Clearances between the wheel and emplacement should be accurately maintained and the wheel should not be allowed to turn at less than 3 r.p.m.
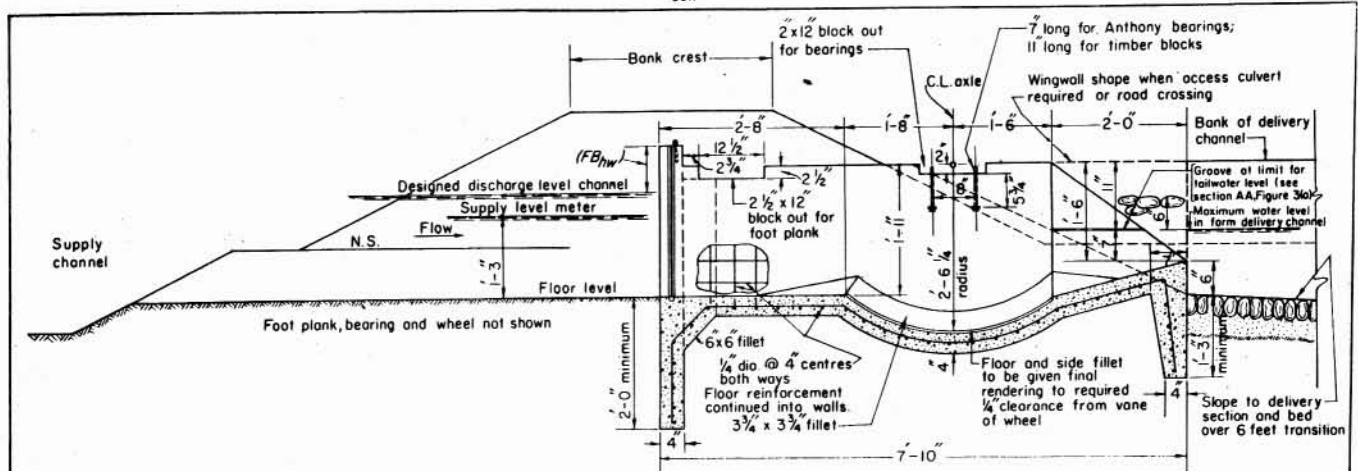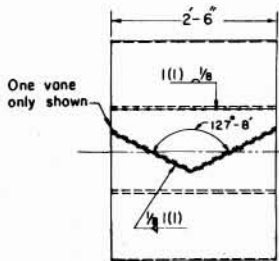
FAO - ICID

SMALL DETHRIDGE METER OUTLET

Project, Region, Country
Australia

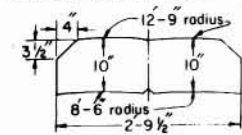Figure No. 5-4 (a)

Section C-C

5/8" dia. galvanised bolts 7" long for Anthony bearings, 11" long timber blocks

Groove at limit for tailwater level (see Section A-A)

Wingwall shape when access culvert required or road crossing

2"x12" block out for bearing

Bank crest

2'-3"

C.L. axle

1'-3"

1'-2"

1'-9"

Bank of delivery channel

Designed discharge level in channel

Supply level meter

(FB)hw

Maximum water level in farm delivery

Supply channel

N.S.

Flow

Floor level

12"

Bearing and wheel not shown.

6" x 6" fillet 1/4" dia. @ 4" both ways. Floor reinforcement continued into walls.

2 3/4 x 2 3/4" fillet Floor and side fillet to be given final rendering to required 1/4" clearance from vane of wheel

Slope to delivery section and bed over 6 feet transition

4"

12 minimum

6'-5"

Section C-C
See Figure 3 (a)

Davies Shephard pendant weight counter

14 B.G. Block steel sheet (0.0185)

Outer spokes

1/2" dia. spokes

Upstream View of Drum

One vane only shown

Side Elevation
Note: Use of aluminium wheels of welded or riveted construction is being considered.

Flow

3/4" lap

Inner spokes

Section D-D

3/8" standard washers 3/16" thick welded to spokes both ends

3/16" dia. hole with 3/16" rod inserted and edge of hole peered

1" internal diameter standard medium pipe (not galvanised, not varnished, not oil painted)

1/2" dia. M.S. spokes welded to drum

Upstream check

Downstream check

D.D.L. Channel

S.L. Meter
Meter outlet
floor level

Setting of Small Meter Outlet in relation to Supply Channel

Development of Vane

Position of Wheel in Emplacement

Flow

Meter Wheel Detail
See note 7 and 8 on Figure 4 (a)

Width of blocks 2 3/4"

11/16" dia. holes for 5/8" dia. bolts in centre of blocks at 8" centres

1/4" x 1/4" x 5" recess for cover plate of C.I. housing (on counter side only)

Block made in halves from seasoned R.G. or other durable timber, dressed to dimensions shown. Slot and housing bolt holes required in one set only.

7/16" dia. holes for fixing of C.I. housing (on counter side only), with 1 1/2" dia. x 5/8" deep recesses.

Hole for axle diameter to suit 1" internal diameter water pipe

4" x 2 3/4" x 12" Timber Axle Block
See note 4 on Figure 4 (a)

Scale 0 2 4 6 inches

FIGURE 5-5. - Large Dethridge meter outlet in operation with free outfall.

The structure is robust in construction and reasonably resistant to illegal tampering. If necessary, gates can be padlocked in position and lead seals fixed to counters. The counter has a ratchet arrangement to prevent winding back if the wheel is rotated in the opposite direction (important against tampering). One vane is painted so that the canal attendant can see from a distance whether the wheel is turning correctly. The meter will pass a fair amount of floating debris in the water without damage to, or stoppage of, the wheel.

Once installed correctly in an accurate emplacement the meter does not require adjustment. Regulation of discharge through the meter is carried out by simple adjustment of the gate opening.

5.5.3   Hydraulic Characteristics

It is important that the meter be installed at the correct level in relation to the designed discharge level of the water in the canal, so as to make the best use of the generally limited head available while at the same time ensuring sufficient head and yet avoiding drowning of the wheel by water that may back up from the farm ditch. The standard setting for the large meter, as shown in Figure 5-6,

is to have the floor of the flume, at entry, 38 cm (15 inches) below supply level
in the canal.   For the small meter this depth is 30.5 cm (12 inches).

The supply level of the canal is fixed by the designed upstream level of each
canal regulator and is taken as a horizontal pool at this level extending upstream
from the regulator to the next regulator.   This level is fixed in relation to the
land to be supplied.

The commanded land is defined as the land 15 cm (6 inches) or more below
the supply level making allowances where necessary for farm canal grades and
structures.   This allows about 7.5 cm (3 inches) drop through the meter to the
maximum allowable level in the farmer's ditch and 7.5 cm (3 inches) depth of
meter over the highest ground.   As the canal running level is above the
horizontal supply level, meters, set some distance upstream of a regulator, have
a water depth over the floor of the flume in excess of the standard 37.5 cm (15
inches) or 30.5 cm (12 inches) respectively and the gate and head wall levels must
be raised accordingly to provide the required free board.   As far as possible,
regulators are located so that meters are within a short distance upstream.

If there were no necessity for clearances between the wheel and the flume,
the meter would give an exact positive measurement of the water passing through
it, as each revolution of the wheel would pass an invariable quantity of the water
between the vanes and the cylinder swept through the emplacement.   With the
provision of the necessary clearance (see section 5.5.4) however, leakage occurs
through the clearance space at a rate dependent not only on the rate of rotation of
the wheel but also on other factors, such as the difference in water levels
immediately upstream and downstream of the wheel, and the depth of submergence.
The quantity of water passed per revolution of the wheel does, therefore, vary to
some extent under operating conditions.   For this reason, the clearance is kept
as small as practical.

5.5.3.1   Relationship between revolutions and discharge

For the conversion of revolutions to acre-feet, constant ratios are
assumed: 1,500 revolutions per acre-foot for the large wheel and 3,500
revolutions per acre-foot for the small wheel.   The dimensions of the wheels
have in fact been designed to provide these simple ratios, and the error is not

more than 5% over the range of normal operating conditions for either meter. This degree of accuracy is considered quite reasonable for the measurement of water deliveries for irrigation.

Similarly revolution counters are geared in the ratio of 15:1 for the large meter, and 35:1 for the small meter, and so the dial reads directly in acre-feet. The counters as manufactured read to two decimal places, but irrigation records are rounded off to one decimal place.

A discharge rate of one acre-foot per day (14 l/s or 0.5 ft^3/s) corresponds closely to one revolution per minute of the larger wheel or $2\frac{1}{3}$ revolutions per minute of the small wheel. These relationships provide a very convenient guide to the canal operator or water-bailiff in setting a meter gate to give whatever discharge rate is required.

5.5.3.2   Design ratings

(a)   Ratings and settings adopted in relation to the supply canal levels for the two meters are as indicated below:

Ratings	Large Meter Outlet	Small Meter Outlet
(i) Free overfall	820 l/revolution or 29.04 ft^3	349 l/revolution or 12.45 ft^3
(ii) Tail water as in (iii)	860 l/revolution or 30.38 ft^3	370 l/revolution or 13.08 ft^3
(iii) Tail Water		
Above downstream sill level	17.8 cm or 7 inches	13.3 cm or $5\frac{1}{4}$ inches
Above upstream sill level	30.5 cm or 12 inches	22.9 cm or 9 inches
(iv) Designed supply level		
Above upstream sill level	38.1 cm or 15 inches	30.5 cm or 12 inches
(v) Clearance between the Meter wheel and the Outlet Structure		
Bottom	0.64 cm or $\frac{1}{4}$ inch	0.64 cm or $\frac{1}{4}$ inch
Side	0.95 cm or $\frac{3}{8}$ inch	0.64 cm or $\frac{1}{4}$ inch

5.5.4   Installation and Operation

Canal regulators are located to regulate supply levels for the meter outlets as shown in Figures 5-6 and 5-7.
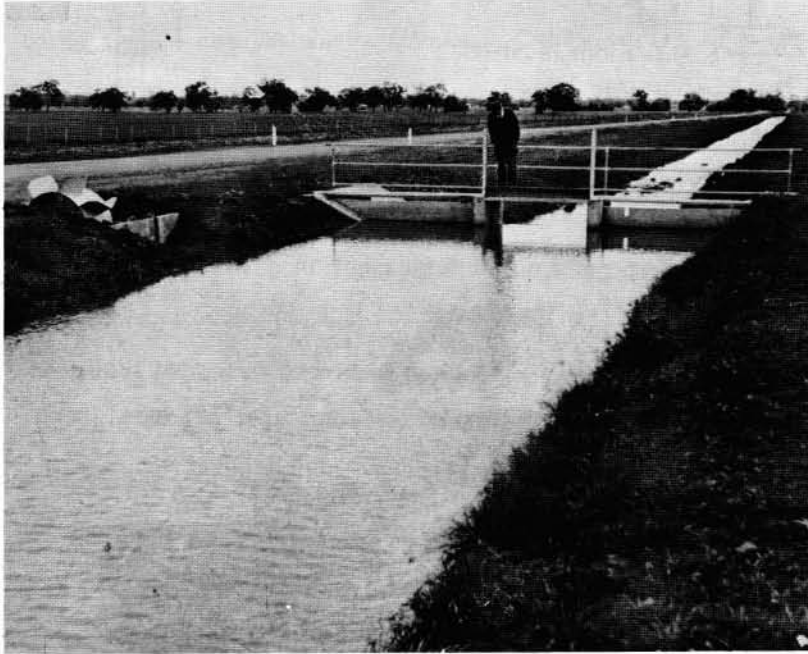

FIGURE 5-6. - Typical setting of Dethridge meter just upstream of a regulator. The supply level of the canal is indicated by the bottom of the slot in the wall on the right hand side of the regulator.

Generally one meter is provided for each farm, a small one for areas up to 40 acres and a large one for larger farms. If the farm area commanded by gravity and suitable for irrigation exceeds 120 ha (300 acres), a second large meter may be provided. These general rules may be modified, if necessary, because of particular farm layouts and topography.

The canal attendant regulates flow to the farm through the meter at required intervals during the irrigation season to provide flows as requested by the irrigator. Regular readings of the counter are taken to keep check on the total volume supplied.

The flow is regulated by adjusting the gate to the appropriate opening as indicated in Figure 5-8. In actual operation the flow is readily checked by
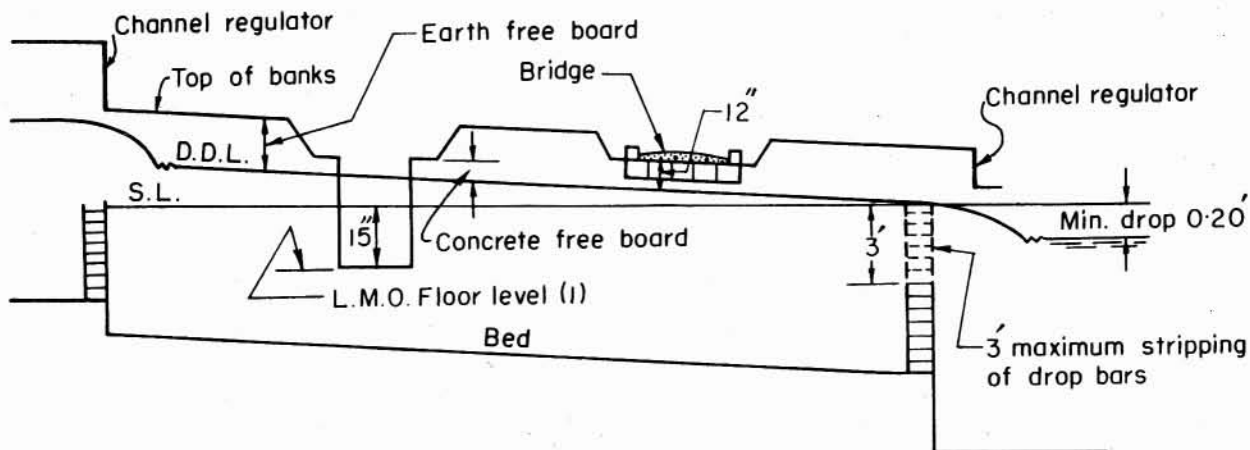
Typical Longitudinal Section

D.D.L. Designed discharge level
Flow profile for maximum discharge with roughness and grade selected

S.L. Supply level for setting meter outlets and spur offtakes

(1) For SMO setting of floor is 12" below supply level

Capacity in ft³/s	Earth bank		Structures	
	Minimum* crest width in feet	Minimum free board in feet	Minimum concrete free board	Prestressed slab bridges **
0 – 30	6	1' – 6"	9"	12"
30 – 490	8	1' – 6"		
> 490	12	2' – 0"	12"	

* The required crest width may be larger than the minimum to meet percolation gradient requirements or construction equipment requirements.
If access is required along the bank a minimum crest width of 12' is required.

** Measured from D.D.L. at abutments to top of prestressed slab.

FAO – ICID
CHANNEL DESIGN, FREE BOARD AND SETTING OF DETHRIDGE METER OUTLETS
Project, Region, Country Australia
Figure No. 5-7

counting the revolutions per minute of the wheel and reference to the table is not necessary.

Accurate ratings have been obtained in the laboratory to give graphs for volume passed per revolution of the wheel over a range of discharges and for different canal levels and tailwater levels. Other series of tests have been carried out for non-standard clearances. These ratings are not required for operations or normal design purposes because average ratings have been adopted for the counter gearing. Detailed ratings are, however, available for special investigations.

5.5.4.1   Costs and quantities of materials

Direct labour and material costs of the installed pre-cast outlet (excluding fittings) are in Australia approximately:

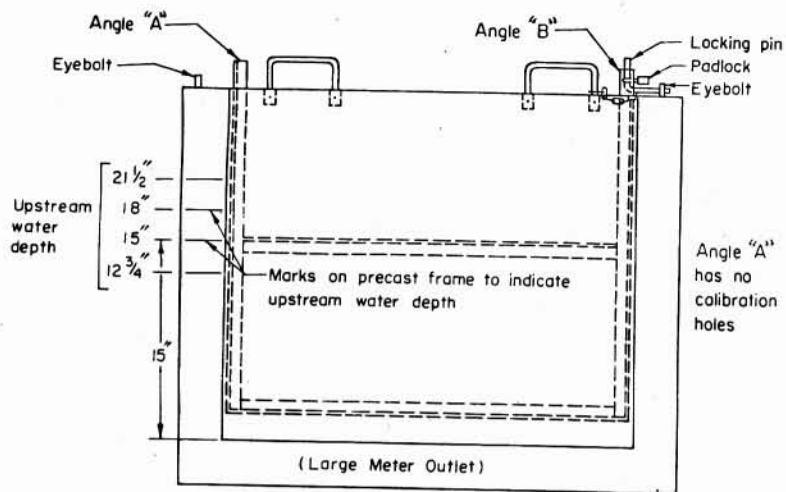|	
small meter outlet	$A 350
large meter outlet	$A 450

These costs vary depending on site conditions and access. The costs of fittings, (wheel, gate, guides, bearings, cyclometer) are $A.43 for a small meter and $A.63 for a large meter and are included in the above cost.

Approximate concrete volumes for pre-cast units are:

small meter outlet	0.65 m^3 or 23 ft^3
large meter outlet	0.99 m^3 or 35 ft^3

5.5.4.2   Maintenance

The main item of regular maintenance for the old type of meter was recoating the wheel with tar, for protection against corrosion. This was done each year in the winter months, when there was no irrigation. This has now been eliminated by using galvanized metal parts. When the canal operator takes a reading of the counter or adjusts the discharge, he should check that the wheel, the bearings and counter are operating correctly. Any damage to the galvanizing should be treated with a cold zinc-rich epoxy paint. The farmer's ditch must be checked to ensure that it is clean of weeds which would increase the water level and cause high tailwater levels on the meter.

Precast frame with gate locked in closed position. Upstream view

(Large Meter Outlet)

Angle "A" has no calibration holes
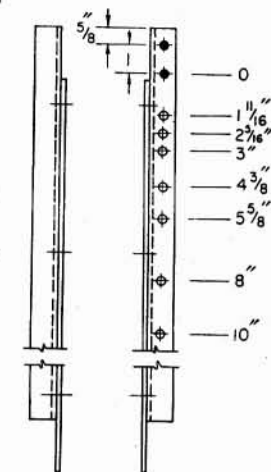
Angle "A"   Angle "B"

Precast frame with gate locked in closed position. Upstream view

(Small Meter Outlet)

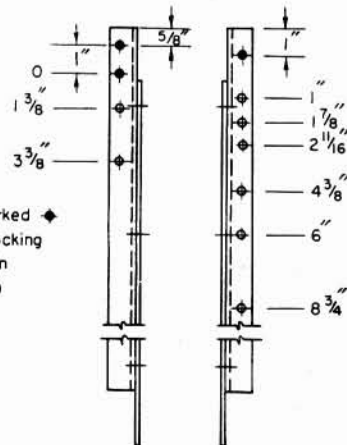Holes marked ◆ are for locking and not in calibration
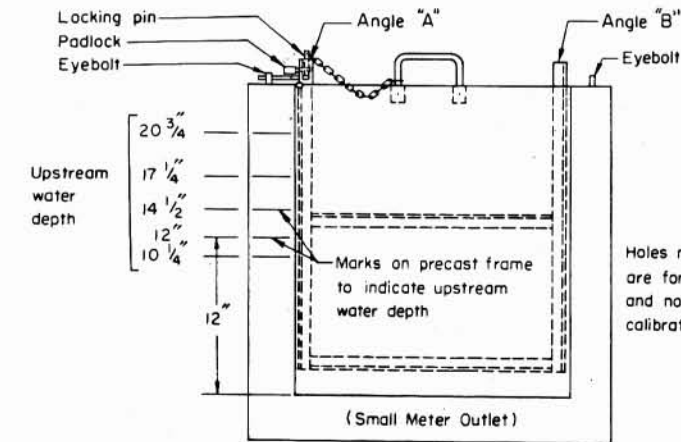
Angle "A"   Angle "B"

(Large Meter Outlet)   12 inches tailwater
Discharge in ft³/s
Approximate No. of r.p.m. of wheel given in brackets

Gate opening	Upstream water depth			
	12¾"	15"(S.L.)	18"	21½"
1¹¹⁄₁₆"	–	–	–	1·59
2⁵⁄₁₆"	–	–	1·68	2·12
3"		1·48 (3)	2·16	2·77
4⅜"		2 (4)	2·99	3·72
5⅝"		2·5 (5)	3·38	4·12
8"	1·52	3·0 (6)	4·27	4·94
10"	1·66	3·56 (7)	4·58	5·38
Maximum discharge – gate fully open = 3·8 ft³/s (8)(15"S.L.)				

(Small Meter Outlet 9 inches tailwater
Discharge in ft³/s
Approximate No. of r.p.m. of wheel given in brackets

Gate opening	Upstream water depth				
	10¼"	12"(S.L.)	14½"	17¼"	20¾"
1"			0·5	0·61	0·76
1⅜"		0·5 (3)	0·68	0·86	1·12
1⅞"		0·75 (3½)	0·95	1·21	1·53
2¹¹⁄₁₆"	0·61	1·0 (5)	1·36	1·65	2·04
3⅜"	0·77	1·22 (6)	1·68	2·06	2·47
4⅜"	0·96	1·5 (7)	1·98	2·30	2·68
6"	1·19	1·75 (8)	2·33	2·65	
8¾"	1·40	1·95 (9)	2·45		
Maximum discharge – gate fully open = 2·05 ft³/s (12" S.L.)					

Notes:
1. The calibrations refer to meter outlets with standard clearances.
2. The calibration tables are based on tests at H.E.S. Werribee during 1964-66.

FAO – ICID

GATE CALIBRATION TABLES
FOR L.M.O. AND S.M.O.
RIVETTED GALVABOND TYPE GATES

Project , Region , Country
Australia

Figure No. 5-8

5.5.5   Numerical Example

Design a Dethridge Meter Outlet under free overfall conditions for standard ratings and setting to deliver during part of the irrigation season 2.5 ft^3/s and in the other part of the season 3.8 ft^3/s.

Refer to the Tables on Figure 5-8, which show that for standard ratings and settings (see 5.5.3.2), the large meter outlet will give a discharge of 2.5 ft^3/s with a gate opening of 5$\frac{5}{8}$ inch and a discharge of 3.8 ft^3/s with the gate fully open.

Other dimensions and details are as given in 5.5.3 and 5.5.4.

5.5.6   Further Research to Improve the Structure

Experiments have been carried out for some time to try and find an arrangement with better accuracy at low discharge. Tests have included rubber flaps on the wheel to seal the clearances. A modified emplacement is at present under test with the downstream section of the flume the same width as the upstream section, instead of flared, and the floor horizontal. A gate is installed on the downstream end to control the flow of water and to ensure filling of the volume between the vanes and drum.

A somewhat similar modified arrangement is used in New South Wales.

Trial aluminium wheels, rivetted and welded, have been installed but are much more expensive than galvanized steel. Fibreglass wheels have been tried in New South Wales.

5.5.7   Summary

The Dethridge meter outlet can operate with a small head loss, as low as 7.6 cm (or 3 inches) for discharges up to 112 l/s (or 4 ft^3/s) for a large meter. A head loss of at least 17 cm (or 7 inches) is required for best accuracy.

The device measures by positive action. The measurement is self-integrating. Accuracy of the measurement is around 5% which is well within reasonable limits for irrigation deliveries.

The accuracy drops off sharply with high tailwater levels. The water level in the farmer's ditch must be controlled to keep it at least 7.5 cm (3 inches) below

Section

199·5

398·9

40

Plastic siphon

Tube

50

H(wt)

Canalette

The detail of tube

A

93

91

78

5

115

198

12

12

93 95

120
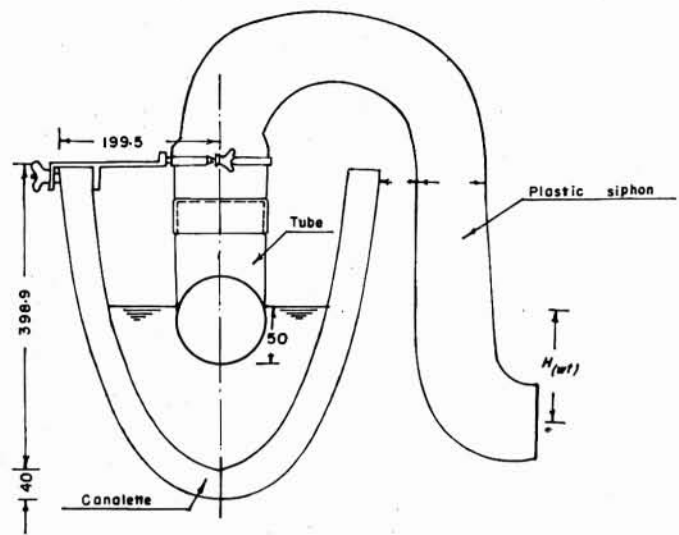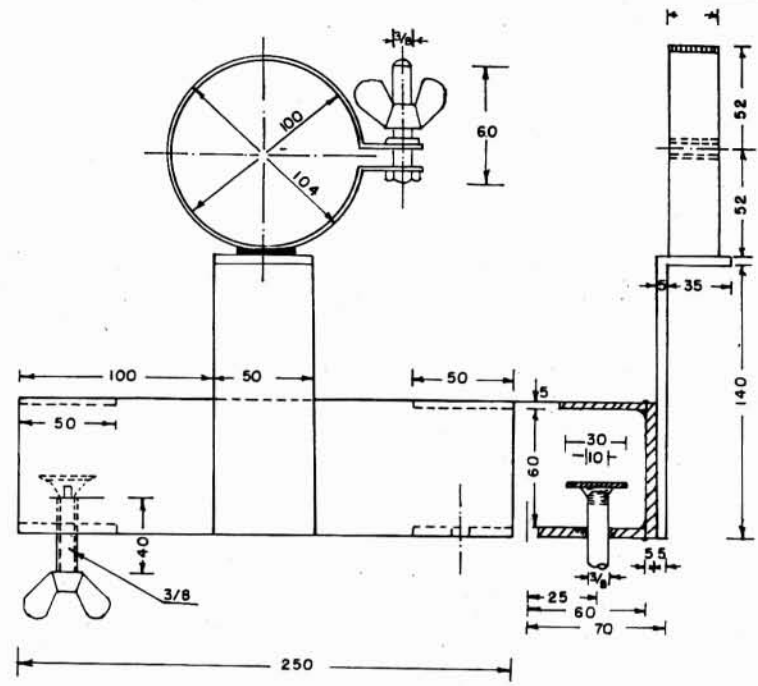
Section A-A

Structural detail to fix the siphon

100

104

60

3/8

52

52

35

140

100

50

50

5

50

60

30

10

40

3/8

5·5

25

60

70

250

3/8

Note :— All dimensions are in m.m.

264
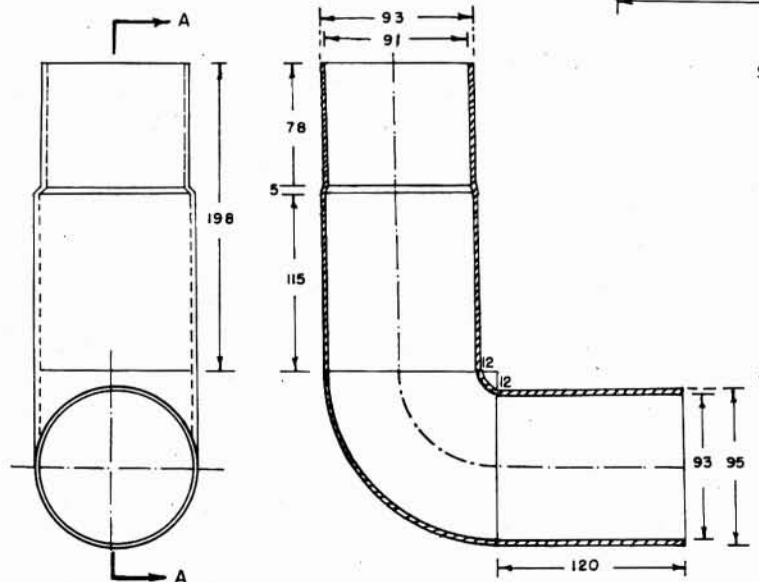
F A O — I C I D

PLASTIC SIPHON OUTLET FITTED

WITH AN INTAKE TUBE

Project, Region, Country

Turkey

Figure 5-9

the meter supply level. Even at the 3 inch control level the error increases to 6.7% at 42 1/s (1.5 ft^3/s).

The accuracy also drops off sharply for discharges lower than the minimum indicated above. For this reason the small meter is not suitable for measuring flows to areas under 2 ha (5 acres).

Considerable care should be taken to provide accurate setting of the wheel and clearance between the wheel and emplacement. The bottom clearance is the most critical. A clearance of 2.54 cm (1 inch) instead of the standard 0.64 cm (0.25 inch) would increase the volume passed by more than 10%.

For free outfall conditions, a "Pelton Wheel" situation occurs if the canal level is more than 76.2 cm (30 inches) above the upstream floor. A jet flow develops under the wheel without completely filling the swept volume beneath the vanes and drum such that the actual volume passed can be as much as 10% less than the rated volume.

This outlet is not suitable for canals carrying silt charged water as it cannot draw its share of the silt.

5.6   PLASTIC SIPHON OUTLET FITTED WITH AN INTAKE TUBE[1]

5.6.1   General

A Plastic Siphon Outlet fitted with an intake tube has been evolved in Turkey for delivering water to farm laterals from elevated flumed tertiary canals running under shooting flow conditions. Elevated flume irrigation systems are being increasingly used in Turkey because of the reduced construction time necessary (through prefabrication) and other economic advantages.

Delivery of the water to individual farms is effected by means of plastic siphons. Plastic siphons enable water to be delivered from any point along

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[1] Based on a note prepared by Ozden Bilen, Turkey, and supplied by the Turkish National Committee, ICID.

PLASTIC SIPHON FITTED WITH AN INTAKE TUBE
RELATIONSHIP BETWEEN DISCHARGE
AND DEPTH OF SUBMERGENCE

FAO-ICID

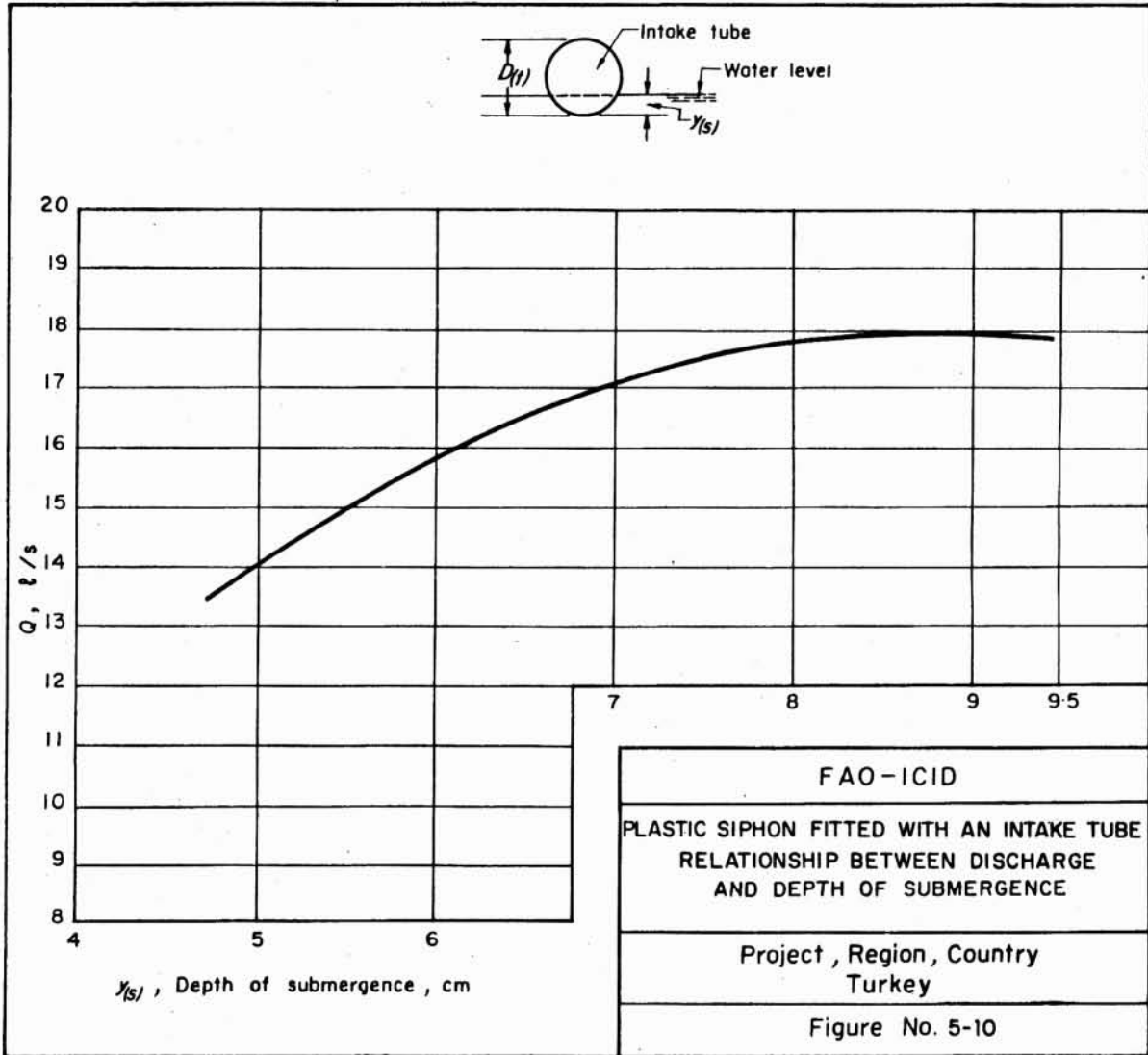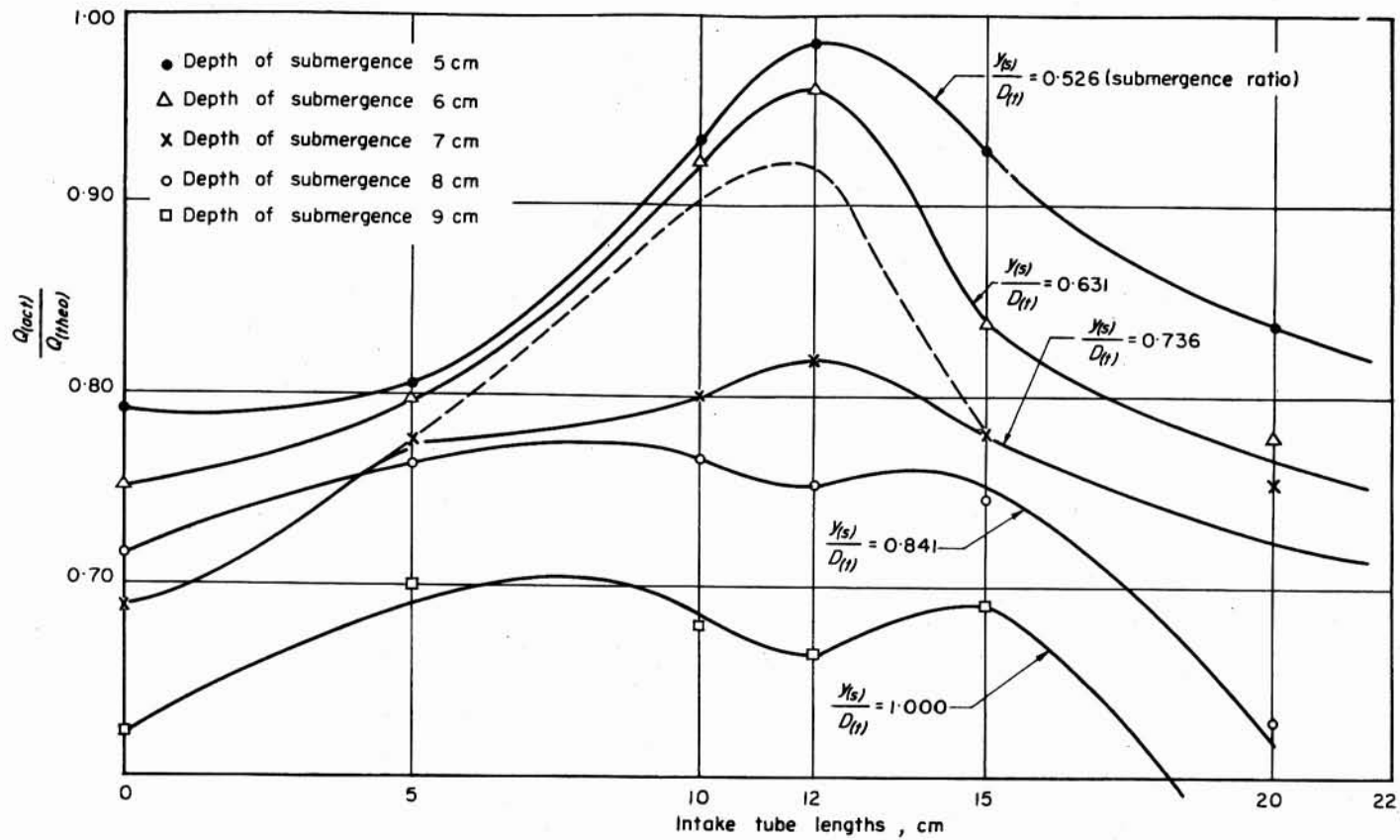Project, Region, Country
Turkey

Figure No. 5-10

FIGURE 5-11. – Plastic siphon fitted with intake tube – Efficiency of siphon.

canals and thus they are adaptable to a wide range of farm layouts.

