HANDBOOK OF IMPROVED IRRIGATION PROJECT OPERATIONS PRACTICES FOR THE KINGDOM OF THAILAND

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HANDBOOK OF IMPROVED IRRIGATION PROJECT

OPERATIONS PRACTICES

FOR THE KINGDOM OF THAILAND

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PREFACE

This study was conducted as part of the Water Management Synthesis II Project, a program funded and assisted by the United States Agency for International Development through the Consortium for International Development. Utah State University, Colorado State University, and Cornell University serve as co-lead universities for the Project.

The key objective is to provide services in irrigated regions of the world for improving water management practices in the design and operation of existing and future irrigation projects and give guidance for USAID for selecting and implementing development options and investment strategies.

For more information about the Project and any of its services, contact the Water Management Synthesis II Project.

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KINGDOM OF THAILAND

ROYAL IRRIGATION DEPARTMENT

HANDBOOK OF IMPROVED IRRIGATION PROJECT OPERATIONS PRACTICES

Operation and Maintenance Division

March 2531 (1988)

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Section I.

INTRODUCTION

A. SITUATION

Thailand is approaching the limit of new land available and suitable for agricultural settlement even with the help of irrigation. Also, the expansion of irrigated rice has slowed, particularly in the dry season. The major role of future irrigation development should, therefore, be to extend dry season irrigation (particularly for upland crops), improve drainage in low-lying areas, and improve the productivity of existing irrigation projects.

One of the current objectives of the Government is to encourage greater diversification of crops, both to: (a) avoid overproduction of rice; and (b) to increase production of crops which Thailand now imports, or which appear to have good export markets. A Royal Irrigation Department (RID) review of irrigation projects was conducted to determine areas suitable for diversification to upland crops. The best prospects appear to be in the North with its cooler dry season climate and lighter soils. There, farmers already have experience in growing upland crops and vegetables and areas exist which are capable of dry season cropping without heavy investment in on-farm development. Potential also exists in: (a) the Central Region's Mae Klong area, although much of the most suitable land there is already under sugarcane; and (b) selected projects in the Northeast, which has suitable soils and surplus dry season reservoir storage water. The Northeast's well-defined dry season is an important climatic advantage compared to other regions.

B. FUTURE APPROACH

The primary purpose of an irrigation system is to deliver water to individual farms on a timely basis that maximizes their welfare through increased crop production. In addition, the irrigation water supplies must be equitably distributed in order that all farmers share equally in project benefits.

During recent decades, the emphasis has been on constructing new irrigation projects in order to increase the amount of cropland that can be served by irrigation networks. Now, increasing attention is being given to making existing irrigation projects more productive. This change in emphasis is reflected in the major focus of the Royal Irrigation Department now being "operation and maintenance" rather than "construction."

To improve the productivity of existing irrigation systems, the major thrusts will be: (1) main system management; (2) tertiary system management; and (3) on-farm irrigation and agronomic practices. Main system management will be the primary responsibility of RID, while the Water Users Groups (farmers) will be responsible for tertiary system management with technical assistance provided by RID. On-farm irrigation and agronomic practices will be targeted on individual farmer's fields with technical support from the Department of Agriculture, Universities, Department of Agricultural Extension and RID.

C. MAIN SYSTEM MANAGEMENT

The "Operations and Maintenance Learning Process" will be the mechanism used for upgrading the maintenance on deteriorated systems and to improve the operation of these systems to provide more equitable and timely deliveries to Water Users Groups. This process begins with project staff receiving field training in "Developing a Maintenance Plan," followed by implementation of the plan during the next fiscal year. This allows the system to be maintained much better so that rehabilitation is not required as frequently. Soon afterwards, the project staff are given training on "Operation of Irrigation Systems," which is immediately followed by field data collection and improved operation within a few seasons that is continually refined each succeeding season. More equitable and timely water deliveries to each outlet will enhance RID credibility with farmers.

The sensitivity gained by project field staff in catching-up on deferred maintenance and improving water deliveries resulting from field data collection will allow them to determine much more accurately those irrigation system improvements that would result in increased agricultural production, as well as provide much better documentation on the cost-effectiveness of alternative improvements.

Section II.

OPERATIONS PHASE OF THE OPERATIONS AND MAINTENANCE LEARNING PROCESS

An irrigation system usually consists of two canals, with a canal to serve lands on each side of the river. Facing downstream, the Left Main Canal (LMC) provides water to croplands on the left side of the river. The Right Main Canal (RMC) delivers water to the farmers on the right side of the river.

Each main canal is located on the highest ground, if feasible, or placed on a fill if necessary, in order that the water surface elevation will be well above the ground surface of all the croplands to be served.

There are a number of laterals that feed from the main canal and then convey water to the far reaches of the lands to be served. Some of these laterals will also divert water into sub-laterals that extend even further the irrigation channel network.

There are many outlets from the sub-laterals and laterals, and even some outlets from the main canal, with each outlet supplying water to a tertiary system. Each outlet serves one Water Users Group. This water is conveyed through a network of irrigation channels within the tertiary system to the individual farmers, all of whom are members of the Water Users Group.

The flow from the tank into each main canal is controlled by separate gates located on the upstream base of the dam. These gates are used to regulate the quantity of water discharged into each main canal. In some cases, there is a diversion structure constructed along the river with gates that control the discharge rate into the canal.

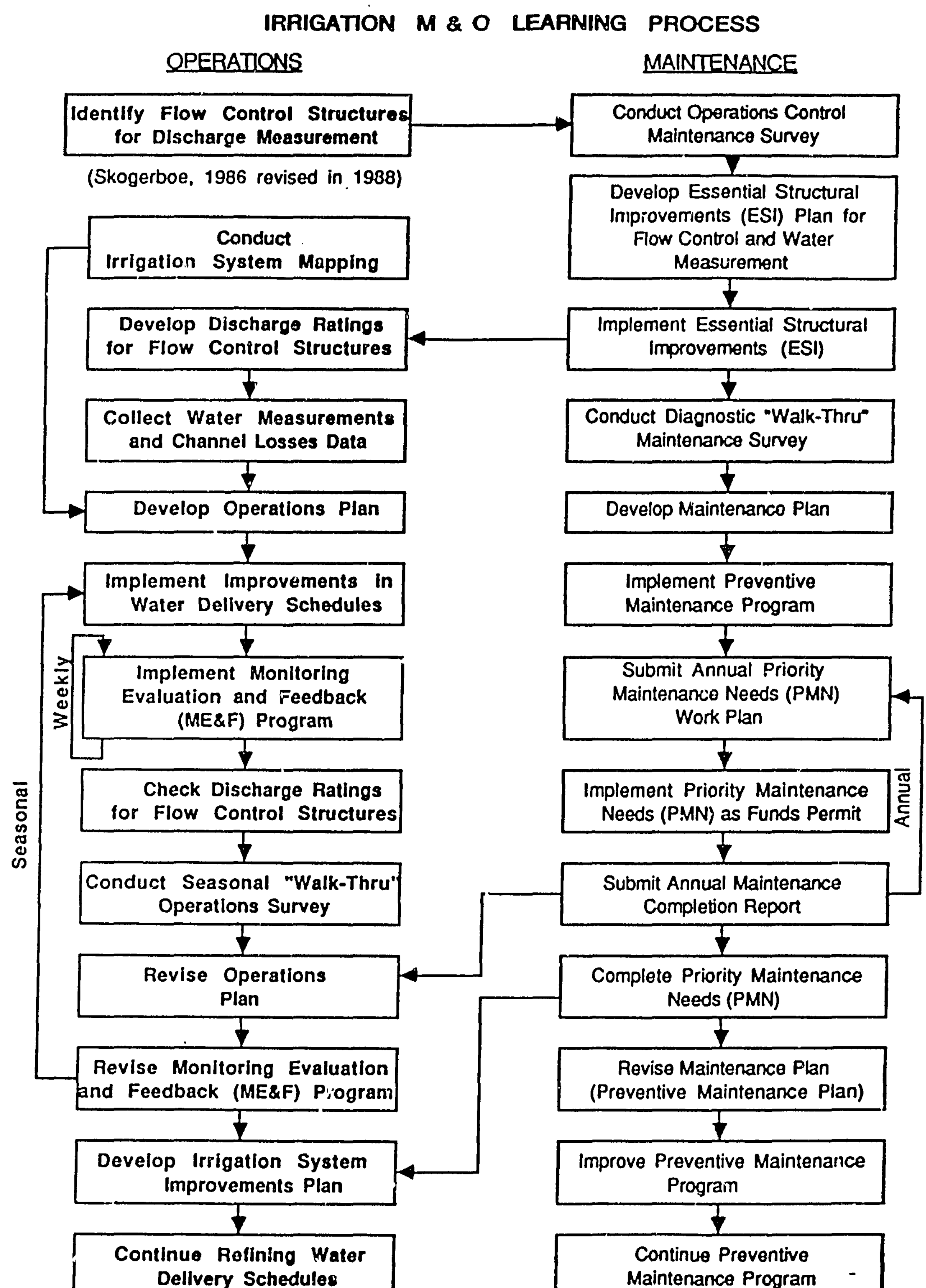

Figure 0-02-01. Operations Phase of the Operations and Maintenance Learning Process.

For each lateral branching off from the main canal, there is a Head Regulator consisting of one or more gates located on the bank of the main canal. This regulates the quantity of water discharged into the lateral. Also, there are Head Regulators located along the laterals that regulate the water supply into sub-laterals.

Many times, Head Regulators have to be placed above the bottom or bed of the irrigation channel so that the water levels are maintained above the ground surface elevation of the croplands to be served. This requires that a check structure be located downstream in the channel feeding the Head Regulator so that the water surface elevation will be sufficiently high to provide adequate flow rates through the Head Regulator.

Most outlet structures serving a Water Users Group consist of a gate structure having a single gate or a Constant Head Orifice (CHO) outlet having two gates. There are many additional structures such as culverts to allow roadways over the irrigation channels, drop structures to lower the elevation of the channel along steep lands, and inverted siphons to convey water underneath natural drainage channels subjected to flooding.

Hydraulic computations establish what the normal depths of flow will be in the various irrigation channels and how these flow depths will change along the length of each channel. However, these normal flow depths occur only in short reaches of some channels because of check structures, Head Regulators, culverts, outlets, etc. As the gate openings are changed in any of these structures, the flow depths immediately downstream will be affected for only a short distance, but the upstream flow depths will usually be affected for a long distance, often more than a kilometer and sometimes more than 10 kilometers. Also, it will likely take a few to many hours for the flow depths to stabilize after a change in any of the gate openings. Consequently, discharge rates and flow depths are frequently changing throughout the day.

The deposition of sediment on the bed of an irrigation channel reduces the cross-sectional area of flow. Consequently, the carrying capacity of the channel is reduced.

The roughness of the channel bed and banks can be increased in many ways, all of which reduces the carrying capacity of the irrigation channel.

Structures containing gates need periodic maintenance to avoid increased leakage, particularly when the gate is closed.

After making necessary structural improvements, a concerted effort will be needed to develop discharge ratings for all of the flow control structures. For the large main canals, a current meter could be used to calibrate each structure, whereas, portable flow measuring flumes can probably be used to calibrate the inlet structures for smaller irrigation channels such as sub-laterals. Lateral channel structures would be rated using portable flow measuring devices wherever feasible; otherwise, a current meter would be used.

Discharge ratings should be developed in the field for all of the major flow control structures: (1) outlets from the dam or river into the main canals; (2) Head Regulators for the laterals and sub-laterals; (3) check structures; and (4) outlets to Water Users Groups. All of these structures have gates that can be calibrated using known hydraulic formulas.

Each outlet to a tertiary system can be calibrated using a standardized flow measuring device, such as Cutthroat flume, located downstream in the tertiary channel. This discharge measurement, combined with measuring the water levels upstream and downstream from the outlet, provide the necessary data for calibrating the gate outlet.

For the other flow control structures, a current meter is usually required to determine the flow rate or discharge. A current meter measures the velocity of the water. By placing the current meter at many locations in the flow cross-section, the discharge can be calculated by also knowing the cross-sectional area of flow. The calculated discharge is combined with water level measurements upstream and downstream from the gate in order to develop the calibration for the gate structure.

To develop all of the necessary discharge ratings in an irrigation system will take many seasons, perhaps two to four years. A plan should be developed for accommodating the data collection as a part of the regular work program of the Water Master and Zoneman.

Primary water losses from irrigation channels are: (1) seepage; (2) leakage; and (3) evaporation. Usually, evaporation losses from the channel water surface is very small when compared with other losses and can be ignored. The primary source of leakage is at closed gate structures; if the leakage is significant, then the appropriate maintenance should be undertaken to reduce the amount of water being lost.

The greatest amount of water loss, even in concrete-lined channels, is due to seepage. Some water loss occurs because concrete is porous, but most of the water loss is through the joint openings

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between concrete panels and the cracks and holes in the concrete lining. Lack of proper maintenance of the channels greatly increases the seepage losses.

For earthen channels, water seeps through the earthen banks near the normal water surface levels because of biological life, which greatly increases the water losses. Also, the seepage loss rate increases rapidly when water levels exceed the usual operating levels.

After discharge ratings have been developed for each flow measurement structure, channel losses can be evaluated for most of the reaches in the irrigation network using the inflow-outflow method. In fact, stage readings collected at each structure prior to developing a discharge rating can be converted to calculated discharge rates that will be fairly accurate provided there has been no significant changes in channel sedimentation or vegetative growth, both of which are problems in many irrigation systems. Channel losses should be measured periodically throughout each irrigation season to determine the effects of channel water depths and surrounding water table depths on seepage rates.

In addition, it is advisable to use the "ponding method" at numerous locations as a check on the inflow-outflow measurements and for channel reaches where sufficient accuracy cannot be obtained by discharge measurements.

In the ponding method, two earthen dams covered with plastic sheets, to prevent seepage losses through the dams, are constructed at the upper and lower ends of a channel reach. Water is allowed to flow slowly over the plastic sheets at the upper dam in order to fill the pond between the two dams. The downstream dam should be constructed to the top of the irrigation channel and the water allowed to reach one to two centimeters of the top of the dam before the water is no longer allowed to pass over the upper dam. The water level in the pond will be level. The drop in water level over time is recorded in order to calculate the change in seepage loss rate with depth of water in the channel.

A combination of inflow-outflow and ponding measurements are used to evaluate the channel losses throughout the main system from the dam to all of the outlets to the tertiary systems.

Each irrigation project needs detailed maps showing all irrigation and drainage channels, along with all cropped lands served by these channels. These maps, along with adequate flow control structures that have been rated for discharge measurement, provide the tools for developing an Operations Plan.

Although the flow of water is from the reservoir to the last outlet in the main system, the development of an Operations Plan goes from the last outlet in each sub-lateral and moves upstream through the system to the reservoir.

A simple Operations Plan can be prepared on paper beginning with some methodology for establishing water requirements or water demands for each tertiary system that is served by an outlet from the main system. This is a function of the cropping pattern and area of each crop being cultivated, planting date, climate, soil type, channel losses in the tertiary system, and some additional factors. Thus, the water requirement will vary from one outlet to another, as well as varying at any single outlet throughout the season. The required flow rate at each outlet can be calculated throughout the irrigation season.

Then, by knowing the conveyance losses in the main system between the canal headworks and the inlet to the tertiary system, the discharge requirements at the canal headworks can be calculated. The remaining data that is needed will be the time lag from the time that the appropriate discharge rate is delivered into the canal headworks until it reaches the main system outlet serving the tertiary system. The time lag can be determined by field experience, but preferably by field measurements.

Implementing the Operations Plan will provide more equitable distribution of irrigation water supplies throughout the irrigation project because channel losses will be taken into account, and the water delivered to each outlet from the main system can be measured. Monitoring can improve water delivery schedules as more field data are collected, and those individuals doing the monitoring will become more sensitive about what is happening within the system.

There should be a Monitoring, Evaluation and Feedback program established so that the discharge measurements at outlets, Head Regulators and other structures are monitored and recorded by each Zoneman. This data should be compiled and evaluated by the Water Master, who should in turn report the evaluation to his supervisor. Feedback should be provided by the Water Master to each of the Zonemen on a weekly basis, while feedback from the supervisor to the Water Master can be done on a weekly, monthly,quarterly or seasonal basis. The Monitoring, Evaluation and Feedback program should also be designed to develop communication between the water users and RID personnel.

Periodically, the discharge ratings should be checked and adjusted, if necessary, which should become a routine operations procedure. Also, detailed field books should be kept to describe the physical condition of the structure and nearby channel each time a discharge rating is made. With periodic maintenance, the discharge rating for each structure will change very little with time. Although this technology is simple, periodic maintenance and attention to details are important in order to have discharge measurements that are accurate within five percent.

An important linkage is to develop communications between RID personnel and the water users. Each season, the Zoneman and Water Master should "walk-thru" their area of responsibility in the irrigation project and discuss operational problems with each Water Users Group Leader and the water users. Every attempt should be made to resolve any difficulties. This is an important activity for strengthening communication with farmers and gaining more sensitivity on how to improve the operation of the system. Using this information to further improve the Operations Plan each season will lead to improved credibility with farmers.

A revised and improved Operations Plan should be developed for the forthcoming irrigation season based upon the results of: (1) the "walk-thru"; (2) the monitoring, evaluation and feedback program for the pervious season; and (3) additional field experience. Likewise, the Monitoring, Evaluation and Feedback program should be revised and improved.

Following this procedure for a number of irrigation seasons will lead to more equitable and timely water deliveries to Water Users Groups in just a few years. In addition, the amount of water being wasted will be reduced significantly while agricultural production will be increased.

If sufficient field data are collected, then the Operations Plan could be placed on a micro-computer model that would adequately simulate the real irrigation system. A simple steady-state, volume balance operations computer program could be utilized consisting of three models: (a) weekly (or any other appropriate time period) irrigation requirements; (b) weekly water delivery schedules; and (c) weekly records and seasonal analysis. Experience has demonstrated that computer modeling leads to more field data collection, greater sensitivity about the system, and more equitable distribution of irrigation water supplies.

The more field data collected, the better the internal workings of the irrigation project will be understood. The field data will lead to some preliminary conclusions as to necessary improvements that would reduce water losses, thereby allowing more water to be available for crop production. A computer model, if developed, can be effectively used to simulate potential irrigation water management improvements in meeting crop water requirements anywhere in the irrigation system. For example, channel losses might be reduced in some high water loss reaches by placing compacted clay lining, soil-cement lining, plastic membrane lining, brick-and-mortar lining, or concrete lining. In other cases, some modifying of the rotation schedules might reduce losses. Providing farmers with technical assistance to improve their water management practices will also be beneficial. In some cases, additional storage on the irrigated lands could lead to more beneficial use of the available water supplies. Then, a cost estimate should be prepared for each potential improvement. The two data sets (water savings and costs) can be combined to formulate options for improving the irrigation scheme that would achieve higher levels of water use. These options can be prioritized by ranking according to cost per unit of water, or analyzing cost-effectiveness, to develop a "package" of technologies that would cost the least to achieve whatever objectives were used in the analysis.

The prioritized options should be documented and presented in an Irrigation System Improvements Plan. The advantage of this document is that justification for the options can be readily understood by government and donor officials. Therefore, these officials can easily decide the level of investment that they consider appropriate at that point in time. Thus, this document makes it easier to seek support and more likely to obtain funding for implementing some, or all, of the recommended improvements. This, in turn, will allow continued improvement in the performance of the irrigation system that will facilitate, rather than hinder, increased agricultural production.

Section III.

JOB FUNCTIONS IN THE OPERATIONS PHASE

The Project Engineer is in-charge of a large-scale irrigation project, while the Provincial Irrigation Engineer (also referred to herein as the Project Engineer) is in-charge of all medium-scale and small-scale irrigation projects in a province. The operation and maintenance of a small-scale project is left to the farmers. For a medium-scale irrigation project, usually a Water Master is looking after the daily operation and maintenance activities, who in turn answers to the Provincial Irrigation Engineer, who then has the Head, Operation and Maintenance Section respond to operational activities of the Water Master. For a large-scale irrigation project, there will be a number of Water Masters, each one of them being the Head of an Operation and Maintenance Section, where each Section has the responsibility for O&M activities over a portion of the irrigation project. These Water Masters will be supervised by the Head, Water Management Section, who in turn answers to the Project Engineer. For the purpose of this handbook, the individual serving between the Project Engineer and the Water Masters, with responsibilities for looking after operations activities, is designated the "Head of Operations" for the irrigation project, whether large-scale or medium-scale, in order to simplify the discussion.

Each Water Master supervises a number of Zonemen. The Zoneman is an extremely important person in the daily operation of the system. He collects the majority of the field data, but even more importantly, he is the RID representative who interfaces with the farmers every working day (7 days a week during the irrigation season), particularly the Leader of each Water Users Group. Also, the Zoneman has Gate Tenders and Laborers working under his supervision. Consequently, in the discussion that follows, the Zoneman is assigned considerable responsibility, but he does have helpers; however, these helpers usually have less technical capability than the Zoneman.

A. CONDUCT IRRIGATION SYSTEM MAPPING

Fortunately, most of the medium-scale and large-scale irrigation projects in Thailand have good topographic maps that are adequate for operations activities. The maps for each irrigation project are done by personnel in the Division of Topographical Survey, which is a division under the Chief Engineer for Civil Engineering at RID headquarters in Bangkok. Any additional surveying required by implementing the Operations Phase of the O&M Learning Process can probably be accomplished by project personnel, such as the Water Masters and Zonemen.

B. DEVELOP DISCHARGE RATINGS FOR FLOW CONTROL STRUCTURES

The identification of the flow control structures to be calibrated for discharge measurement will be done jointly by the Project Engineer and the Head of Operations.

The Water Master and the Zonemen under his supervision will be responsible for doing the discharge ratings in their area of responsibility.

The training for this field work and the data analysis will be done by the Training Division located at RID Headquarters in Bangkok, but the training will be conducted at the irrigation project.

The Water Master will be responsible for insuring that good quality field data is being collected; also, he will be responsible for the data analysis.

The Head of Operations will periodically participate in the field work to be sure that good procedures are being followed and be responsible for reviewing all of the data analysis individually with each Water Master to assure quality control. Most of the problems in field procedures for developing discharge ratings for an irrigation structure can be detected in the data analysis.

The Project Engineer is primarily responsible for the scheduling and completion of the field work, but he also needs to work with the Head of Operations to assure quality control in the field data collection program.

C. COLLECT WATER MEASUREMENTS AND CHANNEL LOSSES DATA

The Head of Operations should develop a schedule for conducting inflow-outflow tests to evaluate channel losses for the various reaches in the irrigation channel network. This may have to be supplemented with ponding tests for some reaches.

The Project Engineer should review and then approve, or modify, the work schedule for conducting the inflow-outflow and ponding tests.

While conducting the inflow-outflow tests, the Head of Operations will coordinate the operation of the main canal, laterals and sub-laterals in conjunction with the field data collection.

The Water Master and Zonemen will be responsible for the field data collection.

The Water Master will be responsible for the data analysis.

The Head of Operations will review the data analysis individually with each Water Master and then be responsible for compiling the results.

The Project Engineer will review the final data compilation and determine if the field results are satisfactory or need to be repeated.

D. DEVELOP OPERATIONS PLAN

The Head of Operations should take the responsibility for developing the Operations Plan. The Water Master(s) can be requested to assist with the computations.

The Project Engineer should review the proposed Operations Plan and approve, or offer suggestions for improvement.

E. IMPLEMENT IMPROVEMENTS IN WATER DELIVERY SCHEDULES

The Project Engineer will direct the implementation of the Operations Plan, with particular attention given during the first few weeks to advise on any necessary adjustments determined from field experience.

The Head of Operations will coordinate the implementation of a new or revised Operations Plan, meet with each Water Master daily in the

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field during the first few weeks of the season, and will inform the Project Engineer daily as to the operations situation.

The Water Master will be in the field daily during the first few weeks of implementing a new or revised Operations Plan, spending some time with each Zoneman, assisting with the field data collection, and analyzing all of the field data, so that he can inform the Head of Operations where the operations performance is satisfactory and where some adjustments may be required.

The Zoneman will be responsible for evaluating whether or not the water demands in his area of responsibility can be satisfied, both by communicating with Water Users Group Leaders and by taking field measurements, with the results being communicated daily to the Water Master throughout the irrigation season.

F. IMPLEMENT MONITORING, EVALUATION AND FEEDBACK PROGRAM

The Zoneman provides daily discharge rates and water volumes to the Water Master for each outlet structure under his jurisdiction, as well as any flow control structures.

The Zoneman provides a weekly report to the Water Master on the cultivated area and crops for each outlet (or half of the outlets each week), plus any special measurements he has made either in the main system or any of the tertiary systems.

The Water Master is responsible for data analysis of all field data and to provide quality control on the field data being collected by the Zonemen under his supervision.

The Water Master is responsible for providing to the Head of Operations a weekly report on water budgets for his area of responsibity in the project.

The Head of Operations must compile the weekly Water Master reports into a "Weekly Operations Performance" report which is submitted to the Project Engineer and feedback provided to each Water Master as to the adequacy of their weekly report and the operational performance in their section.

The Head of Operations prepares the "Monthly Operations Performance and Water Supply Projection" report, which is submitted to the Project Engineer with an information copy to each Water Master so that the water supply projection can be communicated to the Water Users Group Leaders through the Zonemen.

The Head of Operations is also responsible for preparing the "Dry Season Operations Performance" report and the "Wet Season Operations Performance" report, which are largely compiled from the monthly reports, and then submitted to the Project Engineer with an information copy to each Water Master.

The Project Engineer provides feedback to the Head of Operations and other project staff based on the weekly, monthly and seasonal operations performance reports.

The Project Engineer prepares the "Annual Operations Report," which summarizes the seasonal operations reports plus an assessment of the present performance and proposed improvements, which is then submitted to the Regional Director with an information copy to the Head of Operations.

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G. CHECK DISCHARGE RATINGS FOR FLOW CONTROL STRUCTURES

The Water Master should schedule periodic checks of the discharge rating at each of the flow control structures, with at least one discharge measurement (usually with a current meter) on 25 percent, or more, of the structures each season, along with periodic checks of some outlet structures (usually with a Cutthroat flume).

Each Zoneman should include any discharge rating measurements in their weekly data report to their Water Master.

The Water Master should check the results of each discharge rating measurement with previous data for the same structure in order to evaluate if the discharge rating is changing or not, with the results communicated to the Zoneman and appropriate action taken if there is a problem, then cited in the weekly report by the Water Master to the Head of Operations.

The Head of Operations should provide feedback to the Water Masters on the number of discharge rating measurements being made, whether or not a sufficient number are being done, but even more importantly, evaluate the quality of the field data being collected.

Based on the weekly, monthly and seasonal reports, the Project Engineer should determine if this activity is being properly pursued and adequately conducted, or whether improvements are necessary, or the amount of this work load can be decreased.

H. CONDUCT SEASONAL "WALK-THRU" OPERATIONS SURVEY

During each irrigation season, the Water Master should "walk-thru" all of the irrigation channels that are his responsibility in order to determine if there are any problems with the system.

The Zoneman should accompany the Water Master in this "walk-thru" along the channels that are his responsibility and they should definitely talk with the Water Users Group Leader and other farmers at each outlet about any operational problems they are experiencing, as well as when the operation of the system has been satisfactory.

The Water Master should state in his weekly report to the Head of Operations any portion of the system where the "walk-thru" has been completed and the findings from this experience.

I. REVISE OPERATIONS PLAN

The Head of Operations is primarily responsible for revising the Operations Plan after each season using the results from the previous seasonal operations performance reports and the findings from the seasonal "walk-thru."

The Head of Operations is expected to consult the Water Masters and the Zonemen regarding proposed changes in the Operations Plan.

The Head of Operations will consult with the Project Engineer regarding proposed changes in the Operations Plan.

The Head of Operations will submit the "Revised Operations Plan" to the Project Engineer for his approval or modification, then communicate the revised plan to the operations staff.

J. REVISE MONITORING, EVALUATION AND FEEDBACK PROGRAM

After the irrigation season, the Water Master should conduct a meeting with the Zonemen under his supervision to obtain their ideas regarding how the Monitoring, Evaluation and Feedback Program could be improved, and to provide feedback to the Zonemen regarding the quality of performance in this program the previous season.

The Project Engineer and the Head of Operations should conduct a meeting with the Water Masters (after they have received feedback from the Zonemen) regarding improvements that might be made in the Monitoring, Evaluation and Feedback Program.

The Head of Operations should prepare a "Revised Monitoring, Evaluation and Feedback Program," which is then submitted to the Project Engineer for approval, or modification, then communicated to the operations staff.

K. DEVELOP IRRIGATION SYSTEM IMPROVEMENTS PLAN

The Project Engineer is expected to submit after two, three or four years of conducting the Operations Phase of the O&M Learning Process an "Irrigation System Improvements Plan," which is then forwarded through appropriate RID offices for approval and funding.

L. CONTINUE REFINING WATER DELIVERY SCHEDULES

Improving the operational performance of on irrigation project is a never-ending process, so the Project Engineer, Head of Operations, Water Masters, Zonemen, Water Users Group Leaders, and Farmers all have a responsibility to continually seek new and better ways to improve system performance and agricultural productivity.

Section IV.

IDENTIFYING FLOW CONTROL STRUCTURES

There is a growing awareness that agricultural productivity needs to be increased on existing irrigated land. Efficient water management practices are essential in order to accomplish this goal. It has been said that water can be managed only if it can be accurately measured. Thus, it is important to identify the kinds of flow control structures that can be calibrated for measuring discharge as a part of a general procedure for improving the hydraulic operation of the irrigation system.

Water can be utilized effectively for the production of food and fiber only when its quantity is known. Ideally, the discharge should be measured at every division in the irrigation system. In many projects, however, discharge measurements are taken only at the headworks of a canal. The headworks may be an outlet from a dam or other structure which diverts water from a river. While the technology for measuring irrigation water is comparatively simple and readily available, it has not been incorporated into the routine operations practices of many irrigation systems.

A variety of irrigation system structures can be calibrated to measure water. Generally, the most common constriction is a gate structure. Some systems have hundreds of gate structures for flow control. The outlet structures, commonly a Constant Head Orifice (CHO) structure, are also commonly used. Other irrigation structures which can be calibrated are culverts, inverted siphons, and wasteways. Any type of structure that constricts the flow of water can be field calibrated for discharge measurement. By developing field discharge calibrations for existing irrigation structures, irrigation water management will be improved.

When identifying flow control structures, the main system can be subdivided into:

1. Left Main Canal

    a. Lateral L-1 ?
    b. Lateral L-2 ?
    c. ....

2. Right Main Canal

    a. Lateral R-1 ?
    b. Lateral R-2 ?
    c. ....

For each main canal, identify the flow control structures. First, the Head Regular at the dam or river diversion will be the primary flow control structure. Then, there may be some check structures in the main canals that are used to regulate water levels. Finally, the Head Regulator for each lateral is a flow control structure. Also, there may be some outlets (e.g., CHO structures) along the main canal. In addition, there may be some other structures that affect the control of water in the main canals, such as wasteways, that may also be important.

