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USU IRRIGATION MAIN SYSTEM HYDRAULIC MODEL: REPLICATION OF MODELING CAPABILITY IN OTHER COUNTRIES

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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. 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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The program described in this report has been developed by combining: (1) state-of-the-art technology for computer operation of irrigation systems developed at Utah State University; with (2) a program in operation and maintenance training. These two WMS II activities were implemented at two irrigation projects in the Northeast of Thailand. The experience gained from implementing these combined programs has led to the development of an integrated program in Computer-Assisted Irrigation System Management which can be replicated in other countries around the world.

We give much thanks to Dr. Worth Fitzgerald, USAID WMS II Project Officer and to Dr. Jack Keller, USU WMS II Project Director for their continual support in this program. Also, we are very grateful to Khun Nukool Thongtawee, Director, O & M Division, Royal Irrigation Department (RID), Kingdom of Thailand for his leadership and strong support in implementing these programs. Also, we are very thankful to Khun Vira Wongsangnak, Project Field Director, Northeast Small Scale Irrigation (NESSI) Project, RID for his continual organizational support throughout this program. Likewise, we are extremely grateful to Khun Vichai Sanguanpaiboon and Khun Banjong Tanangsuangnun for always being highly supportive of the efforts at the Lam Nam Oon Irrigation Project. Finally, we would like to pay tribute to the person who has frequently expedited our efforts, Khun Kamol Chantanumate, USAID/Thailand.

The Authors

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ABSTRACT

This report concerns the requirements for replicating the capability for hydraulic modeling of irrigation main systems with the USU Main System Hydraulic Model at different project sites and different countries. The computerized model was developed at Utah State University (USU) under the Water Management Synthesis II Project, funded and assisted by the United States Agency for International Development through the Consortium for International Development. The information contained herein complements that which is found in the user's manual for the model (Merkley, 1987).

INTRODUCTION

The USU Main System Hydraulic Model is a mathematical model which is microcomputer-implemented. The model simulates the flow of water in canals and displays results in both graphical and tabular form. It is an interactive and menu-driven software package that has been designed for ease of use by persons who do not wish to be concerned with the mathematical details of hydraulic equations or computer programming. It is a model that was designed for field applications rather than research-oriented activities.

The USU Main System Hydraulic Modeling package consists of the model itself ("CANAL"), a data file editing program ("CDAT"), and two utility programs: (1) a program to compute hydraulic roughness values based on gradually varied flow profiles ("ROUGH"); and, (2) a program to compute pump characteristic curves based on field data for centrifugal pump control structures ("PUMP"). The package also comes with a comprehensive users manual.

A principal application of the model is to improve the operation of canal systems without changing the physical infrastructure or automating the flow control structures. As an operational tool the model can be used to generate control structure adjustment schedules for the routing of flow changes in a canal system. These schedules can be manually implemented in the field by distributing tabular printouts to the operations personnel. The timing and amount of control structure adjustments is defined through the use of the model so that stable water levels are maintained during flow changes. Implementation of the model-generated schedules can also reduce lag times for water deliveries and significantly reduce fluctuations in turnout discharge rates. Through use of the model, canal operations are integrated on a system-wide basis, and operational guesswork is eliminated.

Other modeling applications include assisting canal design or re-design, and training of canal operators through simulations of a real system. As a design tool the model is used to analyze possible system configurations in an operational context. Thus, the final design is based on expected operational conditions, and not only on static peak flow criteria. As a training tool, the model allows users to simulate

canal system operation for a wide variety of flow delivery schemes, and all in a short time. The trainee practices on the model rather than on the real system. Thus, the real system is not jeopardized as a result of trial-and-error experimentations.

PERSONNEL

Use of the model in operational applications requires some knowledge of open channel hydraulics by the person who works on the computer and directs field data collection. An engineering background with one or more university level hydraulics courses is preferable, although exposure to hydraulic principles through short-term training courses can also be adequate. An understanding of at least basic hydraulics will enable a user of the model to correctly interpret the model output and identify the sources of potential simulation errors. Almost anyone who has any inclination to use a computer will be able to run the model, but mistakes are likely to occur when applying the modeling results if the user does not understand the basis for which the model simulates the flow of water in canals. Errors in field data collection and entry of data into the computer can easily result in model simulations that do not reflect the true behavior of the real canal system.