Economically, relatively steep slopes and high water velocities in elevated flume type tertiaries are desirable; on the other hand, this creates difficulty in withdrawing water under shooting flow conditions. To overcome this problem, the dynamic head $(\frac{v^2}{2g})$ developed by the velocity of flow in an elevated flume, has been utilized by connecting intake tubes to the siphons.

5.6.2   Structure

The outlet consists of a plastic siphon with an intake tube 12 cm long and 9.3 cm inside diameter (Figure 5-9).

The siphon is fixed to the sides of the elevated flume by means of a steel ring (2 mm thick) and mild steel plates (5 mm thick) as shown in Figure 5-9. The means for fixing the siphon is easily adjusted for different depths of submergence. The siphon has a free flow and discharges either directly into the farm lateral or into a small masonry or concrete receptacle from where the water flows on to the farm lateral. With this device it is possible to withdraw about 18 l/s of water under shooting flow conditions.

5.6.3   Hydraulics

The angle between the siphon inlet and the intake tube has been selected at 90°. The discharge capacity of the siphon decreases with the increase of this angle.

Figure 5-10 is a graph, developed from experiments, which shows the relation between discharge and depth of submergence. Figure 5-11 is a graph showing the relationship between the length of the intake tube and the efficiency of the siphon (ratio between the actual and theoretical discharges) for depths of submergence from 5 cm to 9 cm. The graphs are valid for an intake tube length of 12 cm and siphon of inside diameter of 9.3 cm. It will be seen from Figure 5-11 that with an intake tube of 12 cm in length and a depth of submergence of 5 cm (corresponding to $\frac{Y_{(s)}}{D_{(t)}} = 0.552$) gives an efficiency of 98%. In the case of complete submergence ($\frac{Y_{(s)}}{D_{(t)}} = 1.00$) efficiency is 66%. It is advisable

to use a submergence depth between 5 cm and 8 cm.

In practice the water velocity under shooting flow conditions in tertiary canals varies between 2.5 m/s to 3.5 m/s, and the graphs in Figures 5-10 and 5-11 give satisfactory results and are valid for this velocity range.

5.6.4   Design

The submergence depth for a given discharge to be passed for a tube with an inside diameter of 9.3 cm is found from Figure 5-11. For this submergence depth and the adopted tube length, the efficiency of the outlet, as explained in 5.6.3, is found. This efficiency should be as high as possible.

For example:

    let Q of the outlet be   15 l/s
         diameter of inlet tube be 9.3 cm
         length of inlet tube   12 cm

From Figure 5-10, submergence depth for a discharge of 15 l/s = 5.5 cm. The efficiency for this design is about 97.5%.

5.7   OPEN FLUME OUTLET (INDIA AND PAKISTAN) [1]

5.7.1   General

The open flume outlet is widely used with success in Punjab, Haryana and some other states of India, and the provinces of Sind and Punjab of Pakistan. (See Figure 5-12)

The earlier types of outlets developed in India - Kennedy's sill outlet, Kennedy's gauge outlet, the Harvey outlet, the Harvey Stoddard irrigation outlet - have been superseded either due to their not having been immune to

[1] Based on a note prepared by A.D. Choudhry, Chief Engineer, Irrigation Works, Haryana (India), and K.C. Gupta, Executive Engineer, Central Designs Irrigation Works, Haryana (India).

tampering or due to improved designs. The open flume outlet, a development of the idea underlying the Harvey outlet, was first introduced by Crump in the Punjab, and underwent improvements and modifications from time to time. The two types which finally emerged are: the open flume as used in the Punjab, and the Jamrao type open flume [1] as used in the Province of Sind, Pakistan.
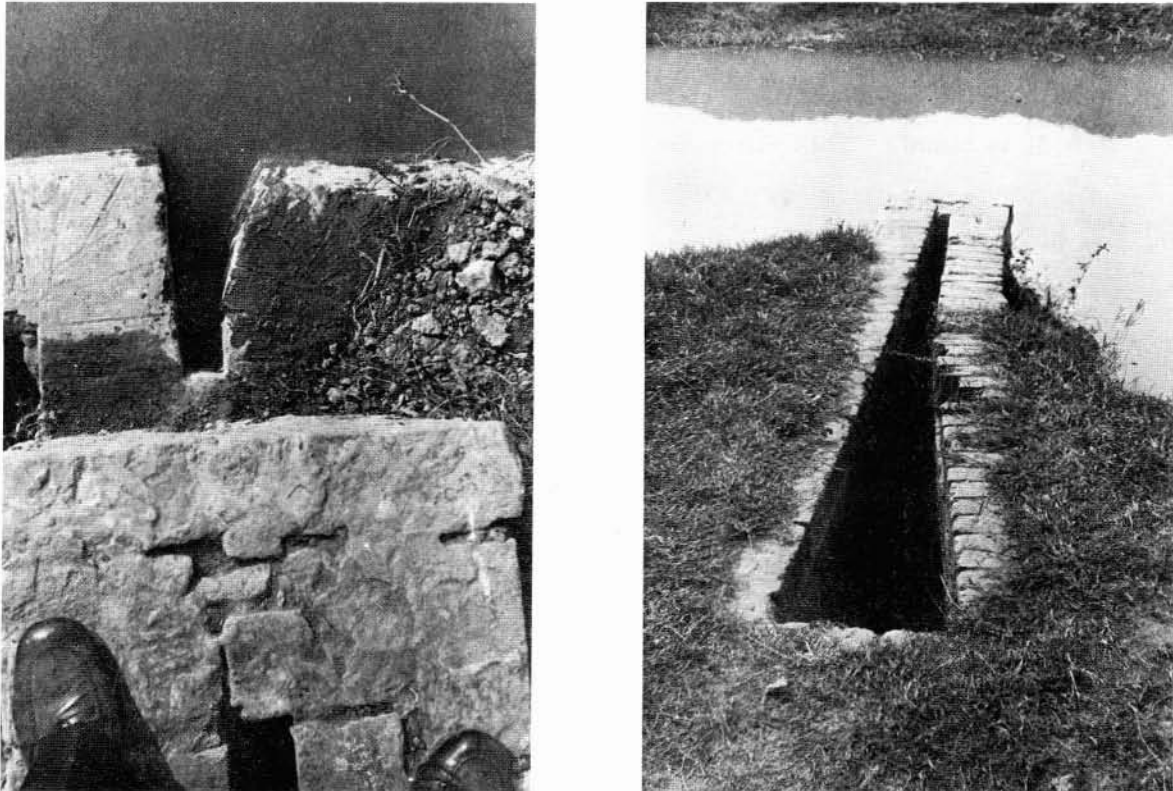
FIGURE 5-12. - Photographs of open flume farm outlet (Punjab type)

5.7.2   Structural and Design Characteristics

The open flume outlet (a typical design is shown in Figure 5-13) is essentially a smooth weir with a throat constricted sufficiently to ensure a velocity above the critical and long enough to ensure that the controlling section

[1] Particulars not furnished in this Handbook. The Jamrao type open flume outlet requires more working head than the Punjab type open flume outlet.
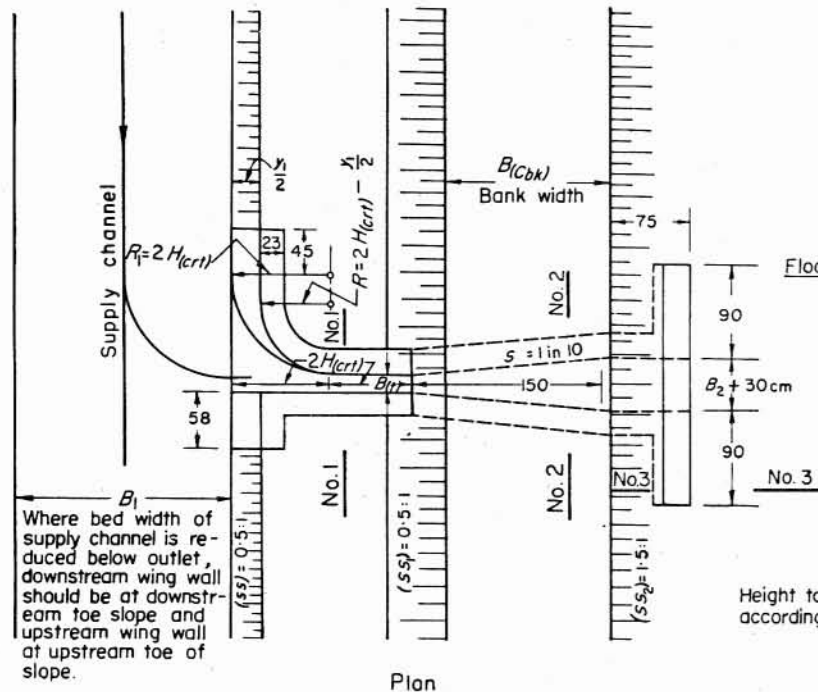
Sections for supply channels 30 to 60 cm depth and $H_{(crt)}$ less than y_1

Section No.1

Section No.2

Top to be in level with upstream wing wall

Section No. 3
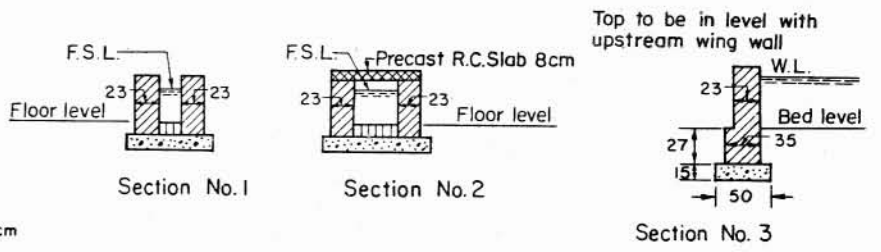
Sections for supply channels above 60 cm depth and $H_{(crt)}$ is less than y_1

Section No.1

Section No. 2

Top to be in level with upstream wall

Section No. 3

Plan

Where bed width of supply channel is reduced below outlet, downstream wing wall should be at downstream toe slope and upstream wing wall at upstream toe of slope.

Longitudinal Section

Floor 12 cm over cement concrete 15 cm

Berm

Bank width

Precast R.C.Slab 8 cm thick
Top should be in level with upstream wing wall
Water surface level

Where bed of watercourse is lower than floor of outlet downstream and wing walls to be taken down 30 cm below bed of watercourse and if necessary a cistern to be provided.

According to y_1 and bank width

120 60 0 120 240

Scale in centimetres

(All dimensions are in centimetres)

FAO — ICID

OPEN FLUME FARM OUTLET

Project, Region, Country
India and Pakistan

Figure No. 5-13

271

remains within the parallel throat at all discharges up to the maximum. A flared upstream head wall and a gradually expanding flume is provided at the outfall to obtain the maximum recovery of head. The entire structure is built in brick masonry; but to prevent tampering and also to help in the construction of a precise section of the throat or gullet, an iron base plate of about 6 mm thickness and about 30 cm long should be fitted in the centre of the controlling section in the gullet. Cast iron plates, 300 mm x 300 mm x 10 mm, should also be provided on the side of the gullet in conjunction with the base plate as shown in Figure 5-14. In case of need to widen or narrow the controlling section at a later stage, check plates ca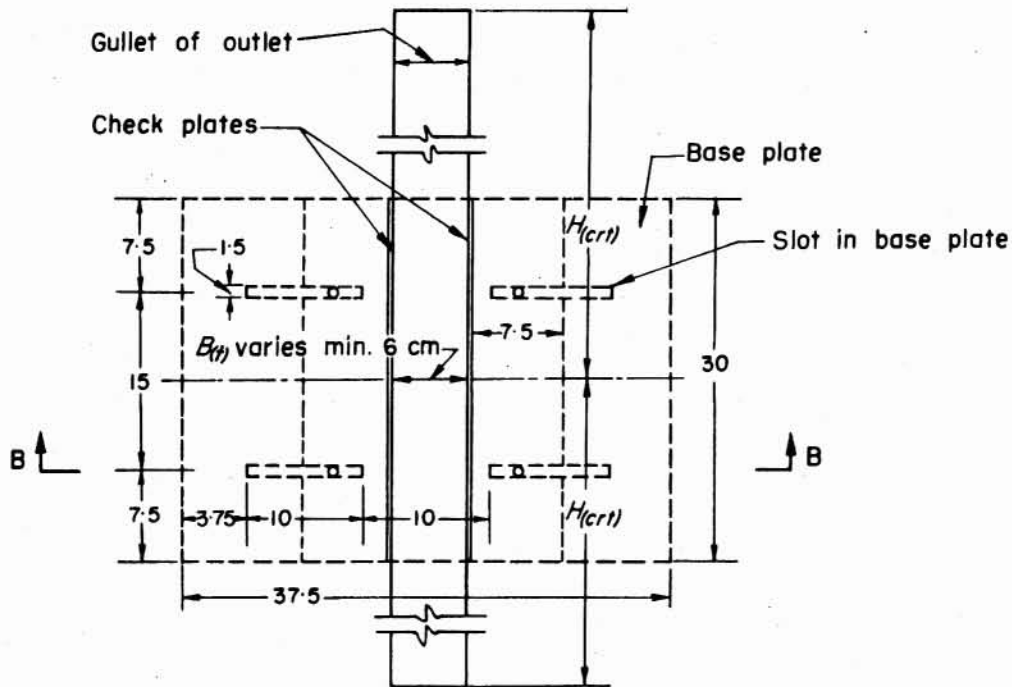n be adjusted by the required width by means of sliding bolts. Mild steel plates may also be used, after welding to size. The thickness of the base plate and the check plate should be 5 to 6 mm when mild steel plates are used and 10 to 12 mm when with cast iron plates.

The length of the throat should be equal to $2H_{(crt)}$ and the controlling section starts at a distance of $2H_{(crt)}$ from the toe of the side slope of the supply canal. In order to obtain the maximum recovery of head, the downstream side walls are splayed at 1:10 for a length of 1.5 m (5 ft), having a width at the end equal to 30 cm (1 ft) plus bed width of the watercourse or farm canal. At the downstream end there are wing walls on both sides. The structure is fitted with a gauge on the upstream side to read $H_{(crt)}$.

The slope of the downstream glacis depends on the bed level of the watercourse or farm canal.

The free board on the supply canal water surface up to the downstream end of the throat is 15 cm (6 inches). Downstream side walls or wing walls are at least 15 cm (6 inches) above the water surface level in the watercourse or farm canal.

The structure is very durable and has a very long serviceable life. It is immune from tampering when cast iron or steel frame and side check plates are fitted in the throat. It can be adjusted, when required, by dismantling one side wall, then either raising or lowering the crest level or reducing or increasing the width of the throat and rebuilding the side wall at the required distance.

Gullet of outlet

Check plates

Base plate

Slot in base plate

7.5

1.5

$B_{(f)}$ varies min. 6 cm

7.5

15

30

B

B

7.5

3.75 10

10

$H_{(crt)}$

37.5

Sectional Plan A-A

Base plates to check plates to be 10 mm to 12 mm thick when
of cast iron and 5 mm to 6 mm thick when of mild steel plates

Face of masonry wing walls

$B_{(f)}$ varies min. 6 cm

A

A

Check plates adjusted at $B_{(f)}$ apart
with the help of sliding bolts and
embeded flush with masonry wing
walls

Back surface to be chequered for
bond with mortar

L-shape check plate to be of cast iron
or mild steel plates welded to shape

30

Base plate

12.5 mm diameter bolt

3.75

Crest level

7.5

37.5
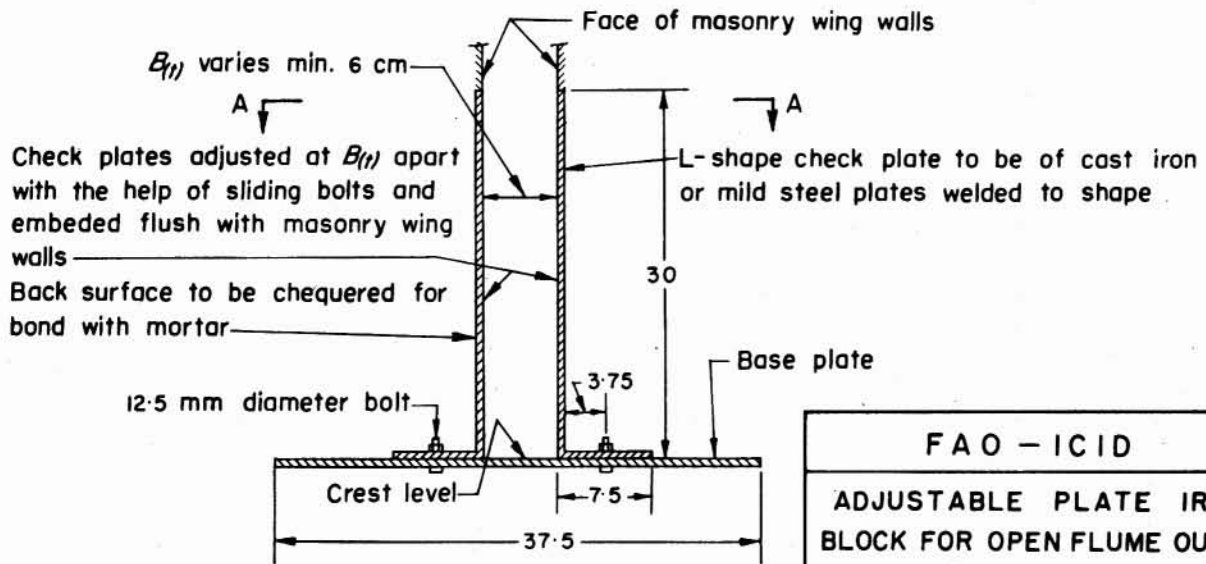
Section B-B

Showing installation of check plates on
base plate

(All dimensions are in centimetres)

FAO — ICID
ADJUSTABLE PLATE IRON BLOCK FOR OPEN FLUME OUTLETS FOR $B_{(f)}$ 6 cm TO 20 cm
Project, Region, Country India and Pakistan
Figure No. 5-14

The structure operates without any control. The gauge at the outlet is read once a month when the canal supervisor carries out inspection of the outlet.

5.7.3   Hydraulic Characteristics

5.7.3.1   Accuracy

In this outlet discharge can be calculated from the design formula (see 5.7.4) so long as a steady standing wave forms downstream and the water surface level in the supply canal does not touch the bottom of the roof block, if the outlet is fitted with one.

5.7.3.2   Flexibility

Flexibility (Fl) (the ratio which the rate of change of discharge of outlet bears to the rate of change of discharge of the supply canal) of the open flume outlet is given by:

$$(Fl) = \frac{\dfrac{dQ_2}{Q_2}}{\dfrac{dQ_1}{Q_1}}$$

$$= \frac{9}{10} \cdot \frac{y_1}{H_{(crt)}}$$

Where Q_2 is the discharge of the outlet, Q_1 is the discharge of the supply canal, and y_1 is the depth at full supply level in the supply canal.

It will thus be seen that proportionality in discharge can be secured by fixing the crest of the outlet at 0.9 of the depth of the supply canal. If the crest be higher than this, the outlet becomes more flexible, i.e. hyper-proportional, and if lower, it tends towards rigidity. With a fall in the full supply level in the supply canal, the flexibility would increase and with a rise in the water level it would decrease.

To decrease the defect of high flexibility, a roof block (Figure 5-15) is fitted in the gullet of an open flume, at the vena contracta, clear of the water surface in the gullet when the outlet is drawing its full supply discharge.
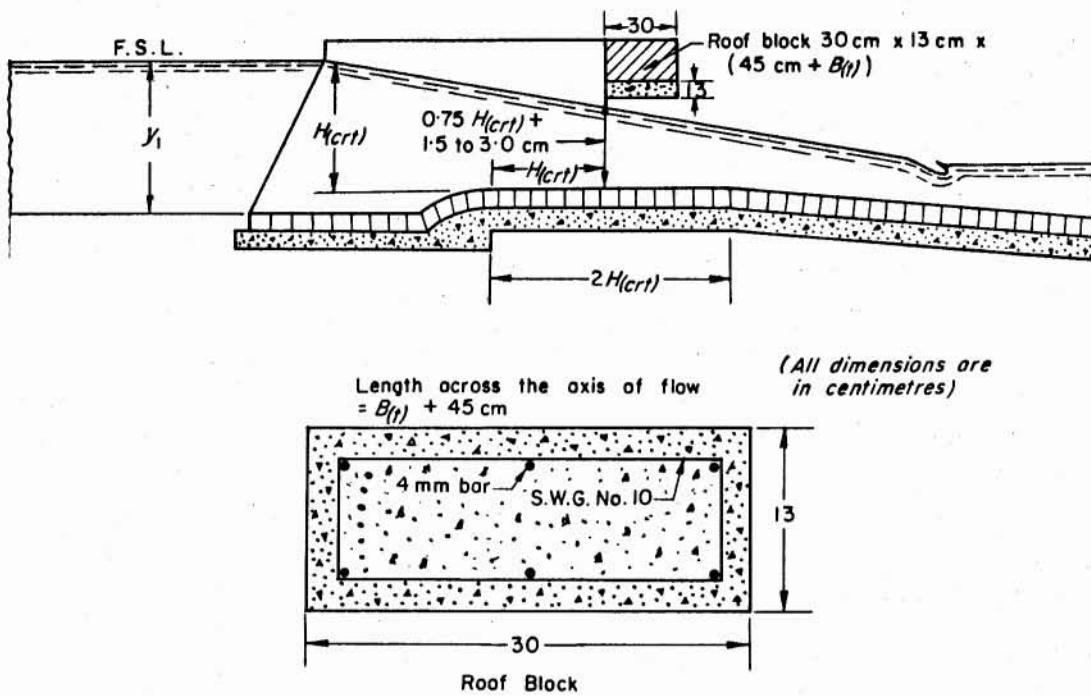
FIGURE 5-15. - Open flume outlet. Details of roof block.

This clearance is generally set at 1.5 cm in the head reach and 3 cm in the tail reach of the distributing canals. The roof block should be fixed at a distance equal to $H_{(crt)}$ below the upstream end of the throat and the bottom of the roof block should be at a height of 0.75 $H_{(crt)}$ above the crest plus the clearance of 1.5 to 3 cm. The roof block should have a square edge at the bottom and it may be of brick masonry or reinforced concrete, the height up to the top of the side walls and length along the flow varying from 12 cm to 23 cm. This device enables the open flume to start working as an orifice as soon as the supply level in the parent canal is above the bottom of the roof block.

5.7.3.3   Silt-drawing capacity

The higher the crest of the outlet compared with the bed level of the supply canal, the less is its silt-drawing capacity. In practice, the width of the throat of the outlet is limited to a minimum of 6 cm (0.2 ft) and, because of this, it often becomes necessary to raise the crest of the outlet much above the bed
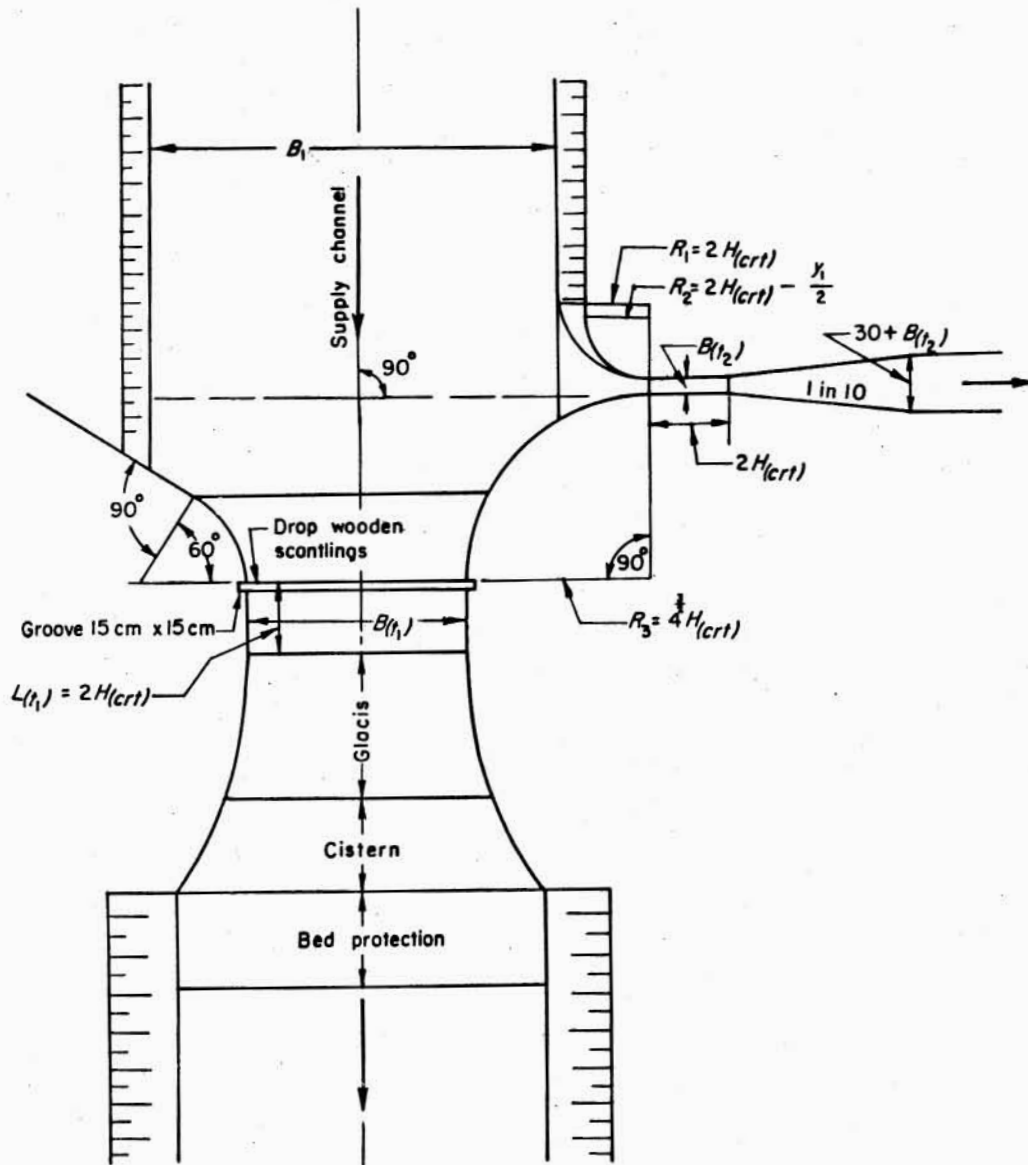
FIGURE 5-16. - Arrangement of open flume outlet upstream of a fall.

level of the supply canal. It is apparent (except in small canals) that it is seldom possible to place the crest of an open flume outlet with a normal discharge of less than 56 l/s (2 ft^3/s) at the bed level of the canal.

5.7.3.4 Range of operation

This outlet can work as a semi-module for all heads over the minimum modular head and for all discharges generally required for outlets.

The open flume outlet with its crest set at $0.9\,y_1$, (provided $B_{(t)}$ is not less than 6 cm), can be used with advantage for proportional distribution of water when the supply canals have to be run below the full supply level. The working head required for modularity under both full and minimum supply conditions should be derived from the expression:

$$h_{(wk)min} = y_1 \left[0.98 - 0.8 \left\{ \frac{Q_{min}}{Q_1} \right\}^{0.6} \right]$$

where

$h_{(wk)min}$ is the minimum working head corresponding to Q_{min}

y_1 is the depth of water in the supply canal

Q_{min} is the lowest limit of discharge at which the supply canal is to be run.

For $\dfrac{Q_{min}}{Q_1} = 0.55$ $\qquad h_{(wk)min} = 0.42\,y_1$

Thus, an open flume outlet with its crest at $0.9\,y_1$ and having a working head of not less than $0.42\,y_1$ will draw proportional discharge within the limits of 55% to 100% supply in the parent canal.

Open flumes are recommended for use within 300 m (1,000 ft) upstream of control points (Figure 5-16), at tail clusters (Figure 5-17) and where adjustable semi-orifice module outlets cannot be designed with their crest at or near bed level of the supply canal. Where banks are wide, it is used in combination with a pipe outlet.

5.7.3.5 Tail clusters

When the discharge of a secondary, tertiary or quarternary canal
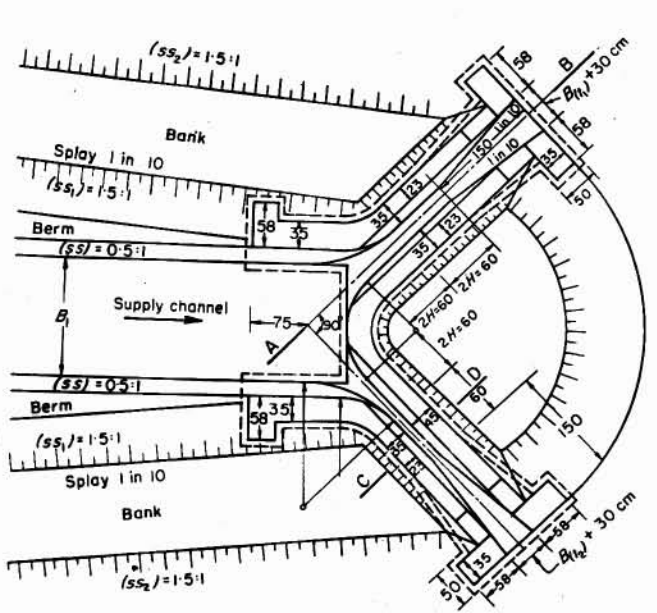
TWO-WAY TAIL CLUSTER
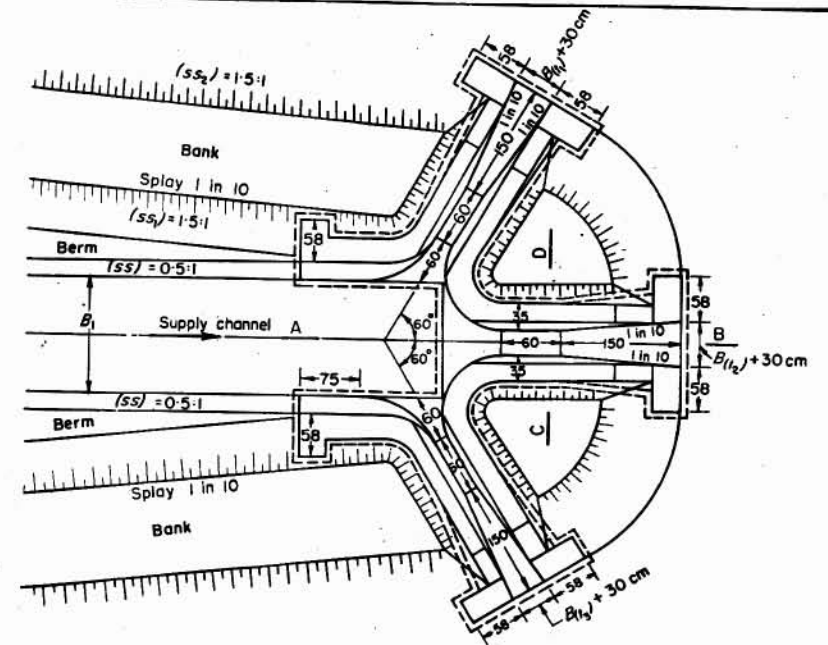
THREE-WAY TAIL CLUSTER
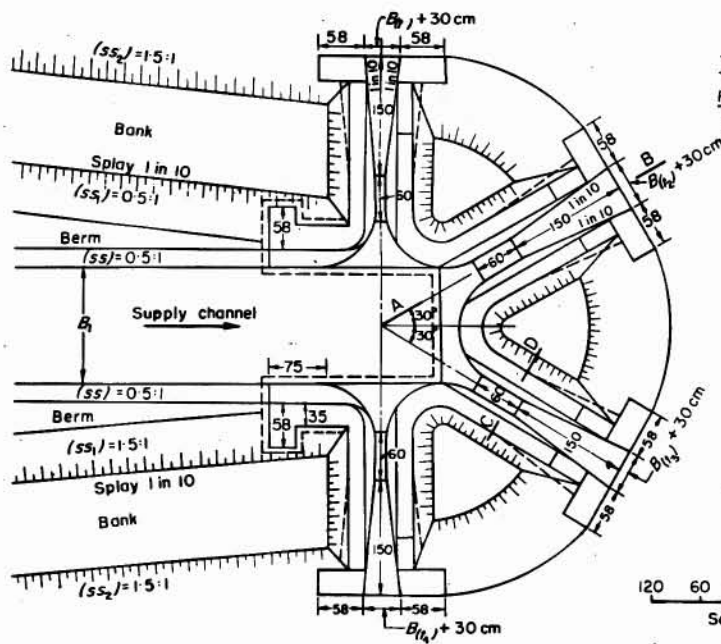
FOUR-WAY TAIL CLUSTER
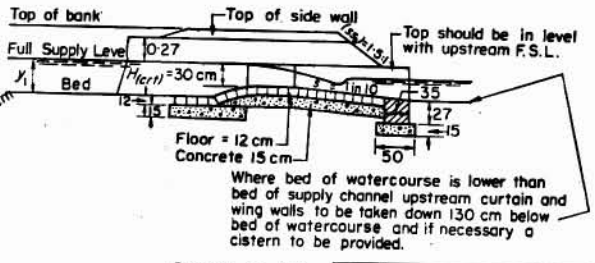
Section on AB

Section on CD

Where bed of watercourse is lower than bed of supply channel upstream curtain and wing walls to be taken down 130 cm below bed of watercourse and if necessary a cistern to be provided.