The inventory of flow control structures for each canal should list all available known information, such as:

1. Location of the structure;
2. Dimensions of the structure;
3. Elevations; and,

4. Existing hydraulic data.

Next, a similar inventory of flow control structures should be prepared for each lateral. The primary structure is the Lateral Head Regulator, followed by the Head Regulator for each sub-lateral, sub-sub-lateral, etc. There will also be some check regulators and other control structures that need to be included. The most common structure will be the outlets, where each outlet serves a tertiary system. Most of these outlets are CHO's but some are single gate structures, and some are gated culvert outlets.

Before discussing the discharge calibration of these flow control structures, some thought should be given to prioritizing the work. If there are two main canals, then probably the largest canal serving the most irrigated land will be of primary importance. However, It may be wiser to undertake the calibration of the smaller canal initially, because: (a) it is important for the project staff to gain experience and develop sensitivity about what is occurring in the system; (b) the work will be completed sooner so that there is a feeling of accomplishment; and (c) the mistakes made on the first canal will provide valuable insights for minimizing errors in the larger canal and will result in better quality work.

After completing the necessary field work for the main canal(s), this same type of thinking should be considered for the laterals. Again, begin with the laterals served by the smaller canal. Then, undertake the field work first of all on the smallest laterals, thereby leaving the largest lateral, which is also probably the most important lateral, for last. Again, sensitivity will be gained under the smaller laterals, particularly with respect to serving individual tertiary systems.

Also, in organizing the work, since many of the structures are standard RID designs, then it may be advisable to begin calibrating identical structures to determine if the results are compatible. If not, then the reasons for such incompatibilities must be investigated, such as checking dimensions and elevations, and even more importantly, the accuracy of the discharge measurements during calibration. Another common difficulty is obtaining accurate measurements of gate openings.

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Section V.

CALIBRATING FLOW CONTROL STRUCTURES FOR DISCHARGE MEASUREMENT

A. DISCHARGE CALIBRATION

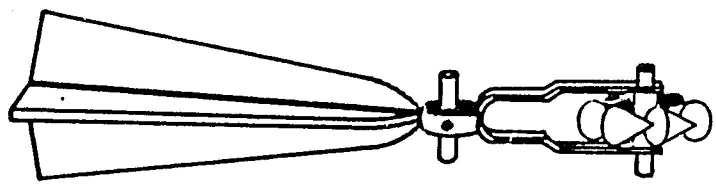
Calibration is the process by which stage-discharge relationships are established. With this process, the hydraulic functioning of constrictions in either the main channel or anywhere also in the system is determined. Standardized primary flow measurement devices can be installed if field conditions permit. While these devices are convenient, they are expensive to purchase and difficult to permanently install in an existing irrigation system. Field calibration becomes necessary: (a) if control structures rated in the laboratory are not identical to structures and conditions existing in the field; or (b) if the dimensions of the field control structures appear incorrect. Under such circumstances, the structures in question need to be recalibrated to correctly describe their hydraulic functioning. When doing field calibration on any of the control structures previously mentioned, water discharge rates corresponding to one or two flow depths are recorded. In order to make a meaningful data analysis, a minimum of four or five readings are necessary. Each reading will contain appropriate upstream, or upstream and downstream, depth measurements and corresponding flow rate information. The manual on "Field Calibration of Irrigation Structures for Discharge Measurement" should be consulted for detailed procedures. When the appropriate data have been collected, a graphical technique is used in analyzing calibration data for establishing stage-discharge relationships. After gaining some experience, a statistical regression technique can be used.

Classification of control structures is based both on their function and their operational nature. Gates, culverts, checks, etc. are designed to operate essentially under either free flow or submerged flow conditions. The terms free flow, critical depth flow and modular flow have identical meaning wherein a change in downstream flow depth does not affect the upstream flow depth because critical depth occurs in the vicinity of the constriction. Likewise, the terms submerged flow, drowned flow, and non-modular flow have identical meaning. A submerged flow condition exists when the downstream flow depth is raised to the extent that flow velocity at every point through a constriction becomes less than the critical value so that an increase in the downstream flow depth results in an increase in the upstream flow depth. Control structures designed to operate under free flow conditions frequently become submerged in response to unusual operating conditions, or with the accumulation of moss and vegetation. Care should be taken to note the operating condition of the control structure in order to determine which flow rating should be used.

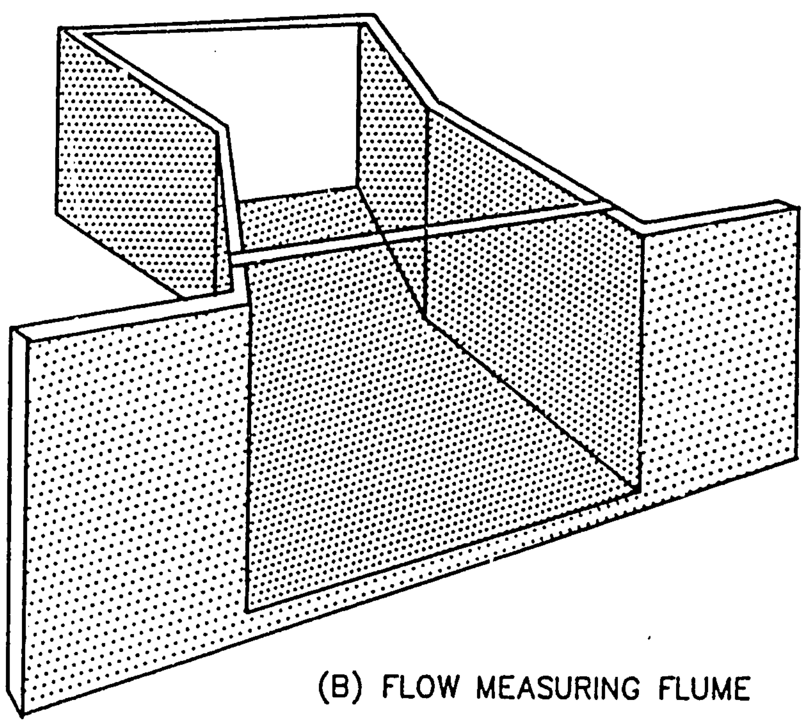
Calibration data is collected differently, depending on whether free flow or submerged flow conditions exist. In a free flow condition, a reading taken at each upstream stage (or depth) must have a corresponding flow rate measurement. In a submerged flow situation, two readings are required, one upstream and one downstream stage (or depth) for each discharge measurement.

B. DISCHARGE MEASUREMENT

There are a number of methods which are commonly used to collect discharge measurements in an irrigation network (Figure 0-05-1). A current meter is generally used for discharge rates greater than 500 liters per second and often for flow rates greater than 200 liters per second. When discharge rates are roughly less than 300 liters per second, flow measuring flumes -- such as the Parshall flume or Cutthroat flume -- are temporarily installed. For larger discharge rates, the dye dilution method can be used. With improved dyes and instruments that measure in parts per billion rather than in parts per million, this method is becoming increasingly useful. Another useful method for measuring discharge rates is to make volumetric measurements (Figure 0-05-2). For example, a small volumetric pan can be used to determine the discharge rate over a small portion of a weir overflow structure. By taking a series of such measurements over the crest width, the total discharge rate can be determined. When needing to measure very small flow rates where the quantity is less than one liter per second, a plastic bag can be used. After collecting the water, it can be repeatedly poured into a graduated volumetric container until the total volume of water has been measured. This is a very helpful method for measuring leakage from gate structures when they are closed.



(A) CURRENT METER



(B) FLOW MEASURING FLUME

Figure 0-05-1. Typical Methods of Collecting Discharge Measurements in Irrigation Channels.

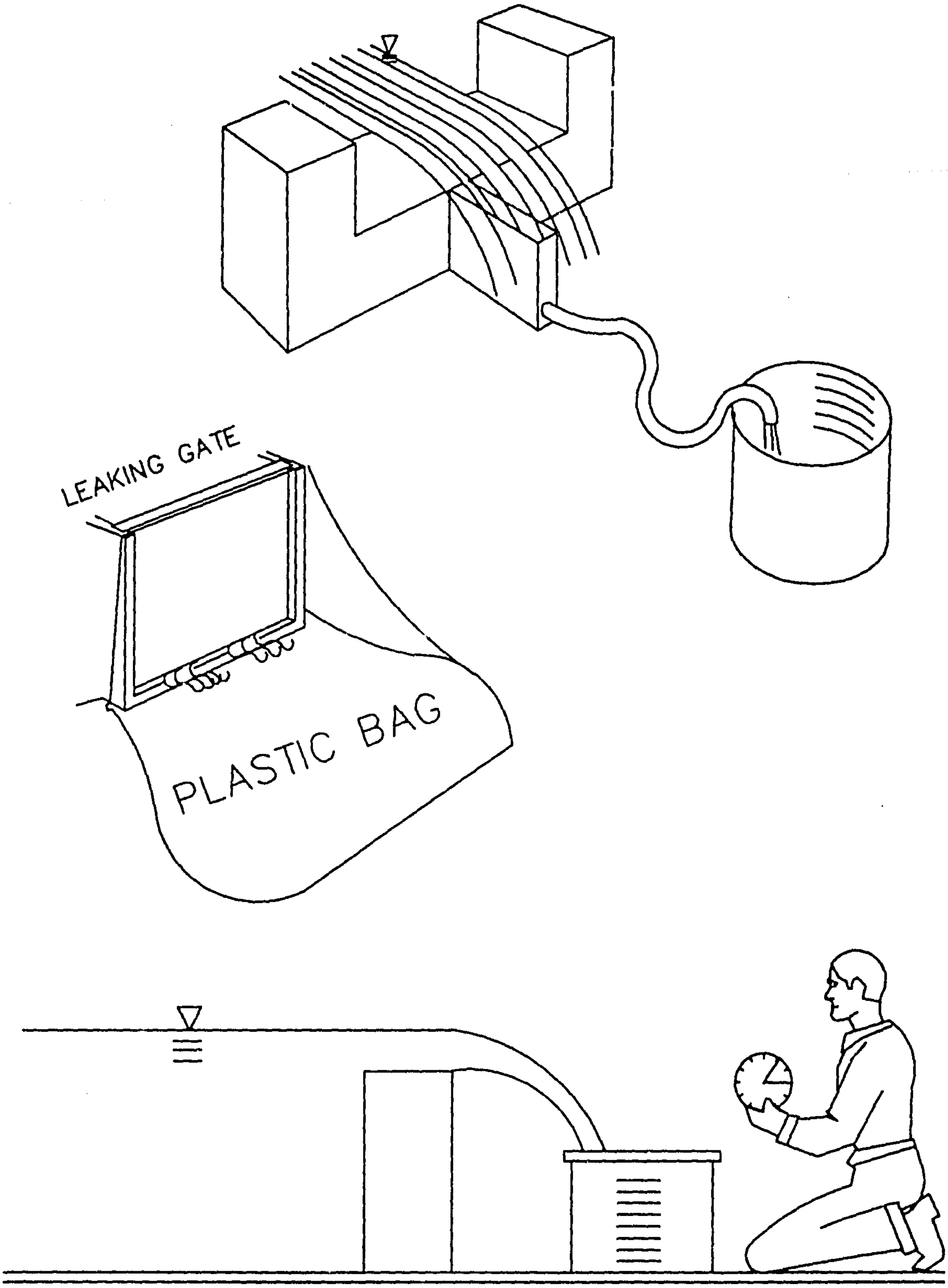
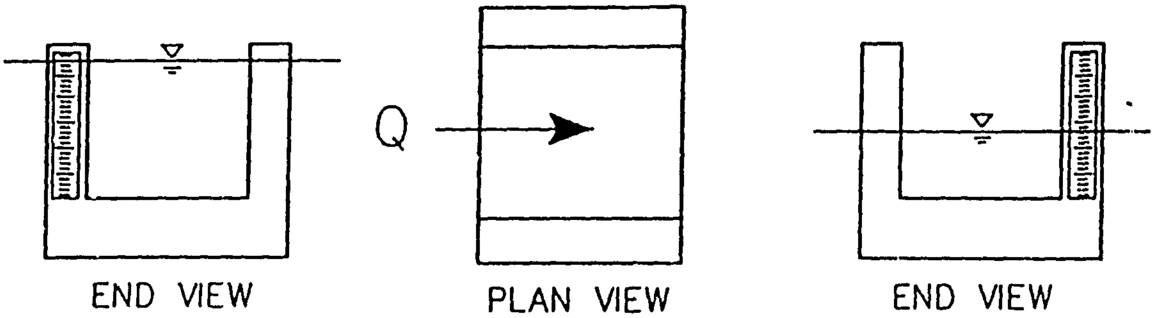


Figure 0-05-2. Some of the Possibilities for Collecting Volumetric Discharge Measurements.

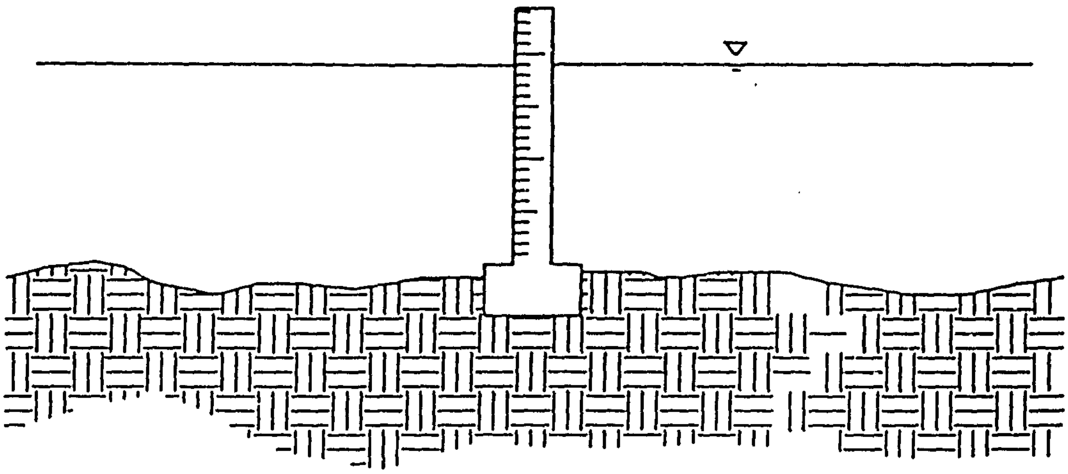
C. STAGE MEASUREMENT

Three techniques for depth or stage measurement are commonly employed. With the first technique, a staff gauge is placed against the wall of an irrigation structure or on a post located in the middle of an irrigation channel (Figure 0-05-3). (Under submerged flow conditions, more than one staff gauge is used -- one upstream and one downstream from the constriction.) The primary advantage of a staff gauge is that everybody can read it, including the farmers. The primary disadvantage of a staff gauge is that it has to be repainted each year because the markings below the surface of the water become obliterated.

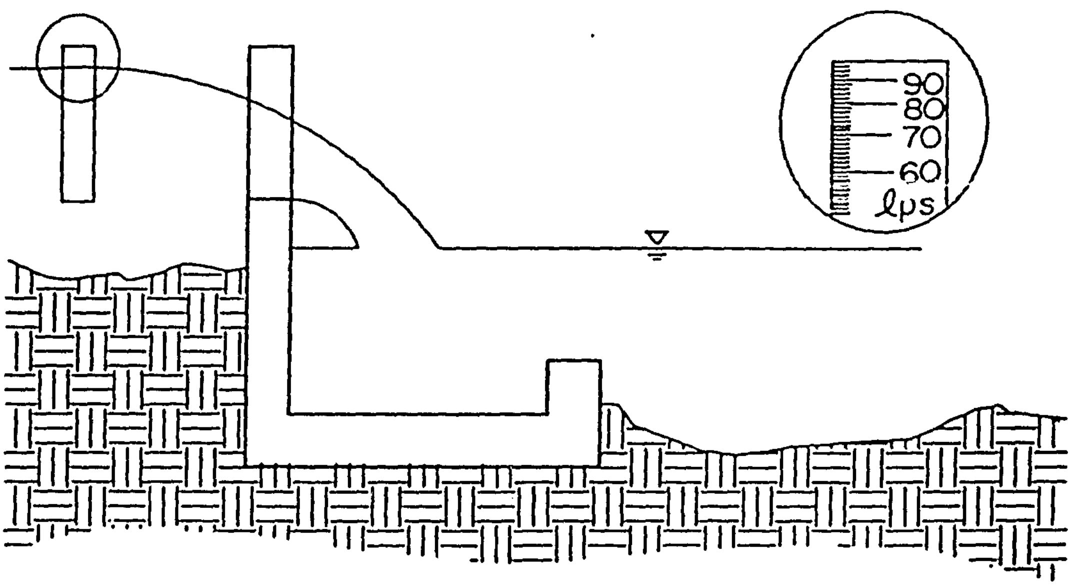
Under some conditions, it is either too expensive to install a staff gauge or the staff gauge is unavailable. When this is the case, the benchmark technique may be used. A mark is drawn on the wall of an irrigation structure and the reading is made by using a tape measure to find the distance from the mark to the water surface below. This mark, known as a benchmark, establishes a reference point from which future readings can be taken. It must be referenced to the appropriate zero flow depth level for the irrigation structure being calibrated. Once the benchmark has been established, it should be etched into, or painted onto, the irrigation structure for preservation. Field notes should include an accurate sketch of the location of each benchmark. If field notes have been carefully prepared, anybody reading them in the future (say 10 years later) should be able both to easily understand the procedure which was followed and to locate the benchmark.



(a) STAFF GAUGE ON UPSTREAM AND DOWNSTREAM WALLS OF AN OPEN CHANNEL CONSTRICTION.



(b) STAFF GAUGE IN AN IRRIGATION CHANNEL



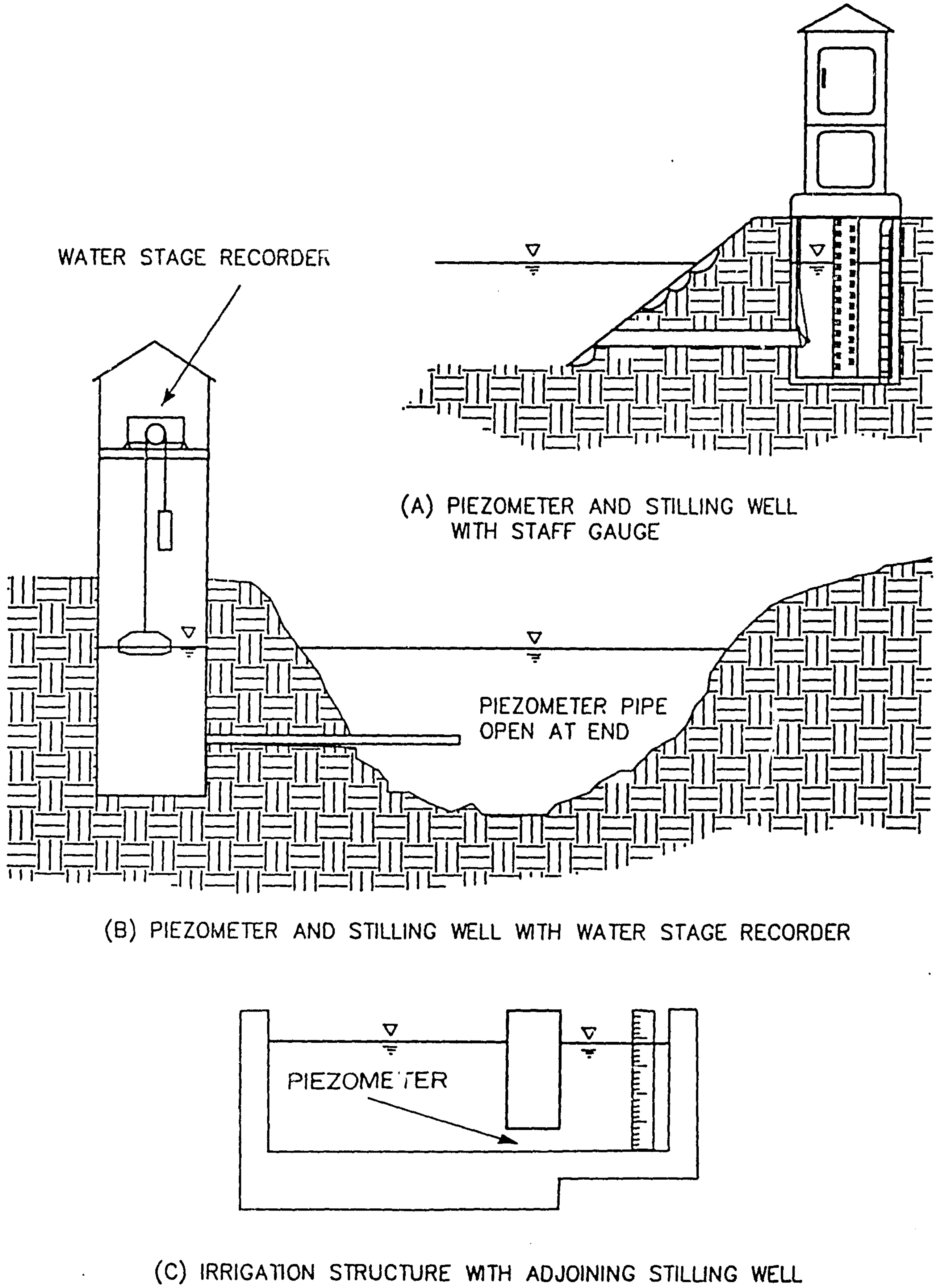
(c) STAFF GAUGE READING DISCHARGE IN LITERS PER SECOND (LPS)

Figure 0-05-3. Various Uses of Staff Gauges.

When the water surface is especially turbulent, the use of a piezometer is helpful (Figure 0-05-4). With this technique, a piezometer pipe is placed through the wall of an irrigation structure and connects a stilling well to the irrigation channel. Piezometer openings are commonly 5 to 10 millimeters in diameter. If piezometer pipe openings are used which are too small, there may be problems with clogging of the openings or slow response time within from the stilling wells.

D. BACKWATER EFFECTS

A simple open channel constriction is shown in Figure 0-05-5. The flow through such constrictions is most often in the tranquil range, and produces gradually varied flow far upstream and a short distance downstream, although rapidly varied flow occurs at the constriction. The effect of the constriction on the water surface profile, both upstream and downstream, is conveniently measured with respect to the normal water surface profile, which is the water surface in the absence of the constriction under uniform flow conditions, and is calculated using Manning's formula. Upstream from the constriction, an M1 backwater profile occurs. The maximum backwater effect, denoted by y^* in Figure 0-05-5, occurs a relatively short distance upstream. For flat gradient irrigation channels, the backwater effect may extend for a considerable distance in the upstream direction, sometimes a few hundred meters in small sub-laterals to more than 10 kilometers in large canals. Immediately downstream from the constriction, the flow expansion process begins and continues until the normal regime of flow has been reestablished in the channel.



(A) PIEZOMETER AND STILLING WELL WITH STAFF GAUGE

(B) PIEZOMETER AND STILLING WELL WITH WATER STAGE RECORDER

(C) IRRIGATION STRUCTURE WITH ADJOINING STILLING WELL

Figure 0-05-4. Typical Piezometer Installations.

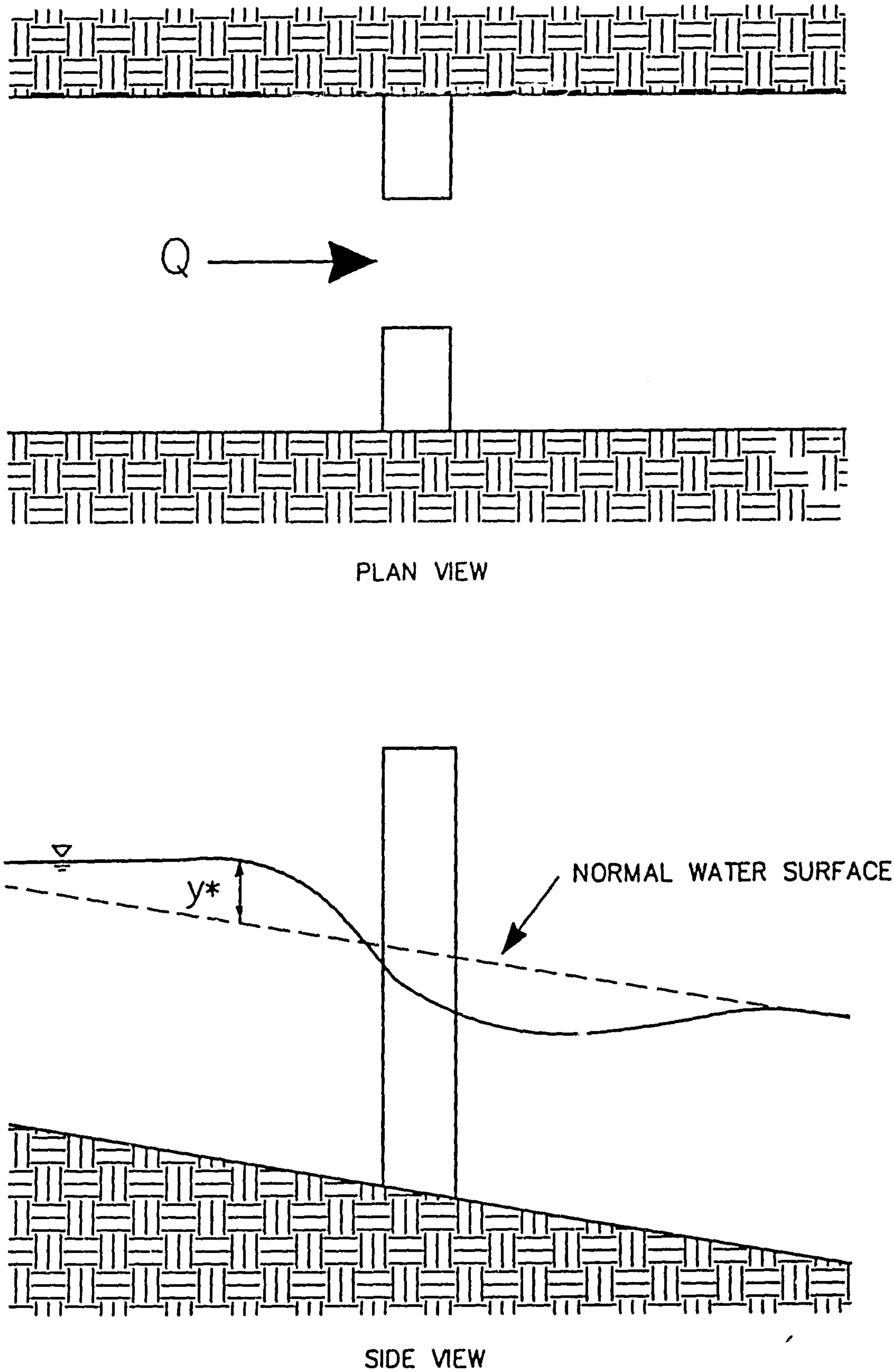


Figure 0-05-5. Definition Sketch for Backwater Effects From an Open Channel Constriction.

E. ORIFICE STRUCTURES

Any type of opening in which the upstream water level is higher than the top of the opening is referred to as an orifice. In this case, if the jet of water emanating from the orifice discharges freely into the air or downstream channel without backwater or tailwater effects, then the orifice is operating under free flow conditions. If the upstream water level is below the top of the opening, then the opening is hydraulically performing as a weir structure. For free flow conditions in an orifice, the discharge equation is:

$$Q_f = C_d \; C_v \; A \; (2g \; h_u)^{0.5} \tag{05-1}$$

where, C_d is a dimensionless coefficient of discharge, C_v is a dimensionless velocity head coefficient, A is the cross-sectional area of the orifice, g is the acceleration due to gravity, and h_u is measured from the centroid of the orifice to the upstream water level as shown in Figure 0-05-6a.

If the downstream water level is also above the top of the orifice (Figure 0-05-6b), then submerged conditions exist and the discharge equation becomes:

$$Q_s = C_d \; C_v \; A \; [2g \; (h_u - h_d)]^{0.5} \tag{05-2}$$

where, $h_u - h_d$ is the difference in water surface elevations upstream and downstream from the submerged orifice.

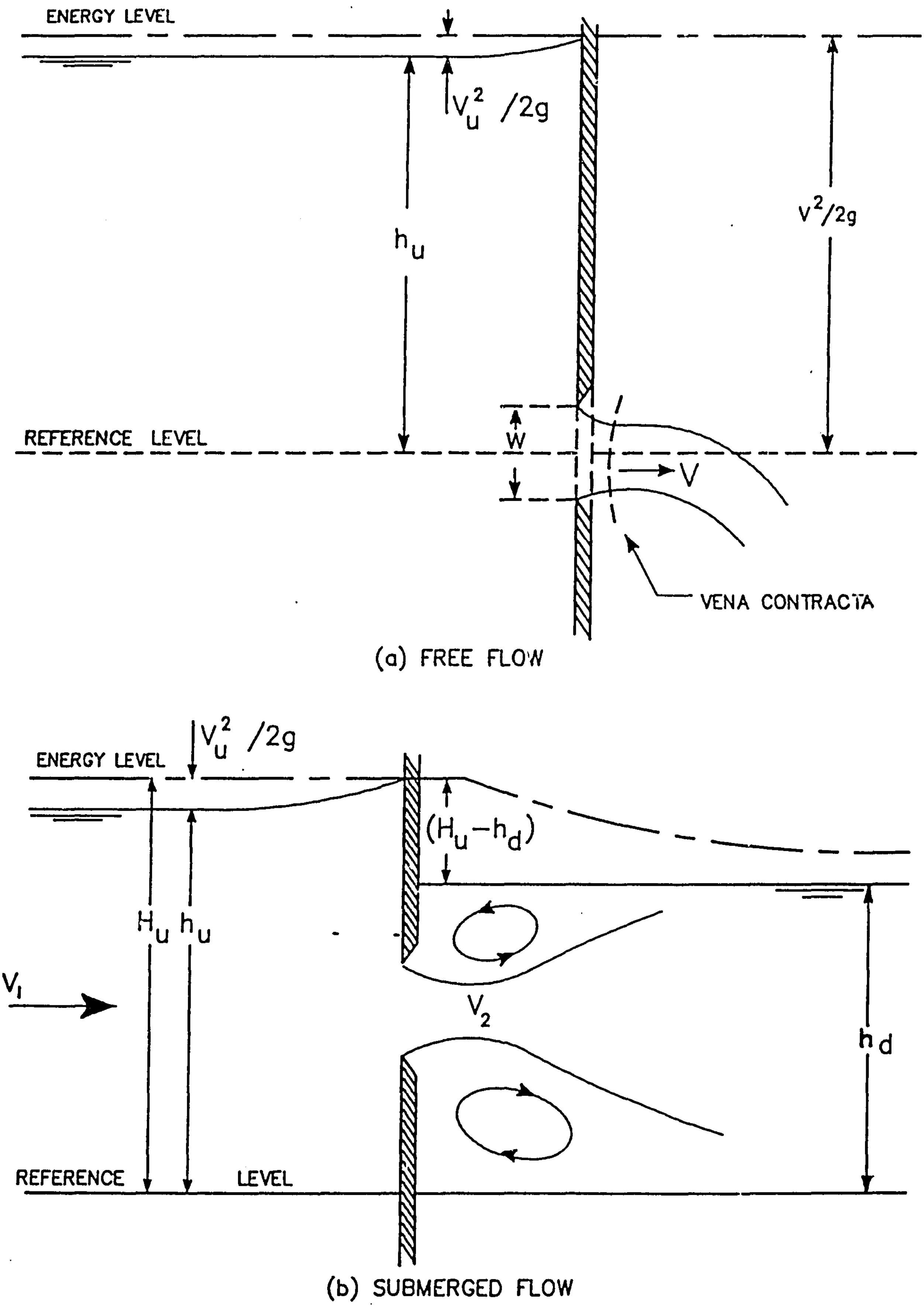


Figure 0-05-6. Definition Sketch of Orifice Flow.