(All dimensions are in centimetres)

Scale in centimetres

120 60 0 120 240
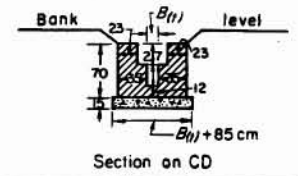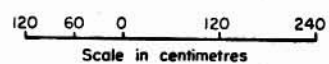
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PLAN OF TAIL CLUSTER
OF TWO, THREE AND FOUR
FARM OUTLETS

Project, Region, Country
India and Pakistan

Figure No. 5-17

279.

diminishes to below 150 l/s, it is desirable to construct all the tail outlets in the form of a cluster for equal distribution of water. Figure 5-17 shows three standard designs for open flume outlets in tail clusters.

The crests of the outer flumes in a tail cluster (three-way and four-way) are sometimes built 0.06 cm lower than the inner flume. To compensate for velocity of approach the standard 30 cm (one ft) gauge should in such cases be fixed with its zero at the crest level of the inner flume. With this arrangement the widths of the flume above the full supply level are made proportional to the designed width of the flume below the full supply level in order to distribute benefits of any excess water reaching the tail of the canal proportionately to all the outlets.

5.7.4   Design Formula

The design formula used for an open flume outlet is:

$$Q = CB_{(t)} H_{(crt)}^{\frac{3}{2}}$$

where Q =   design discharge of the outlet in l/s or ft^3/s;

     $B_{(t)}$ =   width of the throat in cm or ft (throat width less than 6 cm or 0.20 ft must not be adopted);

     $H_{(crt)}$ =   height of the designed full supply level in the supply canal above the crest level of the outlet in cm or ft; and

     C =   a coefficient having the following values for different widths of throat (in metric and British units).

$B_{(t)}$ (cm)	C	$B_{(t)}$ (ft)	C
6 to 9.0	0.0160	0.20 to 0.29	2.90
9.1 to 12.0	0.0163	0.30 to 0.39	2.95
Over 12.0	0.0166	0.40 and over	3.00

The minimum modular head adopted is $0.2 H_{(crt)}$. When designing, it is necessary to assume a particular value of $B_{(t)}$ or $H_{(crt)}$, then calculate the other, and work out the minimum modular head, and then see if the latter is less

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OPEN FLUME FARM OUTLET
Relationships of discharge
to B(t) and H(crt)

Project, Region, Country
India and Pakistan

Figure No. 5-18
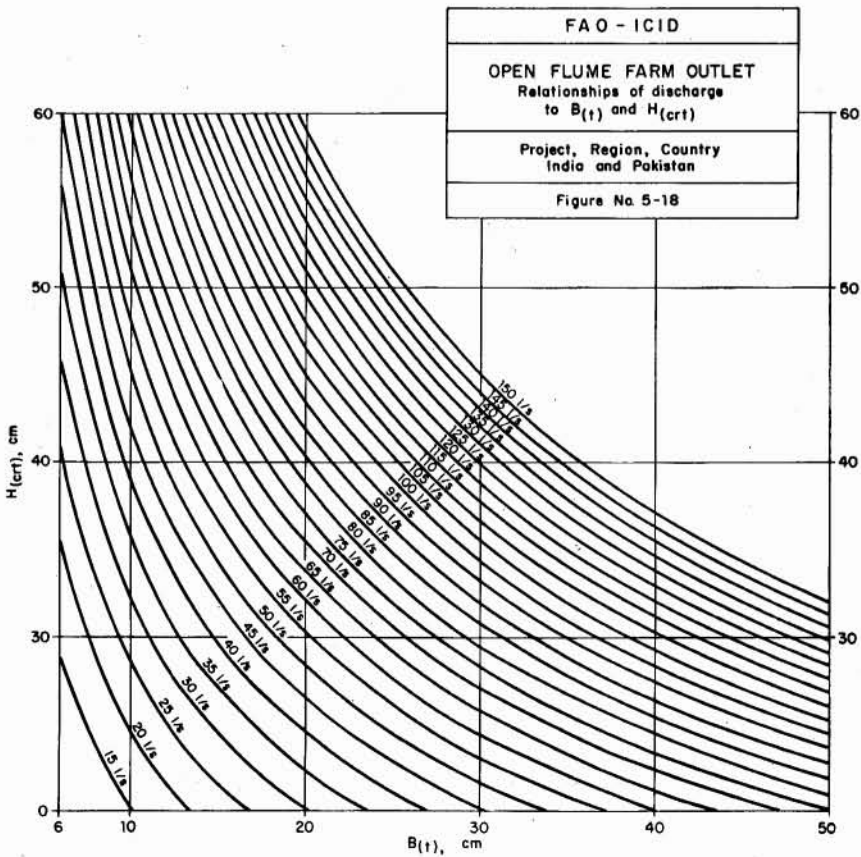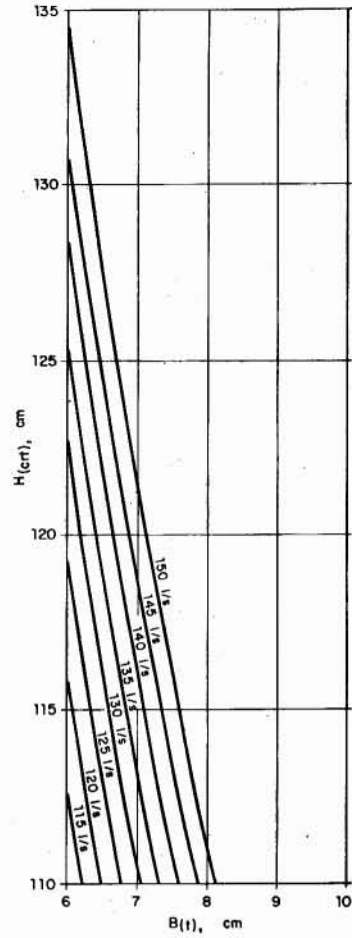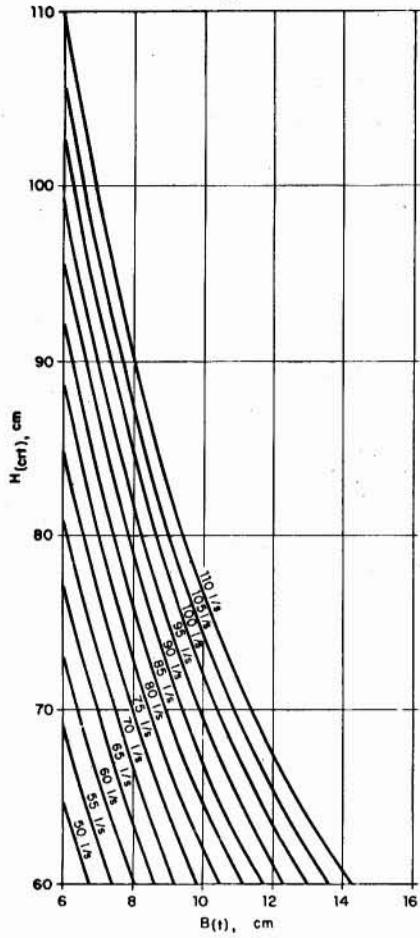
than the available working head. To find $B_{(t)}$ or $H_{(crt)}$ for a given discharge and a working head, Figure 5-18 may be used.

5.7.5   Numerical Examples

Example 1 - Design an open flume outlet, with a discharge of 50 litres per second, for a canal with a full supply depth of 100 cm. The working head available for the outlet is 15 cm.

$$Q \quad = \quad 50 \ \text{l/s}$$

$$y_1 \quad = \quad 100 \ \text{cm}$$

$$h_{(wk)} \quad = \quad 15 \ \text{cm}$$

Since $h_{(wk)_{min}}$ (minimum working head) for an open flume outlet is $0.2 \, H_{(crt)}$, the maximum $H_{(crt)}$ for the outlet can be 75 cm.

With $H_{(crt)} = 75$ cm, and $Q = 50$ l/s, the width $B_{(t)}$ will be 4.8 cm, i.e. less than 6 cm.

$\therefore$ Adopt the minimum value of $B_{(t)} = 6$ cm

Then $50 = 0.016 \cdot 6 \cdot H_{(crt)}^{\frac{3}{2}}$ (refer to formula under 5.7.4).

or $H_{(crt)}^{\frac{3}{2}} = \dfrac{50}{0.096} = 520.83$ or $H_{(crt)} = 64.73$ cm

for which, $h_{(wk)_{min}} = 12.9$ cm against 15 cm available. The design is suitable but setting of the outlet is $\dfrac{64.7}{100} = 0.647$.

This is not too high but the outlet will not take its fair share of silt.

Should it be necessary to conduct silt effectively it is possible, though costly, to build a combined pipe and open flume outlet. (See Section 5.10)

To save mathematical calculation, reference may be made to the diagram in Figure 5-18. It will be seen that for $H_{(crt)} = 75$ cm the value of $B_{(t)}$ is less than 6 cm. For $B_{(t)} = 6$ cm and $Q = 50$ l/s, $H_{(crt)}$ is 64.73 cm.

Example 2 - Assume the outlet in Example 1 is located just above or within a short distance of a fall or drop in the canal which has a depth over crest under full supply condition of 50 cm. Let the available working head of the outlet be 70 cm. Design a suitable outlet.

$$h_{(wk)} = 70 \text{ cm}$$

Since the outlet is above or close to a control point in the canal, $H_{(crt)}$ of the open flume outlet should be the same as that of the fall, viz: 50 cm. This will ensure proportional distribution.

Thus

	Q	=	50 l/s
	$H_{(crt)}$	=	50 cm
	Q	=	$C B_{(t)} H_{(crt)}^{\frac{3}{2}}$
or	50	=	$C B_{(t)} . 353.6$
and	C	=	0.0160
Then	$B_{(t)}$	=	8.84 cm.

This value of $B_{(t)}$ may also be read from Figure 5-18.

The minimum working head required for this size is 10 cm, which is much less than the available working head.

If proportionality is not necessary, a more rigid outlet can be obtained by designing an orifice semi-module.

Example 3 - Design a three way tail cluster open flume outlet in accordance with the following data:

Tail right outlet   : $Q = 50$ l/s and $h_{(wk)} = 12$ cm

Tail centre outlet   : $Q = 80$ l/s and $h_{(wk)} = 30$ cm

Tail left outlet   : $Q = 30$ l/s and $h_{(wk)} = 15$ cm

The full supply depth of the distributing canal at the tail is 40 cm, and full supply depth is 100 m.

An examination of the available working heads of the three outlets shows

that if $H_{(crt)}$ be 30 cm, all the outlets will work modularly.

$$\text{and} \quad h_{(wk)min} \quad = \quad 6 \text{ cm}$$

$$\text{Then for Tail right outlet } Q \quad = \quad C\,B_{(t)}\,H_{(crt)}^{\frac{3}{2}}$$

$$\text{or} \quad 50 \quad = \quad C\,B_{(t)} \;.\; 30^{\frac{3}{2}}$$

$$(\,C \quad = \quad 0.0166\,)$$

$$\text{or} \quad B_{(t)} \quad = \quad 18.3 \text{ cm}$$

$$\text{Adopt crest level} \quad = \quad 99.64$$

$$\text{Tail centre outlet } 80 \quad = \quad C\,B_{(t)} \;.\; 30^{\frac{3}{2}}$$

$$(\,C \quad = \quad 0.0166\,)$$

$$\text{or} \quad B_{(t)} \quad = \quad 29.3 \text{ cm}$$

$$\text{Crest level} \quad = \quad 99.70 \text{ (axis assumed parallel to flow in supply canal)}$$

$$\text{Tail left outlet } 30 \quad = \quad C\,B_{(t)} \;.\; 30^{\frac{3}{2}}$$

$$(\,C \quad = \quad 0.0163\,)$$

$$B_{(t)} \quad = \quad 11.2 \text{ cm}$$

$$\text{Adopt crest level} \quad = \quad 99.64$$

The same values of $B_{(t)}$ can be found for the given data from Figure 5-18.

In case of three and four-way clusters (Figure 5-17) the crest of the central outlet with its axis parallel to the flow in the supply canal will be 0.060 m higher than the other two or three side outlets.

Example 4 - Design an open flume outlet for proportional distribution in a supply canal, the normal supply of which is 55% of the full supply.

$$\text{Data:} \quad y_1 \quad = \quad 120 \text{ cm}$$

$$Q_2 \quad = \quad 45 \text{ l/s}$$

$$h_{(wk)} \quad = \quad 55 \text{ cm}$$

As $h_{(wk)}$ is more than $0.42\,y_1$, i.e. 50.4 cm, the open flume will draw proportional discharge from 55% to 100% of the supply in the main canal.

$$H_{(crt)} = 0.9\ y_1 = 108\ \text{cm}$$

$$B_{(t)} = \frac{Q_2}{C\ .\ 108^{\frac{3}{2}}} = \frac{45}{0.0160\ .\ 1122} = \frac{45}{3.586}$$

$$= 2.51\ \text{cm}$$

Since $B_{(t)min} = 6$ cm, the value of $R_{(crt)}$ is 60.3 cm.

5.7.6   Summary

The open flume outlet requires only small working heads.   It is very suitable for proportional distributors, and for outlets within 300 m of a control point, in tail reach clusters and even in head reaches of a supply canal when the working head available is only small.   The structure requires no manual control. The water surface level in the supply canal above the crest of the outlet is read and the corresponding discharge obtained.

A disadvantage is that the outlet is not provided with any gate arrangement and it is not possible to shut it when the supply canal is running.

The open flume outlet has been developed for systems in which distributing canals are run at full supply level with little fluctuation of discharge in the supply canal.   These systems are usually not equipped with check structures, so that in the case of low discharges, the outlets would not draw their design discharge.

In many cases the open flume outlet has to be either deep and narrow (in which case it is easily blocked) or shallow and wide (in which case it is hyper-proportional and also fails to draw its fair share of silt).   To overcome the defect of high flexibility, a roof block is fitted in the gullet of an open flume, at the vena contracta, clear of the water surface in the gullet when the outlet is drawing its full supply discharge.

This outlet can be tampered with by placing a thin wooden plank, fitting the throat, half way between the crest and the water level.   This increases the discharge if the outlet is working as a free fall one by about 16%.

5.8   ADJUSTABLE ORIFICE SEMI-MODULE (INDIA AND PAKISTAN) [1]

5.8.1   General

Adjustable orifice semi-module outlets are widely used in Punjab and Haryana and other parts of India and in Pakistan.

There are various forms of these outlets but the earliest of them is the one introduced by E.S. Crump in 1922 and called the "Adjustable Proportional Module" (APM). Crump's design aimed at fixing the crest at a setting of 0.6 of the supply depth in the parent canal, which ensured exact proportionality. However, according to past experience in Punjab with this APM canals fitted with it silted up badly. The problem was that it could not draw its fair share of silt, an essential requirement for those Punjab irrigation systems which draw their supplies from rivers. Thus the APM has now been replaced in India and Pakistan by the Adjustable Orifice Semi-Module (AOSM) which is neither proportional nor fully modular, but ensures fair distribution of silt. (See Figure 5-19).

5.8.2   Structural and Design Characteristics

Structurally, the AOSM (Figure 5-20) may be regarded as a long throated flume with a roof block capable of vertical adjustment in the upstream end of the parallel throat. It differs from the open flume outlet as regards the length of the throat and also in that the upstream water level in the supply canal is more than one third above the bottom of the roof block.

5.8.2.1   Upstream and downstream approaches

The upstream face wall or upstream wing wall on the supply canal is curved and flared, the curvature ending 7.5 cm upstream of the start of the crest. The downstream face wall, or downstream wing wall on the supply canal, is set forward inside the canal by a distance, which is generally equal to:

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[1] Based on a note prepared by A.D. Choudhry, Chief Engineer, Irrigation Works, Haryana, and K.C. Gupta, Executive Engineer, Central Designs, Irrigation Works, Haryana.

$$\frac{Q_2}{Q_1} \left(B_1 + \frac{y_1}{2} \right)$$

where Q_2 = discharge of outlet,

Q_1, B_1 and y_1 are respectively the discharge, bed width and depth of the supply canal section just upstream of the outlet.   Setting forward should only be done when the bed width of the canal is reduced below the outlet, keeping the downstream wing wall at the downstream end and the upstream wing wall at the upstream toe slope.
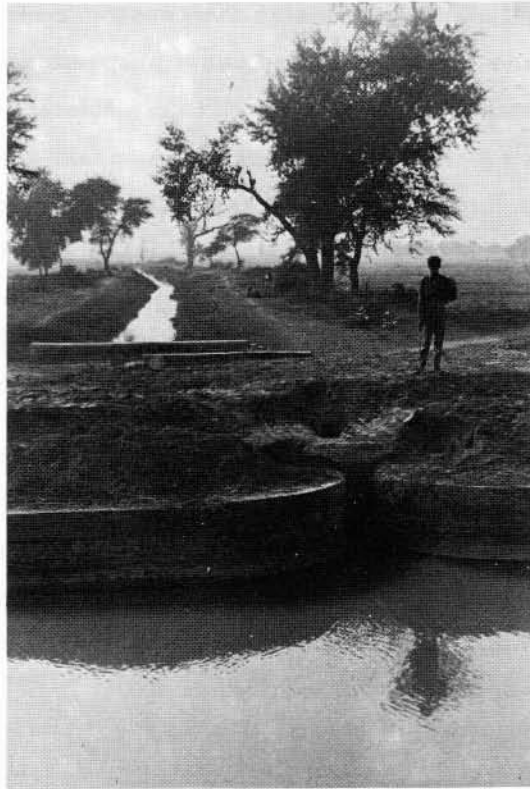
FIGURE 5-19. - General view of an AOSM outlet to a farm watercourse (Haryana, India).

The length of the parallel throat is 60 cm (2 ft) for all cases.   There is no horizontal crest portion of the throat and a glacis sloping at 1 in 15 starts right from the upstream end of the parallel sides of the throat.

Where bed width of channel is reduced below outlet downstream wing wall should be at downstream toe of slope and upstream wing wall at upstream toe of slope.

$R_2 = 2\,H_{(crt)} - \dfrac{y_1}{2}$

$R_1 = 2\,H_{(crt)}$

Bed width

Bank width

Brick masonry

Precast R.C. Roof block

$H_{(crt)}$ $H_{(sft)}$ $H_{(orf)}$

Detail of Roof Block

Bank

Section No. 1

12 cm less than in Section No. 2

Height to be kept according to $H_{(crt)}$

Section No. 2
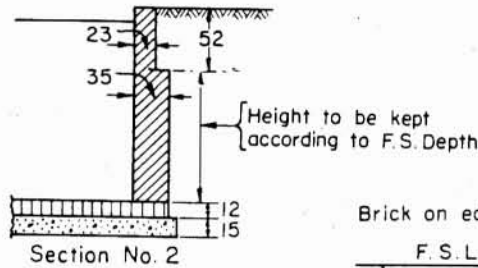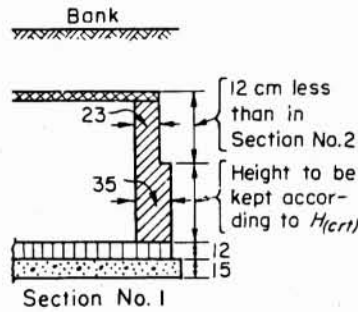
Sections when the full supply depth in the supply channel is above 60 cm.

Height to be kept according to F.S. Depth

Supply channel

No. 2   No. 1

7.5   1 in 10 $B_{(t)}$ + 30 cm

$B_{(t)}$ 90 to 120   90 to 120

Roof block

Plan

R = 65 cm   R = 30 cm   R = 30 cm

13   30   7.5

R.C. Roof Block

Bank   F.S.   Floor

Section No. 1   Section No. 2
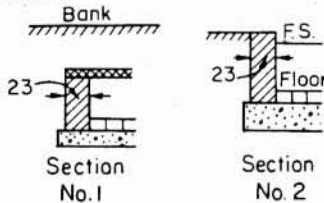
Sections when the full supply depth in the supply channel is below 60 cm.

Open for observation   Bank width   Precast R.C.C. Slab 7.5 cm thick

Brick on edge   Berm   35   23   Top should be in level with upstream wing wall

F.S.L.   Roof block $H_{(crt)}$

y_1 $2\,H_{(crt)} + 7.5$ $H_{(orf)}$ 150 y_2

22   15   35   50

Floor 12 cm over cement concrete 15 cm $R_3 = 2\,H_{(crt)}$ According to y_1 and bank width

Longitudinal Section
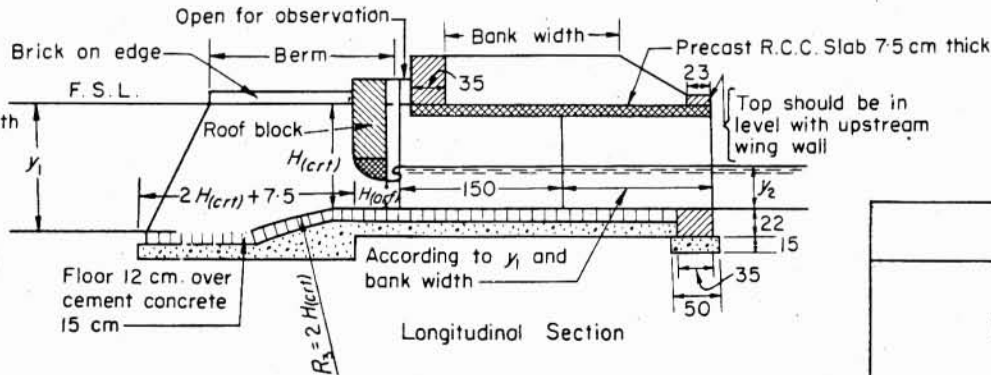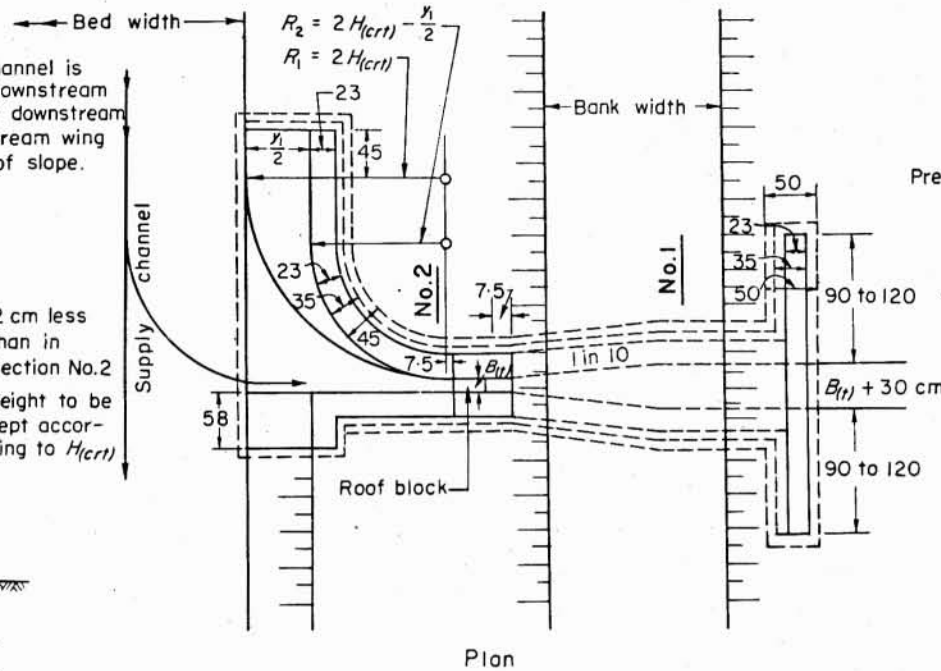
Where bed of watercourse is lower than floor level downstream curtain and wing walls to be taken down 30 cm below bed of watercourse and if necessary a cistern to be provided.

(All dimensions are in centimetres)
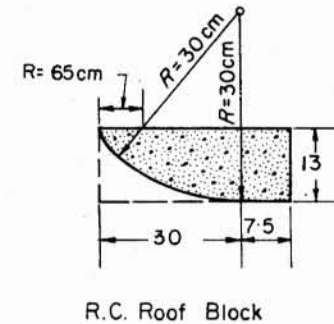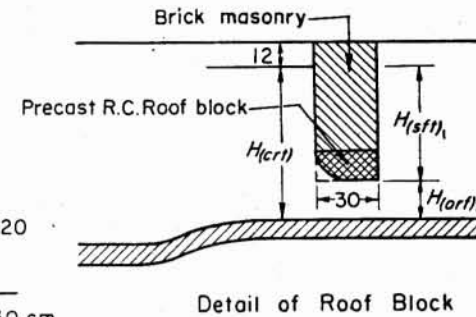
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PLAN OF ADJUSTABLE
ORIFICE SEMI-MODULE

Project, Region, Country
India and Pakistan

Figure No. 5-20
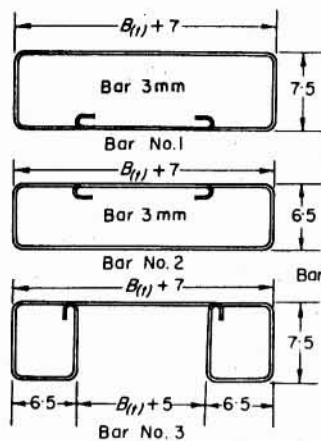
291

Details of precast R.C.Roof

Cross section on B-B
S.W.G. (Standard wire gauge)

Front elevation

Plan

Fixing of precast R.C. Roof block
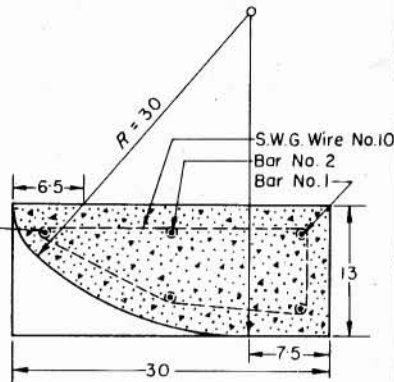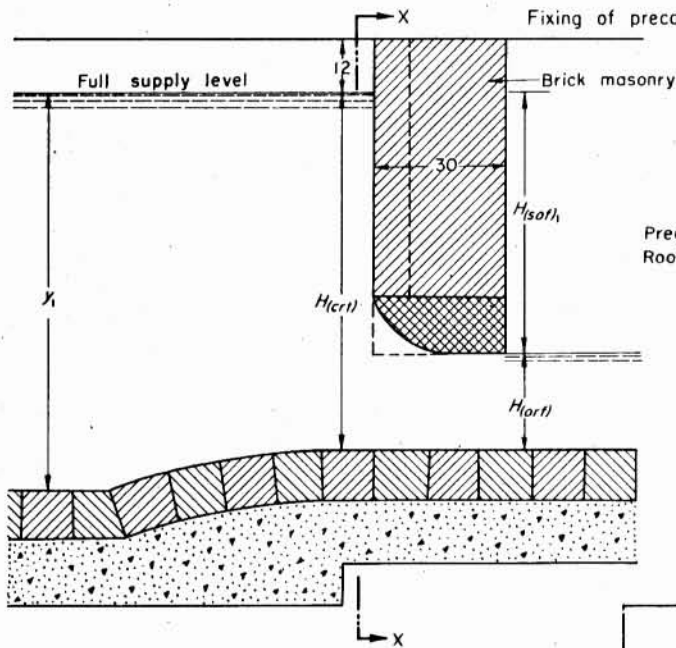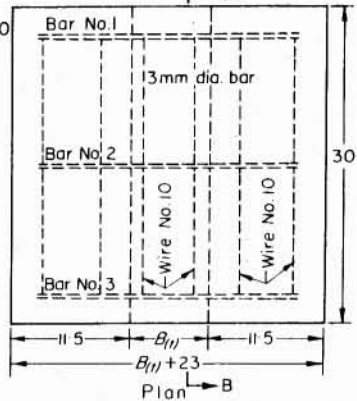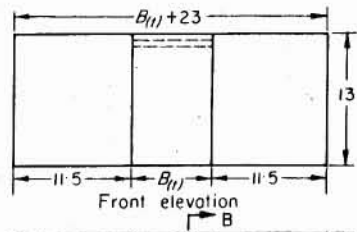
Longitudinal section through outlet

Note: All dimensions are in centimetres
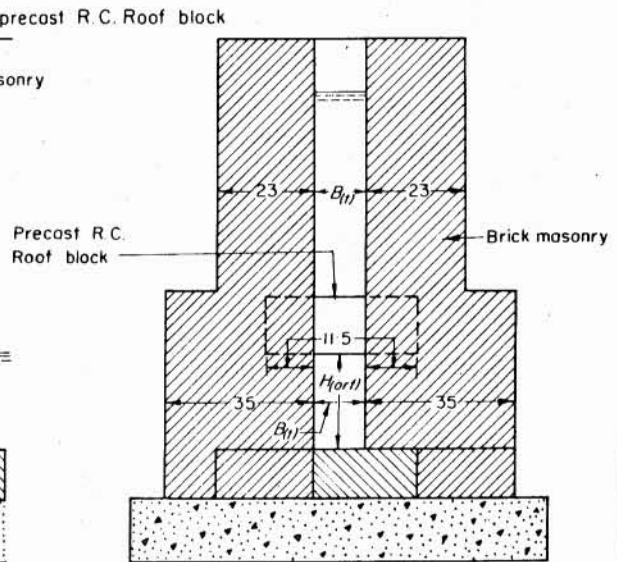
Cross section on X-X

FAO—ICID

ADJUSTABLE ORIFICE
SEMI—MODULE
DETAILS AND FIXING OF ROOF BLOCK

Project, Region, Country
India and Pakistan

Figure No. 5-21

5.8.2.2   <u>Roof block</u>

The roof block may be of cast iron but it is now generally of reinforced cement (see Figures 5-20 and 5-21). The face of the roof block is set 5 cm from the starting point of the parallel throat. It has a lamniscate curve at the bottom with a tilt of 1 in 7.5 in order to converge the water instead of a horizontal base which would diverge it. The cast iron roof block is 30 cm thick. The parallel throat has a cast iron bed and check plates.

5.8.2.3   <u>Side walls</u>

The side walls downstream of the throat are given a splay of 1 in 10, i.e. up to 150 cm (5 ft), after which they are straight up to a length depending on the bank width (Figure 5-20).

5.8.2.4   <u>Susceptibility to tampering</u>

The structure of the outlet is very strong and has a long serviceable life. However, cases of tampering with the outlet are not infrequent. The roof block is sometimes raised bodily and refixed but the tampering is easily detected. A wooden plank is sometimes inserted at the downstream side of the roof block and covered with earth and grass, thus forming an air-tight roof in continuation of the roof block. This increases the discharge due to imperfect aeration of the jet.

5.8.3   <u>Hydraulic Properties</u>

5.8.3.1   <u>Flexibility</u>

This type of outlet is instantaneously proportional when the bottom of the roof block is at 0.3 of the full supply depth of the supply canal. With a rise in the full supply level the flexibility, which is equal to $\dfrac{3y_1}{10\,H_{(crt)}}$, is reduced and the outlet becomes sub-proportional. Similarly with a fall in the full supply level the flexibility is increased and the outlet becomes hyper-proportional.

When the outlet is set near bed level, with any rise in the full supply level, the value of $\dfrac{y_1}{H_{(crt)}}$ falls and the outlet tends to move further from

proportionality in the direction of rigidity.   A fall in the full supply level similarly increases the flexibility and the outlet moves towards proportionality. With the outlet set at bed level, the flexibility remains constant at 0.3.

5.8.3.2   Silt-drawing capacity

This outlet draws at bed level about 14% and below bed level at 12/10th setting) about 29% more silt than it would draw at 6/10th setting when it is proportional.

5.8.3.3   Adjustability

The outlet is easily adjustable, at a small cost, either by raising or lowering the roof block or by dismantling one side wall.

5.8.3.4   Range of operation

The outlet can work semi-modularly for all heads and with all working heads above the minimum modular head and for all discharges from 28 l/s to 150 l/s (1 ft^3/s to 5 ft^3/s) - i.e. the discharge range generally required for outlets.

5.8.3.5   Suitability

This outlet is eminently suitable in head reaches of distributing canals.   In the case of distributing canals carrying silt, setting at bed levels is considered the best for silt-conduction into the outlet.   In the case of distributing canals receiving water free of silt, a setting at 0.6 y_1 is the best for proportionality.

The outlet can also be designed to draw proportional discharge from a distributing canal in which the discharge fluctuates from a certain minimum to full supply discharge.

5.8.4   Design Formula   (According to Crump)

$$Q_2 \text{ or } Q \quad = \quad C\,B_{(t)}\,H_{(crt)}\sqrt{H_{(sof)_1}}$$

where Q_2 or Q =   the discharge of the outlet in l/s

C =   0.0403

$B_{(t)}$ =   width of the throat in cm

$H_{(orf)}$ =   height of the orifice in cm

$H_{(sof)_1}$ =   the depression head or height of the full supply level in the supply canal above the bottom of the roof block   = $H_{(crt)}$ - $H_{(orf)}$

Also, minimum modular head = $h_{(wk)_{min}}$ = $0.82\ H_{(sof)_1}$ - $0.5\ B_{(t)}$

It should be noted that recent research carried out on the adjustable orifice semi-module in Pakistan has shown that the coefficient of discharge varies with the throat width of the structure, i.e. with the ratios $\dfrac{y_1}{H_{(crt)}}$ and $\dfrac{y_1}{B_{(t)}}$.

Consequently the structural shape of the outlet differs slightly from the design presented here. For further information reference may be made to the proceedings of the 7th NESA Irrigation Practices Seminar, Lahore, Pakistan, 1968.

To design an orifice semi-module, it is necessary first to make a trial calculation. Suitable value of $B_{(t)}$ and $H_{(crt)}$ are first assumed; the value of $H_{(orf)}$ and $h_{(wk)_{min}}$ are then calculated. $H_{(orf)}$ has to be less than half $H_{(crt)}$ to ensure the orifice flowing full. If $h_{(wk)_{min}}$, as calculated, is less than the available working head, the process has to be repeated with modified values of $H_{(crt)}$ or $B_{(t)}$. To save this "trial and error" arithmetical work, diagrams have been prepared (Figures 5-22 to 5-26) in which the relationships between outlet discharges and minimum working heads for different values of $H_{(crt)}$ and $H_{(orf)}$ are shown. There are separate diagrams for $B_{(t)}$ = 6 cm, 7.5 cm, 9 cm, 12 cm and 15 cm, which are the most commonly adopted.

5.8.5   Numerical Examples

Example 1

Design an orifice semi-module having a discharge of 60 l/s on a canal with full supply depth of 90 cm. Available working head is 54 cm.

Given Data

Q   =   60 l/s

y_1 =   90 cm

$$h_{(wk)} = 54 \text{ cm}$$

If it is desired that the orifice semi-module should be proportional, then

$$H_{(crt)} = 0.6\, y_1 = 0.6 \cdot 90 = 54 \text{ cm}.$$

An examination of the curves of Figure 5-22 shows that for $B_{(t)} = 6$ cm and $Q = 60$ 1/s, $H_{(crt)}$ must be more than 54 cm. Similarly with $B_{(t)} = 7.5$ cm and $B_{(t)} = 9.0$ cm vide Figures 5-23 and 5-24. For $B_{(t)} = 12$ cm (Figure 5-25) the intersection of $H_{(crt)} = 54$ cm and $Q = 60$ 1/s gives $H_{(orf)} = 21$ cm, $H_{(sof)_1} = 54 - 21 = 33$ cm, and $h_{(wk)min} = 20$ cm.

The setting being $0.6\, y_1$, the outlet is likely to draw its fair share of silt. For canals carrying clear water the setting at $0.6\, y_1$ is ideal for proportionality.

For rigidity and better silt drawing capacity, the outlet should be set at bed level, viz: $H_{(crt)} = 90$ cm. From Figure 5-25, with $B_{(t)} = 12$ cm for a discharge of 60 1/s, $H_{(orf)}$ should be 14.2 cm, but the minimum modular head is a little more than the available working head of 54 cm.

It is, therefore, necessary to reduce $H_{(sof)_1}$. An inspection of Figure 5-22 shows that with $B_{(t)} = 7.5$ cm and $H_{(crt)} = 90$ cm for a discharge of 60 1/s, $H_{(orf)}$ should be 24.5 cm and $h_{(wk)min} = 50$ cm which is less than the available working head.

Another suitable size is $H_{(crt)} = 90$ cm, $H_{(orf)} = 33.0$ cm and $B_{(t)} = 6$ cm (Figure 5-22) with $h_{(wk)min} = 45$ cm.

Should the working head be very low and no suitable size of orifice semi-module with a reasonably deep setting can be designed, it would be necessary to resort to another type of outlet such as a combined pipe and semi-module.

Example 2

An orifice semi-module is found to be working non-modularly during an inspection of the outlet. The actual working head on measurement is found to be 30 cm. How can the outlet be adjusted to give its design discharge?

<u>Given Data</u>

$Q = 60 \, 1/s$, $H_{(crt)} = 90$ cm, $H_{(orf)} = 29.5$ cm and $B_{(t)} = 6$ cm.

Figure 5-22 shows that for the given data, $h_{(wk)min}$ should b' 47 cm against only 30 cm actually available. The first immediate remedy is to clear silt from the watercourse if possible, and to increase the available head to 47 cm. If this is not possible, then the roof block may be raised so that the orifice working under submerged conditions gives the requisite discharge, which can be calculated roughly by adopting $C = 0.0354$ in the formula:

$$Q = C \, B_{(t)} \, H_{(orf)} \sqrt{h_{(wk)}}$$

If neither of the above two alternatives are possible, it is essential to provide a temporary outlet to supplement the discharge of the existing outlet.

The final remedy lies in redesigning the outlet with $H_{(crt)} = 75$ cm and $H_{(orf)} = 36.5$ cm which gives $h_{(wk)min} = 29.0$ cm. This would involve raising the crest of the outlet by dismantling one side wall.

5.8.6   <u>Summary</u>

The outlet is easily adjustable at a nominal cost. It is generally immune to tampering due to its cast iron or reinforced concrete roof block.

The discharge of the outlet is independent of the water level in the watercourse provided a standing wave forms.

The outlet is eminently suitable (provided a minimum working head is available) in head reaches of distributing canals, for drawing its fair share of silt.

It is automatic in operation. The gauge at the outlet is read once a month during routine inspection by the canal supervisor.

This outlet requires more working head than the open flume outlet and is not suitable for use at tails or immediately upstream of control points.

FAO - ICID

$(B_{(t)} = 7.5 cm)$

ADJUSTABLE ORIFICE SEMI-MODULE

Dotted lines denote minimum working
head required for modularity

Project, Region, Country
India and Pakistan

Figure No. 5-22

FAO - ICID

($B_{(t)}$ = 6 cm)

ADJUSTABLE ORIFICE SEMI-MODULE

Dotted lines denote minimum working
head required for modularity

Project, Region, Country
India and Pakistan

Figure No. 5-23

FAO - ICID

$(B_{(t)} = 9 \text{ cm})$

ADJUSTABLE ORIFICE SEMI-MODULE

Dotted lines denote minimum working
head required for modularity

Project, Region, Country
India and Pakistan

Figure No. 5-24

FAO – ICID

B(t) = 12 cm

ADJUSTABLE ORIFICE SEMI-MODULE

Dotted lines denote minimum working
head required for modularity

Project, Region, Country
India and Pakistan

Figure No. 5-25

FAO – ICID

$(B_{(t)} = 15 \text{ cm})$

ADJUSTABLE ORIFICE SEMI-MODULE

Dotted lines denote minimum working
head required for modularity

Project, Region, Country
India and Pakistan

Figure No. 5-26

Minimum setting $H_{(orf)} = \frac{1}{2} H_{(crt)}$

$H_{(crt)}, \text{ cm}$

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5.9   JAMRAO TYPE ORIFICE SEMI-MODULE (SIND, PAKISTAN)

5.9.1   General

The Jamrao Type Orifice Semi-Module (Figure 5-27) has been widely used on the Jamrao Canal in Sind in Pakistan.

The outlet was evolved in Sind by Kirkpatrick in 1925. It has proved to be a successful masonry replacement for the Kennedy Gauge Outlet. (See 5.1.2.)

5.9.2   Structural Characteristics

The upstream approach of the outlet is only 0.6 m (2 ft) long and is shaped like a truncated square pyramid with a convergence of 1 to 4.   The control is a square orifice in an angle-iron frame.   The downstream flume, 3.0 m (10 ft) long, has a horizontal floor with the side walls at a splay of $\frac{B_{(t)}}{300}$ where $B_{(t)}$ is in cm.   The horizontal floor is then sloped down to meet the bed of the watercourse.

Kirkpatrick has stated that the coefficient of discharge for this converging orifice is as nearly constant as that of the elongated bell-mouth and that his design gives the best results as regards the maximum recovery of head.

The most essential feature of this semi-module is the introduction of baffles in the downstream flume, which work as a roof sloping gradually upwards, and through their presence recover considerable head.   At the same time the outlet discharges under free atmospheric conditions.   The optimum slope for the roof (i.e. lower edge of the baffles) is 1 in 15.   The optimum number of baffles is nine, of which the first six are equally spaced, and the last three somewhat spread out.   To protect the baffles and the angle-iron frame from outside interference, an expanded metal sheet is fixed as shown in Figure 5-27.

The outlet works proportionately when the centre of the orifice below the full supply level, $H_{(cnt)}$, is at 0.3 of the depth in the parent canal.

The outlet is very durable and has a long serviceable life.   It is immune from tampering because of the expanded metal protection and angle-iron frame of

the controlling section.   It is automatic in operation and there are no mainten-
ance problems.
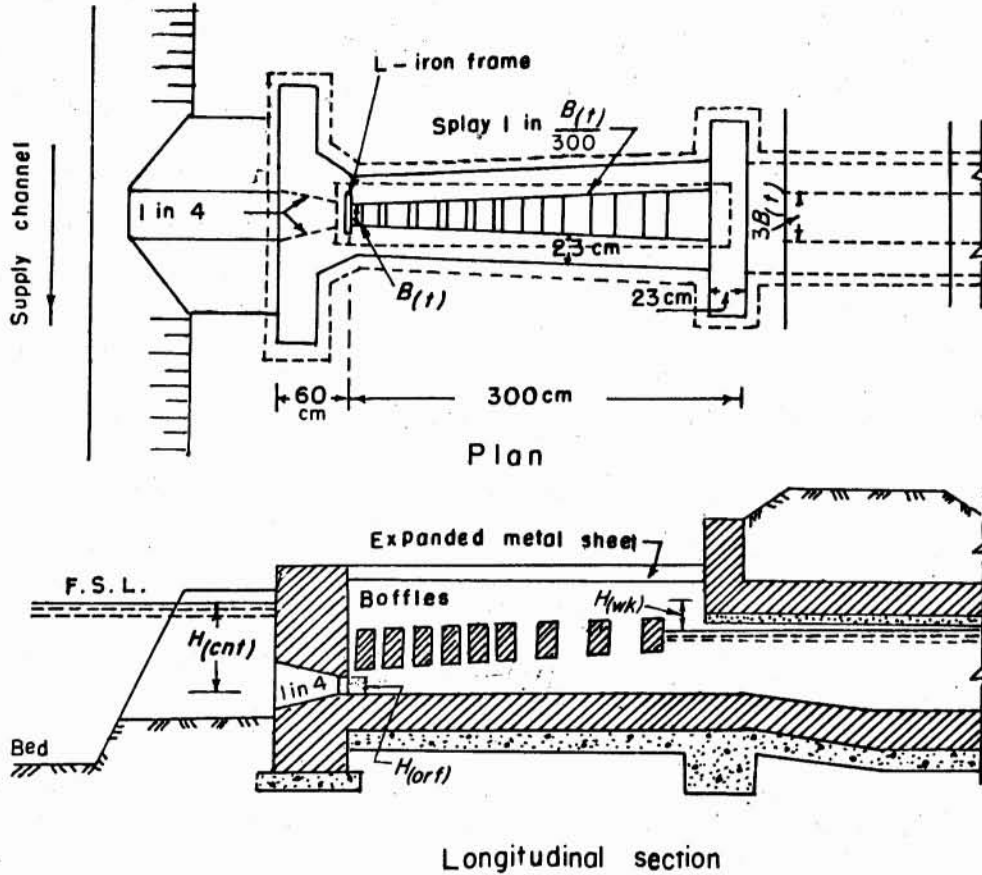
FIGURE 5-27. - Jamrao type orifice semi-module.

5.9.3   Hydraulic Characteristics

This orifice type module with $H_{(cnt)}$ less than 0.45 m (1.5 ft) is
susceptible to upstream variation but as it works proportionately at 0.3 y_1 of
the full supply level, it can be used on canals with depths greater than 1.37 m

(4.5 ft) so as to give $H_{(cnt)}$ of 0.45 m (1,5 ft) at least.

This module requires more working head than an open flume type for the same $H_{(crt)}$ as $H_{(cnt)}$ in this device. Consequently it can be employed where a greater working head is available. This generally happens at heads of large distributaries, and so these semi-modules are used in head reaches of large canals.

The baffles introduced in this type of device help recovery of considerable head. For example, when the centre of the orifice below full supply level is 0.90 m the working head without baffles is 0.52 m, but with 9 baffles for a 12 cm square orifice the working head is only 0.27 m. This reduction is due to the fact that water between the baffles automatically applies the correct pressure to secure a roof which suppresses the standing wave.

The capacity to draw silt will depend on the setting, and in this respect it may be considered similar to the Adjustable Orifice Semi-Module except that the upstream approaches of the AOSM are likely to be more conducive to silt induction than the truncated square pyramid set back in the bank.

5.9.4   Design Formula

The discharge formula applicable to the Jamrao Type Orifice Module was determined experimentally and is

$$Q = 0.0397 \; B_{(t)}^2 \; \sqrt{H_{(cnt)}}$$

5.9.5   Numerical Example

Design a Jamrao Type Orifice Semi-Module with a discharge of 60 l/s, on a canal with a full supply depth of 200 cm. The available working head is 20 cm.

Q = 60 l/s

y_1 = 200 cm

$h_{(wk)}$ = 20 cm

For proportionality $H_{(cnt)}$ = 0.3 . 200 = 60 cm.

By trial and error[1] it is found that for $H_{(cnt)}$ = 60 cm, and an orifice of 14 cm x 14 cm, the discharge is 60.37 l/s, which is satisfactory.

$$h_{(wk)} \text{ for 6 baffles } = \frac{H_{(cnt)}}{3.8} = \frac{60}{3.8} = 16 \text{ cm}$$

$$h_{(wk)} \text{ for 9 baffles } = \frac{H_{(cnt)}}{4.55} = 13 \text{ cm}$$

$\therefore$ use six baffles.

5.10   PIPE SEMI-MODULE (INDIA AND PAKISTAN) [2]

5.10.1   General

The Pipe Semi-Module is widely used in Haryana and Punjab (India) and in Pakistan.   The outlet may be regarded as a development of the Stoddard-Harvey improved irrigation outlet.   This outlet is eminently suitable when the supply canal has wide banks and/or is in high filling because an open flume or an orifice semi-module built in such a bank would be much more expensive. This type of device is also used in lined canals.   Further, the outlet is suitable for drawing its share of silt when it is not possible to achieve a deep setting as required by an open flume or an orifice semi-module.   The lead-in pipe is set at or near the bed level and it opens into a tank on the downstream side to which an open flume or an orifice semi-module (Punjab or Jamrao type) or a Scratchley outlet (with free flow conditions) is fitted.

5.10.2   Structural Characteristics

The outlet (Figure 5-28) consists of a lead-in pipe from the supply canal which discharges into a tank on the outer side of the bank of the supply canal. The upstream end of the lead-in pipe can be placed at any suitable level in the supply canal depending upon the desired silt-draw.   The downstream end of the

[1] Tables are available in (105) and other references.

[2] Based on a contribution by A.D. Chaudhry, Chief Engineer Irrigation Works, Haryana, and K.C. Gupta, Executive Engineer Central Designs, Irrigation Works, Haryana (India).
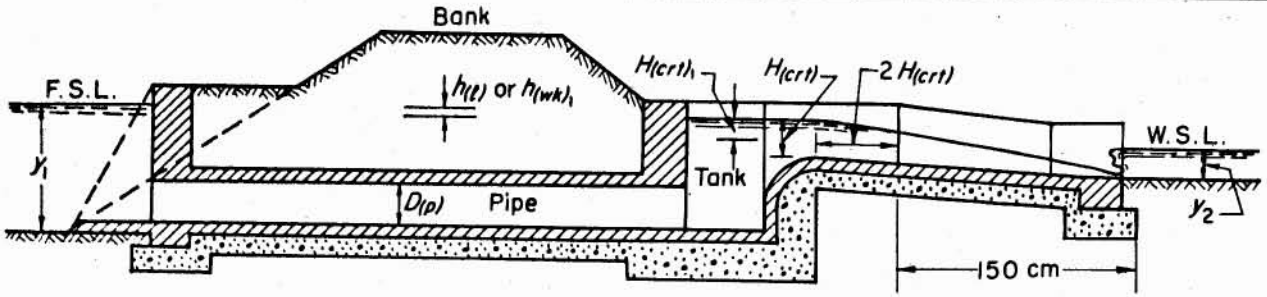
Pipe – cum – Open Flume
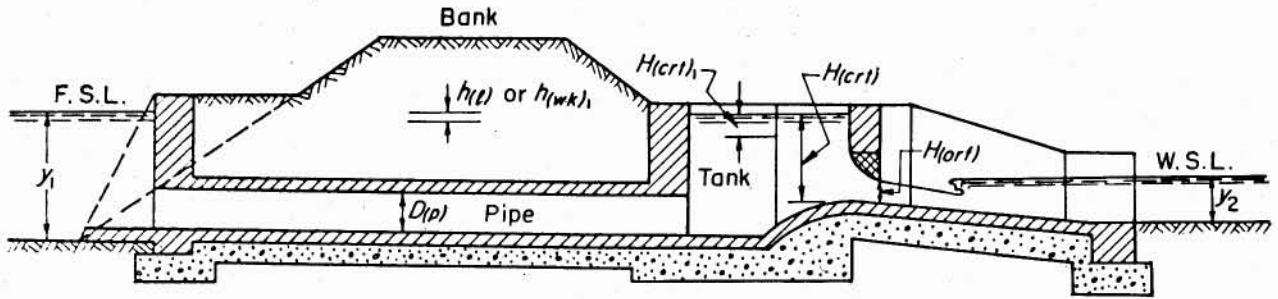
Pipe – cum – A.O.S.M.
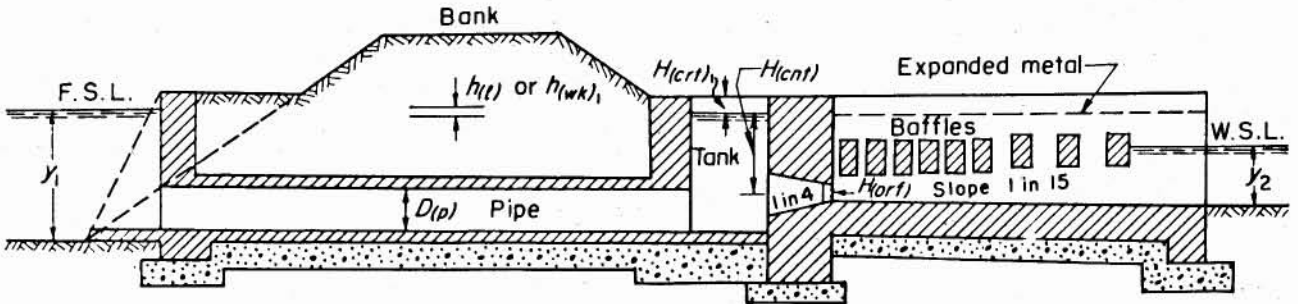
Pipe – cum – Jamrao Type Orifice – Semi – Module
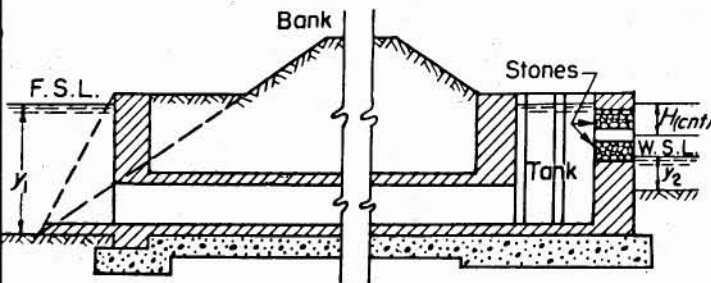
Pipe – cum – Free Fall Scratchley Outlet

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PIPE SEMI-MODULE
FOUR EXAMPLES
Project, Region, Country India and Pakiston
Figure No. 5-28

lead-in pipe may be horizontal or given an upward slope of about 1 to 12 to reduce the depth of the tank. The tank is approximately 60 cm square for a 30 l/s discharge, 80 cm square for 60 l/s and 100 cm square for 90 l/s discharge. In the downstream wall of the tank a semi-module is fitted, four examples of which are shown in Figure 5-28.

5.10.3   Hydraulic Characteristics

5.10.3.1   Flexibility

The flexibility of this outlet depends on the flexibility of the semi-module incorporated with it except that its flexibility will be modified slightly by the action of the pipe between the supply canal and the tank. The flexibility may be expressed thus:

Pipe with open flume
$$\dfrac{y_1}{\dfrac{10}{3} h_{(l)} + \dfrac{10}{9} H_{(crt)}}$$

Pipe with Jamrao-type orifice semi-module
$$\dfrac{y_1}{\dfrac{10}{3}\left[h_{(l)} + H_{(crt)} \right]}$$

Pipe with adjustable orifice semi-module
$$\dfrac{y_1}{\dfrac{10}{3}\left[H_{(crt)} + h_{(l)} \right]}$$

In actual practice, to obtain maximum possible rigidity, a pipe with an open flume is used when the working head available is small, and a pipe with an orifice semi-module when the working head is somewhat greater.

5.10.3.2   Efficiency

The efficiency of this outlet depends on the type of semi-module fixed to it and on the loss of working head through the pipe. As the amount of silt induction into the outlet depends only on the position of the pipe with respect to the bed, the crest of the semi-module can be placed at any level so that the loss in head through the pipe can be more than compensated for by a higher setting of the outlet. The position of the pipe does not affect either the discharge or the proportionality and the pipe can be raised or lowered depending on the silt-draw requirements.

5.10.4 Design Formula

The size of the lead-in pipe is fixed so as to achieve a minimum loss of head, subject of course to obtaining sufficient velocity in the pipe to convey silt from the supply canal.

Various sizes of lead-in pipe for various discharges are given below.

Discharge (l/s)	Pipe size	
	Width (cm)	Height (cm)
Up to 35	30.5	30.5
36-40	36.5	30.5
41-45	43.0	30.5
46-50	46.0	30.5
51-55	46.0	38.0
56-80	46.0	46.0

Let $h_{(\ell)}$ be loss of head

Then $h_{(\ell)}$ (or $h_{(wk)_1}$) $= 926 \dfrac{Q^2}{A_x^2}$

where A_x = cross section of the pipe in cm^2

Q = discharge in l/s.

From this the water level in the tank can be determined as the FSL in the supply canal minus the loss of head through the pipe. The semi-modular outlet (open flume, orifice semi-module, Scratchley etc.) can now be designed, the type depending on the head available and other conditions.

5.10.5 Numerical Example

Design a pipe semi-module for the following data:

Q = 42 l/s

y_1 = 120 cm

$h_{(wk)}$ = 20 cm

Assume any standard size of pipe, say 30 cm diameter.

Then $\quad h_{(wk)_1} \quad = \quad 926\ \dfrac{Q^2}{A_x^2} \quad = \quad \dfrac{926\ .\ 42^2}{0.7854^2\ .\ 900^2} \quad = \quad 3\ cm$

$\therefore$ The working head left for the semi-module is 17 cm.

If $h_{(wk)_2} = 17$ cm, the maximum head $(H_{(crt)})$ for the open flume outlet can be 85 cm. For Q = 42 l/s, $B_{(t)}$ = 6 cm, $H_{(crt)}$ = 57 cm; $h_{(wk)_{min}}$ for this size is 12 cm against 17 cm available.

In the above example, instead of an open flume, an orifice semi-module could also be designed with the following dimensions:

AOSM	Jamrao type orifice semi-module
$B_{(t)}$ = 12 cm	$B_{(t)}$ = 12 cm
$H_{(crt)}$ = 42 cm	
$H_{(orf)}$ = 17.5 cm	$H_{(cnt)}$ = 54 cm
$h_{(wk)_2}$ = 14 cm	$h_{(wk)_2}$ = 14 cm

5.10.6   Summary

The outlet is very durable and has a long serviceable life. It has a high degree of immunity from interference due to the certainty of early detection, and it has a wide range of modularity. The working head required is low, particularly with an open flume outlet attached. It is less costly than the open flume or orifice semi-module outlets built in the bank of wide supply canals. It can be conveniently adjusted while the supply canal is running. This type of outlet can be used with advantage if it is required to work under conditions of high supply. The pipe can be placed with its sill above the low supply level and since the head over the crest is measured in the tank, the design of the outlet is simple. No water will enter the tank until the water level in the supply canal rises above the low supply level.

5.11 FAYOUM STANDARD WEIR FARM OUTLET (ARAB REPUBLIC OF EGYPT) [1]

5.11.1 General

The Fayoum Standard Weir Farm Outlet delivers water from distributing irrigation canals to an irrigator or a group of irrigators. It is a simple accurate device for both measuring and controlling water and is widely used in the Fayoum province of the Arab Republic of Egypt, where the slope of the irrigated area is steep enough to permit the use of weir outlets. It can be used either as a clear overfall weir [2] or as a submerged weir [3]. The irrigation system in the Fayoum province is of great antiquity and the weirs on it are, as originally constructed, of rough masonry; they have now reached a more or less standard section as described in 5.11.2.·

Much research work has been carried out in the past on this weir, taking into consideration the following parameters:

- width of weir, $B_{(t)}$

- height of weir, $H_{(b-c)}$

- length of crest, $L_{(crt)}$

- upstream corners

- velocity of approach, $v_{(app)}$

- aeration.

For example, 500 experiments were carried out on the weir in the Delta Barrage experimental tank by A.D. Butcher in 1920 and more investigations have been made subsequently on a model at the Hydraulic Research and Experiment

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[1] Based on information supplied by A.A. Eldarwish, Inspector General of Irrigation & Director General Hydraulic Research & Experiment Station, Delta Barrage, and M. Kotb Nadar, Deputy Director, HRES Arab Republic of Egypt.

[2] A weir is described as a clear overfall when the downstream water level is below the crest of the weir.

[3] A weir is described as submerged when the downstream water level is above the crest of the weir.

Station in recent years.   As a result of these experiments, the discharge formula
has been deduced.   It has been found that the average variation of an individual
experiment from the corresponding deduced formula is below three per cent.   For
practical purposes, this deviation is acceptable.   Further, the experimental
results have led to the following conclusions:

- by correctly adjusting the radius of the corners to the width of the weir
  a series of weirs of widths from 0.01 m upwards may be constructed, for
  which the discharge per metre width is the same for all widths.

- the discharge can be determined accurately from the upstream gauge with
  a probable error of less than one per cent.

- the velocity of approach may be neglected in practice.

- the downstream conditions (wing walls, etc.) do not affect discharge,
  provided that the downstream slope of the weir is about half to one.

5.11.2   Structural Design

The standard Fayoum type weir (Figure 5-29) is essentially a very simple
construction of rough masonry.   It consists of a masonry wall 0.5 m thick at the
crest, vertical on the upstream face, and with a 1: 0.5 slope on the downstream
side.   The crest is usually of dressed stone and the width of the weir is defined
by blocks of masonry on the crest itself.   Sometimes pre-cast concrete blocks
with a smooth surface are used.   In this way the width of the weir can be
adjusted to a very high degree of accuracy.   The standard height has been
adopted at 0.65 m which, with a full supply depth 0.5 m over the weir, will
give a depth of 1.15 m in the canal, which is believed to be about the mean depth
of Fayoum small canals.   Variations from this depth will inevitably occur, but
this will not result in errors of more than 2%.

The total discharge of any outlet is usually known from the cultivated area
it serves and the crop water duty.   According to the depth of water in the canal
and land levels, the water head (over the crest level) can be easily determined.
As the water head is fixed and the discharge is known, the width of the weir (i.e.
the defined width between the blocks) can be calculated either from the formulae
((1) or (2)) given under 5.11.4, or it can be taken directly from Table 5-1.

Section A-A

Plan
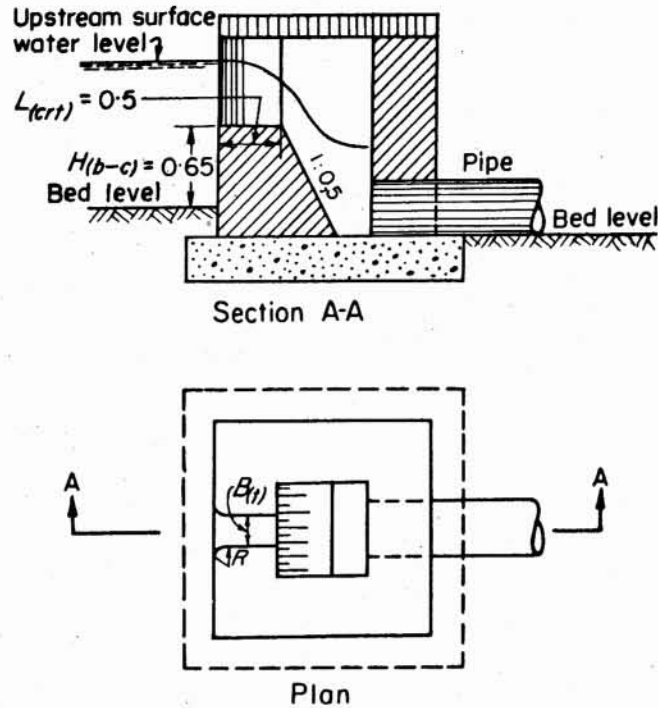
FIGURE 5-29. - Fayoum standard weir farm outlet; general structural design.

The field outlet weirs feeding several farm ditches are usually situated in groups, and called "nasbas" (Figure 5-30); the crest of all the weirs in the group are at the same level and the width of each weir is varied to give the required discharge.

The flowing water, after passing the weir crest, is conveyed to the irrigated land through open field laterals. If there are banks, the flow may first pass through a culvert, such as a pipe or a brickwork arch.

5.11.3   Hydraulic Characteristics

The structure can be used either as a controlling device or for water distribution.

Section A-A

Field outlet

Controlling weir

Field outlet

Plan

Section B-B
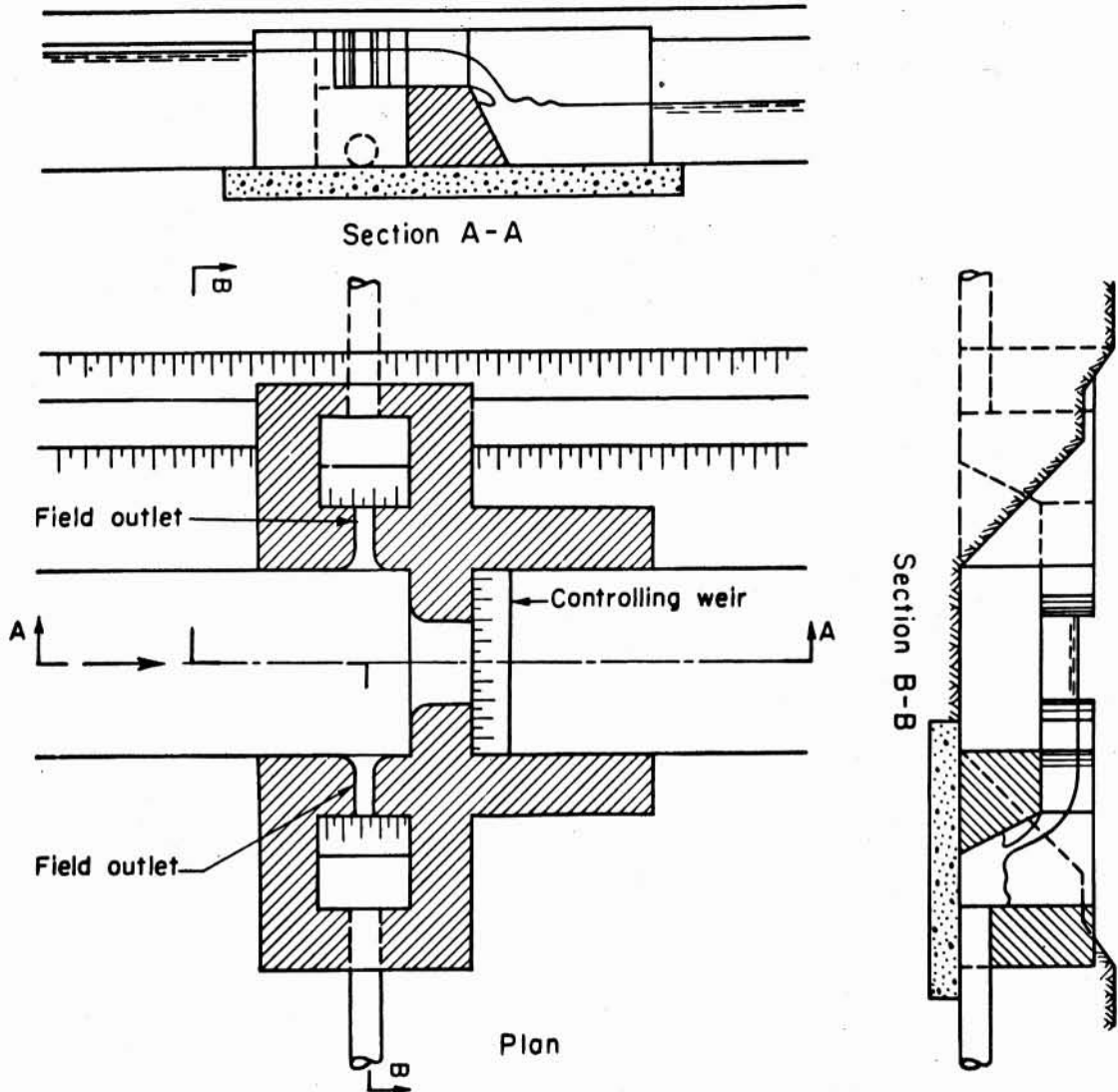
FIGURE 5-30. - Fayoum standard weir farm outlet with a group of field outlet weirs or "nasbas".

It has been demonstrated that the discharge per metre width can be the same for all widths of weir provided the radius of the upstream corner is correctly adjusted to the weir width. Weirs so designed and with their crests 0.65 m above the upstream bed are referred to as standard weirs. The discharges resulting from various depths of water over the weir are shown in Table 5-1.
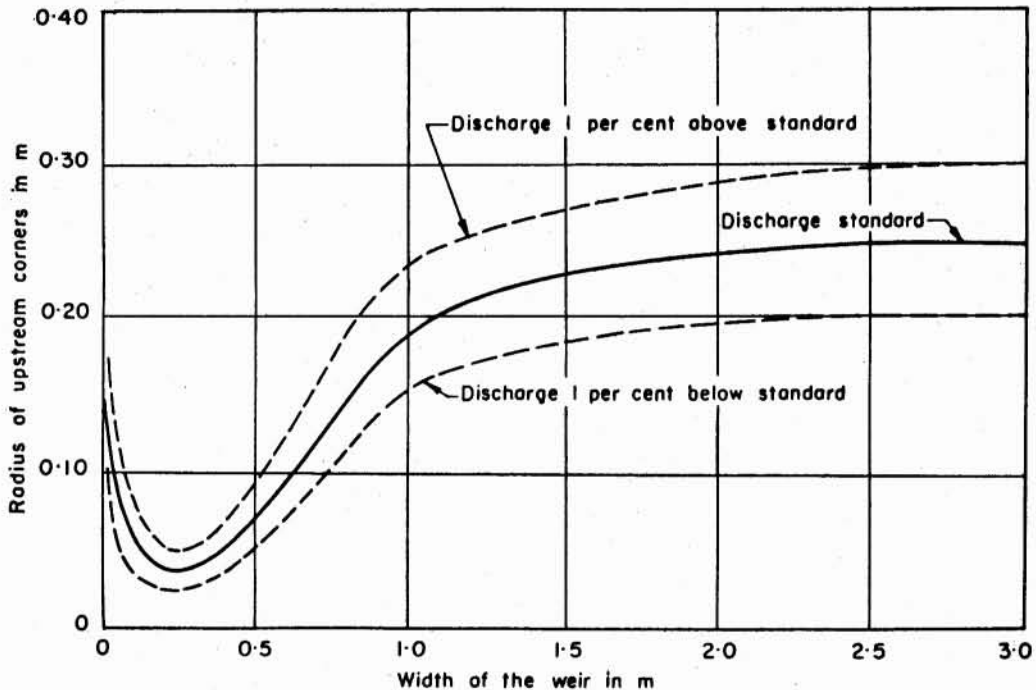
FIGURE 5-31. - Fayoum standard weir farm outlet.
Relation of upstream corners to width of weir.

The correct relation of the radius of the upstream corners to the width of the weir is shown in Figure 5-31. It can be seen from Figure 5-31 that the radius of the corners necessary to give standard discharge decreases with the width of the weir until a width of about 0.2 m is reached. Thereafter, the radius of the corners again increases rapidly. This increase in the radius for small widths is probably due to the increasing action of friction on the sides of

the weir. This friction becomes relatively more important when the weir is narrow, and it is probable that below a width of 0.2 m the discharge depends largely on the degree of roughness of the material used. Above a width of 3 m the action of the corners will be relatively small and a radius of 0.25 m will probably be correct for all wider weirs. This radius is that adopted for all large weirs in Fayoum.

5.11.3.1   Accuracy

The Fayoum weir type is considered to be a very accurate field outlet.

5.11.3.2   Silting

As the standard weir height is 0.65 m above the bed level, it is possible that silting may occur. However, because all crest levels are the same in the "nasba", the discharge passing through each outlet will not be affected at all.