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The velocity head coefficient, C_v, approaches unity as the approach velocity to the orifice decreases to zero. In irrigation systems, C_v can usually be assumed as unity since most irrigation channels have very flat gradients and the flow velocities are low (usually less than 1 m/s).

An orifice can be used as a highly accurate flow measuring device in an irrigation system. If the orifice structure has not been previously rated in the laboratory, then it can easily be rated in the field. The hydraulic head term, h_u or $h_u - h_d$, can be relied upon to have the exponent 1/2, which means that a single field rating measurement, if accurately made, will provide an accurate determination of the coefficient of discharge, C_d. Generally, orifices have C_d values of about 0.6 to 0.8 depending on the geometry of the orifice structure, but values ranging from 0.3 to 0.9 have been measured for various gate structures in Thailand.

A definition sketch for a rectangular gate structure having orifice flow is shown in Figure 0-05-7. For a rectangular gate having a gate opening, b, and a gate width, W, the free flow discharge equation can be obtained from Equation 05-2 and assuming that the dimensionless velocity head coefficient is unity:

$$Q_s = C_d\ (b*W)\ (2g)^{0.5}\ (h_u - h_d)^{0.5} \qquad (05\text{-}3)$$

where, b*W is the area, A, of the orifice.

The upstream flow depth, h_u, can be measured anywhere upstream from the gate, including the upstream face of the gate. The value of h_u will vary a small amount depending on the location chosen for

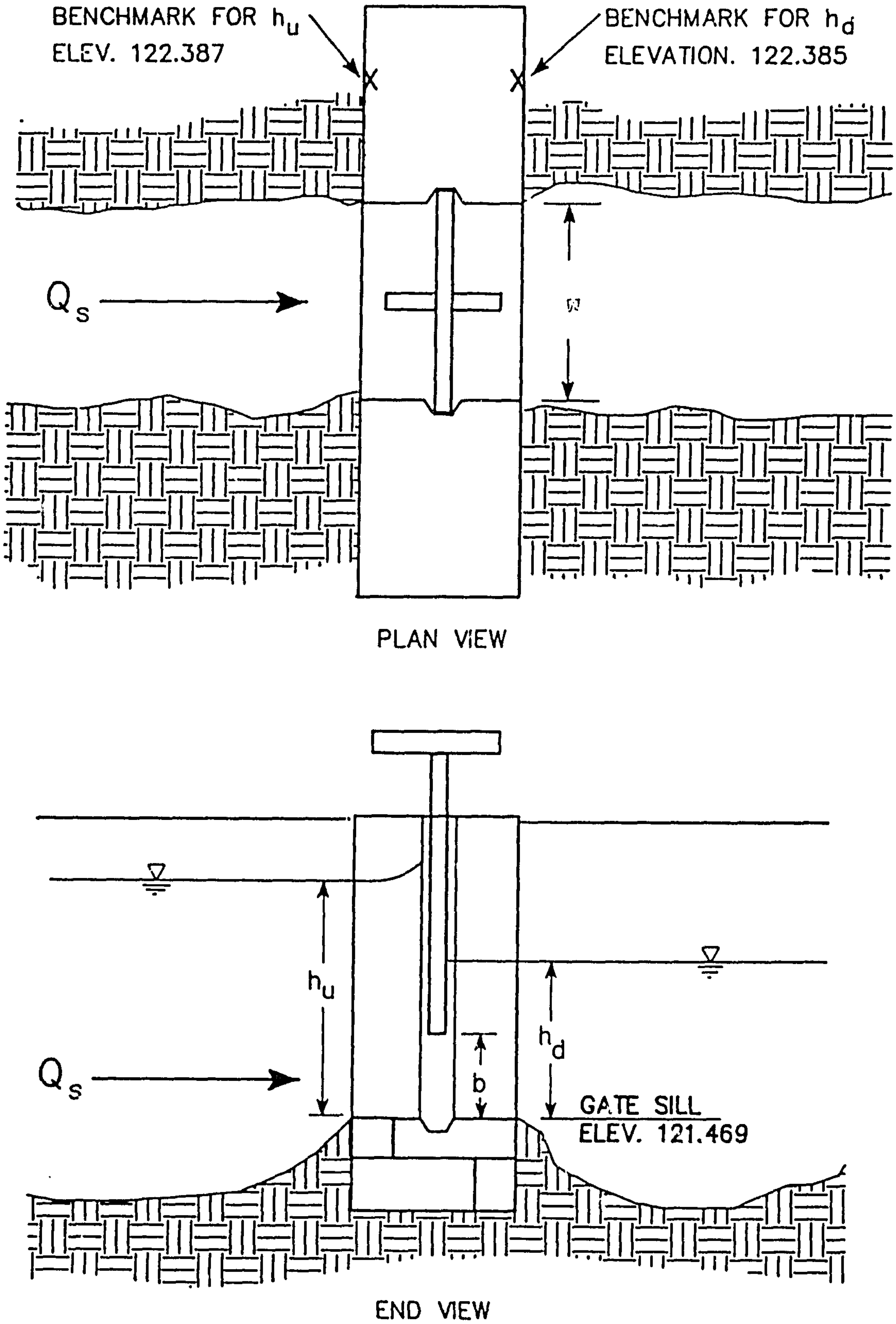

Figure 0-05-7. Definition Sketch and Example of a Rectangular Gate Structure Having Submerged Orifice Flow.

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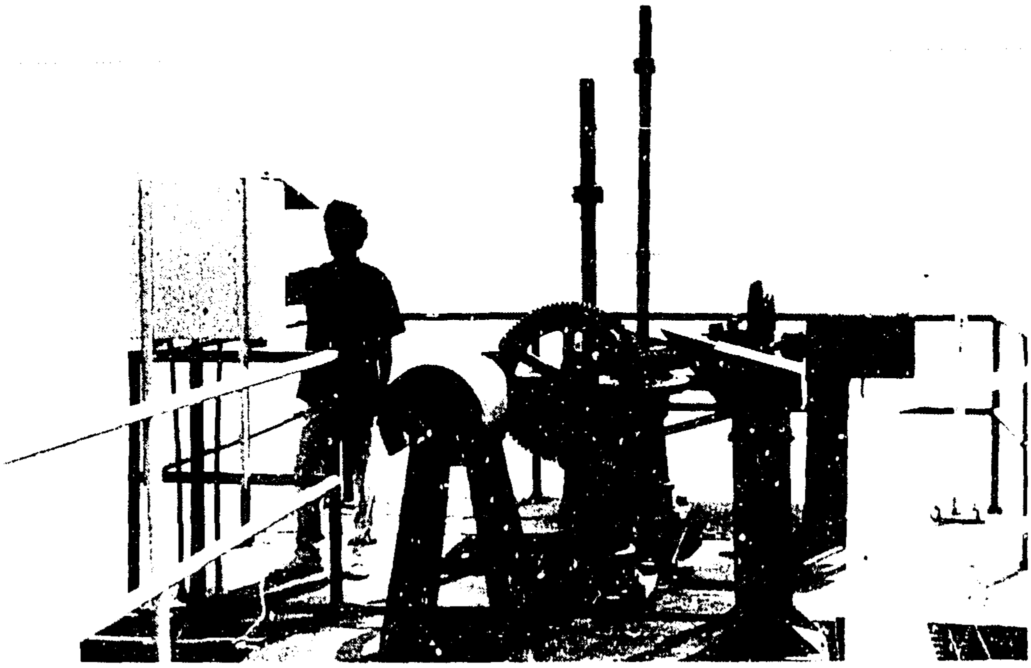
measuring h_u. Consequently, the value of the coefficient of discharge, C_d, will also vary according to the location selected for measuring h_u. This would also be true regarding the location for measuring h_d. The principal criterion for selecting the locations for measuring h_u and h_d is that the water surface is smooth, not turbulent and surging, or bouncing up and down. The second criterion would be to use the same locations for h_u and h_d for similar types of structures so that the C_d values can be compared.

The greatest difficulty in calibrating a gate structure is obtaining a highly accurate measurement of the gate opening, b. For gates having a threaded rod that rises as the gate opening is increased, the gate opening is read from the top of the handwheel to the top of the rod with the gate closed and when set to some opening, b. This very likely represents a measurement of gate opening from where the gate is totally seated, rather than a measurement from the gate sill; therefore, the measured value of b from the threadrod will usually be greater than the true gate opening, unless special precautions are taken to calibrate the threadrod.

Likewise, when the gate lip is set at the same elevation as the gate sill, there will undoubtedly be some flow or leakage through the gate. This implies that the datum for measuring the gate opening is below the gate sill. In fact, there is often leakage from a gate even when it is totally seated (closed) because of inadequate maintenance. Thus, C_d will vary with the gate opening, b. One methodology for analyzing this problem is presented in the manual on "Field Calibration of Irrigation Structures for Discharge Measurement."

Orifices are the most common type of flow control structure encountered in the irrigation systems of Thailand. First of all, the Head Regulator for each main canal is a gate structure having a variable orifice size depending on the height of the gate opening. The Head Regulators for laterals and sub-laterals are also gate orifice structures, as well as the Check Gate Regulators. Then, the CHO's used as Head Regulators and outlets from the main system to the tertiary system have two gates (but sometimes only one). In most cases, these structures operate as submerged orifices, so they are very ideal flow measurement structures.



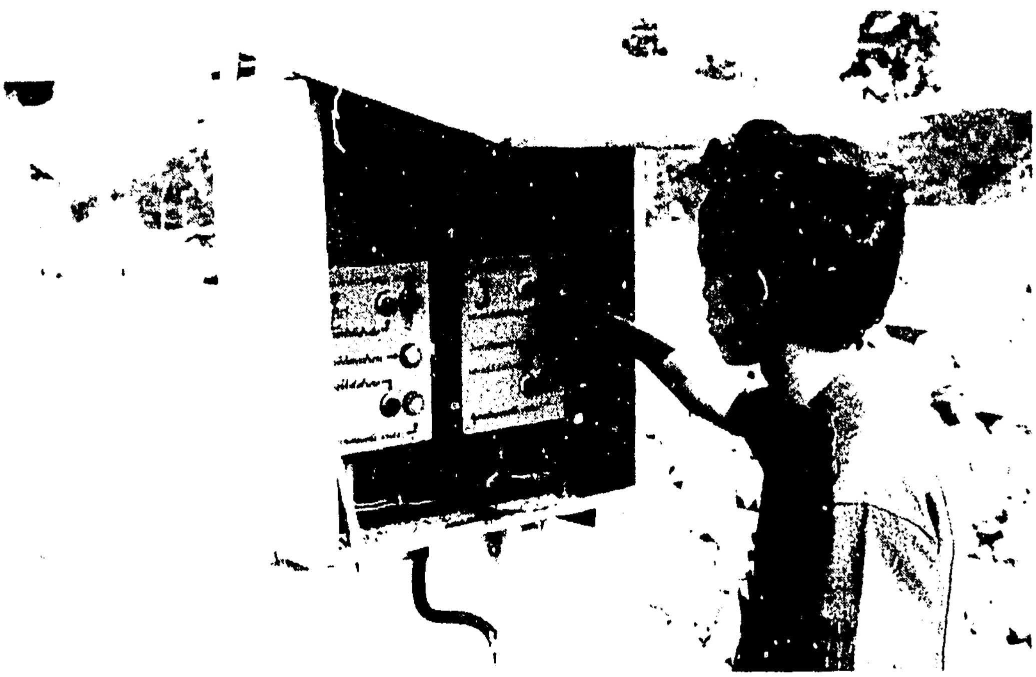
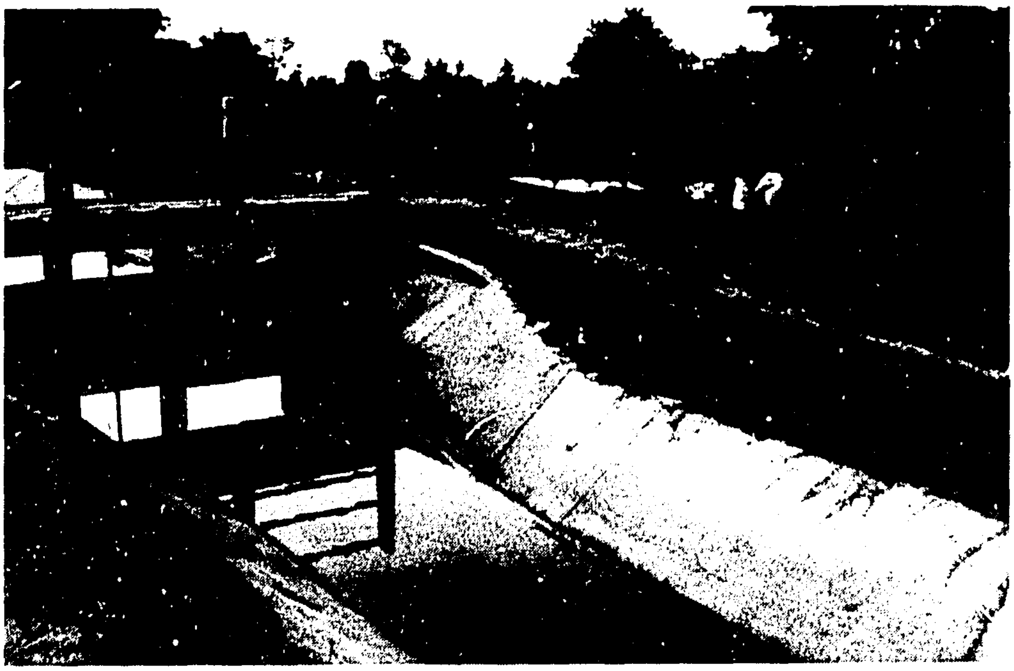


Figure 0-05-8. Control Gates for the Outlet Works From a Dam, which is Also the Head Regulator For the Main Canal.



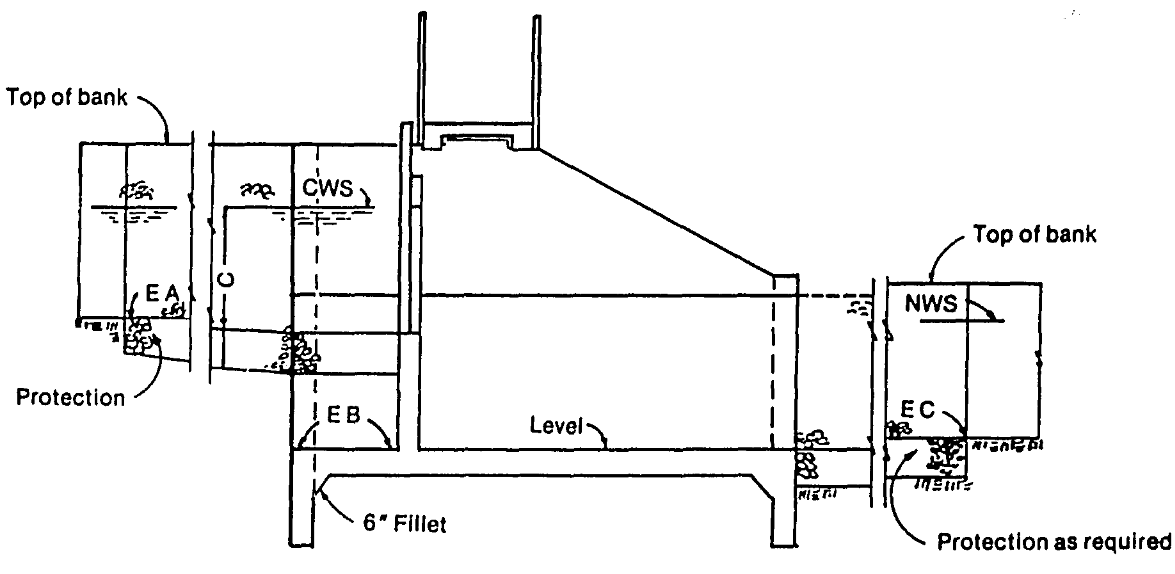


Figure 0-05-9. Examples of Check Gate Structures.

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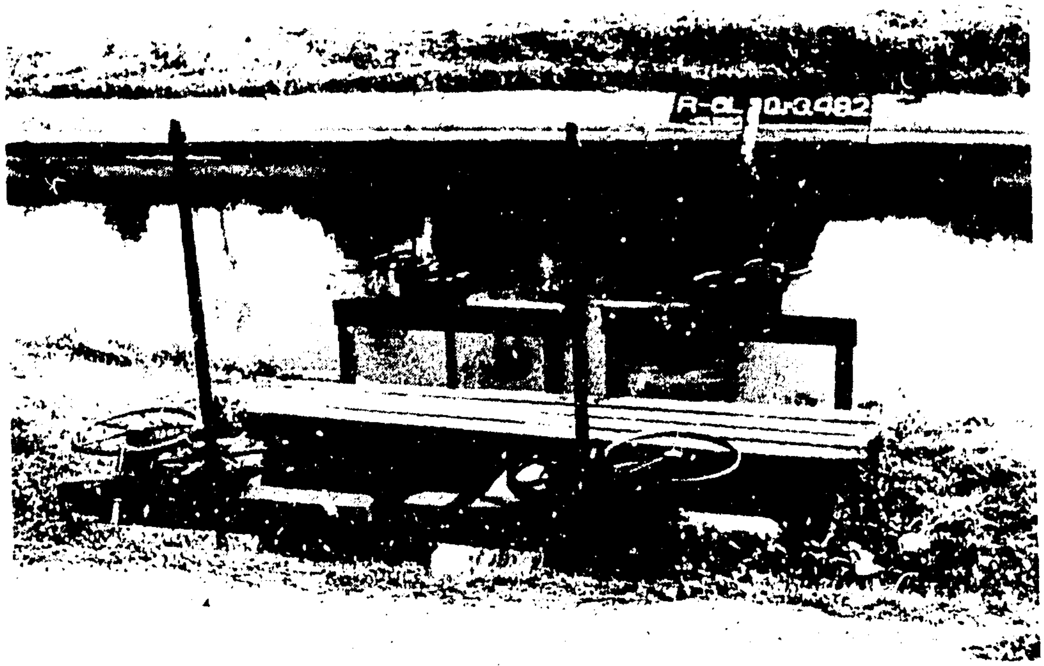

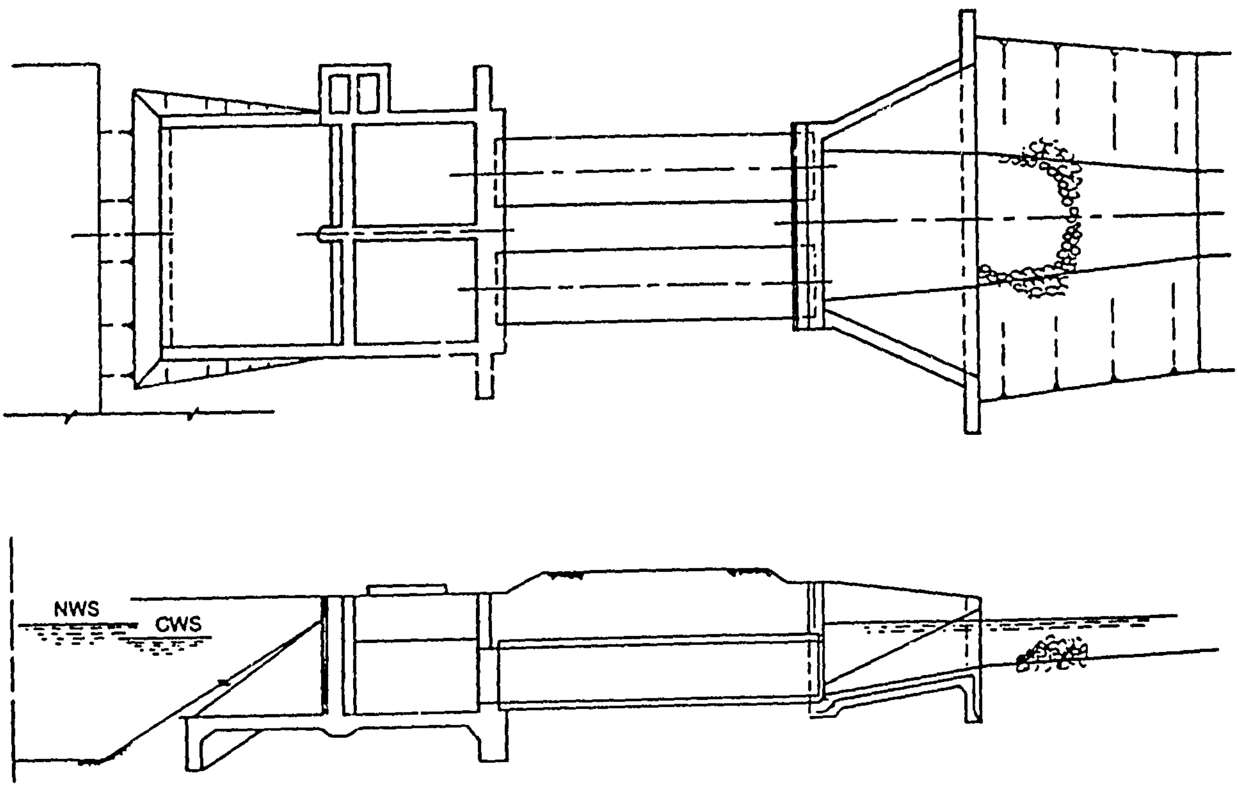

Figure 0-05-10. Examples of Head Regulators For Laterals and Sub-Laterals.

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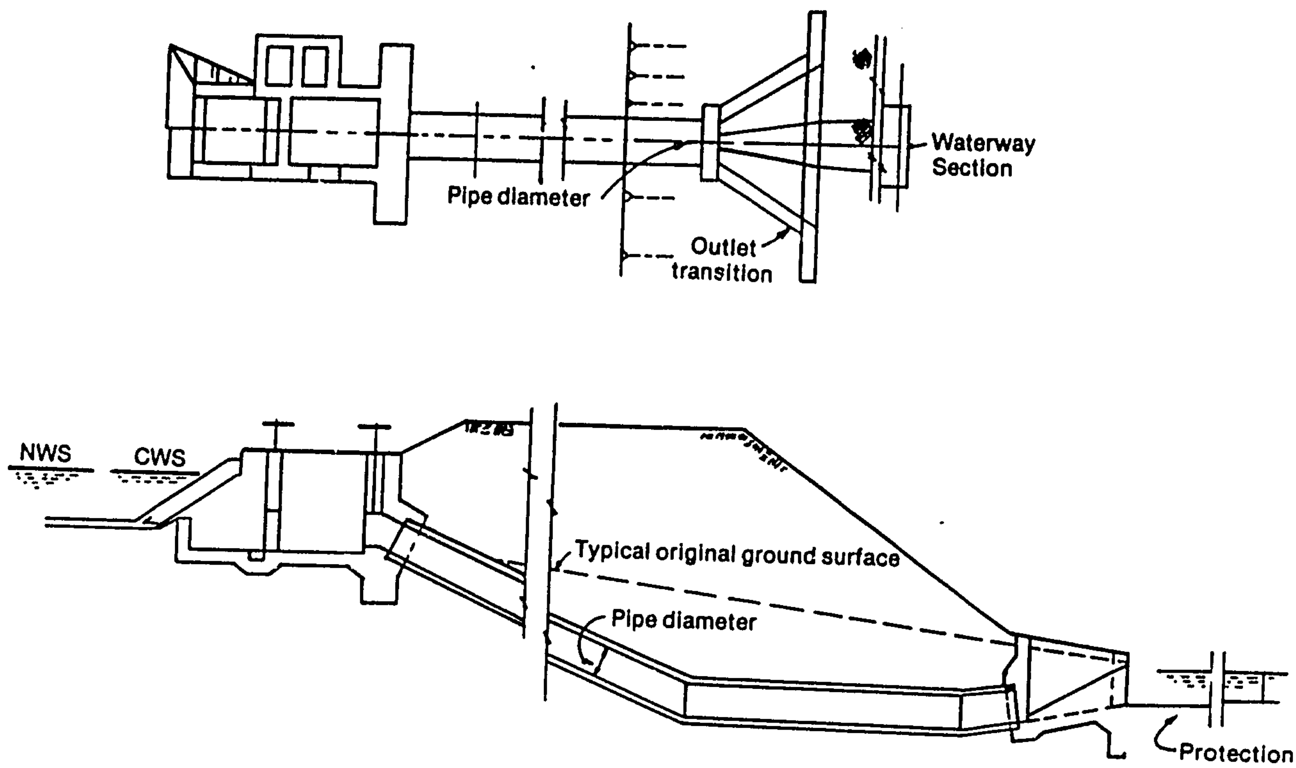
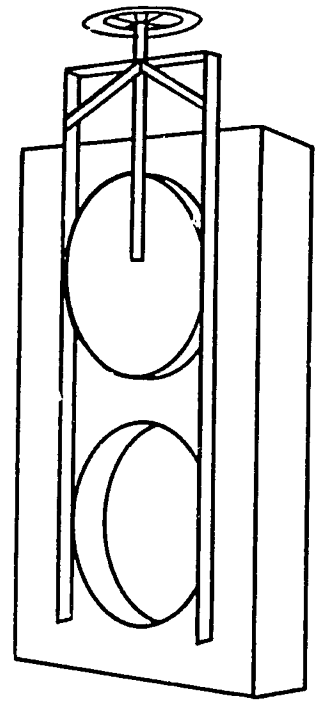


Figure 0-05-11. Constant Head Orifice (CHO) Structures as Head Regulators and Outlet From the Main System Serving a Tertiary System.



NOTE: ALL DIMENSIONS ARE IN METERS

Figure 0-05-12. Gated Pipe Outlet Structures.

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F. CULVERTS AND INVERTED SIPHONS

Culverts can serve as a combination open channel and closed conduit flow measurement structure, depending upon the type of flow condition in the culvert. Most of the research involving the hydraulics of culverts has been concerned with the use of such structures under highways. Most frequently, a highway culvert is designed to operate with full flow (closed conduit) at the design discharge. Much of this research has been concerned with inlet control (free orifice flow) and submerged outlet control (submerged orifice flow).

For culverts placed in an irrigation conveyance channel, often free surface (open channel) flow occurs in the culvert. In addition, downstream conditions will likely control the depth of flow in the culvert. For this particular condition of free surface subcritical culvert flow, the analysis for submerged open channel constrictions would apply.

Hydraulics of Culverts

The classification of the hydraulic performance of culverts can take several forms. Three primary groupings will be used to describe the hydraulics of culverts. The primary groups are based on the three parts of the culvert that exert primary control on the culvert performance and its capacity: the inlet, the barrel, and the outlet.

Inlet Control

Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater, h_u, and the entrance geometry, including the barrel shape and cross-sectional area and the type of inlet edge. With inlet control, the roughness and length of the culvert barrel, as well as outlet conditions (including depth of tailwater), are not factors in determining culvert capacity. An increase in barrel slope reduces headwater to a small degree.

Barrel Control

Under barrel control, the discharge in the culvert is controlled by the combined effect of entrance, length, slope, and roughness of the pipe barrel. The characteristics of the flow do not always identify the type of flow. The usual condition for this type of flow at design discharges is one in which the pipe cross-section flows full for a major portion of the length of the culvert. The discharge in this case is controlled by the combined effect of all hydraulic factors.

Outlet Control

Culverts flowing with outlet control can flow with the culvert barrel full, or part full, for part of the barrel length, or for all of it (Figure 0-05-13). If the entire cross-section of the barrel is filled with water for the total length of the barrel, the culvert is said to be in full flow or flowing full, as shown in Figure 0-05-13. The flow condition in Figure 0-05-13 is called submerged outlet control flow.

Method of Flow Analysis

For culverts in irrigation systems placed on a mild slope and having a short length, three flow conditions should describe the types of flow to be encountered. Beginning with free surface inlet control,

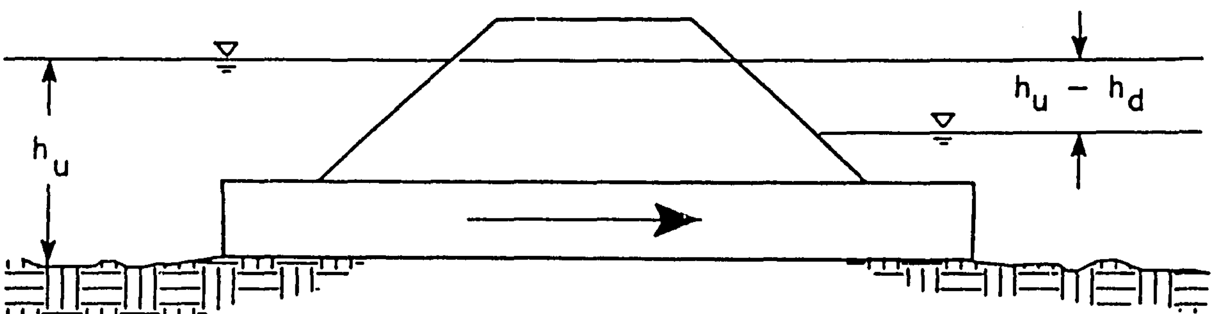

Figure 0-05-13. Outlet Control Flow Conditions For a Culvert.

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the downstream flow depth can be increased until the headwater is increased just slightly. Free surface flow will still exist, but flow conditions are now affected by changes in tailwater. This flow condition can be described as free surface outlet control. Finally, the tailwater can be raised sufficiently to submerge the outlet. For a short culvert installed on a mild slope, a submerged outlet should result in a submerged inlet, with the flow condition being submerged outlet control.

The method of flow analysis is different for each of the three flow conditions mentioned above. The technique for developing the discharge equation describing each of the flow conditions is presented in the manual on "Field Calibration of Irrigation Structures for Discharge Measurement." However, only the simplest case of submerged outlet control is presented below.

When the flow conditions are such that the downstream flow depth, h_d, is raised to the extent that the culvert is completely full throughout the culvert length, resulting in a change in the upstream depth, h_u, then the culvert is operating under submerged outlet control, as shown in Figure 0-05-13. The culvert operating under submerged outlet control flow conditions also requires that two flow depths be measured, one upstream (h_u) at the culvert invert, and one downstream near the end of the culvert (h_d). The reference elevation must be the same for h_u and h_d, and preferably, true elevations should be used. For this hydraulic condition, the absolute values of the flow depths are not important, but rather the difference in water surface elevation, $h_u - h_d$.

For the submerged outlet control flow condition, the submerged orifice equation is valid.

$$Q_s = C_{so}\ A\ (2g\ H)^{0.5} \tag{05-4}$$

where:

Q_s = submerged flow rate, in cubic meters per second;

H = difference between upstream and downstream flow depths, $h_u - h_d$;

C_{so} = submerged outlet control flow coefficient; and,

A = cross-sectional area of the culvert barrel.