5.11.3.3   Range of operation

The Fayoum weir can work for all heads starting from 0.01 m to 1.00 m, and for all widths from 0.01 m to 10 m.

5.11.4   Design Formulae

5.11.4.1   Clear overfall weir

In general, the method used to derive the standard formula is to plot the logarithm of the discharge against the logarithm of the depth of water on the weir for the standard types of weir and to draw a mean curve through the resulting points.

This curve is found to be best presented by a straight line between $H_{(crt)} = 0$ m and $H_{(crt)} = 0.14$ m and a slightly curved line above $H_{(crt)} = 0.14$ m, the corresponding formula being:

From $H_{(crt)} = 0$ m to 0.14 m $q = 1.652 \, H_{(crt)}^{1.54}$ (1)

From $H_{(crt)} = 0.14$ to 1.00 m $q = 1.956 \, H_{(crt)}^{1.72} + 0.014$ (2)

where $H_{(crt)}$ = depth of upstream water level on the weir, and

q = discharge per m^3/s per metre width.

The two formulae overlap in the neighbourhood of $H_{(crt)}$ = 0.14 m and the figures in Table 5-1 are calculated direct from them.

5.11.4.2   Submerged weir

(i) Depression head. The level of the water upstream of the weir measured from the weir crest level is called the depression head (symbol $H_{(crt)}$). The term is usually applied only to a weir working as a clear overfall and for any particular width of weir the discharge Q depends only on $H_{(crt)}$, but it has been used here also for a submerged weir to bring out the equivalent head for a submerged weir for different degrees of submergence in terms of $H_{(crt)}$.

(ii) Depth of submergence. When a weir is submerged the depth of water on the upstream side is referred to simply as the "upstream depth" (symbol $H_{(crt)}$) and the depth of water on the downstream side, measured beyond the region of disturbance, as the "depth of submergence", (symbol $H_{(s)}$).

(iii) Percentage submergence. It is often convenient to express the depth of submergence '$H_{(s)}$' as a percentage of the upstream depth '$H_{(crt)}$' and to call it briefly the percentage submergence '$r_{(s)}$'.

Studies on 1 : 12.5 scale models for submerged conditions have been conducted at the Delta Barrage. The weirs were all of standard shape without wing walls but varied in width from 0.20 m (2.50 m) to 0.01 m (0.125 m) and were tested at all depths from 0.024 m (0.30 m) to 0.08 m (1.00 m).[1]

In practice, two cases of the use of submerged weirs must be distinguished:

(a) a submerged weir fed from a large pool (unaffected by changes in the discharge over the weir);

(b) a submerged weir set in a canal (so that the whole discharge has to pass over the weir).

[1] The figures in brackets would be the corresponding full-size dimensions.

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THE FAYOUM STANDARD WEIR
FARM OUTLET

RELATIONSHIP BETWEEN $H_{(crt)}$ AND
$H_{(s)}$ FOR ALL VALUES OF $H_{(crt)eqv}$ OR OF Q

Project, Region, Country
Fayoum Province, A.R.E.

Figure No. 5-32
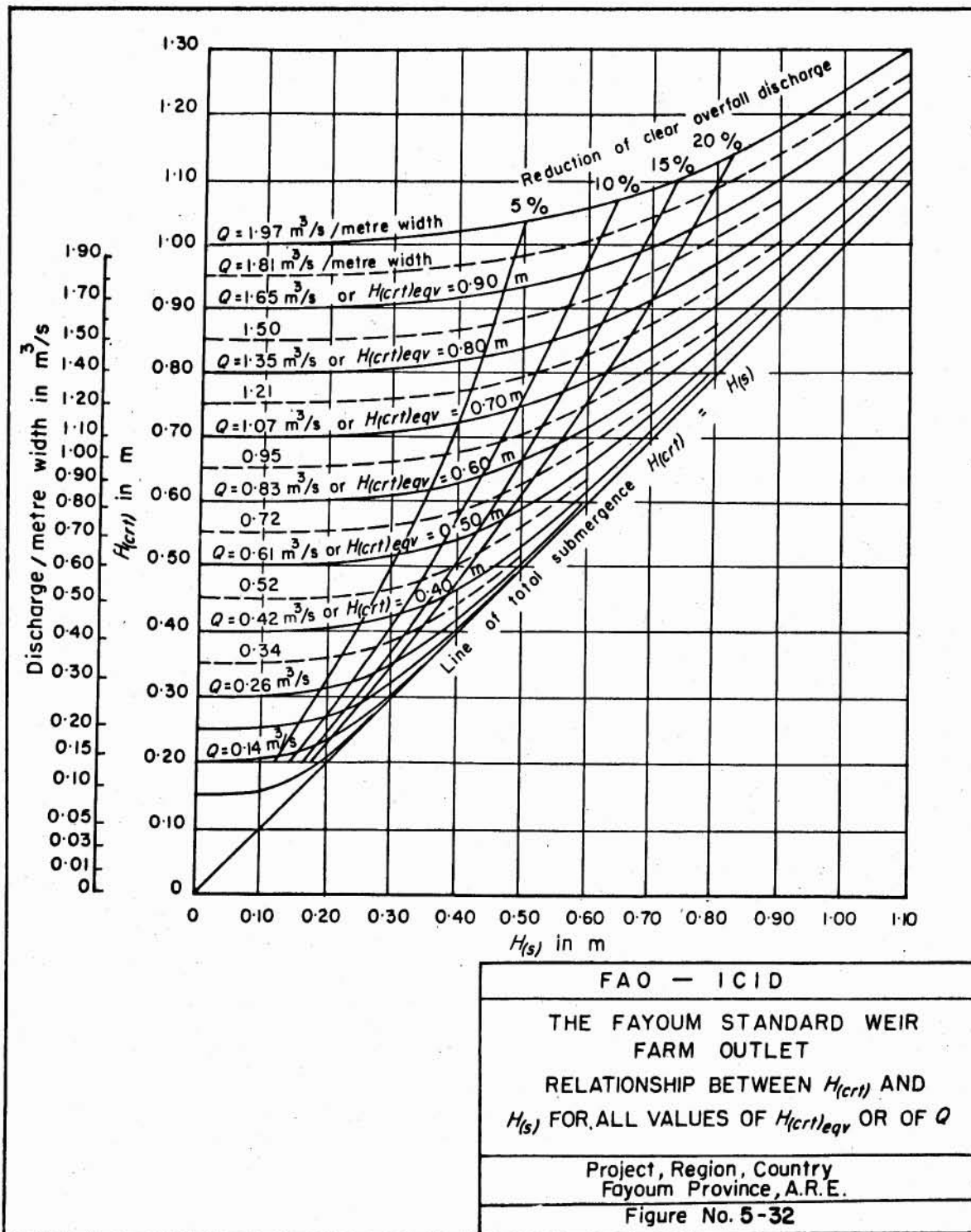
TABLE 5-1

Discharge of Standard Weirs per Metre Width

$H_{(crt)}$ = 0.00 to 0.14 m.   Discharge = 1.652 $H_{(crt)}^{1.54}$

$H_{(crt)}$ = 0.14 to 1.00 m.   Discharge = 1.9555 $H_{(crt)}^{1.72}$ + 0.014

$H_{(crt)}$ m	Discharge m^3/s	$H_{(crt)}$ m	Discharge m^3/s	$H_{(crt)}$ m	Discharge m^3/s	$H_{(crt)}$ m	Discharge m^3/s
0.00	0.0	0.26	0.2068	0.52	0.6490	0.78	1.2894
0.01	0.0014	0.27	0.2197	0.53	0.6702	0.79	1.3177
0.02	0.0040	0.28	0.2330	0.54	0.6916	0.80	1.3462
0.03	0.0075	0.29	0.2466	0.55	0.7133	0.81	1.3750
0.04	0.0116	0.30	0.2606	0.56	0.7353	0.82	1.4040
0.05	0.0164	0.31	0.2740	0.57	0.7576	0.83	1.4333
0.06	0.0217	0.32	0.2895	0.58	0.7802	0.84	1.4628
0.07	0.0275	0.33	0.3045	0.59	0.8031	0.85	1.4926
0.08	0.0338	0.34	0.3198	0.60	0.8262	0.86	1.5227
0.09	0.0405	0.35	0.3354	0.61	0.8497	0.87	1.5530
0.10	0.0476	0.36	0.3514	0.62	0.8734	0.88	1.5835
0.11	0.0552	0.37	0.3676	0.63	0.8973	0.89	1.6143
0.12	0.0631	0.38	0.3842	0.64	0.9216	0.90	1.6454
0.13	0.0714	0.39	0.4012	0.65	0.9461	0.91	1.6767
0.14	0.0800	0.40	0.4184	0.66	0.9709	0.92	1.7082
0.15	0.0888	0.41	0.4359	0.67	0.9960	0.93	1.7400
0.16	0.0976	0.42	0.4538	0.68	1.0213	0.94	1.7721
0.17	0.1068	0.43	0.4720	0.69	1.0469	0.95	1.8044
0.18	0.1164	0.44	0.4904	0.70	1.0728	0.96	1.8369
0.19	0.1264	0.45	0.5092	0.71	1.0990	0.97	1.8697
0.20	0.1368	0.46	0.5283	0.72	1.1254	0.98	1.9027
0.21	0.1475	0.47	0.5477	0.73	1.1521	0.99	1.9360
0.22	0.1586	0.48	0.5673	0.74	1.1790	1.00	1.9695
0.23	0.1701	0.49	0.5873	0.75	1.2062	1.10	2.3179
0.24	0.1820	0.50	0.6076	0.76	1.2337	1.20	2.6898
0.25	0.1942	0.51	0.6282	0.77	1.2614	1.30	3.0847

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In case (a) the effect of gradually submerging a clear overfall weir
will be to momentarily reduce discharge, but ultimately to raise the upstream
level, the discharge returning to its original value.

In case (b) the effect of submergence will be to permanently reduce
discharge, the upstream level remaining constant. In order to ascertain what
reduction in the discharge of a clear overfall weir will be caused by a definite
degree of submergence reference may be made to Figure 5-32 or to Table 5-2.

TABLE   5-2

Depth as clear overfall m	Reduction in discharge				
	5%	10%	15%	20%	
0.20	60	72	81	86	
0.30	60	72	81	86	
0.40	60	72	80	85	
0.50	59	71	77	83	Percentage sub-mergence causing reduction in discharge
0.60	58	68	75	81	
0.70	55	67	74	79	
0.80	53	65	72	78	
0.90	51	63	70	76	
1.00	49	61	68	74	

5.11.5   Numerical Examples

Example 1 - clear overfall weir

Design a field outlet (clear overfall weir) with a discharge of 0.25
m^3/s on a canal with a full supply depth of 90 cm. The available working head of
the outlet is 25 cm.

Formula (2) of section 5.11.4 is applicable

$$q = 1.956 \, H_{(crt)}^{1.72} + 0.014 = 0.194 \, m^3/s \text{ per metre width}$$

From Table 5-1, q for $H_{(crt)}$ =   0.25   can be read off

                                   = $0.1941 \text{ m}^3/\text{s}$ per metre width

As the total discharge   = $0.25 \text{ m}^3/\text{s}$

     the breadth of the weir should be   = $\dfrac{0.25}{0.194}$

                                   =   1.289 m.

Example 2 - submerged weir

(a) What will be the required width of a submerged weir carrying a discharge of $4.00 \text{ m}^3/\text{s}$ having upstream and downstream water depth of 0.93 m and 0.73 m respectively?

From Figure 5-32 the depression head $H_{(crt)eqv}$ can be determined for the upstream and downstream water depths.

For $H_{(crt)}$ =   0.93 m and $H_{(s)}$ =   0.73 m

$H_{(crt)eqv}$ =   0.80 m   (Figure 5-32)

This gives an actual discharge of $1.346 \text{ m}^3/\text{s}$ per metre width.

As the total discharge   = $4.00 \text{ m}^3/\text{s}$

     the width of this submerged weir   = $\dfrac{4.00}{1.346}$

                                   =   3 m.

(b) What will be the reduction in the discharge of a weir 1.00 m wide having an upstream head of 0.50 m, if it is submerged to a depth of 0.40 m?

From Figure 5-32, as $H_{(crt)}$ = 0.5 m and $H_{(s)}$ = 0.4 m, $H_{(crt)eqv}$ = 0.45 which gives a discharge of $0.509 \text{ m}^3/\text{s}$ per metre width.

For a clear overflow weir of $H_{(crt)}$ = 0.5 m, the discharge

     (Table 5-1)   = $0.608 \text{ m}^3/\text{s}$ per metre width.

∴   The reduction percentage in discharge = $\dfrac{0.608 - 0.509}{0.608}$ 100

                                   =   16.3 per cent.

The same reduction percentage can be taken for $H_{(crt)eqv}$ = 0.45 m from Table 5-2 as 16.3 per cent.

(c) A weir of 1.00 m width with an upstream depth of 0.60 m is working freely. What will be the new weir width if it is submerged with $H_{(s)}$ = 0.46 m so as to give the same discharge?

A clear overfall weir of 1.00 m width under a head of 0.60 m gives a discharge (Q) of 0.826 m^3/s.

A submerged weir of 1.00 m width having $H_{(crt)}$ = 0.60 m and $H_{(s)}$ = 0.46 m gives a discharge of 0.705 m^3/s (from Figure 5-32).

To keep the discharge the same as the clear overfall weir (0.826 m^3/s) the submerged weir should be increased in its width by $\Delta B_{(t)}$ which gives a discharge of:

$$0.826 - 0.705 = 0.121 \ m^3/s.$$

i.e. the increase in width, $\Delta B_{(t)} = \dfrac{0.121}{0.705} = 0.172 \ m$

i.e. the new width of the weir should be:

$$1.00 + 0.172 = 1.172 \ m.$$

5.12 SCRATCHLEY OUTLET (INDIA AND PAKISTAN)[1]

5.12.1 General

The Scratchley Outlet is used in Punjab and Haryana in India and in Pakistan when working heads available are small.

This non-modular type of outlet differs from the pipe outlet only at its downstream end. Its advantage over the pipe outlet is its easy adjustment and it has a more or less constant coefficient of discharge.

5.12.2 Structural and Design Characteristics

The Scratchley Outlet (Figure 5-33) consists of an inlet pipe (or barrel)

[1] Based on a contribution by A.D. Chaudhry, Chief Engineer, Irrigation Works, Haryana, and K.C. Gupta, Executive Engineer, Central Designs Irrigation Works, Haryana (India).

Plan

Note: Dimensions are in cm.

Section A-A
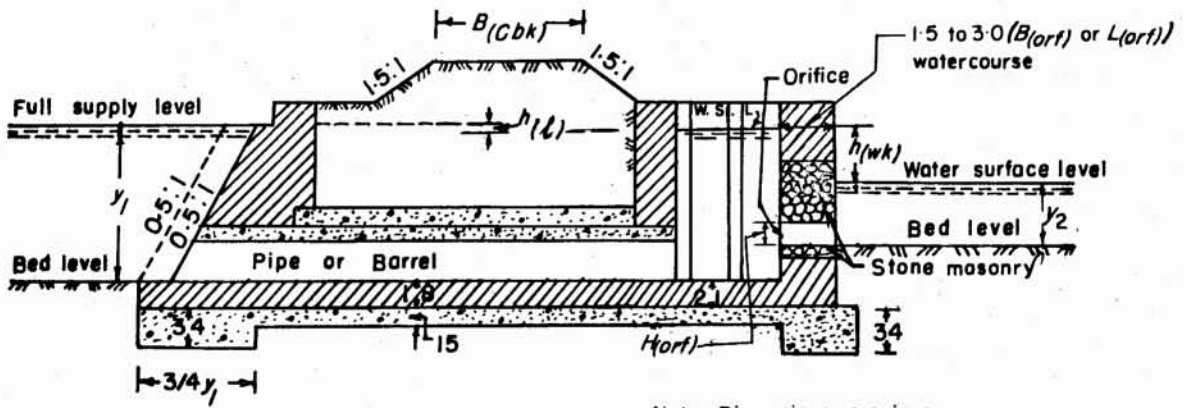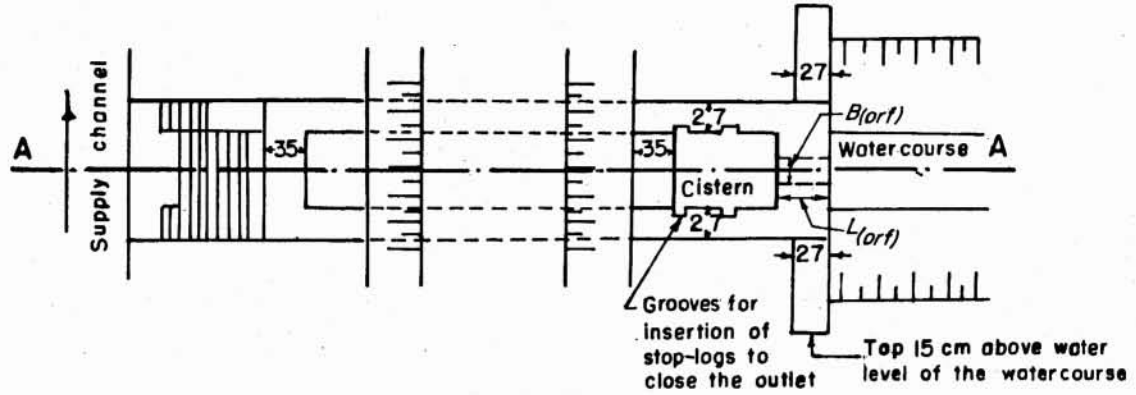
FIGURE 5-33. - Scratchley outlet.

which opens into a small tank or cistern at the outer side of the bank of the distributing canal, at the outer end of which is fixed a cast iron, stone or concrete orifice of the correct dimensions for the required discharge of the outlet. This orifice is kept submerged in the tail water to function as a non-modular outlet. If the orifice is set clear of the tail water, it will function as a semi-module due to free flow conditions.

The size of the cross section of the pipe (or barrel) should be large enough to pass the required discharge at a nominal head loss. The recommended size of the cross section of the orifice and the corresponding cross section of the barrel are as given in Table 5-3.

TABLE 5-3

Cross section of orifice (cm^2)	Cross section of barrel	
	Breadth (cm)	Height (cm)
Up to 550	30.5	30.5
551 to 650	36.5	30.5
651 to 750	43.0	30.5
751 to 850	46.0	30.5
851 to 930	46.0	38.0
931 to 1,000	46.0	46.0

A standard pipe instead of a barrel, approximately equal in area of cross section to the barrel as given in Table 5-3 may also be used.

The dimension of the cistern or tank for outlet discharges below 30 l/s is generally 60 cm by 60 cm. For discharges between 30 l/s and 60 l/s, the cistern is generally 80 cm by 80 cm and for discharge over 60 l/s the cistern is 100 cm by 100 cm.

The sill of the barrel is generally placed at the bed of the supply canal and the orifice at bed level of the farm watercourse, unless it can work as a free fall, when the sill of the orifice is placed higher than the water level in the watercourse. Stone blocks are used on the side of the orifice to discourage tampering. The

structure operates automatically.

The size of the orifice can be modified if required with the channel running. The cost of alteration is small. (It is only the downstream end-wall which has to be dismantled and rebuilt.) The tolerances in the capacity of the barrel are large enough to allow for a small change in the designed discharge.

The coefficient of discharge is the same for all orifices, provided the length of the orifice along the axis of flow is from 1.5 to 3 times the least of the dimensions of width (across the axis of flow) or height of the orifice.

The outlet requires only a small working head although a little more, say 2-3 cm, than the direct pipe outlet, as a small amount of head is lost in the lead-in pipe.

The working head of the outlet can be measured much more easily than in the case of those outlets where the supply water level and the delivery water level are somewhat apart; here, it is the difference in water levels on either side of the same wall.

Silt entry into the outlet can be better controlled by placing the upstream end of the inlet pipe at, above, or below the bed of the supply canal.

The main disadvantage of the Scratchley outlet is that it is not immune from tampering. The discharge can be increased by: lowering the water level in the watercourse; rounding the edges of the orifice; and making holes in the cistern wall.

When the orifice is set for free flow conditions, a comparatively large working head is required. Farmers could thus raise the water level in the watercourse and render the orifice partially submerged. This would result in some increase in the discharge. However, tampering can be fairly easily detected in this type of outlet.

5.12.3   Design Formula

$$Q = C A_{x(orf)} \sqrt{h_{(wk)}}$$

where

$$Q \quad = \quad \text{discharge of the outlet in l/s}$$

$A_{x(orf)}$ = cross-sectional area of the orifice, in cm^2

$h_{(wk)}$ = working head, i.e. the difference in water levels of the cistern and the watercourse, in cm

C = coefficient of discharge = 0.0354

$h_{(\ell)}$ = loss of head through barrel or pipe

$$= \quad \frac{926\, Q^2}{A_x^2}$$

where A_x is the cross sectional area of barrel or pipe. (In working out $h_{(\ell)}$, it would be better to use this formula direct.)

5.12.4 Numerical Example

Design a Scratchley outlet in accordance with the following data:

$$Q \quad = \quad 34 \text{ l/s}$$
$$y_1 \quad = \quad 90 \text{ cm}$$
$$h_{(wk)} \quad = \quad 15 \text{ cm}$$

Assume the water level in the supply canal = 200.00 m

Then the water level in the watercourse = 200.00 - 0.15

= 199.85 m

As an approximation, assume in the first instance a loss of head of 3 cm through the barrel. The working head available for the orifice will then be 12 cm. For the drowned condition,

$$Q \quad = \quad C\, A_{x(orf)} \; \sqrt{h_{(wk)}}$$

$$\text{or } 34 \quad = \quad 0.0354 \; . \; A_{x(orf)} \sqrt{12}$$

$$\text{or } A_{x(orf)} \quad = \quad 277 \text{ cm}^2.$$

The size of the orifice is 18.5 cm x 15 cm and accordingly, vide Table 5-3, the cross section of the barrel will be 30.5 cm x 30.5 cm.

For a barrel size as worked out above, to calculate the exact differences of water levels in the distributing canal and the cistern use the formula:

$$Q = 0.0329 \, A_{x(orf)} \sqrt{h_{(\ell)}}$$

$$\text{or} \quad h_{(\ell)} = \frac{926 \, Q^2}{A_x^2}$$

where A_x is area of barrel or pipe

which gives $h_{(wk)} = 1.2$ cm.

| The water surface level in the cistern | $=$ | $200.00 - 0.012$ |
| | $=$ | 199.988 m |

The working head for the outlet	$=$	$199.988 - 199.850$
	$=$	0.138 m
	$=$	13.8 cm

Now adopt this value of $h_{(wk)}$ in the formula

$$Q = 0.0354 \, A_{x(orf)} \sqrt{h_{(wk)}}$$

and recalculate $A_{x(orf)}$ which comes to 259 cm^2.

∴ The size of the orifice should be 17 cm x 15.25 cm.

5.13 PIPE OUTLET (INDIA AND PAKISTAN)[1]

5.13.1 General

The pipe outlet is the simplest and oldest known type of outlet. Originally the pipes were of earthenware but were gradually replaced, in most places, by rectangular wooden and masonry barrels. Cast iron, steel and concrete pipes were progressively introduced at later stages.

This outlet can be built as a free fall type if sufficient head is available. It is generally used where the silt charge in the supply canal is low and the canal is running in high filling. When the head available is very small, the submerged

[1] Based on a contribution by A.D. Choudhry, Chief Engineer, Irrigation Works Jaryana, and K.C. Gupta, Executive Engineer, Central Designs, Irrigation Works, Haryana (India).

pipe outlet is the obvious choice.

5.13.2   Structural and Design Characteristics

The pipe outlet consists of an upstream headwall, a pipe and a downstream headwall.   In a submerged pipe outlet (Figure 5-34) the upstream end is a little above, or sometimes below, the bed level, depending upon the desired silt draw. The other end of the pipe opens into the farm watercourse below the water surface level.   The pipe can be placed horizontally or sloping slightly up at 1 in 12 downstream.   Both ends of the pipe are built into masonry to prevent tampering and to guard against any leakage along the outer side of the pipe.
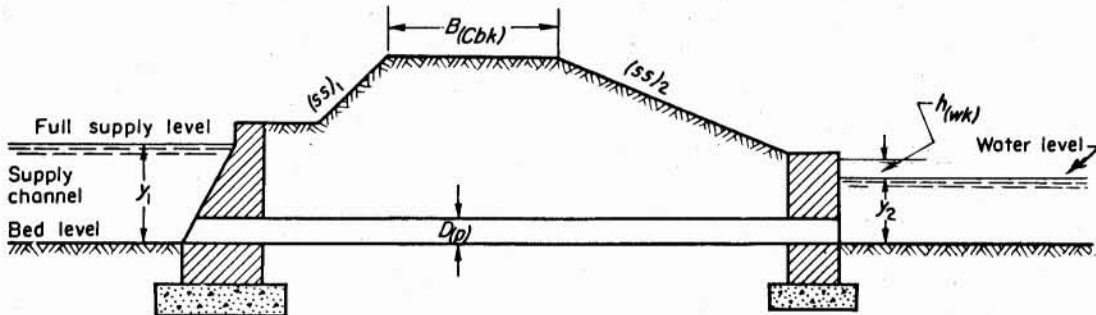
FIGURE 5-34. - Submerged pipe outlet.

The free fall pipe outlet may have a horizontal pipe with its downstream end above the water surface level in the watercourse but in that case it cannot, usually, draw its fair share of silt.   In an attempt to fix the pipe at the bed level of the supply canal and yet obtain free fall conditions (where levels permit), some pipes have been laid with their upstream ends at bed level of supply canals and sloping upwards through the banks so that the downstream lips reach a height of 15 cm above the highest water level in the watercourse.   There are practical limits to the amount of slope that can be given and it should not generally be more than 1 in 12 (based on experience on the Western Yamuna Canal).

5.13.3   Hydraulic Characteristics

5.13.3.1   Flow through pipe outlets

The conditions of flow through a pipe apply equally to pipe outlets.

5.13.3.2   Flexibility

The flexibility of a pipe outlet depends on the ratio which $h_{(wk)}$ or $H_{(cnt)}$ bears to the full supply depth in the supply canal. In Punjab and Haryana (India), wherever pipe outlets are used, it has been the general practice to place them at the bed level of the supply canal. This setting is given to enable the outlets to draw their fair share of silt from the supply canal.

5.13.3.3   Silt drawing capacity

No results of comprehensive experiments on the silt drawing capacity of pipe outlets appear to have been reported, but such experiments would be useful. On the other hand, according to experience so far, canals fitted with pipe outlets at bed level seldom give any silt trouble.

5.13.3.4   Efficiency

Pipe outlets, working under non-modular conditions, suffer from all the defects inherent in the non-modular type. They are important, however, in that they can pass the required discharge with a very small working head, (even only 2.5 cm, with which no semi-module can function).

5.13.3.5   Adjustability

While appreciable adjustment of the design discharge would require dismantling and reconstruction of the structure, or part of it, small changes in discharge can be effected by lowering, or raising, the bed level at the pipe outlet, and this would change the working head.

5.13.4   Design Formulae

5.13.4.1   Submerged pipe outlet

$$Q = C A_x \sqrt{2g h_{(wk)}}$$

Where $\quad$ Q $\quad = \quad$ discharge of the outlet in l/s;

$\quad\quad\quad\quad$ A_x $\quad = \quad$ cross sectional area of the pipe in cm^2;

$\quad\quad\quad\quad$ $h_{(wk)}$ $\quad = \quad$ difference in water surface levels in the supply channel and the water course in cm;

$\quad\quad\quad\quad$ g $\quad = \quad$ acceleration due to gravity in cm/s^2.

The value of C for ordinary cast iron pipe of 15 cm internal diameter has been found to be nearly 0.00074.

A simpler formula for the outlet is

$$Q \quad = \quad C A_x \sqrt{h_{(wk)}}$$

Where $\quad$ C $\quad = \quad$ 0.028 and 0.033 respectively for long (more than 600 cm) and short (less than 600 cm) pipe.

5.13.4.2 $\quad\quad\quad$ Free fall pipe outlet

$$Q \quad = \quad C A_x \sqrt{H_{(cnt)}}$$

Where

$\quad\quad\quad\quad$ Q $\quad = \quad$ discharge of the outlet in l/s;

$\quad\quad\quad\quad$ A_x $\quad = \quad$ cross sectional (internal) area of the pipe;

$\quad\quad\quad\quad$ C $\quad = \quad$ coefficient of discharge $\quad = \quad$ 0.0276

$\quad\quad\quad\quad$ $H_{(cnt)}$ $\quad = \quad$ head of upstream water surface over the centre of the pipe.

The discharge is sensibly correct so long as $H_{(cnt)}$ is greater than $2 D_{(p)}$ (inside diameter of the pipe) and is approximately correct for $H_{(cnt)} = D_{(p)}$ but is appreciably different from that given by the formula for

$H_{(cnt)}$ less than $D_{(p)}$.

5.13.5 Numerical Example

Example 1

Design an outlet for a discharge of 34 l/s on a distributing canal having a full supply depth of 90 cm and with an available working head of 6 cm.

$$Q = 34 \text{ l/s}$$

$$y_1 = 90 \text{ cm}$$

$$h_{(wk)} = 6 \text{ cm}$$

The available working head of 6 cm is low and the outlet must be submerged.

$$Q = C A_x \sqrt{2 g h_{(wk)}}$$

$$C = 0.00074$$

$$34 = 0.00074 \, A_{(p)} \sqrt{2 \, . \, 981 \, . \, 6}$$

or $$A_x = \frac{34}{0.00074 \, . \, 108.5} = 423 \text{ cm}^2$$

or $$\pi \frac{D_{(p)}^2}{4} = 423$$

or $$D_{(p)} = 23.3 \text{ cm}.$$

This may be rounded to the nearest standard size of pipe available.

Example 2

If in Example 1 the available working head is 75 cm, a wide choice would be possible for the type of the outlet to use. If, however, a

pipe outlet is considered desirable, either from the point of view of cost or if the outlet is required only temporarily, it would be desirable to install a free fall outlet so that it might work as a semi-module.

Assuming water level to be 200.00 in the distributing canal, the water level in the watercourse would be 200.00 - 0.75 = 199.25. As a trial, assume a pipe of 15 cm diameter, then:

$$Q = C A_x \sqrt{H_{(cnt)}}$$

$$C = 0.0276$$

$$\therefore 34 = 0.0276 \cdot 176.7 \sqrt{H_{(cnt)}}$$

$$\text{or} \quad H_{(cnt)} = 48.6 \text{ cm.}$$

The centre of the 15 cm diameter pipe should thus be at a level of 200.00 - 0.485 = 199.515 and the outlet would be semi-modular, since the water surface level in the watercourse would be 199.25, (which is well below the bottom level of the outlet, i.e. 199.36).

5.13.6 Summary

The pipe outlet is the simplest and the cheapest type of outlet. It can function with very small working heads, even as small as 2.5 cm. Under submerged conditions the discharge of the pipe outlet depends on the downstream water level (i.e. the level in the watercourse). The discharge can be increased by lowering the water level in the watercourse. On the other hand, if the watercourse should silt up, the working head will reduce, resulting in a decrease in discharge. In a free fall outlet, irrigators may be tempted to raise the water level in the watercourse to make the outlet partially submerged and thus obtain an increased discharge. The discharge coefficient of the pipe outlet is not constant.

5.14 FARM OUTLET (U.S.S.R.) [1]

5.14.1 General

The outlet described here is used to deliver water to a temporary feed-ditch, up to a discharge of 150 l/s and with a working head of up to 60 cm.

The early farm outlets of this type were controlled by wooden flap gates. The main shortcomings of these early outlets were their short life and frequent failures. The wooden gates have now been replaced by steel discs. These outlets are the smallest structures on irrigation systems but they are also the most numerous, accounting for more than half of the total number of structures in a system.

5.14.2 Structural Characteristics

The main parts of the farm outlet are: a pipe; a disc gate; and, for drops over 20 cm, a damper or a stilling basin.

In the submerged farm outlets, types VT-300 and VT-400 (Figure 5-35), horizontal pipes are used for working heads up to 20 cm.

In the free fall farm outlets, types VTP-300 and VTP-400 (Figure 5-36), inclined pipes are used up to working heads over 20 cm but not exceeding 60 cm. In both cases the pipes are of low pressure asbestos cement, 30 cm and 40 cm in diameter.

Regulation of the water supply is accomplished by the manually operated disc gate.

5.14.3 Hydraulics

The discharge capacity of the farm outlet for design purposes is determined by using the following formula.

[1] Based on information provided by A. T. Koshkina, E.P. Martin, A.V. Shatalova, D.D. Aliera and B.V. Kazarinov (U.S.S.R.)

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Cross Section 1-1

Valve · Cast-in-situ concrete · Pipe · Stilling basin · Rock-filling · $h_{(wk)} \leq 60$ · $H_{(crt)}$

Plan

Distributor axis

Cross Section 2-2

Cross Section 3-3

Detail

(All dimensions are in cm)
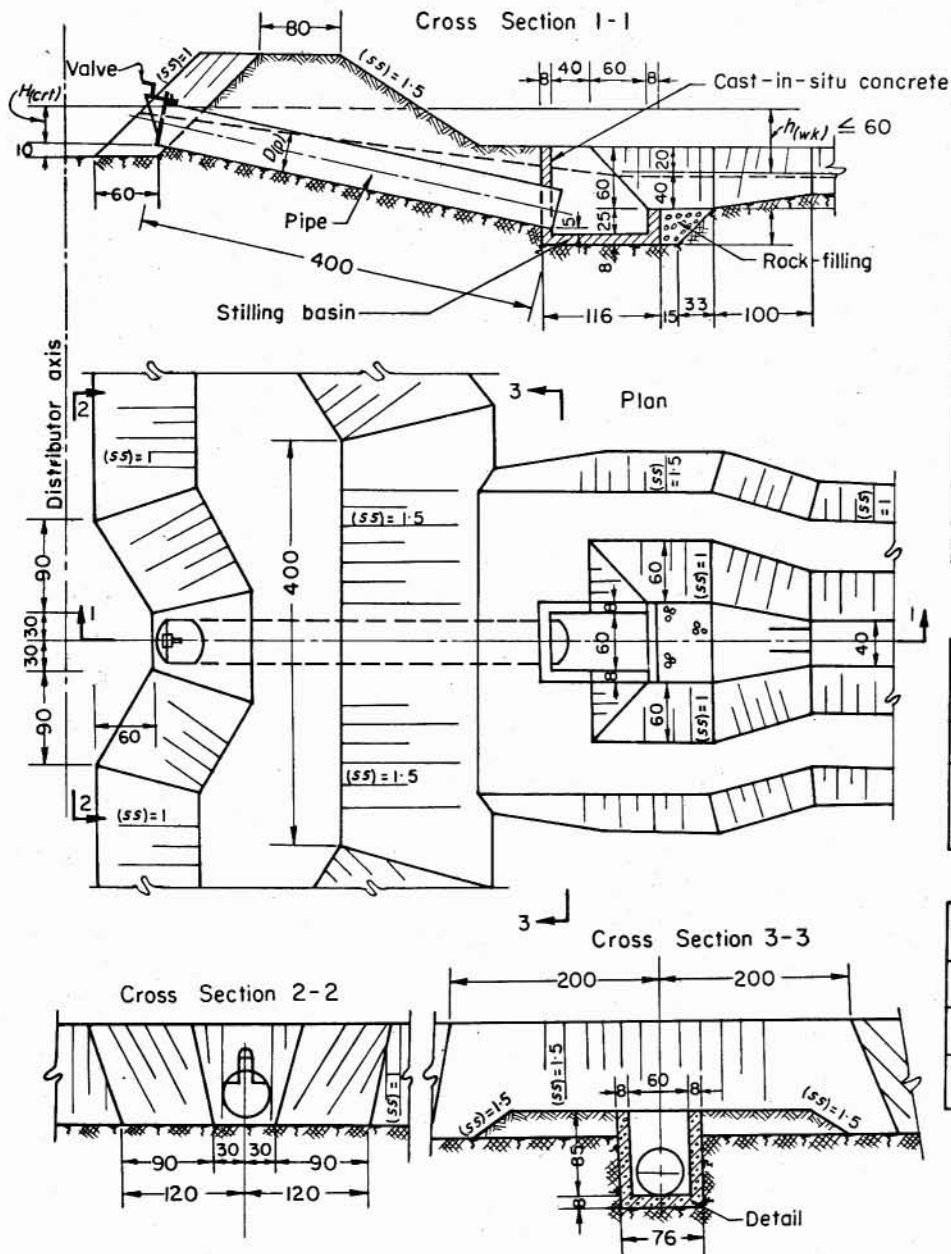
Hydraulic characteristics

Type of structure	Free outfall		
	VTP-300	VTP-400	
Discharge l/s	50	100	150
$h_{(wk)}$ cm	20	$\leq$ 60 cm	
$D_{(p)}$ mm	300	400	
$H_{(crt)}$ cm	27	36	47

Hydraulic characteristics with inlet unsubmerged

Type of structure	Internal pipe dia. mm	Total area of pipe cross section m²	Discharge l/s											
			50				100				150			
			$A_{x(p)c}$ m²	$H_{c(IN)}$ cm	$V_{c(IN)}$ m/s	$H_{(crt)}$ cm	$A_{x(p)c}$ m²	$H_{c(IN)}$ cm	$V_{c(IN)}$ m/s	$H_{(crt)}$ cm	$A_{x(p)c}$ m²	$H_{c(IN)}$ cm	$V_{c(IN)}$ m/s	$H_{(crt)}$ cm
VTP-300	291	0.067	0.042	17	1.19	27	—	—	—	—	—	—	—	—
VTP-400	386	0.117	—	—	—	—	0.074	23	1.35	36	0.094	29	1.60	47

List of details

Type of detail	Concrete volume for detail m³	Detail reinforcement weight kg	Number of details according to structures types	
			VTP-300	VTP-400
Detail	0.25	11.94	1	1
Asbestos cement pipe $L_{(p)}$ = 400 cm	$D_{(p)}$ = 291mm	—	—	1	—
	$D_{(p)}$ = 386 mm	—	—	—	1

Volume of main works

Name	Material	Unit	Type of structure	
			VTP-300	VTP-400
Reinforced concrete details	Concrete	m³	0.25	0.25
	Reinforcement	kg	11.94	11.94
Pipes $D_{(p)out}$ = 300 mm / $D_{(p)out}$ = 400 mm	Asbestos cement gravity pipes	m	4 / —	— / 4
Disc valves	Steel	kg	6.74	8.59

FIGURE 5-35. - Farm outlet (U.S.S.R.) to a temporary feed ditch for discharges of up to 150 l/s - outlet submerged.

Cross Section 1-1

Hydraulic characteristics

Type of structure	Outlet submerged		
	VT-300	VT-400	
Discharge l/s	50	100	150
$h_{(wk)}$ — cm	7	9	20
$D_{(p)}$ — mm	300	400	
$H_{(crt)}$ — mm	49	59	

Hydraulic parameters with inlet submerged

Type of structure	Internal pipe dia. mm	Total area of pipe, m²	Discharge l/s											
			50				100				150			
			$h_{(wk)}$ cm	v m/s	$H_{(sof)_2}$ cm	$H_{(crt)}$ cm	$h_{(wk)}$ cm	v m/s	$H_{(sof)_2}$ cm	$H_{(crt)}$ cm	$h_{(wk)}$ cm	v m/s	$h_{(sof)_2}$ cm	$H_{(crt)}$ cm
VT-300	291	0·067	7	0·75	13	49	—	—	—	—	—	—	—	—
VT-400	386	0·117	—	—	—	—	9	0·86	11	59	20	1·29	0	59

$h_{(wk)} \leq 20$ cm

Plan

List of details

Type of detail		Concrete volume for detail	Reinforcement weight for detail	No. of details according to structures' types	
				VT-300	VT-400
Asbestos pipe L = 400 cm	$D_{(p)}=291$ mm	—	—	1	—
	$D_{(p)}=386$ mm	—	—	—	1

Volume of main works

Name		Material	Unit	Type of structure	
				VT-300	VT-400
Pipes	$d_{(p)}=300$mm	Asbestos gravity pipes	m	4	—
	$d_{(p)}=400$mm			—	4
Disc valves	$y_2 - 291$	Steel	kg	6·74	—
	$y_2 - 386$			—	8·59

Cross Section 2-2

Cross Section 3-3

(All dimensions are in cm)

FIGURE 5-36. - Farm outlet (U.S.S.R.) to a temporary feed ditch for discharges of up to 150 l/s - free outfall.

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<u>With submerged outlet</u>

$$Q = C A_x \sqrt{2gh_{(wk)}}$$

where $\quad C = \dfrac{1}{\sqrt{1 + \Sigma C_{(Hr)} + C_{(f-p)}}} = 0.65$

$\Sigma C_{(Hr)}$ = local hydraulic resistance coefficient $= C_{(f-IN)} + C = 0.976$

with $\quad C = 0.3 + 1.3 (\dfrac{1}{(ss)} - ss)^2 = 0.476$

in which $(ss) = \dfrac{75^o}{90^o} = 0.833$

$C_{(f-p)}$ = frictional resistance coefficient $= C'' \dfrac{L_{(p)}}{D_{(p)}} = 0.399$

with $\quad C'' = 8g N^2 (\dfrac{4}{D_{(p)}})^{\frac{1}{3}} = 0.0672$

with $L_{(p)} = 4 \text{ m}$

$D_{(p)} = 0.389 \text{ m}$

N = coefficient of roughness = 0.015 (for asbestos cement pipes).

The depth of the water surface above the bottom of the pipe at the inlet should be:

$$H_{(crt)} = (D_{(h)} + h_{(wk)} + H_{(sof)_2}) = D_{(p)} + 0.2 \text{ m}$$

with $\quad (h_{(wk)} + H_{(sof)_2}) = 0.20 \text{ m},$

$h_{(wk)} = 0.20 \text{ m}; \qquad H_{(sof)_2} = 0$

<u>With free flow outlet</u>

The depth at the inlet, $H_{(crt)}$, is defined as:

$$H_{(crt)} = H_{c(IN)} + \dfrac{1}{C_v^2} \cdot \dfrac{v^2_{C(IN)}}{2g} \leqslant 1.5 D_{(p)}$$

where $v_{c(IN)}$ = critical inlet velocity,

$H_{c(IN)}$ = critical inlet depth,

C_v = coefficient of velocity = 0.85,

$D_{(p)}$ = pipe diameter.

The critical inlet depth (for a round cross section) is defined by A.M.

Latishenkov's method, as $H_{c(IN)} = H_{c(cir)}\pi$

$H_{c(cir)}$ = critical depth for circular cross section with $D_{(p)}$ = 1 m and discharge $q = \dfrac{Q}{D^{2.5}_{(p)}}$

Critical velocity $v_{c(IN)} = \dfrac{Q}{A_{x_c}}$

The value of the critical depth $H_{c(cir)}$ depends upon q and according to the following table:

q (m^3/s)	$H_{c(IN)}$ (m)	q (m^3/s)	$H_{c(cir)}$ (m)
0.5	0.40	2.0	0.82
1.0	0.58	2.5	0.90
1.5	0.70	3.0	0.94

The design discharges are co-ordinated with the discharges of the temporary feed-ditches and are assumed at 50 and 150 l/s.

The height of the embankment above the water surface on the downstream side is 20 cm and on the upstream side 30 cm or more.

Depending upon the conditions of operation of the disc gates, the upper inlet edge of the pipe should not be submerged by more than 20 cm.

5.14.4   Numerical Example

A farm outlet is required to discharge 90 l/s through a pipe, of 400 mm outside diameter, 5.7 mm thick and 4 m long, under submerged conditions. Find out the working head and design the outlet.

Let inner diameter of the pipe be D = 0.389 m

Let sectional area be A_x = 0.119 m^2

The coefficient of discharge C = 0.65

$Q = C A_x \sqrt{2 g h_{(wk)}}$ = 0.65 . 0.117 . 4.43$\sqrt{h_{(wk)}}$

= 0.34 $\sqrt{h_{(wk)}}$

$$\text{Now} \quad \sqrt{h_{(wk)}} \quad = \quad \frac{Q}{0.34} \quad = \quad 2.94\,Q$$

$$\text{or} \quad h_{(wk)} \quad = \quad 8.64\,Q^2 \quad = \quad 8.64 \cdot 0.09^2 \quad = \quad 0.07\text{ m}$$

The design of the outlet will be as for type VT-300 detailed in Figure 5-35.

5.15   PRE-CAST FARM TURNOUT (TURKEY) [1]

5.15.1   General

The Pre-cast Farm Turnout described herein is widely used in some parts of Turkey to deliver water to farm laterals from distributing canals.   The design of the turnout is adapted from that of a constant head orifice.   All parts of the device are pre-cast units, and the whole structure is very strong and durable.

5.15.2   Structural and Hydraulic Characteristics

As shown in Figures 5-37 and 5-38, the turnout consists of the following parts: a pre-cast concrete pipe of 30 cm diameter provided with a gate at the inlet; in front of this pipe there is an approach box.   The wing walls on both sides, the headwall and the floor of the box are assembled as one pre-cast reinforced concrete unit.   The reinforcing steel bars are of 6 mm diameter and the spacing between them must not exceed 20 cm; there is a pre-cast downstream head wall; there are pre-cast check blocks in the supply canal, downstream of the turnout, provided with grooves for insertion of stop-logs to raise the water level; pre-cast lining is provided in the supply canal on the sides and on the bed; there is coarse gravel protection in the farm lateral just below the turnout structure.

The quantity of cement in the pre-cast concrete mix is 400 kg per cubic metre of concrete.

At present, the turnout is simply a pipe outlet operating under submerged conditions.   However, it can be converted to a constant head orifice should the

[1] Based on a note prepared by Hüseyin Kömürcüoglu (Turkey).

Detail of central plate
3 number

Section D-D

Section C-C

Section E-E

The crest of levee

Section G-G
Detail of the side plate
4 number

Section F-F

Section B-B

Section A-A

Head walls of the entrance box

Front elevation

Bed plate of the entrance box

The details of entrance box of turnout

The detail of downstream head wall of the turnout

Section H-H

Note :- All the dimensions are in cm.

Tertiary levee
Natural soil surface

Check block

Check block

2" Galvanized pipe
Compacted Fill
Natural soil surface
Course gravel lining

Tertiary levee
Natural soil surface

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PRECAST FARM
TURNOUT

Project, Region, Country
Turkey

Figure 5-37

Cross section of precast farm turnout
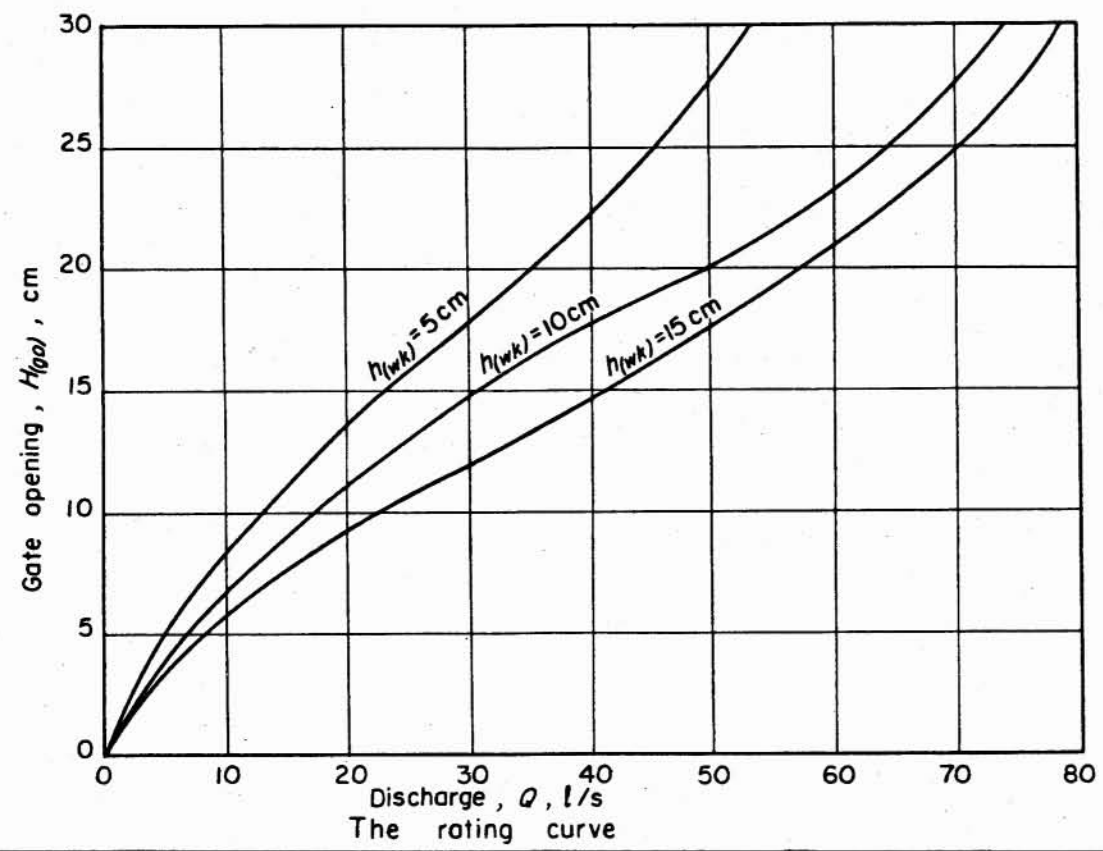
The rating curve

Note: Dimensions are in centimetres.

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RATING CURVE OF PRECAST
FARM TURNOUT

Project, Region, Country
Turkey

Figure No. 5-38

need arise by installing a gate in the guides provided for the purpose.

The turnout works satisfactorily for all discharges up to 60 l/s.

5.15.3   Design Formula

See Figure 5-38 which gives discharges for different openings of the gate.

5.15.4   Numerical Example

Design a pre-cast turnout for a discharge of Q = 45 l/s

Difference of surface water levels in
the supply canal and the farm lateral, $h_{(wk)}$ = 10 cm

From Figure 5-38, for a working head of 10 cm and a discharge of 45 l/s the gate opening $H_{(go)}$ = 19 cm.   Other dimensions are given in Figure 5-37.

5.16   ADJUSTABLE WEIR FARM OUTLET (MALAYSIA)[1]

5.16.1   General

With the advent of double cropping in Malaysia, suitable conditions have to be developed for proper water management, which calls for better water control systems.

The adjustable weir farm outlet, meant for delivering water from a distributing canal to a field canal serving a group of irrigators, has been developed by the Design Branch of the Malaysian Drainage and Irrigation Department to overcome some of the field problems inherent in the earlier types of outlets. These earlier outlets ranged from simple orifices to orifices incorporating some rudimentary forms of regulating valves; they were largely used on irrigation systems drawing water supplies from rivers for irrigating one crop. Basically they suffered from inadequate control in the supply, large head losses and wastages due to tampering.

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[1] Based on information supplied by the Malaysian National Committee, ICID.

5.16.2   Structural Characteristics

The outlet is strong and durable. It consists (see Figure 5-39) of an inlet pipe opening into a well-chamber (inside dimensions 5 ft by 5 ft) at the downstream end of which is an opening with its top 2 ft from the floor of the well-chamber. This opening is 2 ft wide (across the flow) and 3 ft high. The adjustable weir, consisting of mild steel plate, $\frac{1}{4}$ inch thick, $7\frac{3}{4}$ inches wide and 2 ft 10 inches high, slides up and down by means of a screw-down device. A gauge is fixed to the weir to enable the head over the crest to be read.

On the downstream side of the weir there is a small cistern, 5 ft long across the flow and 2 ft wide in the direction of the flow. It is roofed by precast slabs which serve as a platform to operate the screw-down device. At the end of the cistern there is an opening of the same section as the cross section of the field canal.

5.16.3   Hydraulic Characteristics

As regards hydraulic properties of the outlet, it is simply a thin weir structure which has been calibrated in the laboratory. No calculations using formulae are required. The operational procedure of this outlet has been simplified by preparing charts, as referred to in 5.16.4 below. The operator merely sets the head above the weir crest as required for any particular discharge, (which is read off directly from Figure 5-40), by moving the weir up and down by the screw-down device. (For details of this gate see Vol. III of this handbook.)

Additional sets of graphs are being prepared for varying depths of chamber water level to render the operation of the outlet more versatile.

This outlet is not suitable for withdrawal of silt from the supply canal.

The outlet generally operates for a discharge of 85 l/s (3 ft^3/s) but a greater capacity could be achieved if sufficient head were available. The difficulty would be in taking the readings because of the unsteady condition of flow prevailing in the well-chamber for discharges over 85 l/s (3 ft^3/s).

Section A-A

Supply channel

Bank width

Bank level

1·5:1

F.S.L.

1·5:1

1:1

1:1

3" fillet

1·5:1

6"

5'

3"

2'

4½"

Measuring gauge

D/S F.S.L.

12"

F.S.L.

18" dia. x 5' long precast concrete pipework with standard collars

24" block drain

2'

2"

5'

6"

5'-6"

6"

2'-4½"

Section A-A

Section B-B

6"

1·5

2'

1·5

6"

C.L.

¾" dia. M.S. Rung

5'-6"

5'

2"

6"

6'

Section B-B

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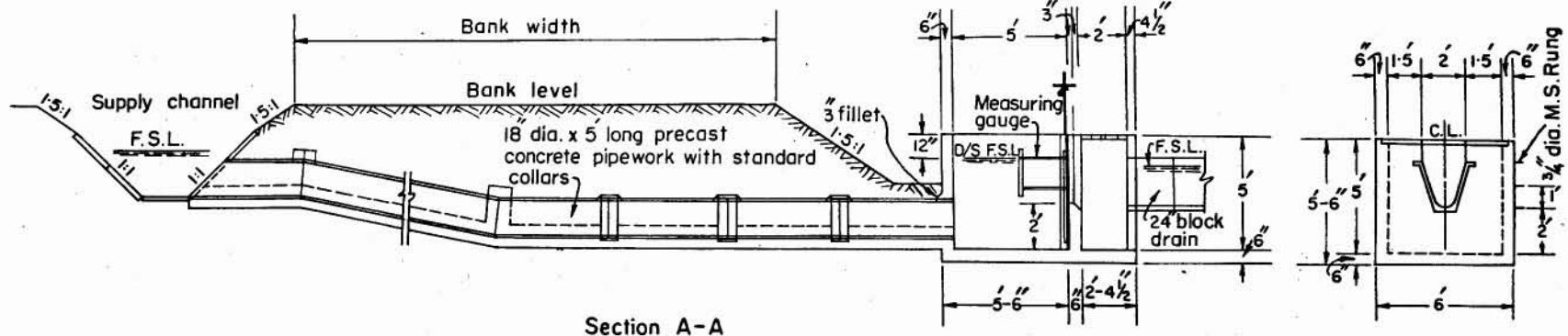
Plan

2" thick cast-in-situ concrete lining

Precast concrete slabs

Turfing

Supply channel

Slope 1·5:1

Bank

Slope 1·5:1

B

C

2½" x 6" x 5'-6" long precast slabs

12" wide field channel bank

6"

12"

6"

6"

6"

1'-9"

2'-6"

1'-9"

C.L.

C.L.

1'-6"

12"

6"

2'-6"

6"

A

A

3"

6"

5'

6"

2'

4½"

8'-7½"

B
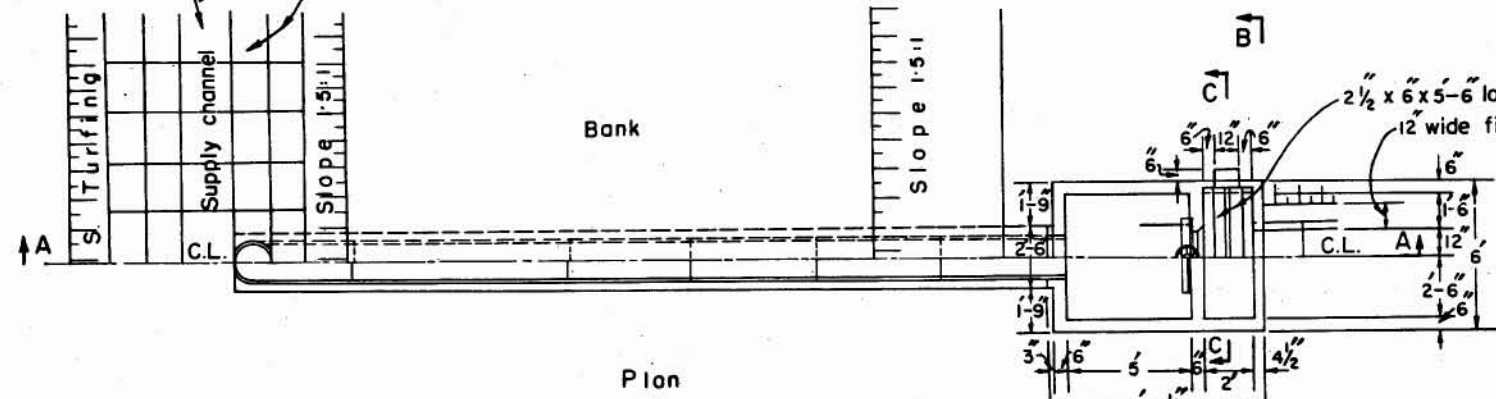
Plan
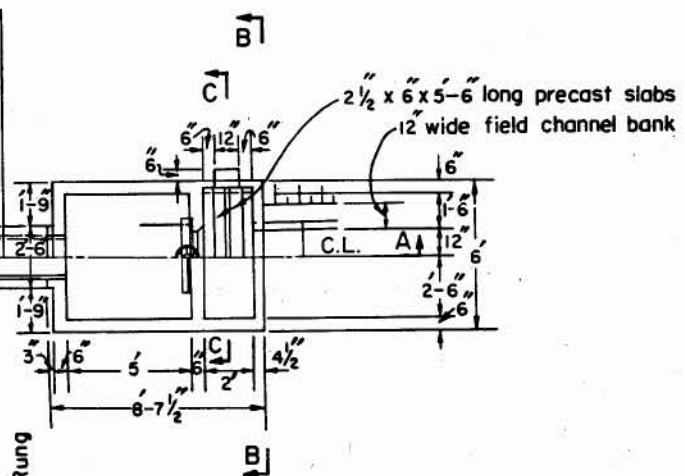
Notes:

1. Cover to reinforcement
   1" for all main reinforcement.

2. Formwork
   Rough faced formwork for surfaces in contact with earth fair faced formwork for all other surfaces.

Section C-C

6"

1-3

3"

2'

1-3

6"

¾" dia. M.S. Rung

2½"

3"

5'-6"

3"

1'-9"

6"

6'

C.L.

Section C-C

FAO — ICID

ADJUSTABLE WEIR

FARM OUTLET

Project, Region, Country
Malaysia

Figure No. 5-39
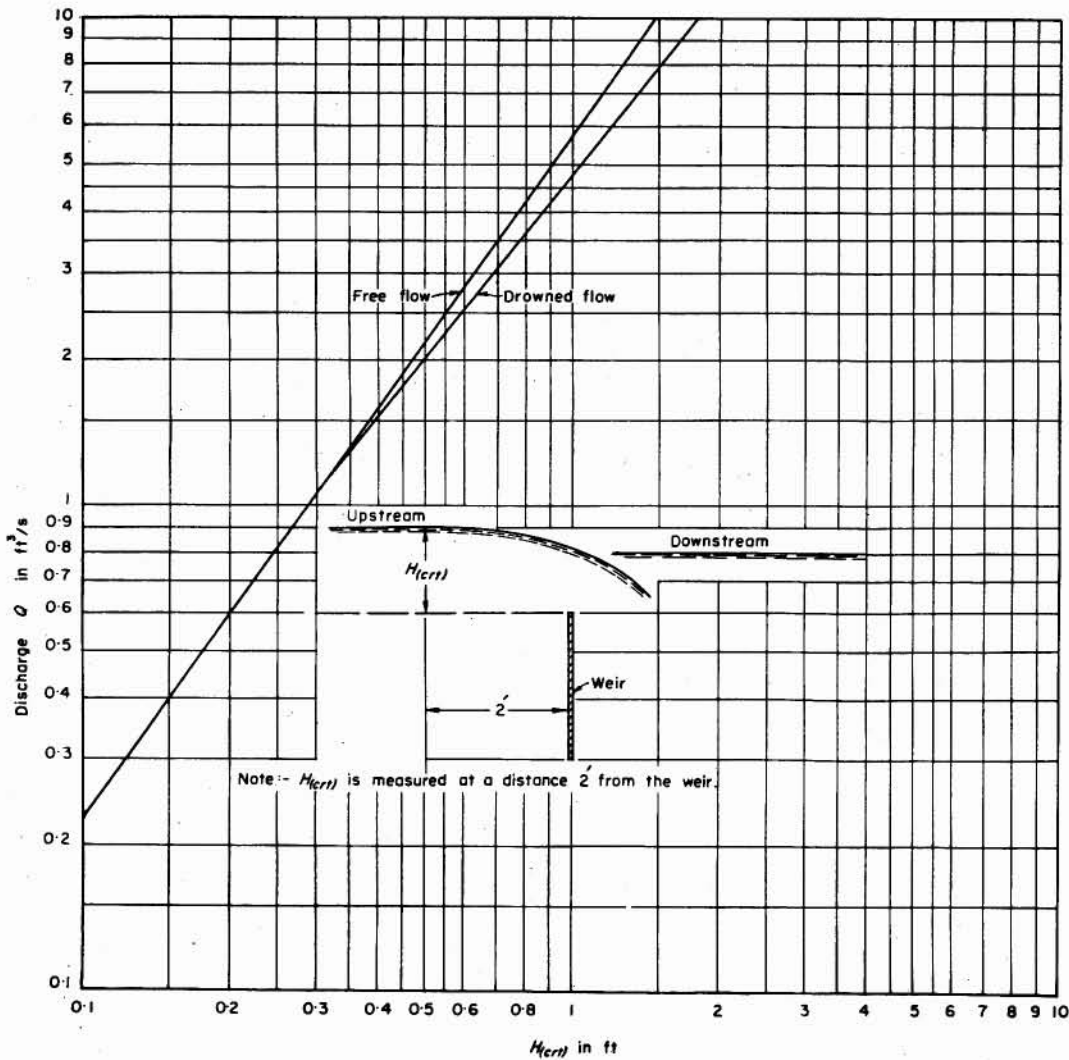
Table I
Minimum free flow condition

Discharge (ft³/s)	Minimum differential head (inches)
· 1·75	4·18
2·00	4·99
2·25	4·92
2·50	5·21
2·75	5·54
3·00	5·81
3·50	6·14
4·00	6·60
4·50	7·37
5·00	7·82
5·50	8·45
6·00	8·88

The difference in water level between the upstream behind the weir and downstream of the weir.

Differential head 'h$_{(wk)}$'

Upstream water level

Downstream water level

Q

4'

Free flow — Drowned flow

Upstream

Downstream

$H_{(crt)}$

2'

Weir

Note :- $H_{(crt)}$ is measured at a distance 2' from the weir.

Discharge Q in ft³/s

$H_{(crt)}$ in ft

356.
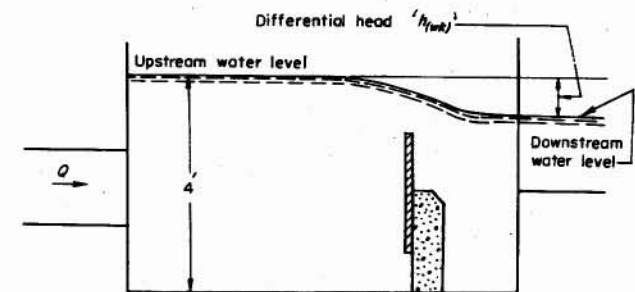
FAO – ICID

ADJUSTABLE WEIR FARM

OUTLET

Q VERSUS $H_{(crt)}$

Project, Region, Country
Malaysia

Figure No. 5-40

5.16.4   Design Procedure

The outlet has been calibrated in the Hydraulics Laboratory, with a constant water depth of 4 ft in the chamber.   Two graphs (Figures 5-40 and 5-41) and one table (given on Figure 5-40) have been developed for operating this outlet. Figure 5-40 shows the relationship between discharge through the weir against the head over the weir for two conditions of flow, i.e. (a) free flow, and (b) downstream level 4 inches below upstream level (submerged).   Figure 5-41 gives the head losses in the inlet pipe for varying discharges.   The Table on Figure 5-40 shows the minimum differential head required for various discharges for maintaining minimum free flow condition.

5.16.5   Numerical Examples

Example 1

Determine the head over the weir $H_{(crt)}$, if

(i)   the discharge Q required is 3 ft^3/s;

(ii)   FSL$^{1/}$ in the irrigation canal is 20.00;

(iii)   water level required in the field canal is 18.00.

From Figure 5-41, head loss through delivery pipe

$$J = 1.4 \text{ inches } (0.12 \text{ ft})$$

From the Table on Figure 5-40, for free flow condition the differential head

$$= 5.8 \text{ inches } (0.48 \text{ ft})$$

.'.   Total head required for free flow condition

$$= (1.4 + 5.8)$$
$$= 7.2 \text{ inches } (0.6 \text{ ft})$$

Head available is 20.00 - 18.00   =   2.0 ft.

.'.   Discharge over weir is in a free flow condition -

From Figure 5-40, $H_{(crt)}$ =   0.62 ft.

Example 2

Determine the head over the weir $H_{(crt)}$, if

$\underline{1/}$ FSL, feet above sea level.
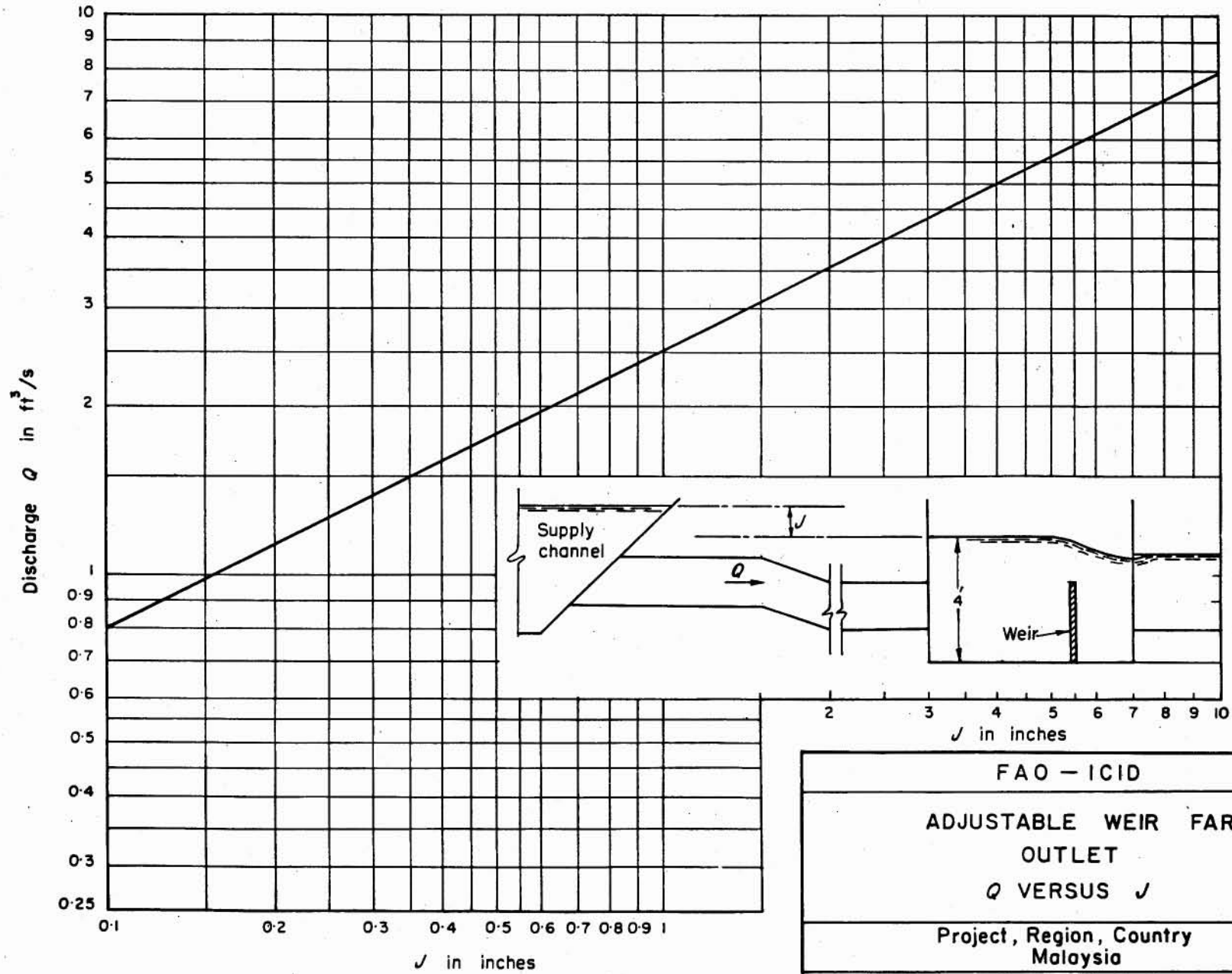
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ADJUSTABLE WEIR FARM
OUTLET

Q VERSUS J

Project, Region, Country
Malaysia

Figure No.5-41

(i)   the discharge Q required is $3 \text{ ft}^3/\text{s}$;

(ii)   FSL in the irrigation canal is 18.45;

(iii) water level required in the field canal is 18.00.

.·.   Head available $h_{(wk)}$ =   0.45   =   (18.45 - 18.00) ft

From Example 1 total head required for the free flow condition

=   0.6 ft

.·.   Flow is submerged

From Figure 5-41,   J   =   1.4 inches (0.12 ft)

Differential head between chamber and field canal water level
is 0.45 - 0.12   =   0.33 ft (4 inches)

Hence from Figure 5-40 (submerged condition)

$H_{(crt)}$ =   0.68 ft.

5.17   PVC PIPE TURNOUT (REPUBLIC OF KOREA) [1]

5.17.1   General

An earlier farm turnout designed in the Republic of Korea consisted of a pair of wing walls and an inlet floor, a concrete conduit and a steel or wooden side gate.   The construction of each turnout involved a significant amount of concrete and mechanical work at considerable cost in materials and time.   Also, the maintenance of the slide gate has been a problem owing to the turnouts being scattered all along the irrigation canals far from villages.   The Agricultural Engineering Research Center, one of the prominent laboratories in Korea, has developed the PVC pipe turnout to overcome the disadvantages of the older type of structure.   It was developed in 1969 and has been recommended for use on new irrigation projects in the country.