The coefficient C_{so} contains the effects of inlet, barrel, and outlet geometry. However, the discharge rating will be affected by the accumulation of sediment or debris in the barrel of the culvert, or in the vicinity of the inlet or outlet.

A culvert with submerged outlet control represents the most ideal case for undertaking a field discharge rating. A single field discharge measurement, accurately done, is sufficient to calibrate the discharge equation so that the culvert can then be used as a flow measuring device by only measuring $h_u - h_d$.

This is also the case for inverted siphons wherein both the inlet and outlet are submerged. However, there is a greater concern about accumulating sediment, gravel and debris at the bottom of an inverted siphon. Such accumulations would reduce the discharge capacity of the inverted siphon, which would result in a lower value of the submerged outlet flow coefficient, C_{so}. Thus, periodic discharge measurements at an inverted siphon, using a current meter, would indicate if C_{so} has

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reduced; if so, then maintenance is required to remove the accumulated material within the inverted siphon.

G. OVERFLOW STRUCTURES

Examples of Overflow Structures

The most common overflow structures used for discharge measurement are weirs. Whereas flumes are open channel structures with the flow constricted from the sides (walls), weirs are open channel structures with the flow constricted from the floor so that the flow must pass over the top of the floor constriction.

In order to prevent the overflow of a canal section and its subsequent failure, canal overflow wasteway structures are often used. If the water level in the canal becomes too high because upstream turnout structures have been closed, then the excess flow will pass over the wasteway structure. A typical canal wasteway structure is shown in Figure 0-05-14. The length of the overflow structure is the weir crest length (or width), W_w.

Drop structures are another common type of overflow structure, such as curvilinear crest drop structures (Figures 0-05-15a and 0-05-15b), inclined drop structure (Figure 0-05-15c), or a vertical drop structure (Figure 0-05-15d). In these examples, the flow is passing through critical depth in the vicinity of the crest, so free flow is occurring.

Check structures are commonly used in irrigation systems. These structures are used to control the water level in the irrigation channel upstream from the check structures. This is often necessary in order to raise the upstream water levels so that there will be

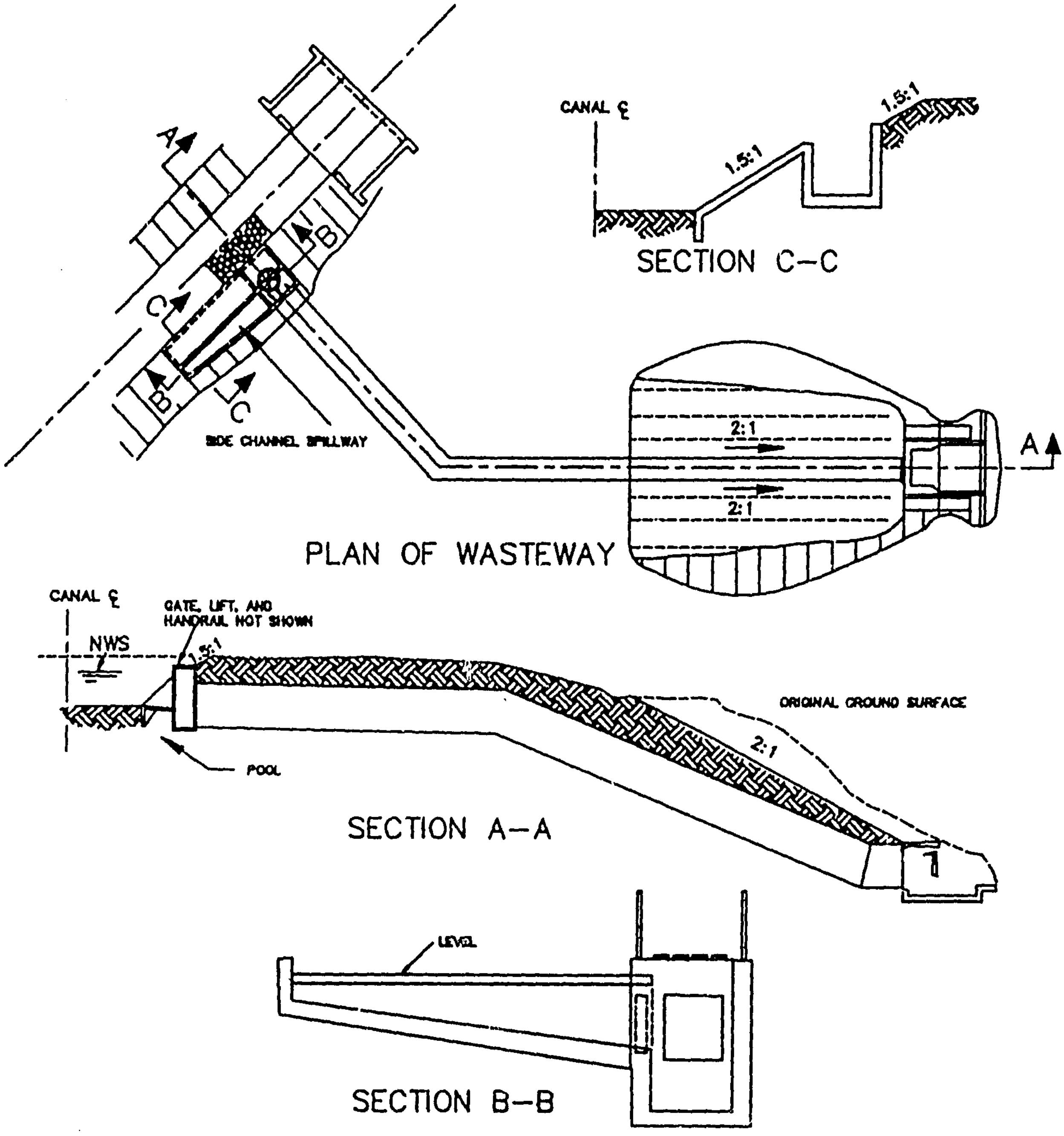

Figure 0-05-14. A Typical Canal Wasteway Structure.

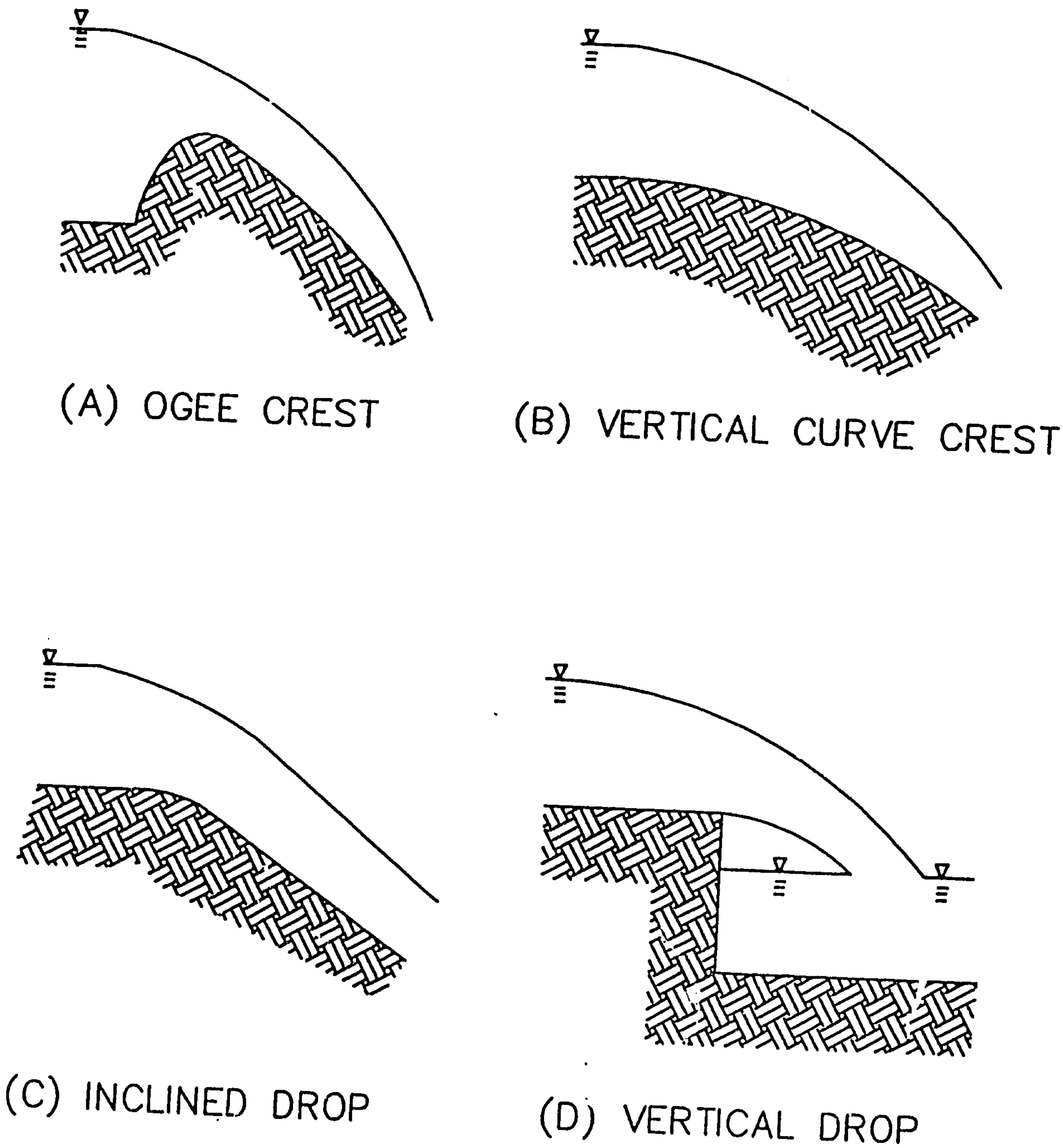

Figure 0-05-15. Examples of Drop Structures.

sufficient head (h_u) to allow an adequate discharge to flow through the upstream turnout structure(s). In some cases, a check structure has no flow passing downstream. More frequently, there will be flow passing over, or through, a check structure to satisfy downstream water delivery requirements.

In many irrigation projects, check structures are installed with a slide gate to control the upstream water level (Figure 0-05-16). There are a wide variety of flow conditions that can prevail at such a structure. Most commonly, the gate will perform as an orifice with either free orifice flow (Equation 05-1) or submerged orifice flow (Equation 05-2). In some cases, the structure is not used to control upstream water levels during certain periods of time, so the gate is raised in order not to constrict the flow. If the downstream bed elevation is about the same as the upstream bed elevation, then the check structure becomes an open channel constriction with either free flow or more likely, submerged flow. If immediately downstream from the gate there is a vertical curve crest (Figure 0-05-15b), inclined drop (Figure 0-05-15c), or vertical drop (Figure 0-05-15d), then the check structure performs hydraulically as an overflow (weir) structure.

Measuring Discharge

For large irrigation channels, usually a current meter is used to measure the discharge when rating an overflow structure. For smaller irrigation channels, a flow measuring flume would likely be used; however, in this case, it may be possible to use a standard calibrated weir, such as a rectangular thin-plate weir or a V-notch weir to measure the discharge at a location downstream from the overflow

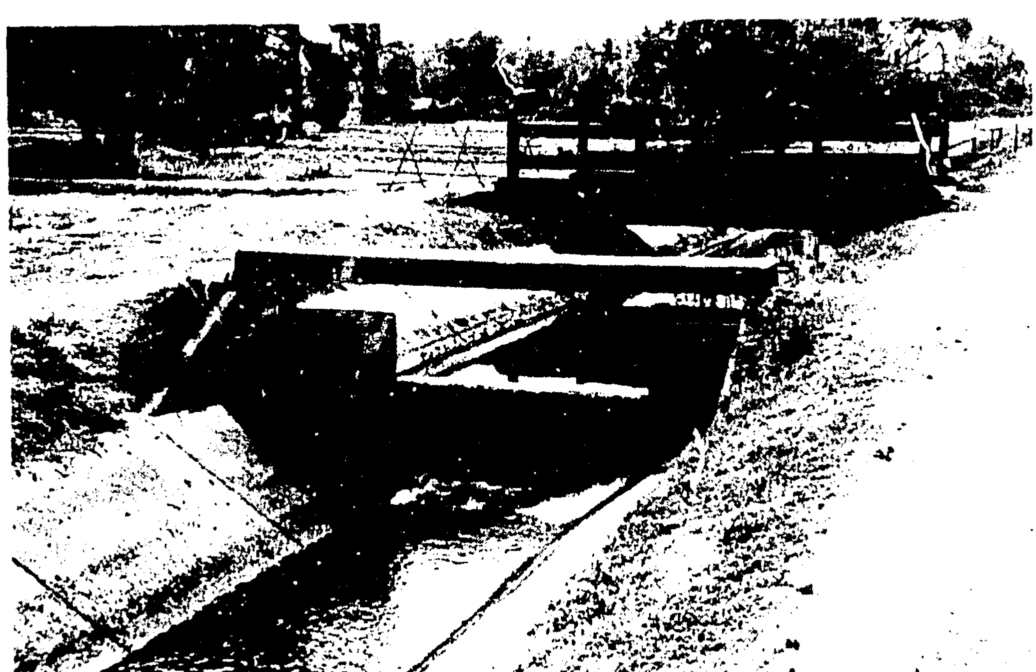

Figure 0-05-16. Typical Check Structures.

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structure. Also, for some overflow structures, such as those shown in Figures 0-05-14 and 0-05-15d, volumetric methods (Figure 0-05-2) can be used to measure the discharge over a small portion of the crest; a series of such measurements can be made in order to determine the variation in discharge across the entire crest width, W_w, which will provide an accurate measurement of the total discharge flowing over the crest.

Free Flow

The general form of the free flow equation for an overflow (weir) structure is:

$$Q_f = (C_d)_f \, W_w \, h_u^{\,n_f} \tag{05-5}$$

where, Q_f is the free flow discharge rate in cubic meters per second, $(C_d)_f$ is the free flow coefficient of discharge, W_w is the crest width of the overflow section, and n_f is the free flow exponent. The upstream flow depth, h_u, must be measured at some location upstream from the overflow crest (the exact location will affect the value of $(Cd)_f$ and must have the zero reference elevation correspond with the overflow crest elevation).

The variation in $(C_d)_f$ will increase only slightly for increasing crest widths if the geometry of the structures are similar. Excellent examples are vertical or inclined drop structures that are used in an irrigation project; usually, the geometry will be very similar and only the width of the structure will be changed according to the design discharge. Certainly, the expected values of $(C_d)_f$ for different values of W_w will be known after field calibration of similar structures at 2 or 3 irrigation projects; but, there will always be some variation from the expected value for each individual structure because of slight differences in construction or approach conditions.

The procedure for developing the free flow discharge rating for an overflow structure is to, preferably, collect 3-5 measurements of Q_f and h_u to verify whether or not nf= 3/2 or a number slightly higher. However, it is a relatively safe assumption that n_f will be equal, or very nearly equal, to 3/2, particularly for overflow structures commonly found in the main system of an irrigation project. If n_f is assumed equal to 3/2, then a single field measurement of Q_f and hu will provide a good estimate of the value of $(C_d)_f$.

Section VI.

MEASURING IRRIGATION CHANNEL LOSSES

A. UNITS FOR EXPRESSING SEEPAGE LOSS RATE

Seepage losses from canals and tertiary irrigation channels are a significant concern for many of the irrigation systems in Thailand. With the increasing emphasis upon improved irrigation water management practices, accounting for the movement of water through a system, including seepage losses, becomes increasingly important. In order to equitably distribute water in an irrigation delivery system, a knowledge of the variation in seepage losses throughout the system is required.

Seepage loss studies can answer such questions as: (a) How much does a given canal or canal reach seep? (b) Where are the major seepage areas? (c) Should a channel be lined? and (d) Is an existing irrigation channel lining effective?

Some of the more obvious factors that affect the rate of seepage loss from an irrigation channel are: (1) permeability of the soil traversed by the channel; (2) surface seal in the channel by silt and clay; (3) depth of water (which is affected by channel roughness, backwater from downstream structures, vegetative growth, aquatic growth, inadequate maintenance, etc.); (4) wetted surface area; (5) location of groundwater table relative to channel invert; (6) soil and water chemistry; and many more factors.

Three common methods for representing seepage loss rates will be presented. The first method calculates the seepage loss rate, Q_1, in liters per second per 100 meters of channel length as the difference in discharge rates in liters per second between the upstream Q_u location and the downstream Q_d location, divided by the length L in increments of 100 meters. Thus, if the length of channel between the Q_u and Q_d measurements is 450 meters, then L would be 4.50.

$$Q_l = \frac{Q_u - Q_d}{L} \times 1000 \qquad (0\text{-}06\text{-}1)$$

where, Q_l = seepage loss rate (lps/100 m);
Q_u = reach inflow rate (m3/s);
Q_d = reach outflow rate (m3/s);
L = reach length (100's of m).

The second method calculates the seepage loss rate Q_{lp} in percent per 100 meters of length, which is Q_l divided by Q_u to represent the seepage loss as a ratio of Q_u, then multiplied by 100 to change from a ratio to percent.

$$Q_{lp} = \frac{Q_u - Q_d}{Q_u \times L} \times 100 \qquad (0\text{-}06\text{-}2)$$

where, Q_{lp} = seepage loss rate (%/100 m);
Q_u = reach inflow rate (m3/s);
Q_d = reach outflow rate (m3/s);
L = reach length (100's of m).

The third method is the most universally acceptable representation of the seepage loss rate Q_{slr} in cubic meters of seepage loss per square meter of wetted surface area per day, denoted by the abbreviation cmd. Likewise, Q_{slr} can be calculated as cubic feet of seepage loss per square foot of wetted surface area per day, which can be abbreviated as cfd. Since there are 3.08 feet in a meter, one cmd equals 3.08 cfd. Often, the seepage loss rate is expressed in millimeters per day (1 cmd = 1000 mm/day).

$$Q_{slr} = \frac{Q_u - Q_d}{WP_{avg} \times L} \times 1000 \qquad (0\text{-}06\text{-}3)$$

where,

Q_{slr} = seepage loss rate (lps/100 m);
WP_{avg} = average wetted perimeter (m);
Q_u = reach inflow rate (m3/s);
Q_d = reach outflow rate (m3/s);
L = reach length (100's of m).

All three methods of representing the seepage loss rate can be used with the Inflow-Outflow Method for measuring irrigation channel seepage, whereas only the third method using Q_{slr} can be calculated from the "Ponding Method."

B. INFLOW-OUTFLOW METHOD

Description of Methodology

The most accurate technique for measuring seepage losses in an irrigation channel is the Inflow-Outflow Method using existing irrigation structures for discharge measurement. The manual on "Field Calibration of Irrigation Structures for Discharge Measurement" provides the necessary information for developing ratings for various types of structures. The seepage loss rate can be evaluated for each reach between two structures. A single structure provides Q_d for one reach and Q_u for the next reach. By having developed discharge ratings for a series of structures along a canal, the seepage losses for the entire canal can be evaluated. In addition, it is very easy to take a series of discharge readings at various times throughout each irrigation season in order to determine the variation in seepage loss rates with time for each reach.

Another technique for using the Inflow-Outflow Method is to install temporary flow measuring devices, such as weirs or Cutthroat flumes. This is usually only satisfactory in small tertiary channels. Also, if a canal outlet or other structure has been field calibrated, then it can be used to obtain Q_u and a temporary flow measuring device could be installed downstream to provide Q_d. The primary disadvantage of installing an additional flow measuring device is that the water surface level upstream is raised, which increases the seepage losses.

Current meter measurements can be made for using the Inflow-Outflow Method. The only difficulty is that the seepage losses have to be much greater than the error in the current meter discharge measurements, about 5 to 10 times greater, or more. Thus, if the seepage loss rate is low, then very long reaches must be used. If this is not feasible, then the "Ponding Method" should be used.

Example of Main Canal Inflow-Outflow Test

Introduction

Training was provided in the field data collection necessary for "Operation of Irrigation Systems" at the Lam Nam Oon Irrigation Project from June 23 to July 3, 1986 under the cooperation of the Training Division and the Operation and Maintenance (O&M) Division of the Royal Irrigation Department (RID). The 37 participants were Water Masters from various irrigation projects and people with an irrigation engineering background from the O&M Division of RID. The trainers were from both the Training and O&M divisions. The inflow-outflow method was adapted to measuring seepage in the canal by using a series of

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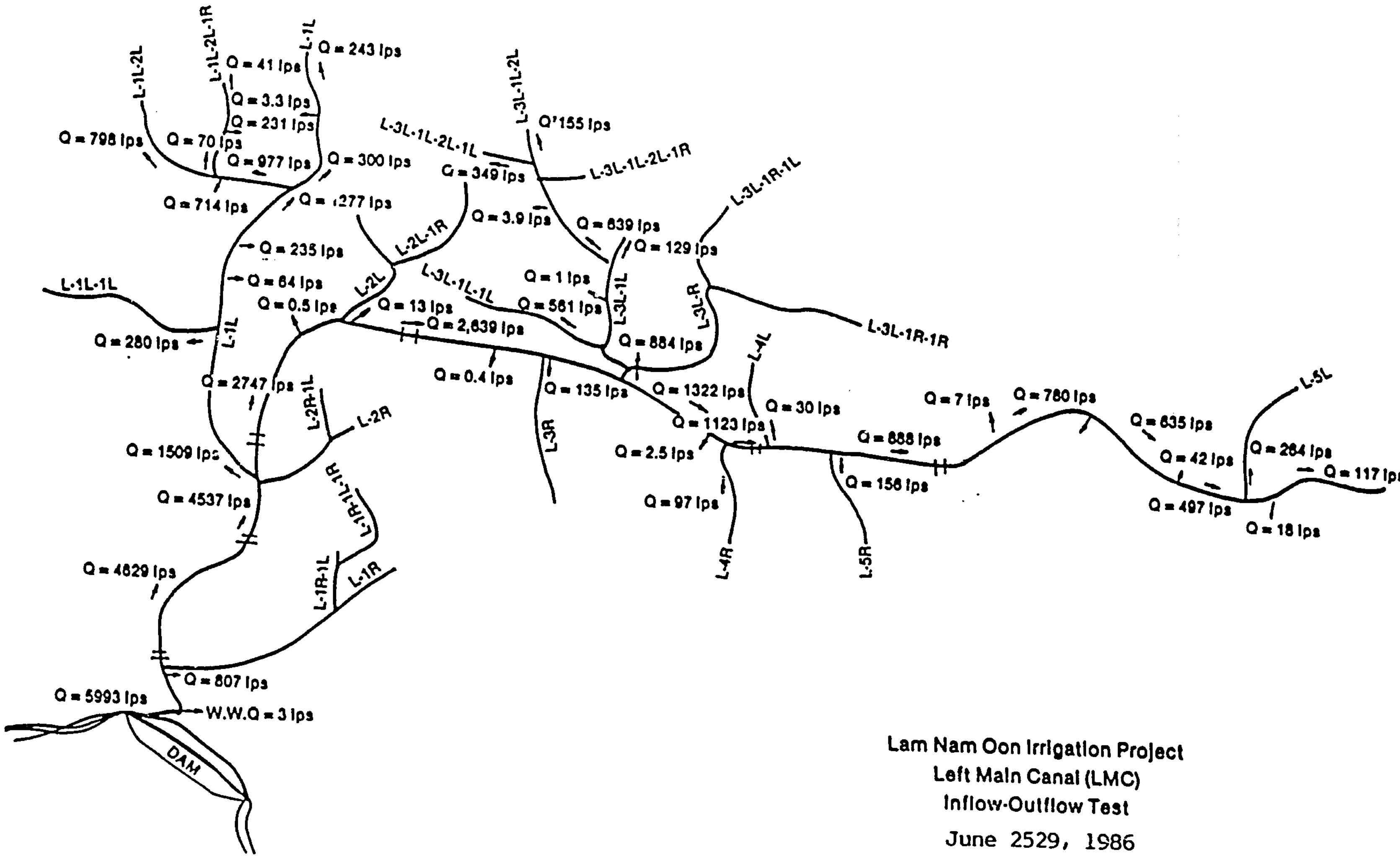

Figure 0-06-1. Inflow-Outflow Test Result for the Left Main Canal at the Lam Nam Oon Irrigation Project During June 2529 (1986).

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control structures in the in the Left Main Canal (LMC) as the boundary of each reach.

This method can be used rather easily and does not interfere with the operation of the canal, but the results depend upon the accuracy of the water measurements. Current meters are generally used to measure the flow rate in large canals. The stage of the canal should be kept constant during the test period in order to eliminate the effect of unsteady flow and bank channel storage. Failure to take this factor into account may introduce large errors into the results. The seepage loss rate should be measured in all reaches of the canal system. The seepage loss rate may have a negative value if the canal reach is in a cut section in which saturated soil conditions cause water to seep into the canal. The leakage that cannot be eliminated can be best measured volumetrically with a container of known volume. The time required to fill the container is measured; therefore, the leakage rate can be determined. If the test is made during periods of precipitation, the rainfall must be measured and considered when computing seepage losses.

Classroom Lectures

The participants were taught the hydraulic theory of calibrating irrigation flow control structures. These included check structures, head regulators, culverts, inverted siphons, overflow structures, and outlet structures. The participants were instructed on how to use a current meter, a Cutthroat flume, and how to perform seepage loss measurements before going to collect the field data.

Field Work

The trainers demonstrated the use of a current meter and Cutthroat flume in the field. After that, individual groups were assigned to practice using the equipment on the real canal system. Then, the participants became familiar with the use of the equipment and how to apply the field data to calibrate the control structures. Each group of about four participants was assigned to calibrate one or more control structures by using a current meter in the LMC and large lateral sections, and using a Cutthroat flume in the smaller channels. It is important that the trainers check the data very carefully after each day of field work to make sure that the information is valid and that no obvious mistakes have been made. When groups are assigned to do the field work in adjacent canal sections, the data from each group can be cross-referenced and any errors can be more easily identified. Also, each group should make all discharge and seepage loss computations at the field site as the data is collected. The true water level elevations at the upstream and downstream ends of each reach were recorded for reference as a datum relating all of the data throughout the system. The gate opening of the control structure should also be precisely measured in the field. This may mean that the person making the measurement will have to dive into the water and use a scale to accurately measure the height of the gate opening.

Conducting Inflow-Outflow Test

The Left Main Canal, and its laterals and sub-laterals, were divided among the nine groups by using control structures as a boundary between each group. Before the group started measuring the water by using the current meter, they had to make sure that steady-state flow conditions existed by observing the water level at the staff gauge. When the stage was no longer changing, the group could start the field work.

The individual groups made a current meter measurement at every important structure such as sluice gates, check structures, drop structures and every outlet. This was necessary to determine how much water is going into and out of the reach. The outflow from the outlet (usually a CHO), or the leakage through the closed outlet, was measured. A very important task is the accuracy of current metering because the seepage loss in the reach sometimes is very small compared with the discharge; therefore, the group must make the discharge computations immediately in the field to make sure that there is nothing wrong with the data. If they find something wrong, they should recompute or perform the measurement again. This strategy helps to assure that meaningful and consistent data will be collected for the entire system.

The seepage loss rate in millimeters per day requires that the average wetted perimeter in the reach be measured. The group should measure the side slope, top width, and bottom width (on the top of the sill) at a couple of stations along that reach. After the computation of wetted perimeter at each station, the average wetted perimeter can be calculated.

The leakage through the closed gates are also considered to be a loss from the canal. The leakage should also be measured volumetrically with a calibrated can.

In the closed constant head orifice gate, the leakage can also be measured by closing the control gate and making sure that both gates are in a completely closed position. After that, take some water out of the well. Start the test by recording the depth of water and begin the time measurement, preferably with a stopwatch. Meanwhile, let the water leak through the closed gate for a while, and record the time and depth of water again. The volume of water is the difference in water depths from the start to the finish, multiplied by the cross-section dimensions of the well.

Results

The final procedure is to make a summary report. After every group calculated the seepage loss, the data was compared. The reasonable results were compiled. If some question or argument was raised, then the group would be sent to re-do the field work at the particular structure in question. So in a couple of days of inflow-outflow measurement, the discharge of every turnout should not be changed (thus, steady state conditions are maintained). When all the results were satisfactory, the individual groups would write their summary report in a clear and easily understood manner. These reports were put together and finalized for the entire inflow-outflow test. The results are summarized in Table 0-06-1 and Table 0-06-2, which show that the average seepage loss rate for the LMC was 851 mm/day. The total seepage loss for the LMC was 1,820 lps, compared to an inflow of 5,993 lps, which gives a total loss of 30 percent.

Now everybody in the project knows how much loss exists in the LMC conveyance system. This allows the operation schedule to take into

Figure 0-06-1. Results of the Inflow-Outflow Measurements for the Left Main Canal, Lam Nam Oon Irrigation Project, June, 2529 (1986)

Group	Canal	Lateral	Sublateral	Reach Begin	Reach End	Length L in m	(W.P)L m^2	Seepage loss ps/100m	Seepage loss %/100m	SLR. ls mm/day	Qu m^3/s	Qd m^3/s	Δ Q m^3/s
1	LMC	-	-	0+210	2+700	2490	17305.3	26.060	0.435	3149.26	5.9932		0.6490
	LMC(Siphon)	-	-									4.6287	
		L-1R	-									0.8067	
	LMC(Waste way)		-									0.0030	
2	LMC	-	-	2+700	6+000	3100	28782.0	9.450	0.204	979.55	4.6287		0.2930
	LMC(Check)	-	-									4.5367	
		L-1L	-	0+020	5+900	5880	29779.0	0	0	0	1.5887		-
		-	L-1L-1L									0.2799	
		L-1L(Pump)	-									1.2270	
		2CHO,WASTE WAY										0.2990	
3	LMC	-	-	6+212	10+798	4586	44567.0	0	0	0	2.7467		0
		L-2L	-									0.1131	
	LMC(Siphon)	-	-									2.6330	
	LMC(CHO+W.W.)	-	-									0.0005	
	LMC	-	-	11+000	16+000	5000	49273.0	5.944	0.225	521.14	2.6390		0.2972
		L-3L	-									0.8840	
	LMC(pump)	KM.16	-									1.3220	
		L-3R	-									0.1354	
	2 CHO	-	-									0.0004	
4	LMC	-	-	[illegible]	16+790	640	2519.0	15.516	1.174	3405.92	1.3220		0.0993
	LMC(Siphon)	-	-									1.1230	
		L-4R										0.0972	
	1 CHO	-	-									0.0025	
	LMC(Siphon)	-	-	16+782	18+800	2064	8502.8	2.370	0.211	497.91	1.1230		0.0490
		L-5R	-									0.1560	
	LMC(Siphon)		-									0.3680	
		L-4L	-									0.0303	
		4 CHO	-									0.0025	

Table 0-06-1 (Continued).