[1] Based on information supplied by U.C. Yeo, Republic of Korea.

5.17.2 Structural Design

The turnout consists of a bell-mouth inlet, a pipe and a specially designed outlet with a screwed stopper (Figure 5-42). The inlet bell-mouth has two horizontal bars to prevent weeds, debris, etc. blocking the pipe, and any material collecting there can be easily removed.
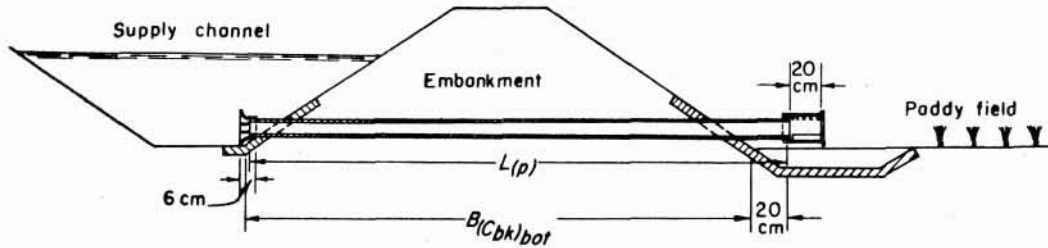
FIGURE 5-42. - Sectional view of the PVC outlet installed through a canal embankment. Note lining of the bed in the field canal below the downstream end of the pipe and in the parent canal.

The PVC pipe has a length equal to the bottom width of the canal embankment plus 20 cm. The bell-mouth inlet and the outlet device are connected to each end of the pipe with synthetic bond.

FIGURE 5-43. - View of the PVC pipe assembly.
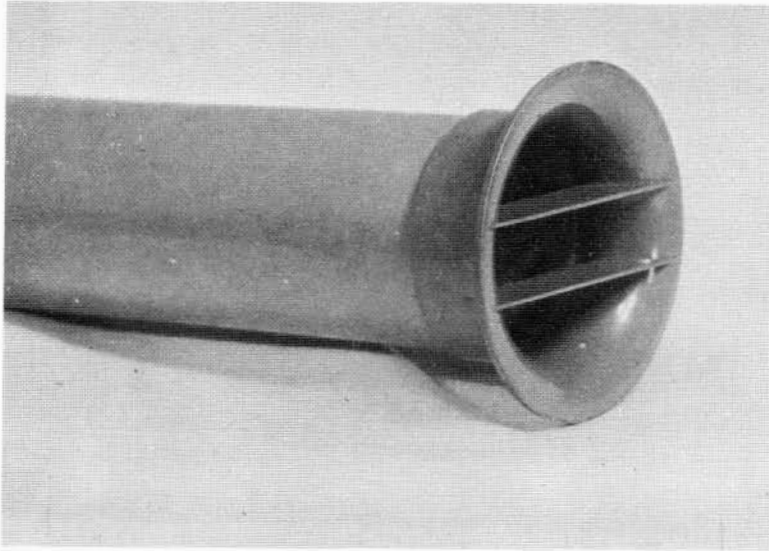
FIGURE 5-44. - View of bell-mouth inlet with cross bars.

Figure 5-43 is a view of the assembled PVC pipe assembly. Figure 5-44 shows the bell-mouth inlet, and Figures 5-45 and 5-46 show the outlet closed and open respectively. This type of outlet can be used for discharges as shown in Table 5-4 for different lengths and diameters of pipe and working heads.
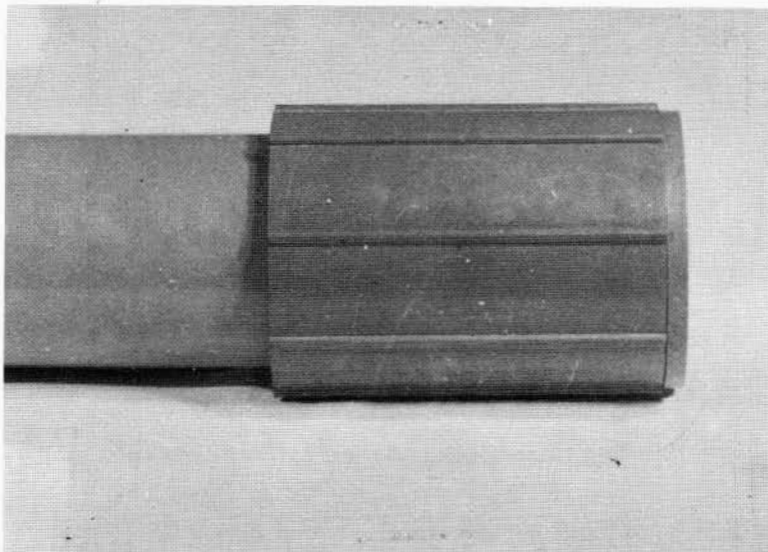
FIGURE 5-45. - View of PVC pipe outlet closed.
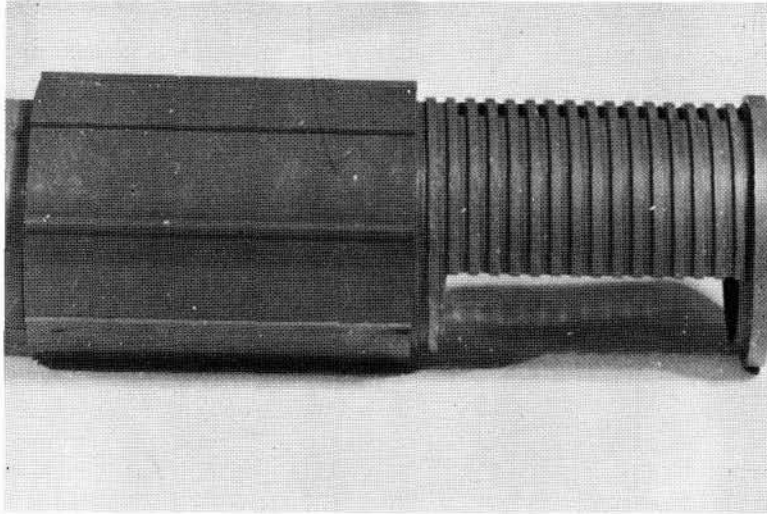
FIGURE 5-46. - View of PVC pipe outlet open.

As the pipe assembly is made of light weight material it is easy to transport and to install. Mass production is possible and its cost is low ($\frac{1}{4}$ to $\frac{1}{6}$ of the cost of existing farm turnouts). The loss of head is low and the discharge from the outlet can be controlled easily by the screwed stopper device. The outlet is susceptible to damage by heat but is not inflammable. It would be weak to mechanical impact at temperatures below freezing point.

Discharge through the outlet is accurate provided the head and the length and diameter of the pipe are carefully designed.

5.17.3   Hydraulics and Design Formula

In the design of this outlet the standard pipe flow formula is used:

$$Q = C A_x \sqrt{2 g h_{(wk)}}$$

where

Q	$=$	discharge of the outlet in m^3/s;
A_x	$=$	internal cross sectional area of PVC pipe in m^2;
$h_{(wk)}$	$=$	difference between designed full supply level in the supply canal and water level of the field channel in m;
g	$=$	acceleration due to gravity in m/s^2;
C	$=$	coefficient derived from the following formula:

$$C = \frac{1}{\sqrt{1 + C_{(f-IN)} + C_{(f-p)}}}$$

Longitudinal Section

Section A-A

Section C-C

Section B-B

Detail of Outlet Port

Plan

Note: All dimensions are in mm
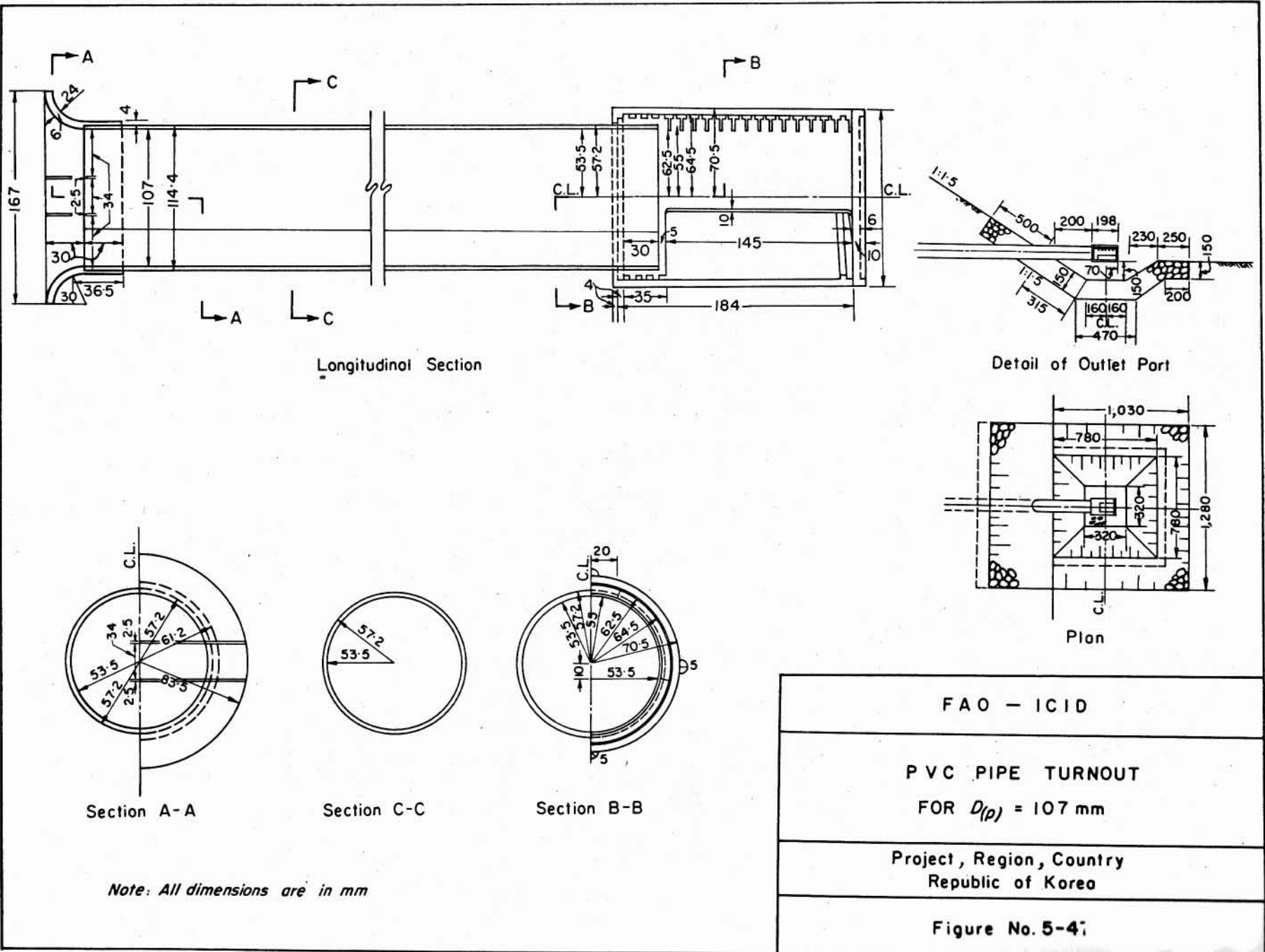
FAO — ICID

P V C   P I P E   T U R N O U T

FOR $D_{(p)} = 107$ mm

Project, Region, Country
Republic of Korea

Figure No. 5-47

363.

where $C_{(f-IN)}$ = loss coefficient of inlet, screen (bars) and outlet part (by test it has been found that the coefficient = 1.135);

$C_{(f-p)}$ = loss coefficient for friction in pipe having a value of
$$\frac{8gN^2}{R_{(H)}^{1/3} \cdot L_{(p)}/D_{(p)}}$$

N = roughness coefficient = 0.01 of this material

$D_{(p)}$ = internal diameter of pipe;

$L_{(p)}$ = length of pipe;

$R_{(H)}$ = hydraulic radius.

Figure 5-47 is a standard design drawing for a PVC pipe turnout with $D_{(p)}$ = 107 mm. Table 5-4 shows velocity and discharge of a PVC turnout for $D_{(p)}$ = 107 mm. Figure 5-48 is a discharge diagram for a PVC turnout with $D_{(p)}$ = 107 mm.
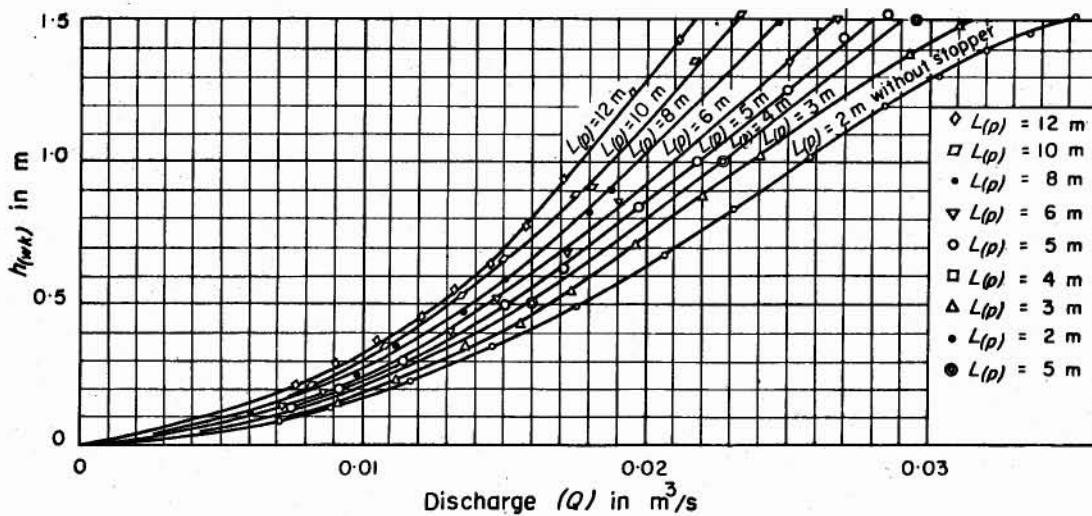
FIGURE 5-48. - Discharge diagram for a PVC pipe turnout for $D_{(p)}$ = 107 mm.

5.17.4   Numerical Example

The difference in the designed full supply level in the supply canal above the water level in the field canal is 0.5 m. The internal diameter of the PVC pipe is 107 mm and its length is 6 m. Calculate the discharge of the outlet.

We have $h_{(wk)}$ = 0.5 m

TABLE   5-4

Velocity and Discharge of PVC Pipe Turn-out, $D_{(p)}$ = 107 mm
D = 0.107 m; A = 0.008992 m²

| Working head $h_{(wk)}$	$L_{(p)}$ = 2 m		$L_{(p)}$ = 3 m		$L_{(p)}$ = 4 m		$L_{(p)}$ = 5 m		$L_{(p)}$ = 6 m		$L_{(p)}$ = 8 m		$L_{(p)}$ = 10 m		$L_{(p)}$ = 12 m	
	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q
0.15	1.057	0.0095	1.101	0.0090	0.946	0.0085	0.912	0.0082	0.857	0.0077	0.790	0.0071	0.757	0.0068	0.690	0.0062
0.20	1.224	0.0110	1.168	0.0105	1.079	0.0097	1.046	0.0094	1.024	0.0092	0.923	0.0083	0.901	0.0081	0.823	0.0074
0.25	1.358	0.0122	1.301	0.0117	1.224	0.0110	1.179	0.0106	1.146	0.0103	1.068	0.0096	1.024	0.0092	0.957	0.0086
0.30	1.502	0.0135	1.424	0.0128	1.346	0.0121	1.291	0.0116	1.257	0.0113	1.179	0.0106	1.118	0.0100	1.068	0.0096
0.35	1.625	0.0146	1.536	0.0138	1.469	0.0132	1.402	0.0126	1.346	0.0121	1.291	0.0116	1.213	0.0109	1.168	0.0105
0.40	1.747	0.0157	1.658	0.0149	1.558	0.0140	1.502	0.0135	1.435	0.0219	1.380	0.0124	1.313	0.0118	1.257	0.0113
0.45	1.858	0.0167	1.758	0.0158	1.658	0.0149	1.591	0.0143	1.524	0.0137	1.469	0.0132	1.390	0.0125	1.345	0.0121
0.50	1.970	0.0177	1.847	0.0166	1.758	0.0158	1.680	0.0151	1.602	0.0144	1.536	0.0136	1.469	0.0132	1.424	0.0128
0.55	2.070	0.0186	1.936	0.0174	1.847	0.0166	1.758	0.0158	1.680	0.0151	1.613	0.0145	1.536	0.0138	1.480	0.0133
0.60	2.170	0.0195	2.014	0.0181	1.925	0.0173	1.847	0.0166	1.758	0.0158	1.691	0.0152	1.602	0.0144	1.558	0.0140
0.65	2.259	0.0203	2.092	0.0188	2.014	0.0181	1.925	0.0173	1.847	0.0166	1.758	0.0158	1.680	0.0151	1.613	0.0145
0.70	2.337	0.0210	2.170	0.0195	2.081	0.0187	1.992	0.0179	1.914	0.0172	1.836	0.0165	1.747	0.0157	1.680	0.0151
0.75	2.426	0.0218	2.237	0.0201	2.159	0.0194	2.070	0.0186	1.981	0.0178	1.903	0.0171	1.803	0.0162	1.736	0.0156
0.80	2.515	0.0226	2.326	0.0209	2.237	0.0201	2.148	0.0193	2.059	0.0185	1.970	0.0177	1.869	0.0168	1.791	0.0161
0.85	2.593	0.0233	2.392	0.0215	2.303	0.0207	2.214	0.0199	2.136	0.0192	2.025	0.0182	1.925	0.0173	1.836	0.0165
0.90	2.682	0.0241	2.481	0.0223	2.370	0.0213	2.281	0.0205	2.203	0.0198	2.081	0.0187	1.981	0.0178	1.869	0.0168
0.95	2.760	0.0248	2.559	0.0230	2.448	0.0220	2.359	0.0212	2.259	0.0203	2.136	0.0192	2.036	0.0183	1.914	0.0172
1.00	2.837	0.0255	2.637	0.0237	2.526	0.0227	2.426	0.0218	2.337	0.0210	2.192	0.0197	2.092	0.0188	1.970	0.0177
1.05	2.915	0.0262	2.715	0.0244	2.593	0.0233	2.492	0.0234	2.403	0.0216	2.248	0.0202	2.136	0.0192	2.014	0.0181
1.10	3.004	0.0270	2.793	0.0251	2.671	0.0240	2.570	0.0231	2.470	0.0222	2.303	0.0207	2.192	0.0197	2.059	0.0185
1.15	3.082	0.0277	2.871	0.0258	2.748	0.0247	2.637	0.0237	2.526	0.0227	2.359	0.0212	2.237	0.0201	2.103	0.0189
1.20	3.160	0.0284	2.949	0.0265	2.815	0.0253	2.704	0.0243	2.593	0.0233	2.415	0.0217	2.281	0.0205	2.148	0.0193
1.25	3.249	0.0292	3.027	0.0272	2.871	0.0258	2.771	0.0249	2.648	0.0238	2.470	0.0222	2.337	0.0210	2.192	0.0197
1.30	3.349	0.0301	3.116	0.0280	2.949	0.0265	2.849	0.0256	2.715	0.0244	2.526	0.0227	2.381	0.0214	2.237	0.0201
1.35	3.461	0.0311	3.205	0.0288	3.027	0.0272	2.915	0.0262	2.771	0.0249	2.582	0.0232	2.437	0.0219	2.281	0.0205
1.40	3.561	0.0320	3.283	0.0295	3.082	0.0277	2.982	0.0268	2.837	0.0255	2.637	0.0237	2.481	0.0223	2.326	0.0209
1.45	3.694	0.0332	3.394	0.0305	3.160	0.0284	3.049	0.0274	2.904	0.0261	2.693	0.0242	2.526	0.0227	2.370	0.0213
1.50	3.895	0.0350	3.494	0.0314	3.227	0.0290	3.116	0.0280	2.971	0.0267	2.748	0.0247	2.582	0.0232	2.415	0.0217

NOTE: Inside of dotted lines, the Table is in the range of less than 3 per cent difference from computed discharges.

$$D_{(p)} = 0.107 \text{ m}$$

$$L_{(p)} = 6 \text{ m}$$

$$\text{Now} \quad R_{(H)} = \frac{A_{(x)}}{P_{(w)}} = \frac{\pi \frac{0.107^2}{4}}{\pi \; 0.107} = 0.02675 \text{ m}$$

$$C_{(f-p)} = \frac{8 \, g \, N^2}{R_{(H)}^{\frac{1}{3}}} \cdot \frac{L_{(p)}}{D_{(p)}} = 1.471, \quad N = 0.01$$

$$C = \frac{1}{\sqrt{1 + C_{(f-IN)} + C_{(f-p)}}}$$

$$= \frac{1}{\sqrt{\left\{1 + 1.135 + 1.471\right\}}}$$

$$= 0.527$$

$$\therefore \quad Q = C \, A \sqrt{2 \, g \, h_{(wk)}} = 0.528 \cdot \frac{\pi \; 0.107^2}{4} \sqrt{19.6 \cdot 0.5}$$

$$= 0.0148 \text{ m}^3/\text{s}.$$

The difference from Table 5-4 is 0.0004 m^3/s and is immaterial for practical purposes.

The discharge can be found easily from the discharge diagram, Figure 5-48 (prepared by hydraulic test).

If this discharge is less than the design discharge, use a larger diameter pipe or two pipes in parallel.

5.18 PIPE OUTLET (PHILIPPINES)[1]

5.18.1 General

A Pipe Farm Outlet with a standard inlet was introduced in the Philippines

[1] Based on information supplied by the Philippines National Committee, ICID.

in 1969 to be used in all National Irrigation Systems in the country.

With the water management programme being undertaken by the National Irrigation Administration (NIA), it became necessary to gate all irrigation turn-outs for proper control of water.   At the start of the programme, the NIA needed to provide about 20,000 gated turnouts.   The existing turnout headwalls in the Philippines were designed to carry concrete pipe conduits.   These headwalls cost about $ 22.00 for a 12-inch diameter pipe, while the corresponding standard hand-operated steel gates cost about $ 22.00, or a total of $ 44.00 per complete unit.   For the 20,000 gated turnouts, the total outlay would be $ 880,000, an amount which the operation and maintenance budget of the NIA could not finance.

Therefore, due to the urgency of installing gated turnouts and the necessity for adjusting the elevations of some of the existing ones, a pre-cast headwall has been designed, which is supported by the pipe conduit instead of vice versa as in the case of the earlier structures.   It is light and can be carried by hand to the site even during the rainy season.   It can be installed in a few hours without substantially affecting the water supply in the canal.   If the elevation of an outlet structure has to be adjusted, it is easy to accomplish and at little cost.   The pre-cast headwall for a 12 inch diameter outlet costs only $2.00 installed while the corresponding steel gate costs only $3.00 or $5.00 per unit.

5.18.2   Structural and Hydraulic Characteristics

The outlet structure consists of a prefabricated headwall and a gate (Figure 5-49) and a pipe.   The gates are designed for manual operation.   The opening of the gate is controlled at the stem of the headwall.   The original design did not foresee that farmers would use the stem for tethering their water buffaloes; the revised design takes this into account.

The structure is on the whole durable although there are some reservations about the concrete stem.   All components are prefabricated and easy to assemble by unskilled labour.   Quality control and mass production is possible using local labour and minimum supervision.   Installation can be done at any time of the year.

The maximum head recommended for this outlet is 20 cm.

FAO - ICID

PIPE FARM OUTLET WITH

STANDARD INLET

Project, Region, Country
Philippines

Figure No. 5-49

The diameter of the pipe is usually dictated by the standard commercial **pipe** available. The nearest larger diameter pipe to the design diameter is usually adopted. The intake of water is controlled by the pre-determined and permanent calibration of the gate stem. The degree of accuracy is on the low side.

The capacity of the turnout is from 20 to 90 l/s, which will serve from 10 to 30 ha of land.

5.18.3 Design Formula

The usual pipe formula is used, namely:

$$Q = CA_x \sqrt{2gh_{(wk)}}$$

where   Q   =   design discharge;

       A_x =   internal cross sectional area of concrete pipe;

       g   =   acceleration due to gravity;

       $h_{(wk)}$ =   head of water - the difference in water level at the inlet and outlet of the structure;

       C   =   coefficient related to transition, contraction, friction and other losses, which may be determined from actual discharge measurements through a typical headwall.

On a fairly well finished headwall, a coefficient of 0.6 is a conservative value.

5.18.4 Numerical Example

A farm having an irrigable area of 15 ha requires water from an irrigation canal. The water duty at maximum demand is about 3 l/s/ha. The difference in the water surface of the supply canal and the farm canal at maximum designed capacity should not exceed 0.05 m. Design the size of outlet required.

       Q   =   0.003 x 15 = 0.045 m^3/s, maximum design discharge of farm canal

       $h_{(wk)}$ =   0.05 cm

       C   =   0.60 (conservative assumption)

       Q   = $CA_x \sqrt{2gh_{(wk)}}$

$$0.045 \quad = \quad 0.6 \ A_x \ \sqrt{19.6 \ . \ 0.05}$$

$$A_x \quad = \quad 0.076 \ m^2$$

$$\therefore \quad D_{(p)} \quad = \quad 2 \ \sqrt{\frac{0.076 \ m}{3.14}}$$

$$= \quad 0.31 \ m$$

Use standard size of concrete pipe with diameter of 31 cm, say, 12 inches.

The crown of the outlet pipe must be set at least 0.05 m below the water surface elevation.

5.19   GATED PIPE OUTLET (FERRARA TYPE, ITALY)

5.19.1   Application and Structural Features

This Gated Pipe Outlet has been developed by the "Consorzio della Grande Bonificazione Ferrarese" in Ferrara, Italy, and is widely used in the irrigation districts of the Po Delta in northern Italy, particularly for rice cultivation.

FIGURE 5-50. - Prefabricated gated outlet (Ferrara type).

As Figures 5-50 to 5-55 illustrate, the structure consists of a covered con-
crete pipe, connecting the distribution canal with the farm watercourse or basin to
be irrigated. At a convenient place along the pipe (normally near the downstream
end) there is a stand which houses a flow regulating gate. As the figures show, the
stand consists of a concrete block fitted to the pipe and a covering slab or box. If
required the stand can be extended by putting a second block on top of the bottom
one. All parts of the stand are prefabricated. The metallic concrete moulds are
hired to contractors by the "Consorzio" for fabrication of the concrete parts as
required. The normal sheet metal slide gate is operated from above, as can be
seen from the Figures. The Figures also show that the stand is equipped with an
anti-tampering locking mechanism.
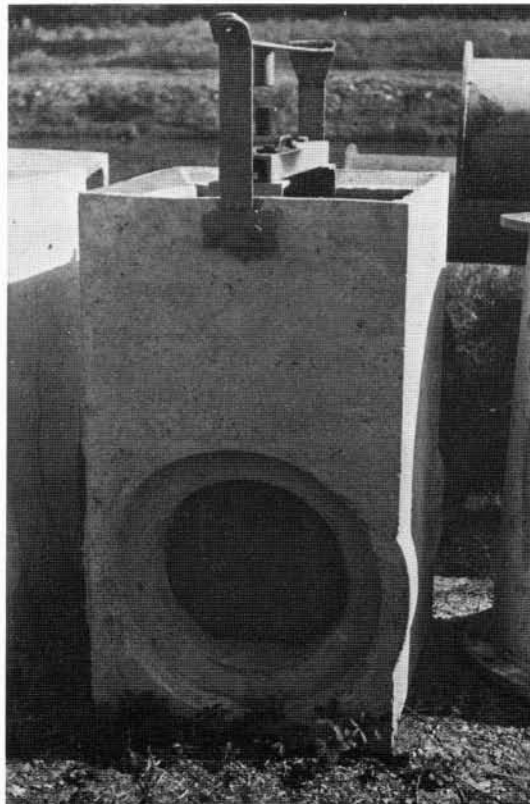
FIGURE 5-51. - Prefabricated gated outlet (Ferrara type)
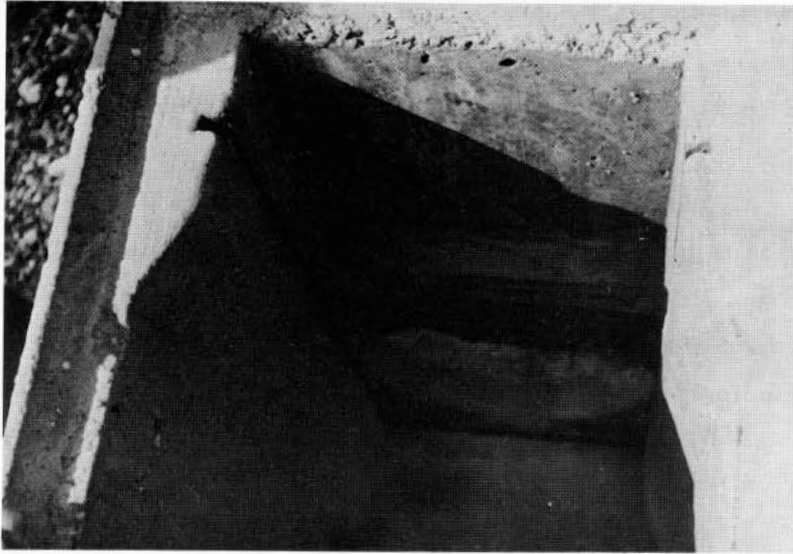
FIGURE 5-52. - Prefabricated gated outlet (Ferrara type).
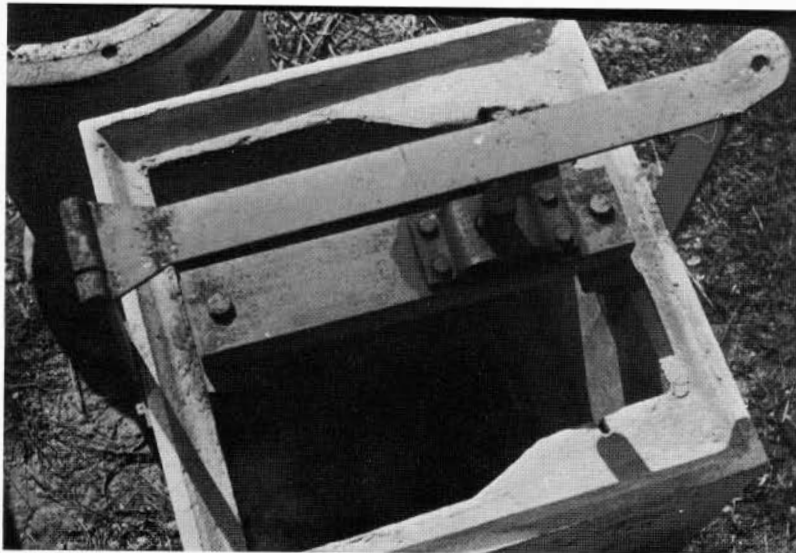
FIGURE 5-53. - Prefabricated gated outlet (Ferrara type).

Longitudinal section
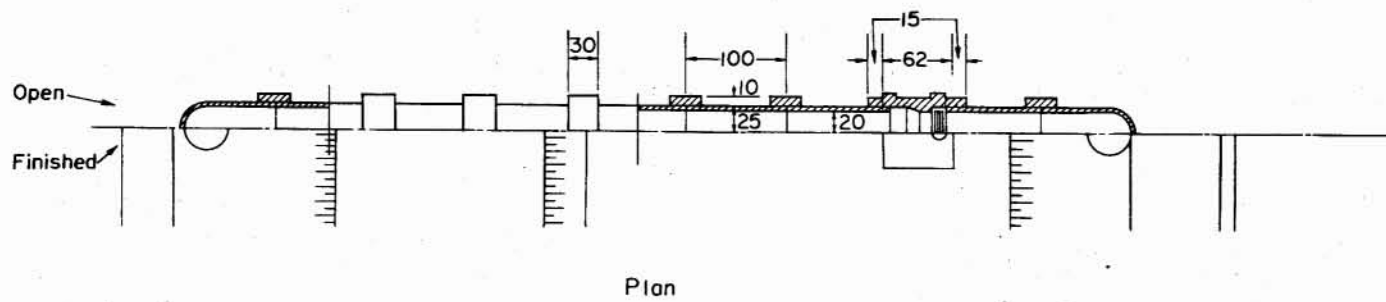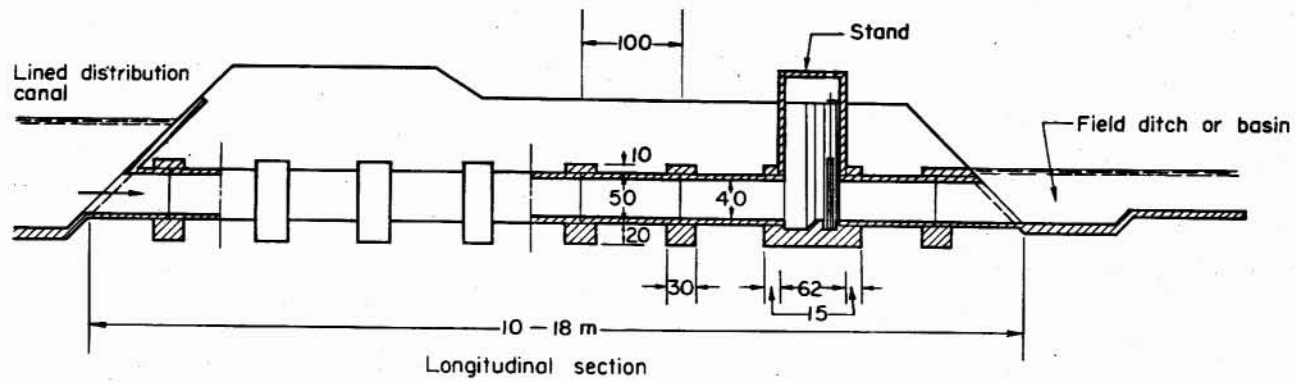
Plan

Note: Unless otherwise stated, all dimensions are in centimetres.

FAO – ICID
GATED PIPE OUTLET (FERRARA TYPE)
Project, Region, Country Italy
Figure No. 5-54 (a)

62

50

74

Extension
block

Pipe stand

104

40
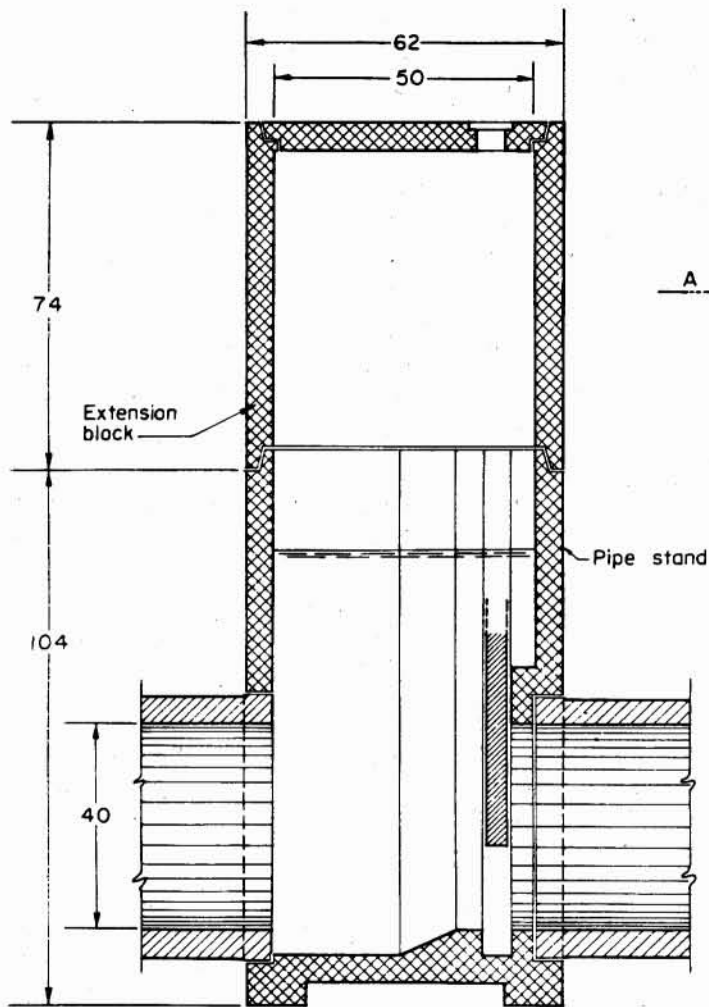
Section A-B

Note: All dimensions are in centimetres.

62

50

25

20

A   62   70   B

12   38   12

62
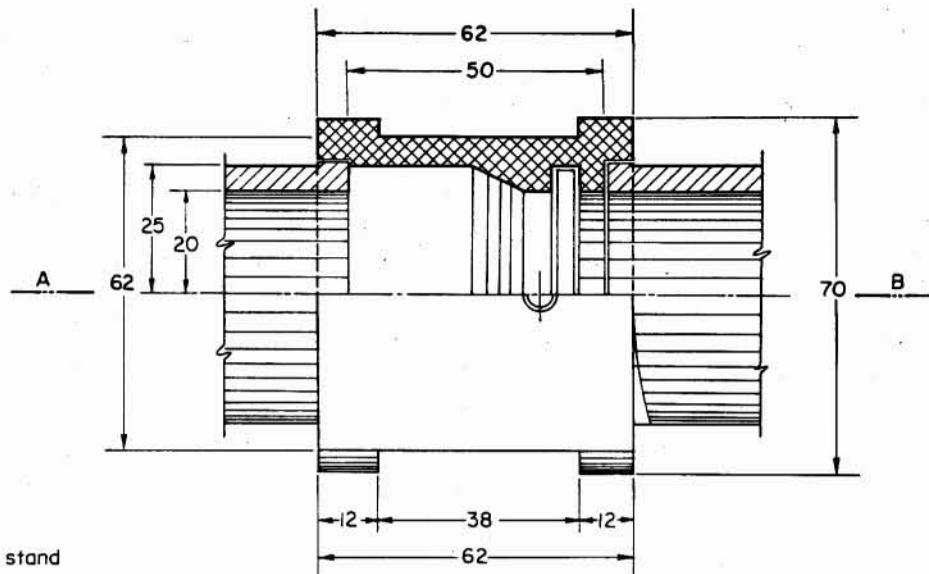
Plan of stand

FAO — ICID

PIPE STAND IN GATED PIPE OUTLET
(FERRARA TYPE)

Project, Region, Country
Italy

Figure No. 5-54 (b)

379.

① Front view

Note: All dimensions are in millimetres.

FAO – ICID

COMPLETE OUTLET STAND (HIGH TYPE)
FOR φ 400 mm PIPE

Project, Region, Country
Italy

Figure No. 5-55 (a)

100 mm

250

1,350

1,100

ϕ 400

Note: All dimensions are in millimetres.

600 x 400 steel gate

620

FAO-ICID

COMPLETE OUTLET STAND (LOW TYPE)
FOR ϕ 400 mm PIPE

Project, Region, Country
Italy

Figure No. 5-55 (b)
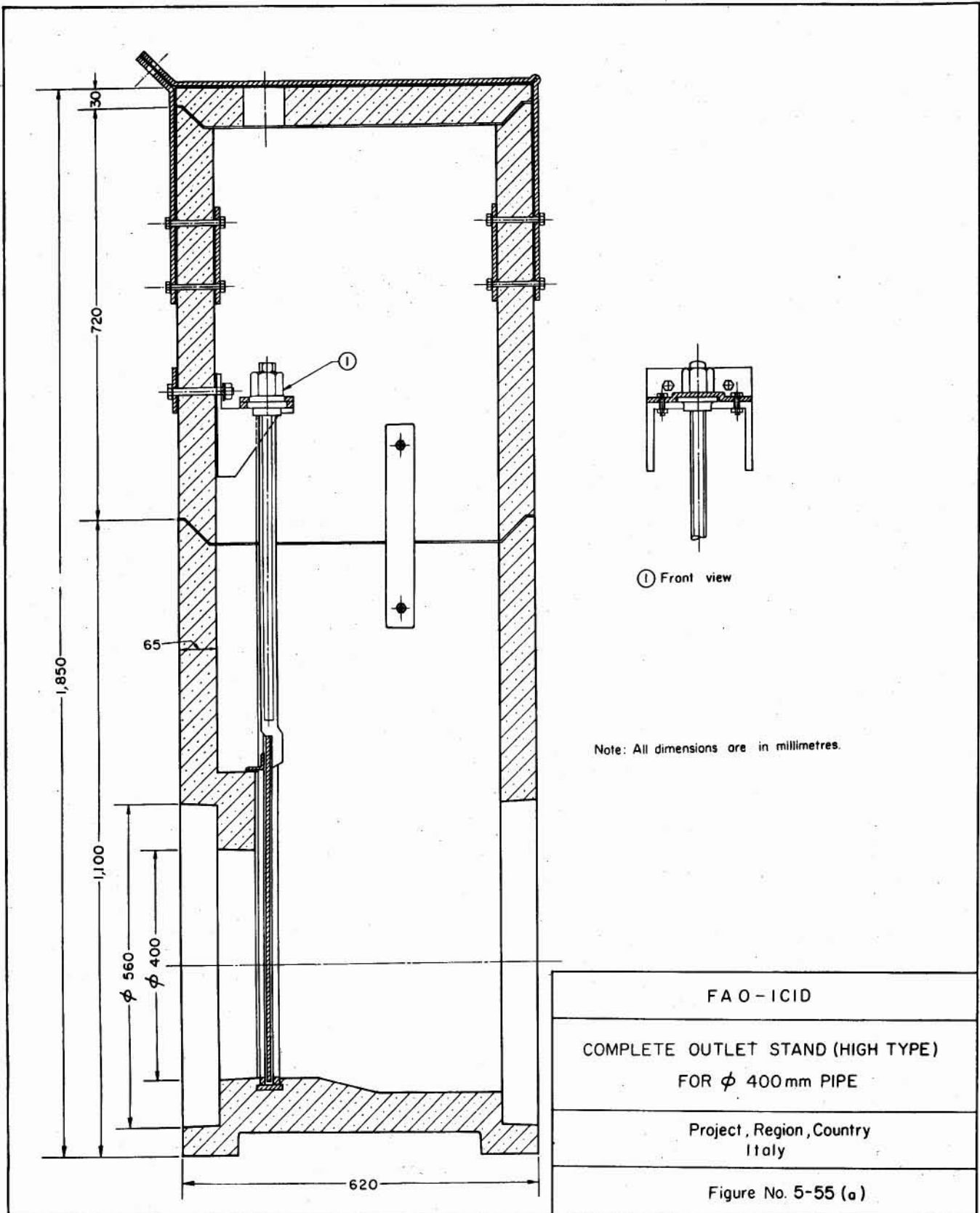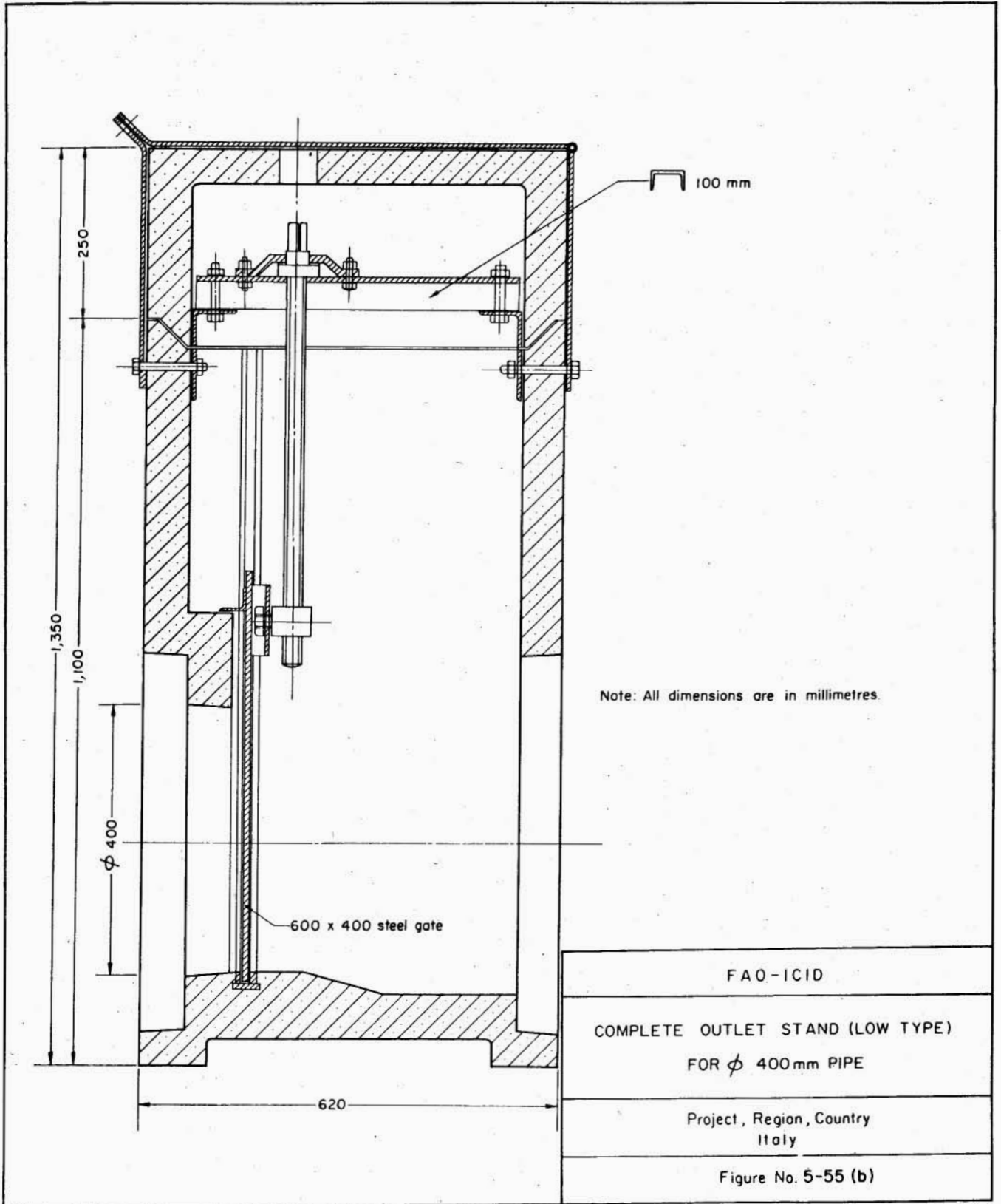
Materials required for a 10 m long outlet are:

stand (low type)   - $0.25 m^3$ concrete; 25 kg reinforcement
                                 steel; 53 kg steel for the gate.

pipe   -   9 m prefabricated concrete pipe; $2.3 m^3$
                                 in situ concrete; 69 kg reinforcement
                                 steel.

The total cost for prefabrication and installation of a 10 m outlet amounted to approximately $ 450 in 1970, of which the gated stand accounted for approximately $ 170.

This outlet is a simple and reliable structure adaptable to any bank width. It is particularly suited to areas where the working head of outlets has to be kept small, i.e. where outlets have to work under submerged conditions. Operation of the structure is simple and tampering is not possible. The gate can be used for instantaneous discharge measurements. Outlet gates for smaller pipe diameters are being developed.

5.19.2   Hydraulic Properties and Operation

The outlet is basically non-modular because the downstream end is usually submerged. The discharge for a given opening is thus a function of the difference in levels between the water surface in the supply canal and the farm or field watercourse. The relationships between working head, gate opening and discharge have been determined experimentally for the standard 400 mm diameter outlet under submerged conditions and are shown in Table 5-5.

In Table 5-5 the gate opening is converted into the number of revolutions of the screw used to lift the gate. This enables the table to be used directly for determining the discharge for any given head. The head is read from staff gauges permanently installed at both ends of the outlet. The table shown is a sample extract from the original table which ranges from 3 to 32 screw revolutions and correspondingly from 2.5 to 407 litres per second. A discharge formula for this outlet is not available.

TABLE 5-5

Discharge Table for 400 mm Diameter Gated Pipe Outlet
(Submerged Condition)

$h_{(wk)}$	Number of screw revolutions										
	14	15	16	17	18	19	20	21	22	23	24
	Discharge in l/s										
0.05	33.0	36.6	40.7	45.0	49.4	53.9	58.4	62.4	66.2	70.0	73.8
0.10	46.8	51.8	57.6	63.7	69.9	76.2	82.7	88.3	93.7	99.0	104
0.15	57.3	63.4	70.5	77.9	85.6	93.3	101	108	115	121	128
0.20	66.1	73.2	81.5	90.0	98.8	108	117	125	133	140	148
0.25	74.0	81.9	91.1	101	110	121	131	140	148	157	165
0.30	81.0	89.7	99.8	110	121	132	143	153	162	172	181
0.35	87.5	96.8	108	119	131	143	155	165	175	185	195
0.40	93.5	104	115	127	140	152	165	177	187	198	209
0.45	99.2	110	122	135	148	162	175	187	199	210	221
0.50	105	116	129	142	156	170	185	197	210	221	233
0.55	110	121	135	149	164	179	194	207	220	232	245
0.60	115	127	141	156	171	187	202	216	230	243	256
0.65	119	132	147	162	178	194	211	225	239	252	266
0.70	124	137	152	168	185	202	219	234	248	262	276
0.75	128	142	158	174	191	209	226	242	257	271	286
0.80	132	146	163	180	198	216	234	250	265	280	295
0.85	136	151	168	186	204	222	241	257	273	289	304
0.90	140	155	173	191	210	229	248	265	281	297	313
0.95	144	159	178	196	215	235	255	272	289	305	322
1.00	148	164	182	201	221	241	261	279	296	313	330
1.05	152	168	187	206	226	247	268	286	304	321	338
1.10	155	172	191	211	232	253	274	293	311	328	340
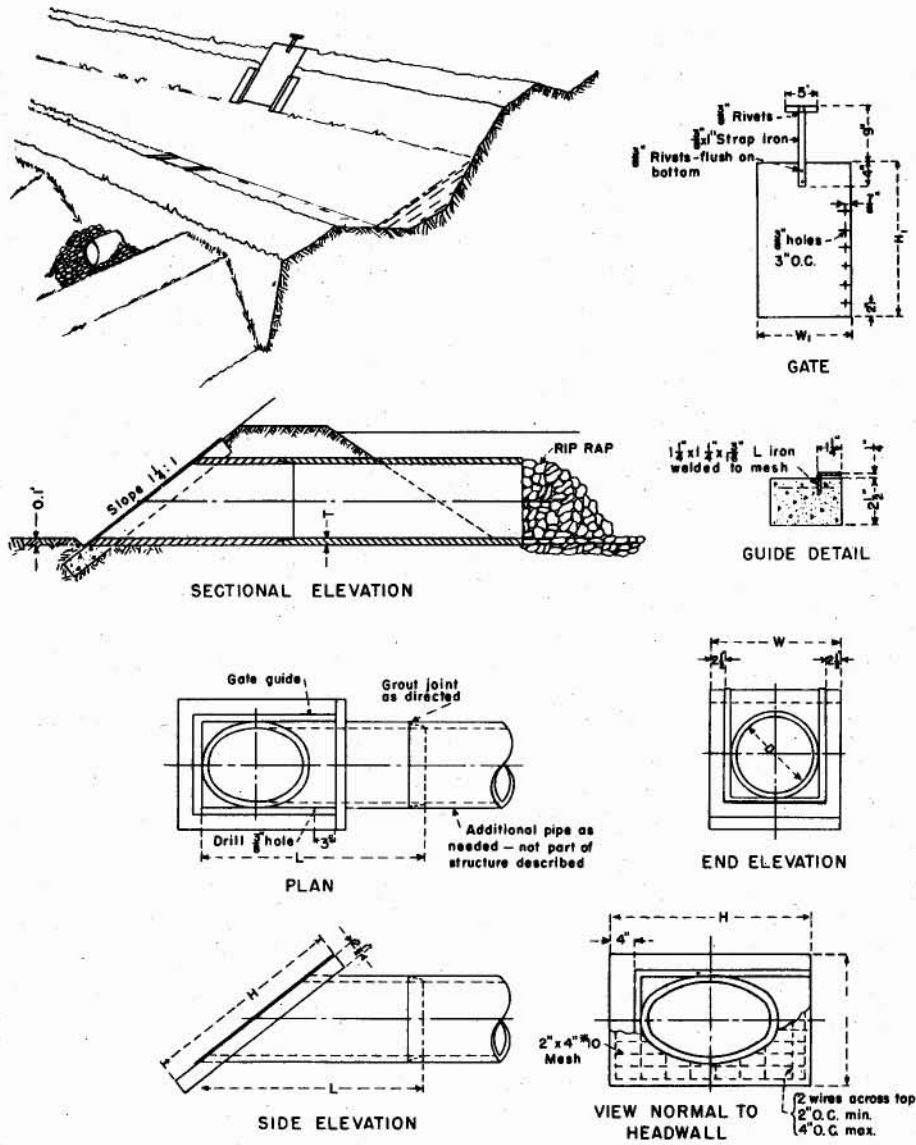
DIMENSIONS & QUANTITIES FOR DIFFERENT CAPACITIES AS INDICATED

PIPE			HEADWALL		GATE			QUANTITIES				CAPACITY
D	L	T	H	W	H_1	W_1	Ga.	Conc. C.F.	wire mesh 2"x4" No. 10	wt. of gate	wt. of iron L & Strap	Cu.ft./Sec.
6"	36"	⅝"	22	17	14	11½	16	0.69	20"x15"	2.8	7.8	0.7 to 1.0
8"	36"	⅝"	25	17	18	11½	16	0.82	23"x15"	3.6	8.5	1.2 to 1.7
10"	36"	⅞"	29	21	22	15½	16	1.12	27"x19"	5.9	10.0	1.6 to 2.4
12"	36"	1"	33	21	25	15½	14	1.35	31"x19"	8.3	11.0	2.4 to 3.1

FIGURE 5-56. - Concrete pipe outlet from primary to secondary farm ditches. (13)

5.20   OUTLET STRUCTURES ON THE FARM

On the farm outlet structures and devices are used to divert water from a primary into a secondary watercourse or ditch or from a head ditch onto the field. Structures for these purposes are usually small with capacities from a few litres per second up to the maximum discharge delivered to the farm.

The type and capacity of field outlets depends primarily on the method of irrigation.   In basin irrigation, outlets are spaced along each side of the supply ditch.   Each outlet may serve one or several successive basins interconnected with control gates.   Capacities of outlets may be as high as 500 litres per second. Outlets should be provided with slide gates or flashboards in order to allow control of flow and basin water level; this is of paramount importance in rice irrigation.

·A commonly used permanent concrete pipe outlet discharging from primary ditches into field ditches is shown in Figure 5-56.   This type can be used as a permanent field outlet as well.   Another type of concrete pipe outlet with a tap at one end is shown in Figure 5-57.

Table 5-6 gives the approximate discharge of these types of outlets. Reference may also be made to the pipe outlets discussed in previous Sections of this chapter.

TABLE   5-6

Flow through Concrete Pipe Field Outlet in Litres per Second

Diameter of pipe (cm)	Pressure head - cm				
	5	10	15	20	25
20	18.7	26.4	32.3	37.3	41.8
25	29.2	41.3	50.5	58.4	65.2
30	42.0	59.4	72.8	84.0	94.0
35	57.2	80.9	99.1	114.4	127.9

PERSPECTIVE

SECTION
OF TAP

metal strip

h
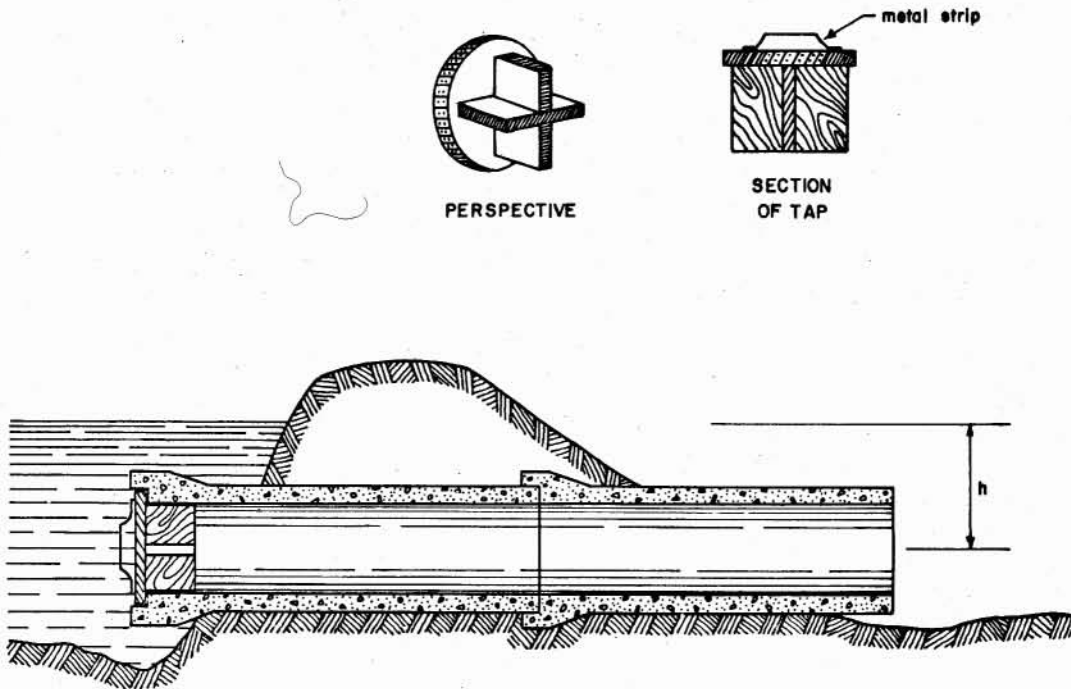
FIGURE 5-57. - Concrete pipe outlet with tap at one
end.

Figures 5-58, 5-59 and 5-60 show commonly used wooden outlet boxes.
For the selection of a suitable width for such outlets reference may be made to
Table 5-7.
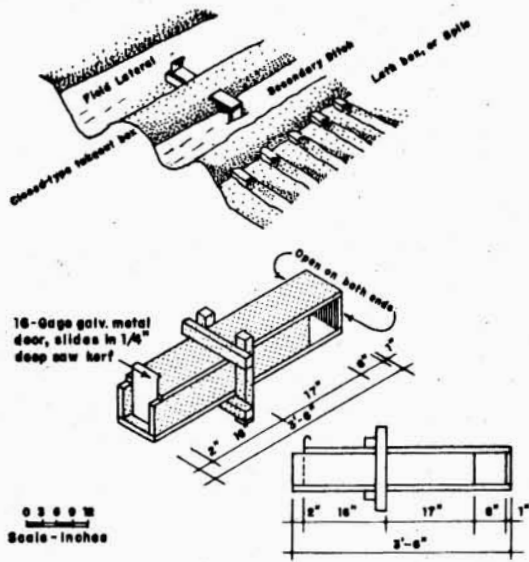
FIGURE 5-58. - Wooden outlet for furrow irrigation (13).
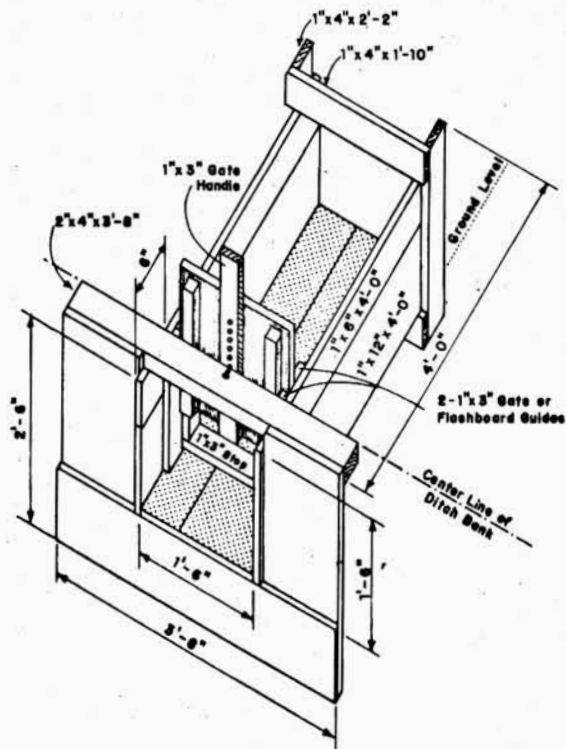
FIGURE 5-59. - Wooden outlet for basin or border
irrigation - maximum discharge around 85 l/s (13).

FIGURE 5-60. - Wooden outlet for basin or border
irrigation.

TABLE   5-7

Approximate Discharge of Small Wooden Field Outlets

Depth of water over the sill at the intake cm	Discharge per 10 cm width of sill [1] litres per second
10	6
15	11
20	17
25	22

[1] Valid in the range of 30 to 100 cm width.

The most popular type of outlet device in furrow irrigation is the siphon. Siphons are usually pre-formed from aluminium or plastic pipe, but are sometimes made of flexible materials such as butyl rubber or corrugated plastic. They have the advantages of easy installation and removal without disturbing the ditch bank and portability reduces the number required. The flow can be regulated by changing the pressure head or varying the size or number of siphons. Commercially available siphons have capacities from 4 l/s up to several hundred l/s. The large ones are sometimes used as field outlets or for diversion of flow from lined into unlined ditches. Large ones require a device for priming.

TABLE 5-8

Flow through Small Siphons in Litres per Second

Diameter of syphon cm	Pressure head - cm						
	5	7.5	10	12.5	15	17.5	20
4	0.75	0.91	1.06	1.18	1.29	1.40	1.49
5	1.17	1.43	1.65	1.85	2.02	2.18	2.33
6	1.68	2.06	2.38	2.66	2.91	3.14	3.36
7	2.29	2.80	3.24	3.62	3.96	4.28	4.58
8	2.99	3.66	4.23	4.72	5.18	5.59	5.98
9	3.78	4.63	5.35	5.98	6.55	7.07	7.56
10	4.67	5.72	6.60	7.38	8.09	8.73	9.34

The flows through various sizes of siphons when operated under different pressure heads are given in Table 5-7. The pressure head is the difference in elevation between the water surface in the farm ditch and either the centre of the outlet if it is free flowing or the water surface above the outlet if it is submerged.

A disadvantage of the siphon is that it may become deprimed during operation due to falling water level or blockage by trash or silt. Recent research carried out by the USDA to remedy this problem has resulted in the Snake River auto-start siphon as shown in Figure 5-61. This siphon is equipped with a cup on each end that holds the water over the ends of the siphon so that air cannot enter the tube when the water supply level recedes. The cups hold enough water to

maintain the water level above the tube ends for 10 to 14 days; thus no repriming is required during this period. Both cups must be at approximately the same elevation. The minimum recommended cup diameter is 2.25 times the tube diameter. The length of the cup from bottom to lip equals $1.41 (E) - (D - d - S)$ where E = nominal 10 day evaporation (inches), D = diameter of cup (inches), d = tube diameter (inches), S = distance between bottom of cup and tube end (inches).
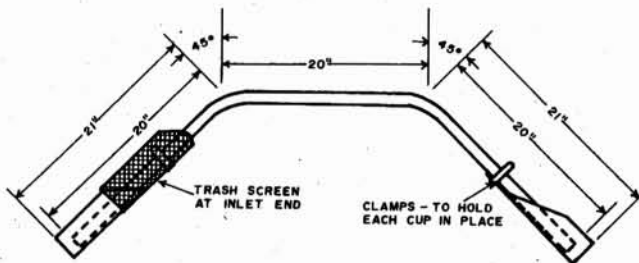
FIGURE 5-61. - Snake River auto-start siphon (89).

In border irrigation the same type of outlets as used for basin irrigation can be installed as well as siphons. Outlet capacity depends on the width of the check between two borders, the slopes and the soil, and may range from a few litres per second up to 300 l/s. Structures may be temporary (portable) or permanently installed and are usually equipped with flashboards or slide gates. Frequently used materials are wood and concrete. The bottom of the outlets should be placed at a lower elevation than the surface of the border so that the water will discharge into a pool at the downstream end.

(a)
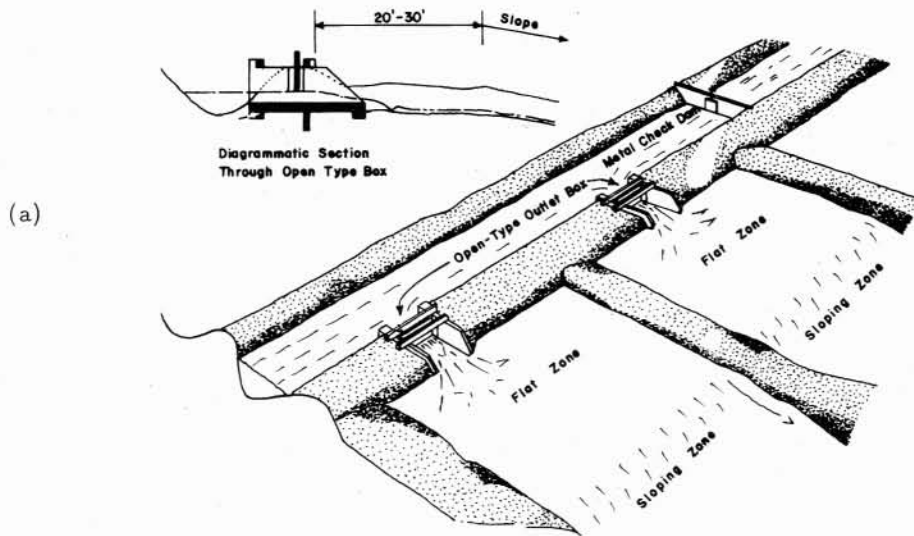
(b)
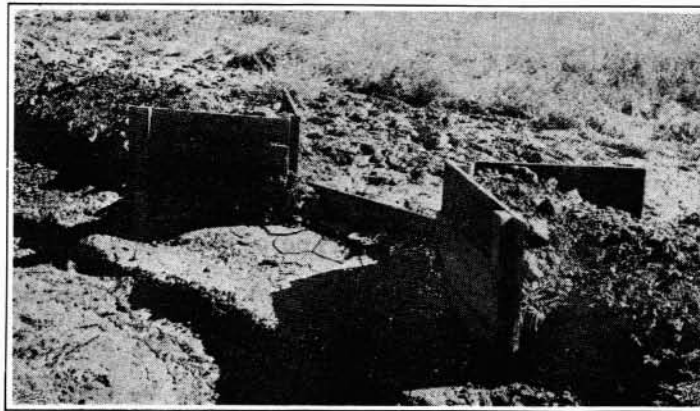
FIGURE 5-62 (a) and (b). - Outlet boxes for border irrigation. (13, 65)

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NOTATIONS AND SYMBOLS[1]

$\underline{A}$

Area	A
Area of cross section	A_x
Area of critical section	A_{xc}

$\underline{B}$

Breadth or width (usually across the axis of flow)	B
Bed width of canal upstream a fall, syphon, aqueduct, etc., and in parent channel in case of intakes and outlets	B_1
Bed width of canal downstream a fall, syphon, aqueduct, etc. Bed width in the offtake channel below the intake or watercourse below the outlet	B_2
Width of throat or controlling section or width of weir crest across the axis of flow	$B_{(t)}$
Width of inlet	$B_{(IN)}$
Width at outlet end	$B_{(OUT)}$
Width of gate opening or sluice opening	$B_{(go)}$
Width of stilling basin, cistern, etc.	$B_{(bas)}$ or $B_{(SB)}$

[1] For terminology and definitions reference should be made to the Multilingual Technical Dictionary on Irrigation and Drainage published by the ICID in 1967.

C

Coefficient of discharge	C_Q
Coefficient of roughness	N
Coefficient of submergence	$C_{(s)}$
Coefficient of submergence of hydraulic jump	$C_{(js)}$
Coefficient in Chezy's formula	C
Coefficient, approach velocity	C_v

D

Depth of canal	y
Designed depth of canal (if distinguished)	$y_{(dgn)}$
Depth of canal upstream of falls, proportional distributors or divisors, syphons, aqueducts, etc., and in parent channels of outlets and offtake channels	y_1
Depth of canal downstream of falls, etc. and depth of offtake channels below intakes and of watercourses below outlets	y_2
Depth of stilling basin	$d_{(bas)}$
Diameter	D
Diameter of pipe	Dp
Discharge	Q
Discharge intensity or discharge per unit width	q
Discharge in the parent canal	Q_1
Discharge of offtake channels or outlets	Q_2, Q_3
Small increment in discharge	dQ, dQ_1, dQ_2
Distances and spacings	x

E

Efficiencies $\quad\eta$

F

Flexibility $\quad$ (FL)

Free board $\quad$ (FB)

Froude number $\quad$ F

H

Height over hardings $\quad$ H

Head over crest, etc. $\quad H_{(crt)}$

Working head $\quad h_{(wk)}$

Head due to velocity of approach $\quad h_v$

Head loss $\quad h_{(\ell)}$

Height of gate opening $\quad H_{(go)}$

Height of crest above upstream bed level $\quad H_{(b-c)}$

Height of crest above bottom level of
stilling basin. $\quad H_{(C-SB)}$

Height of crest above downstream bed level $\quad H_{(c-b)}$

Height of upstream water level above
soffits of orifices, pipes, etc. $\quad H_{(sof)}$

Height of orifice above crest or bottom
level of control section $\quad H_{(orf)}$

Hydraulic drop $\quad H_{(dr)}$

Depth of flow at the beginning of hydraulic
jump or supercritical sequent depth $\quad H_{(j)_1}$

Depth of flow at the end of hydraulic jump
or subcritical sequent depth $\quad H_{(j)_2}$

Critical depth corresponding to minimum
energy $\quad H_c$

L

Length	L
Length of crest along the axis of flow	$L_{(crt)}$
Length of glacis	$L_{(gl)}$
Length of stilling basin	$L_{(bas)}$
Length of pipe	$L_{(p)}$
Length of jump	$L_{(j)}$

P

Proportionality	(PR)

R

Radius	R
Hydraulic radius	$R_{(H)}$
Ratio	r

S

Sensitivity	(SEN)
Shear stress	τ
Slope (longitudinal)	s
Side slope	(ss)

T

Thickness	(T)

V

Velocity	v
Critical velocity	v_c
von Karman's constant	k

<u>W</u>

Weights   W

Specific weight of fluid W_o

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Availability: January 1982

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