Group	Canal	Lateral	Sublateral	Reach Begin	Reach End	Lenght L in m	W.P m^2	Seepage loss lps/100m	Seepage loss %/100m	SLR mm/day	[illegible] m^3/s	[illegible] m^3/s	[illegible] m^3/s
5	LMC	-	-	18+883	20+240	1357	4979.9	7.440	0.819	1752.32	0.9079		0.1010
	LMC(Drop)	-	-									0.7797	
	3 CHO	-	-									0.0070	
	LMC	-	-	[illegible]	[illegible]	[illegible]	17319.8	3.736	0.479	[illegible]97.92	0.7797		0.1379
	LMC(Culvert)	-	-									0.6353	
	3 CHO	-	-									0.0065	
6	LMC	-	-	24+[illegible]	26+[illegible]45	2245	7845.2	4.285	0.675	1059.46	0.6353		0.0962
	7 CHO	-	-									0.0420	
	LMC	-	-									0.4971	
	LMC	-	-	26-28[illegible]	26+[illegible]	[illegible]	3196.7	13.678	2.752	2643.33	0.4971		0.0978
		L-5L	-									0.2641	
	2 CHO	-	-									0.0130	
	LMC(Drop)	-	-									0.1172	
7		L-3L-1L	-	0+020	1+507	1487	65[illegible]3.6	4.035	0.456	[illegible]	[illegible]		[illegible]
		-	L-3L-1L-1L									0.0560	
		-	L-3L-1L-2L									[illegible]	
		-	WEIR									[illegible]	
8		-	L-3L-1L-2L	0+000	4+148	[illegible]	[illegible]	3.156	0.494	[illegible]	[illegible]		[illegible]
		-	L-3L-1L-2L									0.1550	
		-	L-3L-1L-2L-1L									0.3490	
		-	8 CHO									0.0039	
9		L-1L	-	5+958.75	8+660	2701.25	9[illegible]15.0	3.370	0.264	844.09	1.27[illegible]		0.0910
		L-1L-2L	-									0.9770	
		L-1L	-									0.3431	
		[illegible] CHO	-									0.0033	
		L-1L-2L	-	0+000	1+041	1041	[illegible]41.1	9.827	1.010	1990.21	0.9770		0.1023
		L-1L-2L	-									0.7975	
			L-1L-2L-1R									0.0[illegible]98	
			5 CHO									0.0074	
		L-1L-2L-1R	-	0+000	0+550	550	388.2	1.000	1.433	535.02	0.6980		0.0055
		L-1L-2L-1R	-									0.0411	
			3 CHO									0.0232	

Table 0-06-02 : Summary of Seepage Loss Rates for the Left Main Canal, Lam Nam Oon Irrigation Project, June 2529 (1986), and February 2531 (1988).-

Group	Canal	Lateral	Sublateral	Reach Begin	Reach End	Length L in m	(W.P)L m^2	Seepage loss lps/100m	Seepage loss %/100m	SLR. in mm/day	Seepage loss,lps
1	LMC	-	-	0+210	2+700	2490	17805.3	26.060	0.435	3149.26	649.0
2	LMC		-	2+900	6+000	3100	28782.0	9.450	0.204	879.55	293.0
3	LMC	-	-	6+212	10+798	4586	44567.0	0	0	0	0
	LMC	-	-	11+000	16+000	5000	49273.0	5.944	0.225	521.14	297.2
4	LMC	-	-	16+060	16+700	640	2519.0	15.516	1.174	3405.92	99.3
		-	-	16+782	18+800	2064	8502.8	2.370	0.211	497.91	49.0
5	LMC	-	-	18+883	20+240	1357	4979.9	7.440	0.819	1752.32	101.0
	LMC	-	-	20+290	23+981	3691	17319.8	3.736	0.479	687.92	137.9
6	LMC	-	-	24+000	26+245	2245	7845.2	4.285	0.675	1059.46	96.2
	LMC	-	-	26+280	26+995	715	3196.7	13.678	2.752	2643.33	97.8
							184790.7	Average SLR = 351		mm/day	1820.0
2		L-1L	-	0+020	5+900	5880	29779.0	0	0	0	-
9		L-1L	-	5+958.75	8+660	2701.25	9315.0	3.370	0.264	844.06	91.0
			L-1L-2L	0+000	1+041	1041	4441.1	9.827	1.010	1990.21	102.3
			L-1L-2L-1R	0+000	0+550	550	888.2	1.000	1.433	535.02	5.5
7		-	L-3L-1L	0+020	1+507	1487	6503.6	4.035	0.456	797.00	60.0
8		-	L-3L-1L-2L	0+000	4+148	4148	13866.0	3.156	0.494	599.48	130.9

account measured seepage losses, rather than assuming the seepage loss by pulling some number out of the air.

C. PONDING METHOD

Description of Methodology

Several factors will determine the sites selected for conducting ponding tests, such as visual evidence that adjacent cropland is suffering high groundwater levels, evaluating the seepage loss rates for various soil types, or evaluating various irrigation channel lining materials. Ponding sites are preferred that have a minimum of canal outlets, unless they can either be accurately measured or plugged without any leakage.

The length of a pond is dictated mostly by the slope of the irrigation channel, with the ponds being longer for flatter gradient channels. The primary disadvantage of the ponding method is having a level water surface, so that at the lower dike the pond water level should be above the normal water surface or full supply level and will be a like amount below the normal water surface at the upper dike. Another guideline is that the wetted pond end areas should not exceed 3 percent of the total pond wetted area.

For small channels, the dikes or dams may be built of canvas or plastic held in place by a timber at the top and dirt thrown along the edge. More commonly, earth dikes are constructed at each end of the pond; for large canals, the earthen dikes are constructed in layers of 15 to 20 centimeters and each layer compacted. A cutoff trench 30 centimeters deep is recommended. Sometimes, an existing check structure or check-drop structure can be used for one of the dikes by sealing with plastic and earth.

Leakage or seepage through the dikes can be eliminated by covering the pond side of each dike with sheet plastic. The edges can be held in-place by excavating a shallow trench 30 centimeters deep with soil shoveled carefully over the plastic in order not to puncture the plastic sheet.

For canals having a water depth less than 1.5 meters, the water can be allowed to run over the upper dike if protected with plastic sheet. For large canals, a pump is frequently used for filling the pond. For a series of ponds along a canal, a pipe is placed through each dike with a gate at the inlet, or some other mechanism for plugging this pipe.

The test equipment that is commonly used consists of one or two staff gauges, one or two hook gauges, a water stage recorder, stilling wells for the hook gauges and recorder, and in some cases, an evaporation pan. A ponding test can be conducted with only one staff gauge; however, if there is very much wind, then gauges should be used at both the upstream and downstream ends of the pond. Hook gauges should be used when seepage loss rates are low, such as clay soils or lined channels.

Each staff or hook gauge should be referenced to true elevation so that depths of water in the pond can be compared with design operating depths. The gauges can be installed on vertical uprights that have been firmly positioned. The stilling wells for the hook gauges and recorder can be made from metal or plastic pipe, with small piezometer openings of 5 millimeter diameter to dampen water surface disturbances due to wind and wave action. An oil drum can be used for the recorder stilling well.

Good judgment must be exercised in deciding on the necessity for rain or evaporation measurements. If ponding tests are being conducted during the monsoon season, then there must be a capability for measuring precipitation. Evaporation needs to be measured if it will be significant, say 10-20 percent, or more, of the expected seepage loss rate.

An engineering survey to establish the shape of the pond is required before filling the pond. For irregular earthen channels, cross-sections should be measured every 15 meters in length and elevations and widths measured to the closest centimeter. This is usually done with a surveyor's level and rod, along with a tape measure. The survey should establish the shape of the canal to an elevation about 30 centimeters above the anticipated water test level or normal water surface elevation (full supply level).

For more uniform channels, such as well maintained earthen channels, cross-sections can be taken every 30 meters instead of 15 meters. For lined channels, only a few cross-sections will be needed.

From the survey of the pond cross-sections, calculations can be made to determine: (a) the variation in water surface width with elevation; and (b) the variation in wetted perimeter with elevation.

The first step is to plot the canal cross-sections to a scale that will allow the water surface widths and wetted perimeters to be measured within one centimeter. Beginning with an upper elevation about 30 centimeters above the normal water surface at the downstream dike, then the water surface width and wetted perimeter should be scaled from the drawings in increments of 3 to 10 centimeters. This data is placed on two separate tables, each with fixed predetermined elevations across the top of the table and each of the stations are listed in the first column. One of the tables is for water surface widths, which are summed and averaged for each elevation. The other table lists the wetted perimeter for each elevation and station, which are then summed and averaged for each elevation. Having these tables completed prior to filling the pond, allows the seepage loss rates to be calculated in the field during the ponding test.

Generally, a pond should be filled twice. If the test-reach proves to have very high seepage loss rates, such as greater than 0.5 cubic meters per square meter per day (cmd), then the pond should be filled three times.

When conducting a ponding test, a form for recording "Seepage Loss Data" should be used that gives the name of the irrigation channel, the stations for the axis of the upstream and downstream dikes, and the location of the staff gauges, hook gauges, and recorder. The hook gauges are read to the nearest 0.1 millimeters and the staff gauges to the nearest millimeter. Initially, readings are taken quite frequently, say every 15 to 30 minutes, but the time between readings is increased after a few hours to every 1-4 hours according to the rate of fall of the water in the pond.

Between each set of readings, the seepage loss rate in cubic meters of seepage loss per square meter of wetted area per day can be calculated. For the example ponding test described in the following section, upon refilling a pond, readings are taken at 9:00 A.M. and noon, a time increment of 3 hours (Table 0-06-6). At 9:00 A.M, . the water surface elevation in the pond was 82.445 meters, which had dropped to 82.415 at noon, a difference of 30 millimeters. Based on the pond survey, the interpolated values of the water surface width are 3.90 meters and 3.84 meters, while the wetted perimeter decreased from 4.39 meters at 9:00 A.M. to 4.27 meters at noon. The volume of seepage loss is the average water surface width of 3.87 meters multiplied by the drop in water surface elevation of 0.030 meter multiplied by the length of the pond of 270 meters, which results in 31.347 cubic meters of seepage loss. The wetted surface area is the average wetted perimeter of 4.33 meters multiplied by the length of the pond of 270 meters, which is 1169.1 square meters. The time period between 9 A.M. and noon is 3 hours, which is 0.125 day. Thus, the seepage loss rate is 31.347 cubic meters divided by the terms 1169.1 square meters and 0.125 day, which is 0.214 cubic meters per square meter per day. Note that the length of pond is both in the numerator and the denominator, so it is not necessary in the computations.

The purpose of the ponding test is to measure the seepage loss rate. Low seepage loss rates are 0.009 to 0.03 cubic meters per square meter per day. A poorly lined canal or an unlined canal with significant losses would have a seepage loss rate of 0.15, or higher, cubic meters per square meter per day. Seepage loss rates greater than one have been measured.

Example of Main Canal Ponding Test

This example problem has been taken from the training manual, "Measuring Seepage in Irrigation Canals by the Ponding Method." This is an unlined earthen canal that illustrates the methodology described above. Tables 0-06-3 and 0-06-4 represent the field survey of the pond prior to conducting the ponding test. Table 0-06-5 is the actual field data measurements beginning with the first filling on September 16, and continuing through the second filling on September 18. Finally, the seepage loss rate computations are shown in Table 0-06-6.

Table 0-06-3. Table of Water Surface Widths in meters for the Example Ponding Test.

Canal High Line Pond No. AT STA 3 + 480 to 3 + 735

Elevation / Station	Water Surface Widths for Various Elevations							
	582.13	582.19	582.25	582.31	582.37	582.43	582.49	582.55
3 + 480	2.62	2.81	2.96	3.17	3.32	3.54	3.69	3.90
3 + 495	2.65							
3 + 510	2.68							
3 + 525	2.74							
3 + 540	2.77							
3 + 555	2.84							
3 + 570	2.80							
3 + 585	2.74							
3 + 600	2.90							
3 + 615	2.93							
3 + 630	2.99							
3 + 645	3.05							
3 + 660	3.05							
3 + 675	2.96							
3 + 690	3.26							
3 + 705	3.35							
3 + 720	3.42							
3 + 735	3.32	3.35	3.38	3.42	4.15	4.30	4.48	4.73
Total	53.07							
Average	2.95	3.08	3.17	3.29	3.72	3.90	4.09	4.36

The figures in this table are used as an illustration. The table is not completely filled out. Interpolate for water surface widths not calculated.

Table 0-06-4. Table of Wetted Perimeters in meters for the Example Ponding Test.

Canal High Line Pond No. AT STA. 3 + 480 to 3 + 735

Station \ Elevation	Wetted Perimeters for Various Elevations							
	582.13	582.19	582.25	582.31	582.37	582.43	582.49	582.55
3 + 480	3.35	3.54	3.66	3.69	3.75	4.06	4.57	4.73
3 + 495	3.38							
3 + 510	3.35							
3 + 525	3.51							
3 + 540	3.45							
3 + 555	3.42							
3 + 570	3.63							
3 + 585	3.72							
3 + 600	3.69							
3 + 615	3.57							
3 + 630	3.54							
3 + 645	3.42							
3 + 660	3.57							
3 + 675	3.63							
3 + 690	3.66							
3 + 705	3.54							
3 + 720	3.72							
3 + 735	3.66	3.69	3.75	4.06	4.57	4.73	4.94	5.31
Total								
Average	3.55	3.60	3.72	3.87	4.24	4.39	4.82	5.00

The figures shown in this table are used as an illustration. Table is not completely filled out. Interpolate for wetted perimeters not calculated.

Table 0-06-5.

Field Data Measurements for Seepage Loss in Example Ponding Test.

NOTE: Assume two staff gages and a recorder used on this pond.

Canal High Line

Dike: sta. 3 + 480 and sta. 3 + 750

Hook gages: sta. NONE and sta. ___

Staff gages: sta. 3 + 495 (S_u) and sta. 3 + 720 (S_d) Recorder: LMT

DATE	TIME	HOOK		STAFF		TEMPERATURE		add 581.823 to staff gage for elevation	REMARKS
		H_n	H_d	S_u	S_d	AIR	WATER		
9/16	9:00am			0.707	0.707	16	13	582.476	Normal w.s.
9/16	3:00pm			0.654	0.651	29	14	582.476	582.44 at sta.3 + 735
9/16	11:00pm			0.567	0.567	21	18	582.390	
9/17	9:00am			0.461	0.461	18	14	582.284	
9/17	5:00pm			0.385	0.382	32	16	582.207	
9/18	1:00am			0.321	0.318	18	18	582.143	Refilled pond
9/18	9:00am			0.619	0.625	20	14	582.445	During night.
9/18	12:00noon			0.592	0.592	29	16	582.415	
9/18	5:00pm			0.543	0.543	32	18	582.366	

Table 0-06-6.

Seepage Loss Rate Computations for Example Ponding Test.

1	2	3	4	5	6	7	8	9	10	11	12
Date	Time	Elapsed time, hours	Water surface elevation	Drop in water surface meters	Water surface width meters	Average water surface width, meters	Product of Columns 5 & 7 m^3	Wetted perimeter, meters	Average wetted perimeter, m	Product of Columns 3 & 10 m – hr.	Seepage rate cmd $\frac{\text{Col. 8} \times 24}{\text{Col. 11}}$
9/16	9:00 am		582.530		4.21			4.85			
		6.0		0.054		4.135	0.223		4.415	26.49	0.202
9/16	3:00 pm		582.476		4.06			3.98			
		8.0		0.086		3.920	0.337		4.110	32.88	0.246
9/16	11:00 pm		582.390		3.78			4.24			
		10.0		0.106		3.490	0.370		3.710	37.10	0.239
9/17	9:00 am		582.284		3.20			3.78			
		8.0		0.017		3.140	0.242		3.690	29.52	0.197
9/17	5:00 pm		582.207		3.08			3.60			
		8.0		0.064		3.020	0.193		3.553	28.42	0.163
9/18	1:00 am		582.143		2.96			3.506			
	—	REFILL —									
9/18	9:00 am		582.445		3.90			4.39			
		3.0		.030		3.87	0.116		4.330	12.99	0.214
9/18	12 noon		582.415		3.84			4.27			
		5.0		0.049		3.735	0.183		4.225	21.13	0.208
9/18	5:00 pm		582.366		3.63			4.18			

BASIC EQUATION:

$$(\text{cmd}) = \frac{\text{Length of Pond} \times \text{Drop in Water Surface} \times \text{Average Width of Water Surface} \times 24}{\text{Length of Pond} \times \text{Average Wetted Perimeter} \times \text{Hours of Run}}$$

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Section VII.

TERTIARY SYSTEM EVALUATION

A. EMPHASIS

A tertiary system is the land served by an outlet from the main system. This outlet is the last flow structure in the main canal, laterals or sub-laterals. There are many of these outlets, each serving a tertiary system. In most cases, a single Water Users Group is organized with every farmer served by the tertiary system being a member.

The tertiary system is quite complex because it contains three irrigation subsystems: (1) water delivery; (2) the farm; and (3) water removal.

The tertiary subsystem is the most neglected portion of new irrigation projects in Thailand. Increased emphasis on improving the performance of the tertiary channel network will enhance RID communication and credibility with farmers, and even more importantly, strengthen the organizational effectiveness of Water Users Groups.

The initial emphasis will be assisting each Water Users Group in providing more equitable water distribution to individual farmers. This will be accomplished by evaluating channel losses throughout the tertiary network, with participation by leadership in the Water Users Group, followed by communicating the results in open meetings with all Water Users Group members (farmers). First of all, these results will disclose whether improved maintenance needs to be performed by the Water Users Group. Also, this data will give an indication as to whether or not any improvements in the tertiary system are needed for providing an adequate water supply, or for equitably distributing water among farmers. This will be a highly participatory process of farmer involvement, with RID personnel providing technical assistance.

B. MEASURING CHANNEL LOSSES

Description of Approach

The first step is to design and conduct diagnostic field studies by taking field measurements on discharge rates, channel losses, farm water deliveries, etc. The main system outlet structure, if not already calibrated, should have a discharge rating developed so that the discharge rate, and consequently volumes of water delivered to the tertiary system, can be measured and recorded.

Often, the main system outlet is a gate structure, such as a CHO, that can be quickly calibrated by placing a temporary flow measurement structure, such as a Cutthroat flume, downstream in an earthen channel. Other water control structures in the tertiary system can be calibrated in a similar manner. If adequate control structures do not exist at junctions in the channel network, then consideration should be given to constructing better control structures, or temporarily installing standard flow measuring devices to make the necessary discharge measurements.

An important element of field evaluations is to determine the channel losses in various reaches of the tertiary network. The methods described in Section VI on "Measuring Irrigation Channel Losses" would apply. Wherever possible, the Inflow-Outflow Method should be used; however, frequently two Cutthroat flow measuring flumes will have to be temporarily installed for 2-8 hours, which will result in a slightly

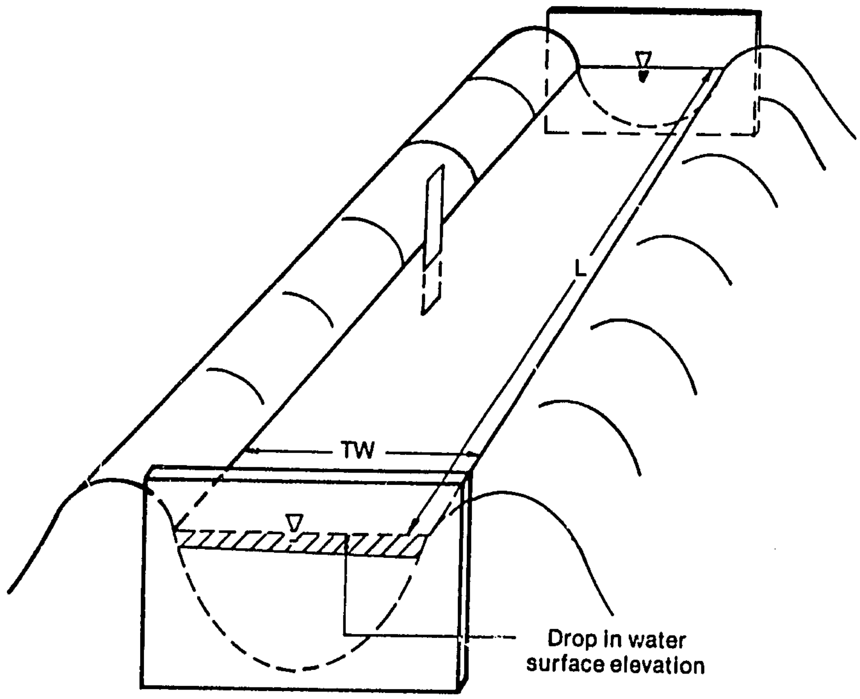

Figure 0-07-1. Ponding Test For a Tertiary Channel Using Metal End-Plates.

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higher water level in the reach and a higher measured value of the seepage loss rate. Also, it is very feasible to construct dikes at the upstream and downstream ends of a reach for conducting a ponding test, which could be repeated at a number of locations in the tertiary system.

If the water losses are considerable, then the causes of these losses must be established. For example, perhaps the earthen channel banks have become narrow and weak because of encroachment by adjoining farmers, or heavy vegetation or sedimentation have caused the water levels to rise so that the seepage loss rate is much greater, or some of the farm outlets are leaking water, or animals crossing the channels or wallowing in the channel have weakened the banks, or rodent holes are causing leakage at many locations. Determining the causes of the water losses is crucial to developing appropriate solutions.

Example of the Inflow-Outflow
Method Using CHO and Cutthroat Flume

Since the outlet structure serving a tertiary system should already have been calibrated as part of the process for improving water deliveries in the main system, this becomes a valuable structure for measuring the inflow, Q_u, to the tertiary system. If there is an existing structure located downstream that can be calibrated, then it can be used for the measurement of Q_d. Often, there is a lack of structures in the tertiary system that can be calibrated, so the use of a portable flow measuring device is required. This portable device might be installed for a few hours and then removed, or it may be left in-place for a few days, a season, or one or two years.

This example utilizes a CHO structure that has already been calibrated, which controls the outflow from the main system, that corresponds with the inflow to the tertiary system. A Cutthroat flume was installed downstream. The procedure for the work is as follows:

1. Select the channel and try to maximize the length of channel that is being evaluated;

2. Install the Cutthroat flume (in this example, 30 x 90 cm was used) at the downstream end of the reach (which is 992 meters in this example);

3. Set the difference in water levels upstream and downstream of the control gate in the CHO at 6 cm and then measure and record the gate opening;

4. Observe both h_u and h_d in the Cutthroat flume to be sure that the water depths are not changing and steady-state flow conditions exist. This may take a few hours, or even more time if submerged flow is occurring in the Cutthroat flume. After the flow has stabilized, measure and record the values of h_u and h_d;

5. Measure the wetted perimeter along the test reach every 15-30 meters depending upon the degree of variation in the channel cross-section and then calculate the average wetted perimeter for the reach, WP_{avg}, in meters;

6. Calculate the discharge, Q_u, at the CHO (a gate width of 0.60 m was used in this example, and the coefficient of discharge was equal to 0.667). The formula that was used in this calculations was:

$$Q_u = C_d \; W \; G_o \quad 2_g \quad h$$

7. Determine the discharge, Q_d, through the Cutthroat flume. For this example, free flow conditions occurred, so only h_u, the upstream flow depth, had to be used to determine the discharge from Table 5 in the manual "Cutthroat Flow Measuring Flumes";

8. Calculate the seepage loss rate, Q_{slr}, in mm/day using:

$$Q_{slr} = \frac{(Q_u - Q_d)\ 86{,}400{,}000}{(WP_{avg})\ (\text{Reach Length}, L)}$$

9. Plot Q_u (lps) vs. Q_{slr} (mm/day) on rectangular coordinate graph paper.

Table 0-07-1. Seepage Loss Computations by the Inflow-Outflow Method (CHO and Cutthroat flume).

CHO (upstream)					Cutthroat Flume (downstream) size 30x90 cm		Qu-Qd (cms)	Average Wetted Perimeter	Length of Reach	SLR (mm/day) Col 8 x 86,400,00
W (m) 1	Cd 2	Go (m) 3	h (m) 4	Qu = Cd W Go 2g h (cms) 5	hu (m) 6	Qd (cms) 7	8	(m) 9	(m) 10	Col 9 x Col 10 11
0.60	0.667	0.060	0.228	0.099	0.243	0.082	0.017	1.682	992	880
0.60	0.667	0.060	0.210	0.091	0.237	0.078	0.013	1.578	992	718
0.60	0.667	0.060	0.147	0.064	0.200	0.057	0.007	1.435	992	425
0.60	0.667	0.060	0.115	0.050	0.175	0.045	0.005	1.262	992	345

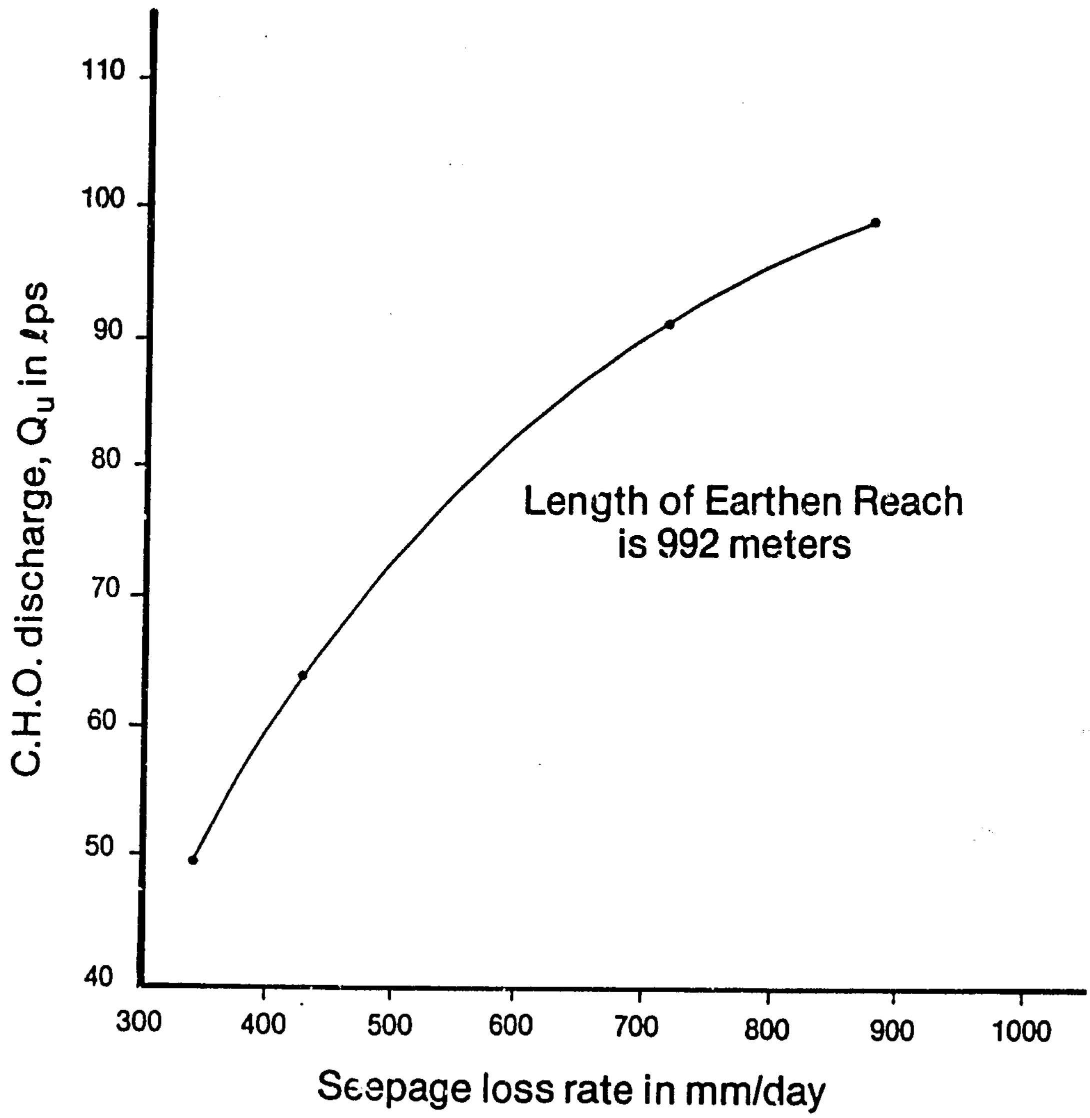

Figure 0-07-2. Variation of Seepage Loss Rate with CHO Discharge in Example of Inflow-Outflow Method in Tertiary System.

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Example of Inflow-Outflow
Method Using Two Cutthroat Flumes

Very commonly, in order to evaluate the seepage losses from many of the channels in the tertiary system network, two portable flow measuring devices will be required. Usually, relatively long reaches are needed in order to use the inflow-outflow method, because the seepage loss has to be significantly greater than the likely errors in discharge measurements upstream, Q_u, and downstream, Q_d.

In this example, two Cutthroat flumes are used as the portable flow measuring devices. For installing and using the Cutthroat flume, the training module on "Measuring Discharge in Irrigation Channels with a Cutthroat Flume" should be consulted.

The following procedure would be used:

1. Select the channel and try to maximize the length of channel that is being evaluated;

2. Install the Cutthroat flumes (in this example, two 30 cm x 90 cm flumes were used) at the upstream and downstream ends of the reach (which is 958 meters in this example);

3. Observe both h_u and h_d in the Cutthroat flumes to be sure that the water depths are not changing and steady-state flow conditions exist. This may take a few hours or even more time if submerged flow is occurring in either of the Cutthroat flumes. After the flow has stabilized, measure and record the values of h_u and h_d in both flumes;

4. Measure the wetted perimeter along the test reach every 15-30 meters, depending upon the degree of variation in the channel cross-section, and then calculate the average wetted perimeter for the reach, WP_{avg}, in meters;

5. Determine the discharge, Q_u and Q_d, through the Cutthroat flumes. For this example, free flow conditions occurred in both flumes, so only h_u, the upstream flow depth, had to be used to determine the discharge from Table 5 in the manual "Cutthroat Flow Measuring Flumes";

6. Calculate the seepage loss rate, Q_{slr}, in mm/day as shown in Table 0-07-2.

$$Q_{slr} = \frac{(Q_u - Q_d)(86,400,000)}{(WP_{avg})(\text{Reach Length}, L)}$$

7. Plot Q_u (lps) versus Q_{slr} (mm/day) on rectangular coordinate graph paper as shown in Figure 0-07-3.

For this example, a plot of the average wetted perimeter in meters was plotted against the seepage loss rate in mm/day as shown in Figure 0-07-4. This graph illustrates a typical situation in earthen channels. When the water level approaches the usual water levels in the channel, the seepage loss rate begins to rapidly increase. Then, small increases in water levels result in much higher seepage loss rates. The primary reason for this sudden increase is the vast amount of biological life that occurs in the moist capillary fringe about the phreatic line in the channel banks. There are also other factors such as cuts in the banks by farmers, thin embankments, and many more.

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Table 0-07-2. Seepage Loss Computations by the Inflow-Outflow Method (two Cutthroat flumes).

Cutthroat Flume 1 (upstream) size 20x90 cm		Cutthroat Flume 2 (downstream) size 20x90 cm		Qu-Qd (cms)	Wetted Perimeter Average (m)	Length of Reach	SIR (mm/day) Col 5 x 86,400,000 / Col 6 x Col 7
hu1 (m)	Qu (cms)	hu2 (m)	Qd (cms)	Col 2-Col 4			
1	2	3	4	5	6	7	8
0.158	0.025	0.145	0.022	0.003	1.130	958	240
0.173	0.030	0.155	0.024	0.006	1.556	958	348
0.218	0.046	0.173	0.030	0.016	1.743	958	828

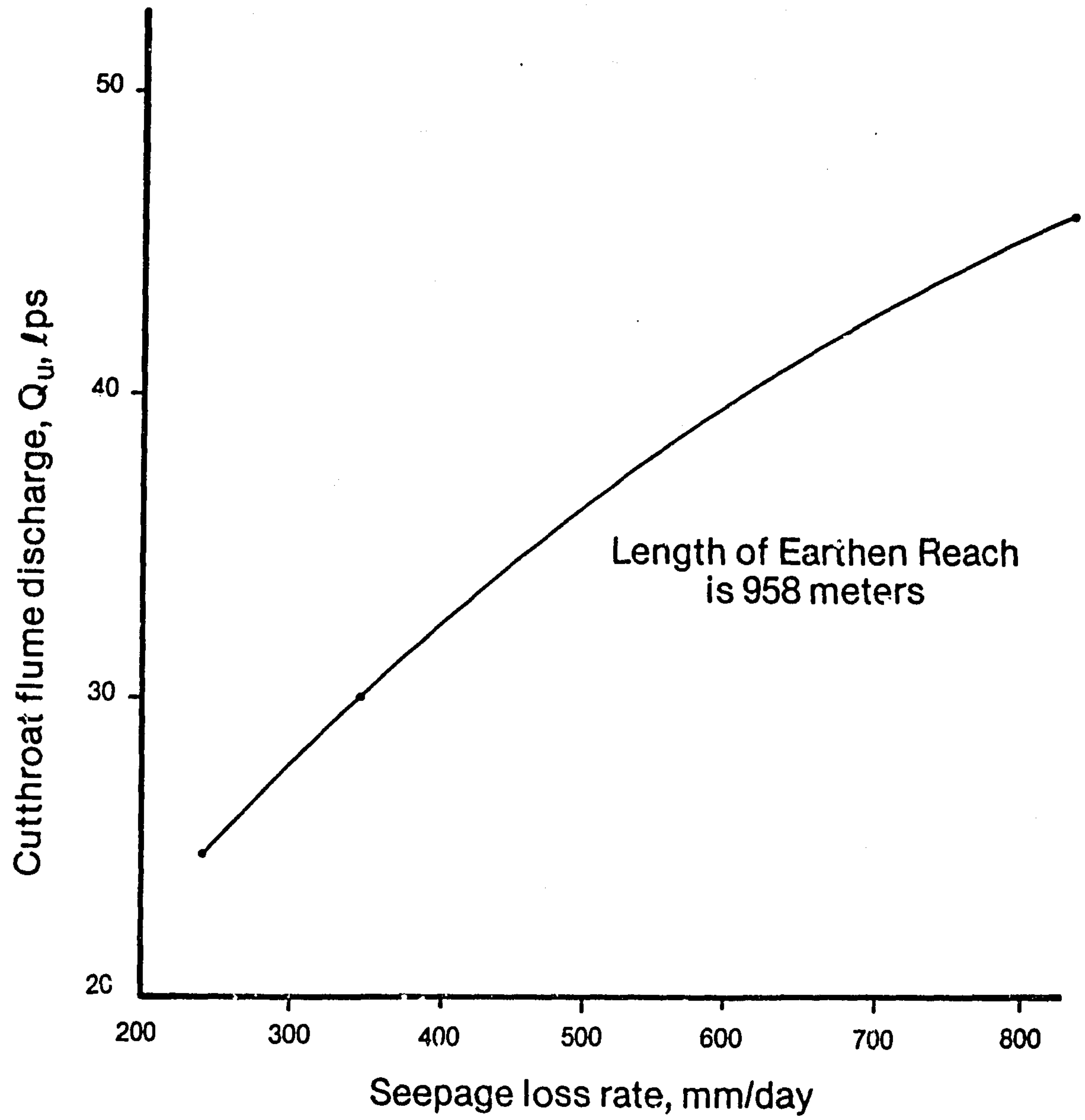

Figure 0-07-3. Variation of Seepage Loss Rate with Inlet Discharge, Q_u, Using Two Cutthroat Flumes for Inflow-Outflow Tests in a Tertiary Irrigation Channel.

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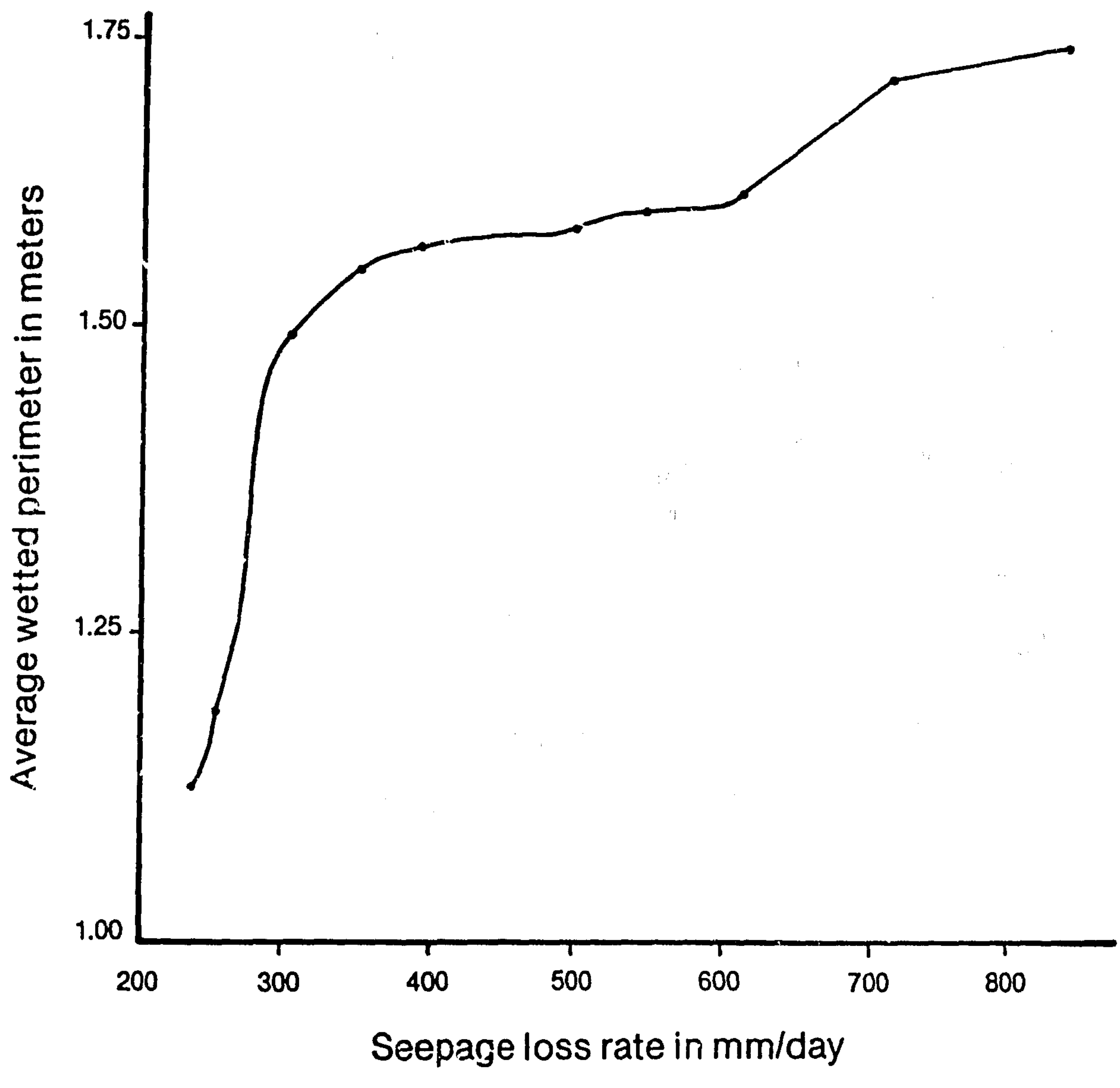

Figure 0-07-4. Variation of Seepage Loss Rate with Average Wetted Perimeter for Inflow-Outflow Tests Using Two Cutthroat Flumes in a Tertiary Irrigation Channel.

Example of the Ponding Method

Often, in the tertiary system, using the Ponding Method is necessary because there are many short channel reaches. Fortunately, conducting a ponding test is relatively easy and quick in a tertiary channel as compared with a main canal or large lateral.

The manual, "Measuring Seepage in Irrigation Canals by the Ponding Method" can be consulted. The procedure is as follows:

1. Select the channel reach for the ponding test taking into account the channel slope in determining the length of the pond.

2. Construct a small dike at each end of the pond and cover with plastic to prevent seepage or leakage through the dikes.

3. Install a staff gauge near each end of the pond for measuring water depths.

4. Place water in the pond up to the crest level of the downstream dike by letting water flow over the plastic covering the upstream dike.

5. Record the time and staff gauge readings periodically according to the deceasing rate of water depth in the pond.

6. Measure the water surface width and wetted perimeter for various flow depths in the pond every 15-30 m along the pond length, and calculate the average water surface width and wetted perimeter for the various water depths.

7. Calculate the seepage loss rate in cubic meters per day of water loss per square meter of wetted surface area from the equation,

$$Q_{slr} = \frac{\text{Water Surface Drop x Average Surface Width x 24}}{\text{Average Wetted Perimeter x Hours Elapsed}}$$

8. Plot the average water depth in meters versus the seepage loss rate in mm/day on rectangular coordinate graph paper.

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Table 0-07-3. Seepage Loss Computations by the Ponding Method.

Date	Time	Elapsed Time	Depth of Water (m)			Drop in Water Surface (m)	Water Surface Width (m)			Average Width of Water Surface (m)	Wetted Perimeter (m)		
			Staff Gage 1	Staff Gage 2	Average Depth		At Staff Gage 1	At Staff Gage 2	Average		At Staff Gage 1	At Staff Gage 2	Average
1	2	3	4	5	6	7	8	9	10	11	12	13	14
2 July 86	13.39		0.100	0.230	0.165		1.270	1.105	1.188		1.390	1.339	1.365
		28 min (0.47 hrs)			(0.160)	0.010				1.167			
	14.07		0.090	0.220	0.155		1.235	1.055	1.145		1.293	1.282	1.288
		63 min (1.05 hrs)			(0.150)	0.010				1.138			
	15.10		0.078	0.212	0.145		1.220	1.042	1.131		1.252	1.230	1.241
		62 min (1.03 hrs)			(0.1425)	0.005				1.126			
	16.12		0.074	0.206	0.140		1.200	1.040	1.120		1.250	1.230	1.240
		48 min (0.80 hrs)			(0.1385)	0.003				1.114			
	17.00		0.071	0.203	0.137		1.185	1.030	1.108		1.212	1.187	1.200
		45 min (0.75 hrs)			(0.136)	0.002				1.102			
	17.45		0.069	0.201	0.135		1.160	1.030	1.095		1.184	1.150	1.167

Basic Equation:

$$\text{Seepage Loss Rate in cmd} = \frac{\text{Col 7 x Col. 11 x 24}}{\text{Col. 15 x Col. 3}} = \frac{\text{Length of Pond x Drop in Water Surface x Average Width of Water Surface x 24}}{\text{Length of Pond x Average Wetted Perimeter x Hour of Run}}$$

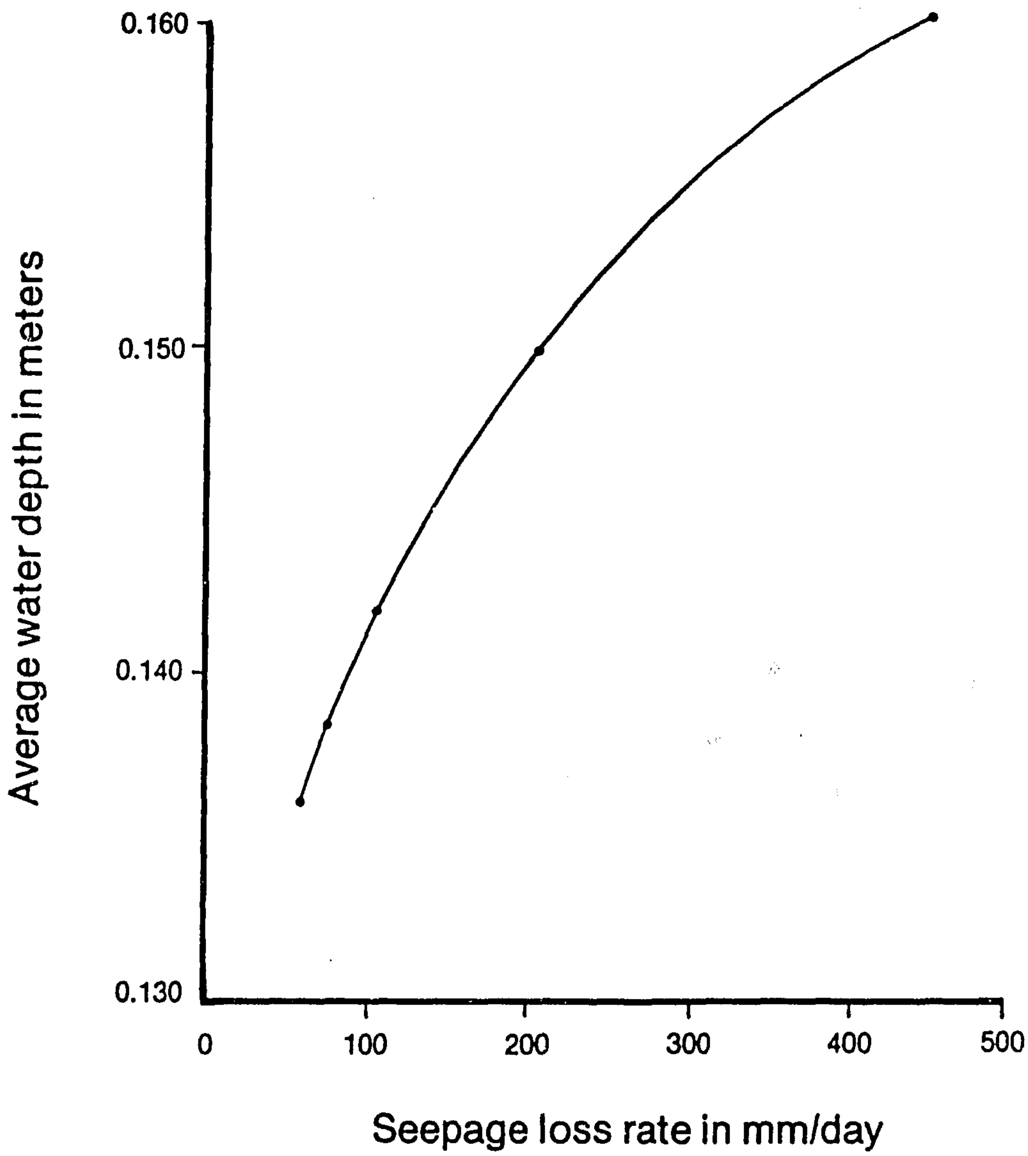

Figure 0-07-5. Variation of Seepage Loss Rate with Average Water Depth for Ponding Test in a Tertiary Irrigation Channel.

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C. MEASURING IRRIGATION APPLICATION EFFICIENCIES

Description of Approach

Another important consideration is to determine how much of the farm delivery is consumed as evapotranspiration by the plants, and how much is deep percolation below the root zone. To do this accurately, serious consideration should be given, particularly for dry season crops, to using lysimeters and weather stations to determine crop water use. Or, values of crop water requirements determined at RID experimental stations can be utilized.

To conduct field studies on each farm would require considerable resources and time. Consequently, the usual practice is to select a sample of the farms, probably no more than five percent, for initial field investigations. Sample farms should be selected that are representative of farms located at the head, middle and tail of the tertiary channels.

The simplest procedure is to install a Cutthroat flume at the inlet of a single bunded field, or preferably, in a channel that serves a number of bunded fields (say 2-8) in rotation. Then, the volume of water delivered to each bunded field must be monitored for each irrigation event during the season. Also, the area of each field should be measured. Then, knowing the crop evapotranspiration (by calculation or by lysimetry), the volume of deep percolation loss can be calculated for each irrigation event and for the season. A point to remember is that any inaccuracies in measuring the volume of water applied, the area of the field, and the crop evapotranspiration are all reflected in the calculated values of deep percolation losses. Taking into consideration that the likely errors in these measurements are about 5 percent for the volume of water applied, 1-2 percent for the field area (and sometimes more), and 10-30 percent for the crop evapotranspiration, then it can be appreciated that the calculated values of deep percolation losses are likely to be in error by 20-40 percent.

Example of Measuring
Irrigation Application Efficiencies

One of the simplest techniques to measure irrigation application efficiency on a bunded field is to just measure the water applied during each irrigation event of the season. Many fields could be evaluated during an irrigation season if one individual were available to measure staff gauge readings and the time of application.

A relatively simple example is illustrated in Figure 0-07-6, where one Cutthroat flume was installed for the irrigation season to measure the water applied onto two fields side-by-side. The same farmer owned both fields, plus additional fields nearby that he irrigated during the wet season only. The tomatoes were grown on beds with deep furrows, which allowed the water to advance to the end of the field rapidly. The sweet corn was also planted on the top of furrows, but because the furrows were not as large as in the tomato field, and because the field was longer, then larger depths of water had to be applied during each irrigation event for the field growing sweet corn.

These crops were grown during the dry season, so measurements of precipitation were not collected, primarily because this was the first season these tests were conducted; however, in the future, precipitation measurements will be collected. Also, there is always

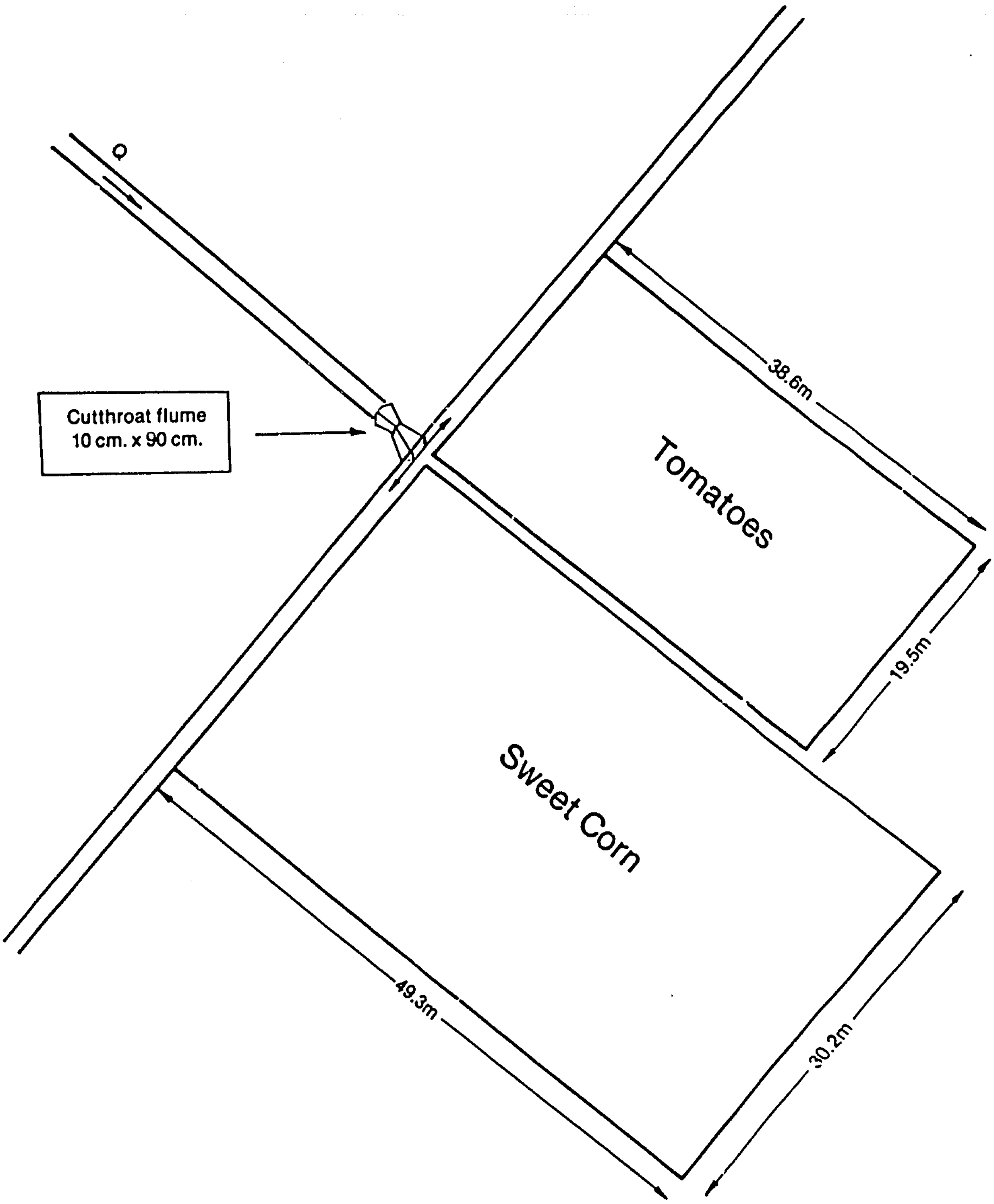

Figure 0-07-6. Layout of Fields and Flow Measuring Device for Measuring Irrigation Application Efficiencies.

Table 0-07-4. Irrigation Application Efficiency Measurement and Calculation for a Bunded Area (38.6 m x 19.5 m) with Tomatoes.

Week	Consumptive Use (mm/day)	Depth of Water Requirement (mm)	Cutthroat Flume 10 cm x 90 cm hu (m)	Q (cms)	Time of Water Supply (min)	Water Supply (m^3)	Depth of Water Supply (mm)
1	3.38	24	0.210	0.020	25	30.00	40
2	3.38	24	0.220	0.022	32	42.24	56
3	3.38	24	0.230	0.024	45	64.80	86
4	3.38	24	0.235	0.025	30	45.00	60
5	4.18	29	0.240	0.026	35	54.6	73
6	4.18	29	0.235	0.025	25	37.50	50
7	4.18	29	0.230	0.024	35	50.40	67
8	4.18	29	0.230	0.024	30	43.20	57
9	4.85	34	-	-	-	-	-
10	4.85	34	0.225	0.023	25	34.50	46
11	4.95	34	-	-	-	-	-
12	4.85	34	0.215	0.021	35	44.10	59
13	5.35	38	-	-	-	-	-
		386					594

$$\text{Irrigation Application Efficiency, Ea} = \frac{\text{Depth of Water Requirement}}{\text{Depth of Water Supply}} = \frac{389 \text{ mm}}{594 \text{ mm}} = 65\%$$

Table 0-07-5. Irrigation Application Efficiency Measurement and Calculation for Bunded Area (30.2 m x 49.3 m) with Sweet Corn.

Week	Consumptive Use (mm/day)	Depth of Water Requirement (mm)	Cutthroat Flume 10 cm x 90 cm hu (m)	Q (cms)	Time of Water Supply (min)	Water Supply (m3)	Depth of Water Supply (mm)
1	3.38	24	0.210	0.020	90	108.00	73
2	3.38	24	0.220	0.022	85	112.20	75
3	3.38	24	0.230	0.024	95	136.8	92
4	3.38	24	0.235	0.025	98	147.00	99
5	4.18	29	0.240	0.026	88	137.28	92
6	4.18	29	0.235	0.025	92	138.00	93
7	4.18	29	0.230	0.024	100	144.00	97
8	4.18	29	-	-	-	-	-
9	4.85	34	0.220	0.022	125	165.00	111
10	4.85	34	-	-	-	-	-
11	4.85	34	-	-	-	-	-
		314					732

$$\text{Irrigation Application Efficiency, } Ea = \frac{\text{Depth of Water Requirement}}{\text{Depth of Water Supply}} = \frac{314 \text{ mm}}{732 \text{ mm}} = 43\%$$

some question as to the accuracy of the estimated consumptive use, which can easily be in error by more than 10 percent unless local calibrations for crop evapotranspiration are conducted using lysimeters.

The results for the field of tomatoes is listed in Table 0-07-4. The seasonal irrigation application efficiency of 65 percent is very good. The results for the field of sweet corn is listed in Table 0-07-5, where the seasonal irrigation application efficiency of 43 percent is good, but could be better.

This is only one example of an approach for measuring irrigation application efficiencies, but one of the simplest techniques. Consult the manuals for the training course, "On-Farm Water Management for Tertiary Systems" for more detailed approaches and techniques.

D. PREPARING WATER BUDGETS

Description of Approach

Another important tool for assessing a tertiary system is to prepare a water budget for each portion of the channel network, including the cropped lands served by the channel. Such water budgets begin with the lower portions, or branches, of the channel network and progress upstream to the main system outlet.

When the water deliveries and channel losses have been determined, and the water budgets for the tertiary system are completed, the results should be presented in a meeting open to all farmers served by the tertiary network. There may be serious inequities in the amount of water being received by various farmers. In some cases, certain inequities can be easily resolved, but often the water users will have to discuss at length the measures that they might employ to improve the equity of water deliveries. Although options can be presented to the Water Users Group, it will usually be important that they decide among themselves what appropriate remedies might be implemented, if any.

A major advantage in preparing water budgets for a tertiary system is to provide much more reliable estimates of the time distribution of water supply required at the main system outlet throughout the irrigation season, both the wet season and the dry season. When this task is completed for many of the tertiary systems in an irrigation project, then more equitable water distribution will occur and crop yields can be expected to increase.

The Zoneman responsible for delivering water at the main system outlet should have participated in the field evaluations. Then, he will also have a much better understanding and sensitivity about the situation below the main system outlet. This will allow the Zoneman to be more responsive to the needs of the Water Users Groups and to better communicate with the Water Users Group Leaders.

If the Operations Plan calls for a predetermined rotation schedule, then this schedule can be posted on a water distribution board located at the main system outlet. If computerized irrigation system management is being employed, then the Zoneman will be expected to be in almost daily contact with the Leader of the Water Users Group in order to respond to changing demands in the tertiary system.

Example of Preparing a Seasonal Tertiary System Water Budget

There are many ways to prepare a seasonal water budget for a tertiary system. The simplest is to use the crop survey data and published values of crop consumptive use to estimate the total water consumed by the crops, which can then be compared with the total volume of water delivered at the outlet for the tertiary system, to determine the overall tertiary system efficiency. However, this does not provide any information about how much of the losses are due to channel seepage and how much is due to deep percolation losses on the croplands.

This example tertiary system (Figure 0-07-7) illustrates the use of seepage loss studies and measuring irrigation application efficiencies to develop a better understanding of what is occurring within the tertiary system. There are many more measurements that could be made as compared with this particular example. Usually, additional data is collected each year, so that the internal functioning of the tertiary system is better understood each succeeding season. For this example, the water distribution in the system begins from the CHO which diverts water from the sub-lateral. The water will pass through the main tertiary channel, to the branch channel, and then to the agricultural lands. The basic data that are needed include:

1. Cropping area and crop type;
2. Irrigation application efficiency for each crop type, if possible, but probably only some fields will have been measured; and,
3. Seepage loss rates for as many reaches within the tertiary channel network as possible.

The calculations for the tertiary system began with the reach along the main channel and continued up to where two branch ditches start from the same location (Point B in Figure 0-07-7). In this case, the agricultural areas at the left and right sides received water from

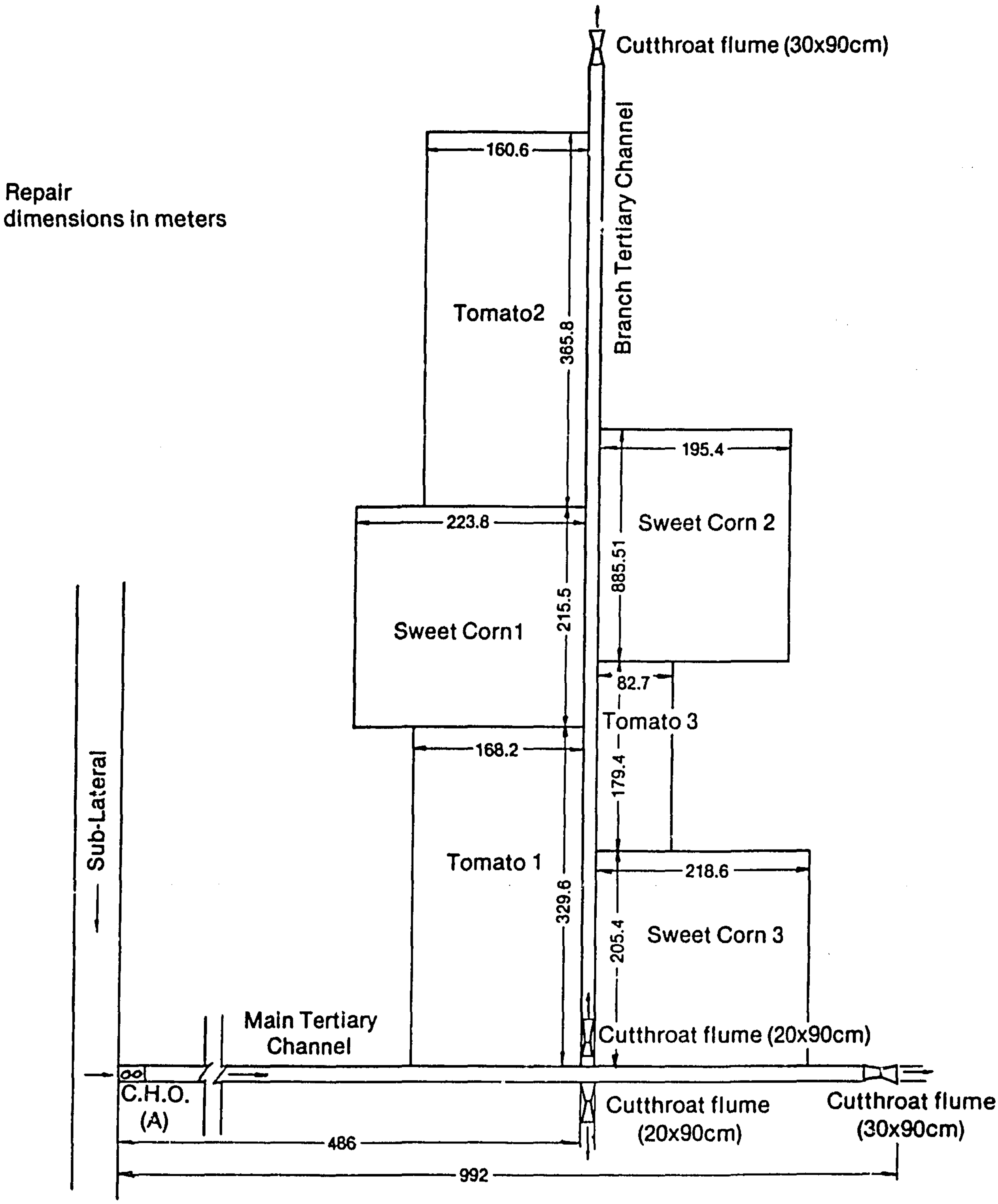

Figure 0-07-7. Physical Layout of Dry Season Cropping for the Example Tertiary System.

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the same main channel. It was assumed that the same discharge served both the left and right agricultural areas.

One of the agricultural areas had been divided into six bunded areas which were planted in tomatoes and sweet corn. The water outlets were located at the center of each bunded area.

The calculations began with the estimation of the weekly water requirement for each crop. The water which had to be delivered through the outlet was estimated by taking the irrigation application efficiency into account for each crop. For example, in the first week the water diverted to the branch channel was 0.030 cms. The seepage loss was estimated up to each outlet location. In this case, the area T_1 received 0.029 cms of water and the seepage loss was 0.001 cms. For the area S_1 the water delivery was 0.027 cms and the seepage loss was 0.003 cms. The seepage loss calculations are shown in Table 0-07-7. The irrigation schedule was adjusted for the whole area in seven weeks. The efficiency was close to the efficiency for the experimental plots. Due to the tertiary system having two branches at Point B, 0.060 cms of water was needed. The calculations for seepage loss in the main channel (Table 0-07-6) shows that the water releases through the CHO should be about 0.064 cms.

The water budget computations are summarized in Table 0-07-9. For the total dry season, the crop water requirement for the half of the tertiary system being evaluated was 92,675 cubic meters, which is then divided by the total water supply delivered to the fields of 178,077 cubic meters, to arrive at the tertiary system irrigation application efficiency of 52 percent. Likewise, the farm deliveries of 178,077 cubic meters are divided by half of the total water discharged from the

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Table 0-07-6. Seepage Loss Computations from CHO A to Control Structure B in Example Tertiary System.

Week	CHO Station A					Total Length (m)	Qd (cms)	Length A to B (m)	Q at Stat B (cms)	Q loss A to B (cms)	SIR (mm/day)
	W (m)	Cd	h (m)	Go (m)	Qa=Cd W Go 2g h (cms)						
1	0.60	0.667	0.060	0.147	0.064	992	0.057	496	0.060	0.004	425
2	0.60	0.667	0.060	0.159	0.069	992	0.060	496	0.064	0.005	470
3	0.60	0.667	0.060	0.136	0.059	992	0.053	496	0.056	0.003	395
4	0.60	0.667	0.060	0.216	0.094	992	0.079	496	0.086	0.008	775
5	0.60	0.667	0.060	0.200	0.087	992	0.074	496	0.080	0.007	665
6	0.60	0.667	0.060	0.189	0.082	992	0.070	496	0.076	0.006	595
7	0.60	0.667	0.060	0.180	0.078	992	0.067	496	0.072	0.006	555
8	0.60	0.667	0.060	0.147	0.064	992	0.057	496	0.060	0.004	425
9	0.60	0.667	0.060	0.200	0.087	992	0.074	496	0.080	0.007	665
10	0.60	0.667	0.060	0.159	0.069	992	0.060	496	0.064	0.005	470
11	—	—	—	—	—	—	—	—	—	—	—
12	0.60	0.667	0.060	0.180	0.078	992	0.067	496	0.072	0.006	555
13	—	—	—	—	—	—	—	—	—	—	—

Table 0-07-7. Seepage Loss Computations from Control Structure B to the Outlet at Each Bunded Area in the Example Tertiary Systems.

Week	Cutthroat Flume (B) size = 20 x 90 cm		Qo (cms)						
	hu (m)	Qu (cms)	Tomato T1 L = 165 m	Tomato T2 L = 728 m	Tomato T3 L = 295 m	Sweet Corn S1 L = 437 m	Sweet Corn S2 L = 498 m	Sweet Corn S3 L = 103 m	SIR (mm/day)
1	0.173	0.030	0.029	0.025	0.028	0.027	0.027	0.029	348
2	0.180	0.032	0.031	0.027	0.030	0.029	0.028	0.031	390
3	0.168	0.028	0.027	0.024	0.026	0.026	0.025	0.028	300
4	0.210	0.043	0.041	0.032	0.039	0.037	0.036	0.041	718
5	0.203	0.040	0.038	0.032	0.037	0.035	0.034	0.039	615
6	0.198	0.038	0.036	0.030	0.035	0.033	0.033	0.037	556
7	0.190	0.036	0.034	0.029	0.033	0.032	0.031	0.035	495
8	0.173	0.030	0.029	0.025	0.028	——	——	——	348
9	0.203	0.040	——	——	——	0.035	0.034	0.039	615
10	0.180	0.032	0.031	0.027	0.030	——	——	——	390
11	——	——	——	——	——	——	——	——	——
12	0.190	0.036	0.034	0.029	0.033	——	——	——	495
13	——	——	——	——	——	——	——	——	——

Table 0-07-8a. Calculations of Dry Season Water Budget for the Example Tertiary System.

Week	Consumptive Use (mm/day)	T1 (Tomatoes: 168.2 m x 329.6 m)				T2 (Tomatoes: 160.6 m x 365.8 m)				T3 (Tomatoes: 82.7 m x 179.4 m)			
		Water Req (cu m)	Water Supply (cms)	Water Supply Time (hr)	Water Supply (cu m)	Water Req (cu m)	Water Supply (cms)	Water Supply Time (hr)	Water Supply (cu m)	Water Req (cu m)	Water Supply (cms)	Water Supply Time (hr)	Water Supply (cu m)
1	3.38	1,312	0.029	24.82	2,591	1,390	0.025	31.51	2,836	351	0.028	6.98	704
2	3.38	1,312	0.031	23.57	2,630	1,390	0.027	30.82	2,996	351	0.030	6.70	724
3	3.38	1,312	0.027	24.02	2,335	1,390	0.024	29.78	2,573	351	0.026	7.94	743
4	3.38	1,312	0.041	22.85	3,373	1,390	0.032	32.54	3,749	351	0.039	8.04	1,129
5	4.18	1,622	0.038	26.18	3,581	1,719	0.032	31.49	3,628	434	0.037	7.01	934
6	4.18	1,622	0.036	26.59	3,446	1,719	0.030	30.17	3,258	434	0.035	7.58	955
7	4.18	1,622	0.034	24.29	2,973	1,719	0.029	32.50	3,393	434	0.033	7.49	890
8	4.18	1,622	0.029	31.94	3,335	1,719	0.025	38.26	3,443	434	0.028	9.48	956
9	4.85	1,882	—	—	—	1,994	—	—	—	504	—	—	—
10	4.85	1,882	0.031	34.08	3,803	1,994	0.027	43.66	4,244	504	0.030	9.12	985
11	4.85	1,882	—	—	—	1,994	—	—	—	504	—	—	—
12	4.85	1,882	0.034	57.45	7,032	1,994	0.029	37.39	3,904	504	0.033	8.21	975
13	5.35	2,076	—	—	—	2,200	—	—	—	556	—	—	—
		21,340			35,099	22,612			34,024	5,712			8,995

$$\text{Efficiency} = \frac{\text{Water Req}}{\text{Water Supply}} \qquad \text{Efficiency} = \frac{\text{Water Req}}{\text{Water Supply}} \qquad \text{Efficiency} = \frac{\text{Water Req}}{\text{Water Supply}}$$

$$\text{Efficiency} = \frac{21,340}{35,099} = 60.8\ \% \qquad \text{Efficiency} = \frac{22,612}{34,024} = 66.5\ \% \qquad \text{Efficiency} = \frac{5,712}{8,995} = 63.5\ \%$$

Table 0-07-8b. Calculations of Dry Season Water Budget for the Example Tertiary System.

Week	Consumptive Use (mm/day)	S1 (Sweet Corn: 215.5 x 223.8 m) Water Req (cu m)	S1 Water Supply (cms)	S1 Water Supply Time (hr)	S1 Water Supply (cu m)	S2 (Sweet Corn: 225.5 x 195.4 m) Water Req (cu m)	S2 Water Supply (cms)	S2 Water Supply Time (hr)	S2 Water Supply (cu m)	S3 (Sweet Corn: 205.4 x 218.6 m) Water Req (cu m)	S3 Water Supply (cms)	S3 Water Supply Time (hr)	S3 Water Supply (cu m)
1	3.38	1,141	0.027	37.63	3,658	1,043	0.027	34.42	3,346	1,062	0.029	32.64	3,408
2	3.38	1,141	0.029	39.26	4,099	1,043	0.028	34.22	3,449	1,062	0.031	33.43	3,731
3	3.38	1,141	0.026	38.06	3,562	1,043	0.025	37.55	3,380	1,062	0.028	30.65	3,090
4	3.38	1,141	0.037	36.89	4,914	1,043	0.036	34.49	4,470	1,062	0.041	33.19	4,899
5	4.18	1,411	0.035	36.70	4,624	1,289	0.034	34.37	4,207	1,314	0.039	32.26	4,529
6	4.18	1,411	0.033	37.15	4,413	1,289	0.033	34.70	4,122	1,314	0.037	31.80	4,236
7	4.18	1,411	0.032	38.26	4,408	1,289	0.031	34.15	3,811	1,314	0.035	31.32	3,946
8	4.18	1,411	—	—	—	1,289	—	—	—	1,314	—	—	—
9	4.85	1,637	0.035	61.47	7,745	1,496	0.034	40.99	5,017	1,524	0.039	20.62	2,895
10	4.85	1,637	—	—	—	1,496	—	—	—	1,524	—	—	—
11	4.85	1,637	—	—	—	1,496	—	—	—	1,524	—	—	—
12	4.85	—	—	—	—	—	—	—	—	—	—	—	—
13	5.35	—	—	—	—	—	—	—	—	—	—	—	—
		15,119			37,423	13,816			31,802	14,076			30,734

$$\text{Efficiency} = \frac{\text{Water Req}}{\text{Water Supply}}$$

$$\text{Efficiency} = \frac{15{,}119}{37{,}423} = 40.4\ \%$$

$$\text{Efficiency} = \frac{\text{Water Req}}{\text{Water Supply}}$$

$$\text{Efficiency} = \frac{13{,}816}{31{,}802} = 43.4\ \%$$

$$\text{Efficiency} = \frac{\text{Water Req}}{\text{Water Supply}}$$

$$\text{Efficiency} = \frac{14{,}076}{30{,}734} = 45.8\ \%$$

Table 0-07-8c. Calculations of Dry Season Water Budget for the Example Tertiary System.

Week	Consumptive Use (mm/day)	Q_B, Water Supply at Stat B (1 way) (cms)	Q_B, Water Supply at Stat B (2 way) (cms)
1	3.38	0.030	0.060
2	3.38	0.032	0.064
3	3.38	0.028	0.056
4	3.38	0.043	0.086
5	4.18	0.040	0.080
6	4.18	0.038	0.076
7	4.18	0.036	0.072
8	4.18	0.030	0.060
9	4.85	0.040	0.080
10	4.85	0.032	0.064
11	4.85	-----	-----
12	4.85	0.036	0.072
13	5.35	-----	-----

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Table 0-07-9. Summary of Water Budget Computations for Example Tertiary System.

Week	Total Crop Water Requirement For One Branch Tertiary Canal (m3)	Total Water Supply At Fields (m3)	Total Water Supply Delivered At B (m3)	Seepage Loss From CHO To B (m3)	Total Water Supply At CHO (m3)
1	6,299	16,543	18,144	2,419	38,707
2	6,299	17,629	19,354	3,023	41,731
3	6,299	15,683	16,934	1,815	35,683
4	6,299	22,534	26,006	4,839	56,851
5	7,789	21,503	24,192	4,234	52,618
6	7,789	20,430	22,982	3,630	49,594
7	7,789	19,421	21,773	3,628	47,174
8	7,789	7,734	18,144	2,419	38,707
9	9,037	15,657	24,192	4,234	52,618
10	9,037	9,032	19,354	3,023	41,731
11	9,037	—	—	—	—
12	4,380	11,911	21,773	3,628	47,174
13	4,832	—	—	—	—
Grand Total	92,675	178,077	232,848	36,892	502,588

$$\text{Tertiary System Irrigation Application Efficiency} = \frac{92{,}675}{178{,}077} = 52\ \%$$

$$\text{Tertiary System Conveyance Efficiency} = \frac{178{,}077}{(502{,}588/2)} = 71\ \%$$

$$\text{Tertiary System Efficiency} = \frac{92{,}675}{(502{,}588/2)} = 37\ \%$$

CHO (i.e. half of 502,588 cubic meters) to arrive at the tertiary system conveyance efficiency of 71 percent. The overall tertiary system efficiency is the crop water requirements of 92,675 cubic meters divided by half of the 502,588 cubic meters discharged from the CHO, which is 37 percent (or, 0.52 multiplied by 0.71).

Section VIII.

DEVELOPING A MAIN SYSTEM OPERATIONS PLAN

A. PRESENT SCHEDULING PROCEDURES

In some small-scale and medium-scale irrigation projects, the Operation Plan depends on the demand of water that is required by the farmer. The farmer would ask for water at the beginning of the land preparation period through the Zoneman. The Water Master only delivers water in the canal at the same amount for the entire season, except the Head Regulator would be decreased according to the precipitation. The farmers will use water according to their needs and will control the outlets by themselves. The Water Master also has to check the water available at the beginning of the season and estimate the cultivated area before starting water delivery to determine how much area can be served.

For many medium-scale and large-scale irrigation projects, the Operations Plan depends on the available water resources. The Water Master has to estimate the cultivated area. The weekly cultivated area and stage of cultivation should be observed by the Zoneman. The water requirement depends on the stage of growth, which is estimated from the design criteria. So, the demand at the Head Regulator is estimated by using the design coefficient of discharge, design seepage losses and design farm efficiency. The water requirement for each week would also be adjusted according to the effective rainfall. Some irrigation projects may estimate the water demand from the previous year's information. For each irrigation period, after the needed data in the field is collected, the Water Master will estimate the water demand and the appropriate gate opening at each Head Regulator. The required gate openings would be conveyed to the Zoneman, who is the individual that adjusts the Head Regulator. After the gates are adjusted, the Zoneman has to periodically observe whether or not the water supply at each regulator is sufficient. If the farmers complain that the water supply is too much, or too little, the Zoneman has to adjust the gate again. If every regulator has to be adjusted, then there will be considerable fluctuations in water levels in the main canal. The discharge rate for every Head Regulator and outlet will be disturbed. This type of Operations Plan may result in a lack of water in some areas and surplus water supplies is other areas of the irrigation project.

For some particular irrigation projects, the operation is undertaken with the advice of a consultant. The Operation Plan will depend on each individual consultant, which are all different in detail, but the same in principle -- to use water effectively. New technology will be introduced to the project. Actual field data in the system will have to be collected. The procedures after collecting field data and for analyzing the data would be discussed. Recently, computer simulation is also being introduced to improve the irrigation Operations Plan.

B. USE OF DISCHARGE RATINGS AND CHANNEL LOSS MEASUREMENTS

Description of Methodology

In comparison with the present scheduling procedures, the development of discharge ratings for flow control structures and the measurement of irrigation channel losses discussed in this handbook allows a more precise Operations Plan to be developed.

For determining the water demand at each outlet, the crop water requirement can be estimated using standard RID procedures. If seepage loss and deep percolation loss measurements have been collected in the tertiary system, then more knowledge will be available for estimating the correct water demand at the outlet. This allows a more precise determination of the flow required at the outlet.

In the main canal, laterals and sub-laterals, the seepage loss rate would also be determined in order to estimate the seepage losses and conveyance efficiency more precisely. Usually, the seepage loss rate would be determined for 2 or preferably 3 different discharge rates if the Inflow-Outflow method is used. The Ponding method provides information on how the seepage loss rate varies with flow depth.

The actual coefficient of discharge at each outlet, flow control structure, and all head regulators should be determined in order to have accurate discharge ratings. The more precisely that the discharge through the outlet structure can be measured, the better the control that can be provided at the head regulator.

After the rating curves have been developed, then during actual operations the discharge at the outlet structures and head regulators can be checked as to whether or not the water supply is adequate at the outlets. If not, then the discharge rates at the upstream flow control structures and head regulators can be checked to evaluate the problem. This provides more information for improving the water deliveries the next day or the next week.

The seepage loss rate does change according to the depth of water in the irrigation channels. The Inflow-Outflow or Ponding tests should be used for measuring the seepage loss rate for different discharge and water depths in various reaches of the main canal, laterals, and sub-laterals. For the next irrigation period, the discharge and depth of water in the irrigation channels should be observed in order to use the appropriate seepage loss rate. The determination of the variation in seepage loss rate with water depth for all of the reaches in the irrigation network is very useful. Every irrigation project needs these relationships in order to predict the amount of water required at each flow control structure so that water can be distributed equitably and adequately to the farmers served by each outlet.

These field measurements of discharge ratings and seepage loss rates need some time to complete. By programming these measurements into the regular work program during the operation of the system, the amount of data, and consequently knowledge, will steadily increase. After 3-4 seasons, very good data should be available for preparing a canal Operations Plan. However, the first season that the field data is collected, such data is used immediately to improve water deliveries. Thus, the capability for equitably distributing water to outlets will improve each season, until finally a good Operations Plan is being implemented after 3-4 seasons.

These field measurements need to be as accurate as possible so that the calculated discharge requirements at various flow control structures will come close to the actual values required to meet the water demand at each outlet. As the needed data are collected, the Operations Plan can be calculated manually. An example will be used to illustrate the development of an Operation Plan using this field data.

Example of Manually Calculating an Operation Plan

The discharge ratings and seepage loss rate measurements were collected in the field during June, 1986, as part of the "Operation of Irrigation Projects" training course. The Inflow-Outflow method was applied to the Left Main Canal (LMC) of the Lam Nam Oon Irrigation Project in Northeast Thailand (Figure 0-08-1). The discharge at the head regulator was 5.993 m^3/s. The seepage loss for the total length of the LMC from Km 0+000 to Km 26+995 was 1.848 m^3/s. The seepage losses for lateral 1-1L from Km 5+959 to Km 8+660 and sub-laterals L-1L-2L, L-1L-2L-1R, L-3L-1L, and L-3L-1L-2L were also determined. For the rest of the laterals and sub-laterals, the seepage loss rate was estimated according to the known water depths and the average measured values of seepage loss rate in the other laterals and sub-laterals. The seepage loss for the entire LMC sub-system (main canal, laterals, and sub-laterals) was 3.119 m^3/s, which gives a conveyance efficiency of 48 percent.

This example is illustrated in Tables 0-08-1, 0-08-2, and 0-08-3, which shows the procedure for determining the water demand at the LMC head regulator for June 1986. The procedure starts from the lowest end of the sub-system. After the crop water requirement, and deep percolation and seepage losses, have been estimated for each tertiary system, the required discharge of each outlet will be known. The resulting discharge at each outlet in the tertiary system, plus estimated seepage losses in the sub-lateral, will add up to the water requirement for each sub-lateral. This procedure will be repeated for every sub-lateral. Then, the discharge of every head regulator (CHO) in the sub-lateral will sum up to be the discharge at the head

0-08-06

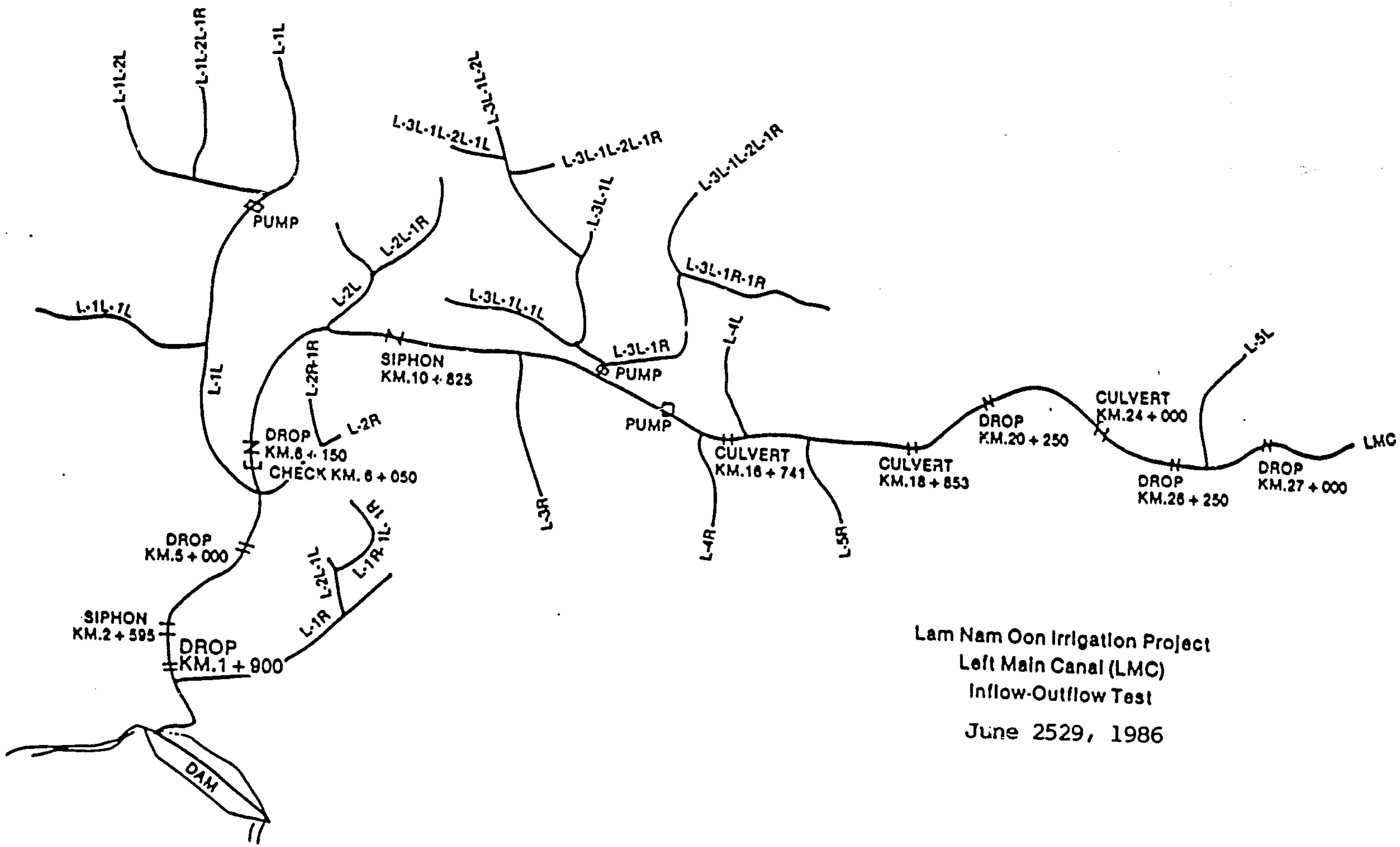

Figure 0-08-1. Layout of Left Main Canal, Laterals and Sub-Laterals at the Lam Nam Oon Irrigation Project Used in Preparing an Plan for June 2529 (1986)

Table 0-08-1. Measured and Estimated Seepage Losses for the Left Main Canal Sub-System at the Lam Nam Oon Irrigation Project for June, 1986.

Canal	Lateral	Sub-Lateral	Reach Begin	Reach End	Seepage Loss (m3/s)
LMC			0+000	2+700	0.649
LMC			2+700	6+000	0.293
LMC			6+000	10+798	0.000
LMC			10+798	16+000	0.297
LMC			16+000	16+700	0.099
LMC			16+700	18+800	0.049
LMC			18+800	20+240	0.101
LMC			20+240	23+981	0.138
LMC			23+981	26+245	0.096
LMC			26+245	26+995	0.098
LMC			26+995	28+040	0.028
	L-1L		0+000	5+900	0.000
	L-1L		5+900	9+310	0.091
		L-1L-1L	0+000	3+400	0.088
		L-1L-2L	0+000	6+500	0.101
		L-1L-2L-1R	0+000	1+500	0.006
	L-1R		0+000	6+624	0.179
		L-1R-1L	0+000	1+170	0.023
		L-1R-1L-1R	0+000	1+569	0.026
	L-2L		0+000	3+500	0.052
		L-2L-1R	0+000	2+880	0.037
	L-2R		0+000	3+010	0.071
		L-2L-1L	0+000	1+250	0.027
	L-3R		0+000	2+500	0.042
	L-3L		0+000	0+835	0.000
		L-3L-1L	0+000	2+020	0.060
		L-3L-1L-1L	0+000	1+900	0.025
		L-3L-1L-2L	0+000	6+320	0.131
		L-3L-1L-2L-1L	0+000	2+000	0.036
		L-3L-1L-2L-1R	0+000	2+155	0.030
		L-3L-1R	0+000	3+173	0.091
		L-3L-1R-1L	0+000	1+920	0.036
		L-3L-1R-1R	0+000	3+500	0.065
	L-4R		0+000	1+500	0.018
	L-4L		0+000	0+530	0.007
	L-5R		0+000	1+650	0.034
	L-5L		0+000	1+600	0.031

Table 0-08-2. Water Demands for Sub-Laterals and Laterals in the Left Main Canal Sub-System at the Lam Nam Oon Irrigation Project for June, 1986.

Lateral	Sub-Lateral	Reach Begin	Reach End	Reach Outflows (cms) Turnout Discharge	Seepage Loss	Total	Water Demand (cms) Reach End	Reach Beginning
L-5L		0+000	1+600	0.233	0.031	0.264	0.000	0.264
L-5R		0+000	1+650	0.122	0.034	0.156	0.000	0.156
L-4L		0+000	0+530	0.023	0.007	0.030	0.000	0.030
L-4R		0+000	1+500	0.079	0.018	0.097	0.000	0.097
L-3R		0+000	2+500	0.083	0.042	0.135	0.000	0.135
	L-3L-1R	0+000	3+173	-	0.091	-	-	-
	L-3L-1R-1L	0+000	1+920	-	0.036	-	-	-
	L-3L-1R-1R	0+000	3+500	-	0.065	-	-	-
	L-3L-1L-2L-1R	0+000	2+155	-	0.030	-	-	-
	L-3L-1L-2L-1L	0+000	1+920	0.313	0.036	0.349	0.000	0.349
	L-3L-1L-2L	0+000	6+320	0.353	0.131	0.484	0.155	0.639
	L-3L-1L-1L	0+000	1+900	0.028	0.025	0.056	0.000	0.056
	L-3L-1L	0+000	2+020	0.695	0.060	0.755	0.129	0.884
	L-3L	0+000	0+835	0.000	0.000	0.884	-	0.884
	L-2L-1R	0+000	2+880	-	0.037	-	-	-
	L-2L	0+000	3+500	0.061	0.052	0.113	0.000	0.113
	L-2L-1L	0+000	1+250	-	0.027	-	-	-
	L-2R	0+000	3+010	-	0.071	-	-	-
	L-1R-1L-1R	0+000	1+569	-	0.026	-	-	-
	L-1R-1L	0+000	1+170	-	0.023	-	-	-
L-1R		0+000	6+624	0.628	0.179	0.807	-	0.807
	L-1L-2L-1R	0+000	1+500	0.023	0.006	0.029	0.041	0.070
	L-1L-2L	0+000	6+500	0.078	0.101	0.179	0.798	0.977
	L-1L-1L	0+000	3+400	0.192	0.088	0.280	-	0.280

Table 0-08-3. Water Demands at Various Locations Along the Left Main Canal at the Lam Nam Oon Project for June, 1986.

Reach Begin	Reach End	Reach Outflows (cms) Turnout Discharge	Seepage Loss	Total	Water Demand (cms) Reach End	Reach Beginning
26+995	28+000	0.089	0.028	0.117	-	-
26+245	26+995	0.282	0.098	0.380	0.117	0.497
23+981	26+245	0.042	0.096	0.138	0.497	0.635
20+240	23+981	0.007	0.138	0.145	0.635	0.780
18+800	20+240	0.007	0.101	0.128	0.780	0.888
16+700	18+800	0.126	0.049	0.235	0.888	1.123
16+000	16+700	0.100	0.099	0.199	1.123	1.322
10+798	16+000	1.020	0.297	1.317	1.322	2.639
6+000	10+798	0.108	0.000	0.108	2.639	2.747
2+700	6+000	1.790	0.293	1.882	2.747	4.537
0+000	2+700	0.807	0.649	1.456	4.537	5.993

Total Seepage loss 3.119 cms Water demand 5.993 cms

Conveyance efficiency = 48 percent

regulator (CHO) in the lateral, and so on, until finally the water demand at the head regulator for the main canal is known. Remember, the seepage loss rate in the laterals and sub-laterals must also be considered. This procedure is not complicated, but it needs to be done carefully and patiently.

C. MICRO-COMPUTER STEADY-STATE SIMULATION

For initial use of a micro-computer in operating the main subsystems, it is recommended that a steady-state simulation be used, unless the irrigation project has a highly experienced irrigation engineer with substantial computer capability. A steady-state simulation is quite simple because it fits very well with the logic presently used in operating an irrigation project. However, a steady-state simulation does require discharge ratings for flow control structures and channel loss measurements.

The manual "Micro-Computer Steady-State Modeling of Irrigation Delivery Systems" should be consulted. This manual describes in detail how to develop the model for any particular irrigation project. The various models, which are really sub-models or programs, are set up as modular units, meaning that most programs have a short controlling section from which the calculations and activities are directed to subroutines. There are four main programs: (1) The Weather Simulation and Evapotranspiration (WSEM) Model; (2) The Irrigation Requirements Model (IRM); (3) The System Operations Model (SOM); and (4) The Records Model (RM). These programs can each operate individually. Any one of the programs can be used individually, as long as the required data files are available, or the programs can be used in combination.

The four principal models are similar in the way they are executed and in the way they handle data. Each model is accessed from the computer keyboard and operated interactively. A menu is displayed listing the functions that are available. The user selects the desired function from the menu and during execution supplies certain data and references to files where data are stored. After the function is completed, program control returns to the program menu and another function can be executed, or the program execution terminated.

Input data for each model is normally stored on the disk and referred to as an input data file. Data files can be named, created and revised using functions listed in the program menu. This data storage method minimizes the data entry requirements and simplifies data editing procedures.

All outputs from the models are also stored in data files on the disk. The programs do not print results directly to the line printer during the program execution, but instead print the results to an output data file. If desired, the output can then either be viewed on the monitor or printed on the line printer by selecting the appropriate option on the menu.

Some of the model functions require more than one input data file. For example, the function calculating planned canal flows uses a data file containing canal dimensions and a data file containing planned irrigation requirements. The former is created by entering the data from the keyboard and storing it on the disk. The latter is an output data file created by the Irrigation Requirements Model. In this case, storing the output from the Irrigation Requirements Model minimizes the data input effort or the System Operations Model.

The Record Model (RM) is a separate program that allows the user to record measurements of canal flow and rainfall and compare planned and actual flows. This is useful in preparing the weekly, monthly, and seasonal operations performance reports.

D. MICRO-COMPUTER HYDRODYNAMIC SIMULATION

Under the Water Management Synthesis II Project, with funding from the U.S. Agency for International Development, the Department of Agricultural and Irrigation Engineering at Utah State University (USU) has been developing computer software to assist in the plannings design, and operations of irrigation projects. Four submodels have been under development: (1) a watershed catchment model for predicting precipitation-runoff processes; (2) a river and reservoir model for addressing water management issues associated with the collection and storage of water resources; (3) a main system model to evaluate water management in conveyance networks; and (4) a command area (tertiary system) model for analyzing aggregate crop water requirements, irrigation application efficiencies, on-farm practices, and crop yields. Two of the models, the Unit Command Area Model and the USU Main System Hydraulic Model are completed. They are being implemented on an experimental or trial basis on two irrigation projects in the Northeast of Thailand.

The "Users Manual for the Pascal Version of the USU Main System Hydraulic Model" should be consulted. This computer model performs hydrodynamic simulations of water flow in irrigation channels. Its applications are in the areas of canal operation, analysis, design, and operator training. The hydrodynamic simulation capability means that unsteady flow conditions can be simulated, such as the filling and emptying of irrigation channels, or the changes in water levels with time after changing a gate setting. Thus, the real system is more closely simulated than using the steady state simulation described in the previous section.

The Utah State University (USU) Main System Hydraulic Model is a mathematical model capable of simulating actual hydraulic conditions in a canal system and of optimizing the operation of a system by calculating control structure settings which best maintain constant flow levels. This optimization feature is implemented during a hydraulic simulation by using the "Gate Scheduling" mode. Through gate scheduling, the model can be used to determine appropriate control structure settings which will optimize the daily operation of a canal system. When frequently changing demands are imposed on a canal system, the actual supply can better meet these demands by implementing gate scheduling, and the water levels in the canals will remain as stable as possible.

The USU Hydraulic Model can also be very useful and effective as a training tool for canal operators. Many different operational schemes can be quickly and safely evaluated using the model and the canal operator can in this way become more familiar with the response of the real canal system to varying flow conditions and water distribution schedules. After spending some time with the model performing simulations, the canal operator should be better able to operate the real canal system effectively because he will be more familiar with the hydraulic behavior of the system. The canal operators' knowledge of the real canal system will extend beyond the normal operational modes of the canal system and thereby sensitize the operator to extra-ordinary flow conditions.

As a design tool, the model can be used to evaluate proposed canal system designs under real-time operating conditions. The final design of a canal system can not only be based on static flow criteria, but also on the performance of the system in conveying and distributing water. Such a design is more likely to perform satisfactorily under actual operating conditions. Locations and types of control structures and turnouts can be evaluated, the need for canal lining can be assessed, and the ability of the system to deliver water according to proposed allocation schedules can be analyzed. Thus, this model is a powerful tool in assessing alternative improvements when developing an "Irrigation System Improvements Plan."

Section IX.

MONITORING, EVALUATION AND FEEDBACK PROGRAM

A. MONITORING

Monitoring is a routine function done at all irrigation projects. However, there are significant differences between irrigation projects as to the amount of monitoring data collected. In order to continually improve the operation of an irrigation system, fairly extensive monitoring data is required, particularly for a few years. Then, as more knowledge is gained about the hydraulic functioning of the irrigation system, then the amount of monitoring data can be reduced.

Tertiary Subsystems

The collection of monitoring data for the tertiary subsystems is entirely the responsibility of Zonemen. They are commonly graduates from a technical school, which requires 9 years of public education, rather than 12, plus 2-4 years of schooling at a public or private technical school.

Each day during the irrigation season, staff gauge readings and the corresponding gate opening must be measured at each main system outlet structure (which serves a single Water Users Group). The number of staff gauge readings collected daily at each outlet should be dependent upon the degree of discharge variation and water surface fluctuation in the vicinity of each outlet. Based upon the staff gauge readings and gate opening, the discharge rate can be determined based upon the field discharge rating. The discharge rate should be determined and recorded before the Zoneman leaves the site.

The other essential data that must be collected by the Zoneman for each tertiary system is the cultivated area. Usually, this data would be collected weekly, particularly during the early portion of the irrigation season; however, once the crops have been planted it is an easy job to report the cropped area weekly for each crop.

The Zoneman can provide valuable assistance to the Water Users Group by periodically collecting additional data within their tertiary system, such as channel losses, time lags for the conveyance of water from one portion of the system to another, and deep percolation losses on croplands. The Zoneman should look for opportunities during his schedule to conduct such measurements. This program should begin with perhaps only one or two tertiary systems so that a better understanding of a tertiary system can be gained. Sufficient measurements of channel losses and deep percolations losses should be made over a few irrigation seasons so that water budgets can eventually be prepared for the tertiary system. The Water Master should work with the Zoneman periodically to check the quality of the data. Usually, poor data is worse that no data at all, because it is misleading and results in improper conclusions.

For some irrigation projects, there will be some unique problems that require special attention. For example, ground water levels may be high in some portions of the project area, which affects the yield of many field crops. Thus, observation wells might be installed so that measurements can be made periodically, perhaps once a month, of the depth of groundwater below the ground surface. The Zoneman would be responsible for making these measurements. If drains exist in the tertiary system, then it may be important to measure the drainage flows.

Also, in some areas, it may be necessary to periodically collect soil samples that are forwarded to a central laboratory for analysis because of soil salinity, soil acidity, or some chemical toxicity for certain crops. The Zoneman could advise the Water Users Group Leader to request that the Kaset Tambon collect these soil samples and forward to the appropriate laboratory, then depending upon the results, request assistance from the appropriate Subject Matter Specialist in the Department of Agricultural Extension. Many times, the solutions to these problems includes improved irrigation water management practices.

<u>Main Subsystems</u>

The main subsystems are: (a) the Left Main Canal and network of laterals, sub-laterals, etc.; and (b) the Right Main Canal and network of laterals, sub-laterals, etc. Most of the monitoring data is collected by the Zonemen who are supervised by a Water Master. Most Water Masters have graduated from the RID School of Irrigation, which requires 3 years of academic training followed by 6 months of practical training. Applicants for this school must have completed 12 years of public education and graduated with an emphasis in science.

The most important monitoring data in a main subsystem are the daily discharge rates, which are read at many locations throughout the day. Again, after reading the staff gauges, gate openings, etc., then the flow rate should be determined by calculation or from a rating table before leaving the site.

Although discharge ratings will already be available for each flow control structure, there should be periodic checks, using either a current meter or flow measuring flume, depending upon the particular flow control structures. A schedule of periodic checks should be developed for each irrigation season, with the discharge rating being checked on at least 25 percent of the flow control structures each season.

Experience should be gained by the Zonemen and Water Master on the Time Lags for different discharge rates to move from one location in the main subsystem to another location downstream. This is done by monitoring changes in water levels at various locations in the irrigation channel network when Head Regulators are opened, or when gate openings are changed. This monitoring should be continued for more hours than deemed necessary to be sure that the system has reached a steady-state condition.

Although channel losses will have been measured in order to develop an Operations Plan, there is always a need for additional data to further improve and refine the Operations Plan. Also, channel seepage loss rates can change with time due to vegetative growth, aquatic growth, and improvement or deterioration of channel boundary conditions (e.g., surface sealing of earthen channels or the quality of concrete lining). A schedule for periodic measurements of channel losses should be made for each irrigation season. This can easily be done in conjunction with monitoring of time lags; once steady-state flow conditions have occurred between two structures with known discharge ratings, then the discharge measurements can be read and wetted perimeters measured between the two structures. So, with proper planning, this task is relatively easy to accomplish.

Changes in the boundary conditions of irrigation channels, as well as sedimentation, vegetative growth and aquatic growth, affects the hydraulics of flow in the channels, such as the flow depths. Thus, periodic measurements of the hydraulic roughness of various reaches should be made. In most irrigation channel networks, there are some reaches where normal flow depth occurs; consequently, these are ideal reaches for evaluating the change in the hydraulic roughness (Manning's n). However, there are many reaches, particularly in the main canals, where the flow depths are always greater than normal flow depth because of backwater effects from downstream flow control and regulating structures. In such cases, the principles of gradually varied flow must be used in order to calculate the hydraulic roughness based upon field measurements of discharge rate and variation of flow depth along a reach.

Some irrigation projects maintain a weather station. As a minimum, daily measurements of temperature and precipitation are made. Often, there is also a standard evaporation pan and daily measurements of evaporation are collected.

Watershed, Storage and River

The source of water for most of the irrigation in Thailand is surface runoff from watersheds. Most small-scale, medium-scale and large-scale irrigation projects have storage facilities in order to capture the surface runoff, which is used during the wet (monsoon) season and the dry season. In many cases, the available storage limits the amount of land that can be cultivated in the dry season as compared with the wet season.

For every nation, water is a valuable natural resource that must be properly managed in order to maintain it's utility for future uses and changing needs. Water diverted for irrigation use should also be properly managed so as to minimize the water quality degradation in the return flows back to the river, or those subsurface return flows percolating into the underlying ground water.

Daily records are kept on the reservoir water surface elevation, so that the volume of available storage is known. If a precipitation station and an evaporation pan are located nearby, then readings are also taken daily.

The reservoir serves as an integrator of both surface and subsurface runoff from the watershed. However, it may be desirable to maintain a discharge rating station on each of the major sources of surface inflow to the river. If reservoir sedimentation is a concern, then periodic water samples should be collected at the discharge rating station(s). Also, consideration should be given to having both chemical analyses and biological analyses undertaken on these water samples in the laboratory.

Serious consideration should be given to maintaining a discharge rating station in the river downstream below the irrigated lands. This outflow station serves two major useful purposes: (a) it facilitates the computation of monthly, seasonal and annual water budgets for the irrigation project; and (b) periodic water samples can be collected and analyzed in the laboratory for water quality, which combined with discharge measurements, allows the computation of chemical or biological loading into the river.

B. EVALUATION

Evaluation is primarily the analysis of monitoring data. Some data analysis should be done everyday. Then, it becomes very important to evaluate the performance of the irrigation project for each irrigation season. The multitude of evaluations that must occur during each season has been subdivided into weekly and monthly tasks for two purposes: (a) to provide feedback among operations personnel and farmers; and (b) to provide an information base and evaluation on the performance of the system.

Daily

Each Zoneman takes many discharge readings each day. The most basic of evaluations is taking the discharge readings and converting then into a daily volume of water passing each control point in the irrigation channel network. Thus, daily discharge rates and water volume is recorded for each outlet serving a tertiary system, each control structure regulating the flow of water, and any other flow control structures designated for use in operating the system. The Zoneman should preferably deliver this information to the Water Master at the end of the day, but no later than the following morning, unless there are extenuating circumstances.

Weekly

The Water Master should compile the daily discharge records received from the Zonemen under his supervision and prepare a weekly water balance for his area of responsibility, which might be a medium-scale irrigation project, or a portion of a large-scale irrigation project. For a large-scale project, the Water Masters should forward their Weekly Water Balance Report to the Head of Operations, who serves under the Project Engineer, and he would prepare the report for the irrigation project. This water balance would show the volume of water delivered in cubic meters to each outlet, the amount of cultivated land in rai (1,600 square meters) under each outlet, and the average depth of water delivered per cultivated rai expressed in millimeters. Then, the water balance would show the volume of water diverted into each main canal, the volumes of seepage losses for each reach between flow control structures, and the volumes of water diverted into each lateral and outlets along the main canal. This same process of water balance computations would be shown for each lateral subsystem showing the distribution of seepage losses, outlet flows, and sub-lateral diversions. All of this information can be summarized into the total diversion into each main canal, total seepage losses, and outlet diversions, plus the calculated crop evapotranspiration for the week in order to estimate the total losses for the tertiary subsystems.

Each week, the Zoneman should submit to the Water Master any special measurements made during the week. For example, if the Zoneman made a current meter measurement at a water control structure, he should submit the measurement forms, including staff gauge readings, so this information can be analyzed by the Water Master and also placed in the file for that particular structure. Likewise, any Time Lag or Channel Loss data collected during the week should be submitted on appropriate forms for analysis by the Water Master and then filing.

The same procedure would be used in reporting discharge rates and water level measurements along a channel reach in order to analyze Hydraulic Roughness.

The Zoneman must also submit a weekly report to the Water Master on the cultivated area for each crop under each outlet structure. Also, the Zoneman should report any measurements of channel losses conducted in any of the tertiary systems, along with a good description of the reaches measured, and the computations. Also, any measurements of discharge rate and time of application on any bunded croplands should be reported, along with the computed depth of application in millimeters.

Monthly

The most important monthly evaluation is a projection of available irrigation water supply. First of all, this requires a water balance analysis of the storage reservoir. Also, a summary of the weekly water balance reports should be compiled into a monthly water budget analysis for the irrigation system. Also, the daily records of precipitation and evaporation should be reported and compared with the same month for previous years. Also, the crop reports should be compiled so that projections can be made of expected crop water requirements for the remainder of the season, along with a statistical projection of expected rainfall. For a medium-scale irrigation project, the Water Master will have the responsibility for preparing this report, or it could be the responsibility of the Head, Operation and Maintenance Section, Provincial Irrigation Project Office. For a large-scale irrigation project, the Head, Water Management Section would have the responsibility for preparing this monthly report.

Seasonal

An evaluation of the hydraulic performance of the irrigation project for the entire season is extremely useful for providing insights as to how the system performance might be improved in the future. These seasonal evaluations become the primary information base for: (a) Revising the Operations Plan; and (b) Revising the Monitoring, Evaluation and Feedback (ME & F) Program.

The seasonal evaluation of the performance of the irrigation project is also an important historical record and forms the basis for comparing performance in future years. In other words, it becomes a record of how the performance of the project is improving from one year to the next. Likewise, the seasonal evaluation could disclose that the irrigation system is deteriorating.

Most of the information required for either the "Wet Season Operations Performance" or the "Dry Season Operations Performance" evaluation can be drawn from the monthly reports on watershed balance, irrigation water budgets, and croplands. Thus, seasonal water budgets would be reported, along with cropping data, climatic data, ground water levels, water quality data and any special investigations or studies. For example, the status of investigations on the hydraulic performance of any tertiary systems should be reported. Any special studies on the variation of hydraulic roughness during the season for particular channel reaches should also be reported. Then, the conclusions about the overall performance of the project should be stated, along with projections of pending improvements or problems. Finally, recommendations should be made for further improvement in the project performance, if deemed necessary.

The same individual responsible for preparing the monthly report on reservoir water balance, irrigation system water budgets, and projected irrigation water supply should prepare this seasonal evaluation report. For a large-scale irrigation project, this would likely be the Head, Water Management Section. For a medium-scale irrigation project, this would either be the Water Master responsible for the operation of the project, or the Head, Operation and Maintenance Section, Provincial Irrigation Project Office.

<u>Annual</u>

An "Annual Operations Report" should be prepared for each irrigation project. This report should be prepared by the Project Engineer. The intent is to write a brief assessment of the performance of the irrigation project. The "Wet Season Operations Performance" and the "Dry Season Operations Performance" reports would provide the data base for preparing the "Annual Operations Report". This brief report would summarize the seasonal evaluations. The Project Engineer should carefully review the Conclusions and Recommendations in the seasonal reports and include the most important issues in this annual report. In addition, the Project Engineer should include his personal assessment of the operational performance of the project, including any suggestions for changes.

C. FEEDBACK

Feedback is the communication of the evaluation of monitored data. In this case, feedback is the communication of operational performance of the irrigation project. Successful performance is highly dependent upon effective communication between all individuals that play a role in the operation of the irrigation system.

Water Users Group Leaders

The Water Users Group (WUG) Leader communicates to the Zoneman the water needs of the farmers in the tertiary system, whether recent water supplies have been adequate, and any particular operational difficulties. In turn, the Zoneman informs the Water Users Group Leader about the amount of recent deliveries, any anticipated difficulties in meeting the immediate water requirements for the tertiary system, and discusses every month the water supply projection for the remainder of the irrigation season.

Zonemen

The Zonemen play the key role in the operation of the irrigation project. Their ability to communicate with farmers is highly important. Also, the majority of the field data is collected by them. They provide to the Water Master every day the discharge rates and water volumes for each of the outlet structures and flow control structures under their jurisdiction. Also, they provide a weekly report to the Water Master that contains any special measurements they have made. The Zoneman should also verbally communicate to the Water Master any unusual problems for any of the tertiary subsystems, or in the portion of the irrigation channel network that is his responsibility.

Water Masters

One of the most important functions of the Water Master is to provide quality control on the field data being collected by the Zonemen under his supervision. This can only be done if the Water Master periodically participates with each Zoneman in the collection of field data and if he carefully analyzes the field data provided to him by the Zonemen. This requires that the field data be evaluated as quickly as possible so that the Zoneman can be provided rapid feedback regarding the quality of the field data.

The Water Master also has a major responsibility for data analysis. He needs to provide a weekly report on water budgets for the area of the project under his jurisdiction to the Head of Operations (Head, Water Management Section for large-scale irrigation projects or Head, Operation and Maintenance Section, Provincial Irrigation Project Office for medium-scale irrigation projects).

Head of Operations

A proper title for this position is difficult to describe because it varies between large-scale irrigation projects and medium-scale irrigation projects. For a medium-scale project, a Water Master may be the highest ranking individual at the project on a daily basis. He answers to the Project Engineer at the Provincial Irrigation Office who assigns operations responsibilities to the Head of the Operation and Maintenance Section in this office. In contrast, for a large-scale project, there will be a number of Water Masters, each one having jurisdiction over a portion of the system, and they are supervised by the Head of the Water Management Section. This title of "Head of Operations" is used to describe the individual between the Project Engineer and the Water Masters who is responsible for operations.

This individual receives a weekly report from each Water Master, which should be reviewed and feedback provided to the Water Master as to the strengths and weaknesses in their report. The Head of Operations should also spend some time with each Water Master and some of the Zonemen when field data is being collected in order to advise on proper procedures to ensure good quality data. Also, the Head of Operations must compile the weekly Water Master reports into a "Weekly Operations Performance" report.

Another important assignment for the Head of Operations is the preparation of the "Monthly Operations Performance and Water Supply Projection" report. This compilation of data, reservoir water balance, and irrigation system water budgets is submitted to the Project Engineer with information copies provided to the Water Masters.

The Head of Operations also has the responsibility for preparing the "Dry Season Operations Performance" report and the "Wet Season Operations Performance" report. Most of the information in these reports are obtained from the monthly reports. These reports are submitted to the Project Engineer with information copies to the Water Masters.

Project Engineer

The most important role played by the Project Engineer is in establishing the "attitude" towards continual improvement in the performance of the irrigation project. He establishes the expectations for irrigation system performance and the criteria for good job performance. The weekly, monthly and seasonal operations reports should be quickly reviewed by the Project Engineer and feedback provided to project staff with emphasis on what work is being done well and what work needs to be improved.

The Project Engineer prepares the "Annual Operations Report" for the project. This report summarizes the seasonal operations reports, plus provides Conclusions and Recommendations regarding the overall operations performance during the past two seasons and any new activities that will be undertaken to further improve the performance of the irrigation project. Copies of this report are submitted to the Regional Director with an information copy to the Head of Operations.

Section X.

IRRIGATION SYSTEM IMPROVEMENTS

One of the important advantages in implementing the Operations Phase of the O&M Learning Process is that considerable field data is collected. Also, this data is analyzed and evaluated so that considerable knowledge is obtained about the internal functioning of the irrigation system. Certainly, after a few years, the main subsystems (canal, laterals, sub-laterals, etc.) will be well understood. If the Project Engineer and the Head of Operations have taken an active interest in this process, then there will be at least a fair knowledge about what is occurring within the tertiary subsystems. The Monitoring, Evaluation and Feedback Program will provide significant sensitivity about the irrigation project.

After two, three or four years, the series of "Wet Season Operation Performance" reports and "Dry Season Operations Performance" reports will provide a strong data base regarding the hydraulic performance of the system. The channel losses throughout the main subsystems will be known, as well as the deliveries to individual outlets. Thus, at this point, there would be considerable insight as to how the main subsystems might be improved.

For example, there would be a good understanding about which reaches in the main canal, laterals, and sub-laterals have the greatest seepage loss rates; an obvious set of improvements would be to reduce seepage losses in these reaches through some type of lining program. The costs of various types of linings could be calculated. Or, it might be desirable to experiment with different types of lining materials on some of the reaches. Also, some concrete-lined channels have high seepage loss rates through cracks, joints and holes; so different methods of reducing water losses for this situation might be tried on an experimental basis on some reaches.

One of the significant characteristics about irrigated agriculture is that each irrigation project is "site specific." In other words, each irrigation project is different and unique. Thus, there can be no universal prescribed solutions that will be highly beneficial for every irrigation project. Therefore, the solutions for existing problems have to also be unique to each particular project. Thus, experimenting with various solutions and "evaluating" their performance can be expected to result in much more cost-effective irrigation system improvements.

The Project Engineer will play a significant role in determining the emphasis upon improvements in the irrigation project, whether most of the effort is placed on the main subsystems, or the tertiary subsystems, or both. The most significant "feeling" or "sensitivity" about the functioning of on irrigation project comes from an understanding of the tertiary subsystems, and most importantly, the actual application of water on the cropland. Only by gaining such knowledge can an individual really understand an irrigation project. Thus, project personnel are strongly encouraged to devote as much time as possible in evaluating the tertiary subsystems.

As more and more knowledge is gained about an irrigation project, then it can be expected that more options for improvement will come to mind. Also, instead of thinking primarily in terms of physical improvements, which are often refered to as "hardware" solutions, it is more likely that a number of "software" solutions will become more obvious. One obvious solution is the use of computer software to simulate the hydraulic operation of the main subsystems in particular, but also the tertiary subsystems are possible.

The tertiary subsystems are more likely to need a combination of both hardware and software solutions, as compared with the main subsystems. A natural sequence of events in the "evolution" of an irrigation project over a time span of roughly 50 years is to move from a focus on hardware solutions, to a combination of hardware and software solutions, and finally, to a predominant focus on software solutions.

During the early years of project development, it can be expected that most of the proposed irrigation system improvements will be physical solutions. But, in later years, particularly as more knowledge is gained about the total system, then software solutions in combination with physical improvements will become more obvious.

The Project Engineer has the responsibility for deciding what irrigation system improvements should be investigated. Also, he is responsible for the development of the "Irrigation System Improvements Plan." A suggested outline for this report would be:

1. Physical Description of Irrigation Project
2. Results of Reservoir Water Balance
3. Irrigation Project Water Budgets
    a. Main Subsystems
        (i) Wet Season
        (ii) Dry Season
    b. Tertiary Subsystems
        (i) Wet Season
        (ii) Dry Season

4. Irrigation Project Performance

   a. Present Performance
   b. Historical Trends in Performance
   c. Potential Performance

5. Alternative Irrigation System Improvements

   a. Proposed Alternatives
   b. Impact of Alternatives Upon Project Performance
   c. Costs of Alternative Improvements
      (i) Capital Costs
      (ii) Operation and Maintenance Costs

6. Cost-Effectiveness of Alternative Improvements

7. Priority of Alternative Improvements

8. Recommendations on Implementation of Priority Improvements

Copies of this report would be forwarded to the Regional Office for approval and subsequently the Operation and Maintenance Division at RID headquarters, the Division of Program Coordination and Budget, the Director-General of RID, and then sent to the Bureau of the Budget for funding. This approval process will be facilitated by the documentation involved in preparing the "Irrigation System Improvements Plan."

WATER MANAGEMENT SYNTHESIS PROJECT REPORTS

WMS 1 Irrigation Projects Document Review

Executive Summary
Appendix A: The Indian Subcontinent
Appendix B: East Asia
Appendix C: Near East and Africa
Appendix D: Central and South America

WMS 2 Nepal/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 3 Bangladesh/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 4 Pakistan/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 5 Thailand/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 6 India/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 7 General Asian Overview

WMS 8 Command Area Development Authorities for Improved Water Management

WMS 9 Senegal/USAID: Project Review for Bakel Small Irrigated Perimeters Project No. 685-0208

WMS 10 Sri Lanka/USAID: Evaluation Review of the Water Management Project No. 383-0057

WMS 11 Sri Lanka/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 12 Ecuador/USAID: Irrigation Sector Review

WMS 13 Maintenance Plan for the Lam Nam Oon Irrigation System in Northeast Thailand

WMS 14 Peru/USAID: Irrigation Development Options and Investment Strategies for the 1980's

WMS 15 Diagnostic Analysis of Five Deep Tubewell Irrigation Systems in Joydebpur, Bangladesh

WMS 16 System H of the Mahaweli Development Project, Sri Lanka: 1980 Diagnostic Analysis

WMS 17 Diagnostic Analysis of Farm Irrigation Systems on the Gambhiri Irrigation Project, Rajasthan, India: Volumes I-V

WMS 18 Diagnostic Analysis of Farm Irrigation in the Mahi-Kadana Irrigation Project, Gujarat, India

WMS 19 The Rajangana Irrigation Scheme, Sri Lanka: 1982 Diagnostic Analysis

WMS 20 System H of the Mahaweli Development Project, Sri Lanka: 1983 Diagnostic Analysis

WMS 21 Haiti/USAID: Evaluation of the Irrigation Component of the Integrated Agricultural Development Project No. 521-0078

WMS 22 Synthesis of Lessons Learned for Rapid Appraisal of Irrigation Strategies

WMS 23 Tanzania/USAID: Rapid Mini Appraisal of Irrigation Development Options and Investment Strategies

WMS 24 Tanzania/USAID: Assessment of Rift Valley Pilot Rice Project and Recommendations for Follow-On Activities

WMS 25 Interdisciplinary Diagnostic Analysis of a Work Plan for the Dahod Tank Irrigation Project, Madhya Pradesh, India

WMS 26 Prospects for Small-Scale Irrigation Development in the Sahel

WMS 27 Improving Policies and Programs for the Development of Small-Scale Irrigation Systems

WMS 28 Selected Alternatives for Irrigated Agricultural Development in Azua Valley, Dominican Republic

WMS 29 Evaluation of Project No. 519-0184, USAID/El Salvador, Office of Small-Scale Irrigation - Small Farm Irrigation Systems Project

WMS 30 Review of Irrigation Facilities, Operation and Maintenance for Jordan Valley Authority

WMS 31 Training Consultancy Report: Irrigation Management and Training Program

WMS 32 Small-Scale Development: Indonesia/USAID

WMS 33 Irrigation Systems Management Project Design Report: Sri Lanka

WMS 34 Community Participation and Local Organization for Small-Scale Irrigation

WMS 35 Irrigation Sector Strategy Review: USAID/India; with Appendices, Volumes I and II (3 volumes)

WMS 36 Irrigation Sector Assessment: USAID/Haiti

WMS 37 African Irrigation Overview: Summary; Main Report; An Annotated Bibliography (3 volumes)

WMS 38 Diagnostic Analysis of Sirsia Irrigation System, Nepal

WMS 39 Small-Scale Irrigation: Design Issues and Government Assisted Systems

WMS 40 Watering the Shamba: Current Public and Private Sector Activities for Small-Scale Irrigation Development

WMS 41 Strategies for Irrigation Development: Chad/USAID

WMS 42 Strategies for Irrigation Development: Egypt/USAID

WMS 43 Rapid Appraisal of Nepal Irrigation Systems

WMS 44 Direction, Inducement, and Schemes: Investment Strategies for Small-Scale Irrigation Systems

WMS 45 Post 1987 Strategy for Irrigation: Pakistan/USAID

WMS 46 Irrigation Rehab: User's Manual

WMS 47 Relay Adapter Card: User's Manual

WMS 48 Small-Scale and Smallholder Irrigation in Zimbabwe: Analysis of Opportunities for Improvement

WMS 49 Design Guidance for Shebelli Water Management Project (USAID Project No. 649-0129) Somalia/USAID

WMS 50 Farmer Irrigation Participation Project in Lam Chamuak, Thailand: Initiation Report

WMS 51 Pre-Feasibility Study of Irrigation Development in Mauritania: Mauritania/USAID

WMS 52 Command Water Management - Punjab Pre-Rehabilitation Diagnostic Analysis of the Niazbeg Subproject

WMS 53 Pre-Rehabilitation Diagnostic Study of Sehra Irrigation System, Sind, Pakistan

WMS 54 Framework for the Management Plan: Niazbeg Subproject Area

WMS 55 Framework for the Management Plan: Sehra Subproject Area

WMS 56 Review of Jordan Valley Authority Irrigation Facilities

WMS 57 Diagnostic Analysis of Parakrama Samudra Scheme, Sri Lanka: 1985 Yala Discipline Report

WMS 58 Diagnostic Analysis of Giritale Scheme, Sri Lanka: 1985 Yala Discipline Report

WMS 59 Diagnostic Analysis of Minneriya Scheme, Sri Lanka: 1986 Yala Discipline Report

WMS 60 Diagnostic Analysis of Kaudulla Scheme, Sri Lanka: 1986 Yala Discipline Report

WMS 61 Diagnostic Analysis of Four Irrigation Schemes in Polonnaruwa District, Sri Lanka: Interdisciplinary Analysis

WMS 62 Workshops for Developing Policy and Strategy for Nationwide Irrigation and Management Training. USAID/India

WMS 63 Research on Irrigation in Africa

WMS 64 Irrigation Rehab: Africa Version

WMS 65 Revised Management Plan for the Warsak Lift Canal, Command Water Management Project, Northwest Frontier Province, Pakistan

WMS 66 Small-Scale Irrigation--A Foundation for Rural Growth in Zimbabwe

WMS 67 Variations in Irrigation Management Intensity: Farmer-Managed Hill Irrigation Systems in Nepal

WMS 68 Experience with Small-Scale Sprinkler System Development in Guatemala: An Evaluation of Program Benefits

WMS 69 Linking Main and Farm Irrigation Systems in Order to Control Water (5 volumes)

WMS 70 Integrating Strategies for Improving Irrigation System Design and Management

WMS 71 The USU Unit Command Area Model

WMS 72 Development of a Branching Canal Network Hydraulic Model

WMS 73 User's Manual for the FORTRAN Version of the USU Main System Hydraulic Model

WMS 74 Hydraulic Modeling Applications in Main System Management

WMS 75 User's Manual for the Pascal Version of the USU Main System Hydraulic Model

WMS 76 Formulation and Evaluation of the USU Main System Allocation Model

WMS 77 Irrigated Land Use and Irrigation Distribution Systems for Four Schemes in the Polonnaruwa District of Sri Lanka

WMS 78 Classification of Gravity Irrigation Systems and their Operation

WMS 79 Development and Management of Small Marais

WMS 80 Baskets of Stones: Government Assistance and Development of Local Irrigation in a District of Northern Sumatra

WMS 81 Implementing the Irrigation Maintenance and Operations (M & O) Learning Process Regionally or Nationally

WMS 82 Handbook of Improved Irrigation Project Operations Practices for the Kingdom of Thailand

WMS 83 Handbook of Improved Irrigation Project Maintenance Practices for the Kingdom of Thailand

WMS 84 USU Irrigation Main System Hydraulic Model: Replication of Modeling Capability in Other Countries

WMS 85 Development of the Centre International de l'Irrigation

WMS 86 Forum on the Performance of Irrigated Agriculture in Africa: Papers and Proceedings

WMS 87 Niger Irrigation Scheme Case Studies (English & French)

WMS 88 Irrigation Management for Development

WMS 89 Bureaucratic and Farmer Participation in Irrigation Development

WMS 90 Irrigation System Management: An Interdisciplinary Synthesis of Water Management Studies

WMS 91 Assessment Report: Maharashtra Irrigation Program. USAID/India

WMS 92 Irrigation System Operation Intensity and Relative Water Supply: The Asian Case

WMS 93 Methodologies for Interdisciplinary Diagnosis of Irrigation Systems

WMS 94 Management-Focused Improvement of Irrigated Agriculture

WMS 95 Diagnostic Analysis for Improving the Management of Irrigation Systems