The model user must also occasionally exercise sound judgement with respect to the handling of modeling data, always recognizing that the model is only a tool for improving canal operation. As with any tool, the model should be used in appropriate applications, without exceeding its inherent limitations. For example, the model does not consider the possibility of surges or hydraulic jumps. Some of the model's limitations are of a numerical nature and these are discussed in the user's manual (Merkley, 1988). Misuse of the model can generate simulation results which appear reasonable, but which are actually false.

The model user should also possess some field experience in the actual operation of real canals so that he can more easily identify operation schemes that cannot be practically applied. This means that hydraulic modeling applications require not only technical judgement, but also consideration of the actual logistics for implementing the computer-generated operational schedules. The model can sometimes generate operational schedules for the control structures and turnouts which are technically feasible, but are not realistic from a practical point of view. Thus, a model user with field experience is better able to make adjustments in operational schedules to accommodate practical considerations.

The model user should also direct and participate in field data collection activities. This requires an understanding of basic open channel hydraulics, flow control structure calibration, and methods for seepage loss measurement. An understanding of how to calibrate flow control structures is particularly important since every irrigation main

system has its own peculiarities, and a "cookbook" method for performing the field calibrations will inevitably fail to answer all of the potential uncertainties. If field calibration work is done on a step-by-step basis with no real understanding of what is being done, then it should be expected that significant errors will be unintentionally introduced into the data. This is not to say that the calibration of flow control structures is complicated or difficult. Training courses given to junior level engineers or technicians about how to perform this calibration work can provide sufficient understanding for directing the field activities.

The total number of people required to implement the model will depend on the size of the project and the workload of the existing personnel. For the collection of calibration data, a single team of two or three people may be enough for an entire system. In many cases it is not necessary to hire new people for the purposes of hydraulic modeling. This is because the model is used to complement existing operational methods, and not to automate the system. Model-generated operational schedules can be implemented by the same field personnel who normally do the monitoring of flow levels and adjust the control and turnout structures. Therefore, the only potential increase in the workload for the existing field personnel would be to assist in collecting calibration data for the model.

Field personnel may even have more free time available for other tasks after using the model because they do not need to constantly check the flow levels in the system and make estimated control structure adjustments. With the model, control structure adjustments are not estimated, they are calculated. Thus, the field personnel know when adjustments should be made and do not need to constantly patrol the system to check whether the water levels are correct or not.

After initial installation of the model, only periodic checking of the calibration data is required. The person responsible for running the model could be expected to spend from one to three hours on the computer each time that the system's flow rate or distribution of flows are changed. These system flow changes could be daily or weekly, depending on the water allocation policy used at the irrigation project. Usually it is not necessary to sit and monitor a hydraulic simulation as the model runs on the computer. It is only necessary to begin the run by entering an inflow hydrograph and turnout demands, then review the simulation results and obtain printouts.

EQUIPMENT

Computer

Versions of the model are available for IBM XT and AT microcomputers, and for Hewlett-Packard 9836 workstations. The XT and AT computers can be IBM compatibles provided that all of the hardware components are truly compatible with the IBM machine. The RAM memory must be at least 640K, and it is preferable that the computer have a hard (fixed) disc drive. A hard disc makes using the model more convenient because of the greater access speed and storage capacity. One of the three following graphics cards must also be installed: (1) EGA (enhanced graphics adapter); (2) CGA (color graphics adapter); or, (3) Hercules graphics card. The model's software automatically recognizes any of these three graphics cards and makes the necessary adjustments for the screen display. The model will display color graphics for the EGA card, but the display is monochrome for both the CGA and the Hercules. The CGA's color capability is sacrificed so that the display will be of higher resolution.

It is recommended that a math co-processor (80287 chip) be installed to provide fast program execution. The numerical computations are intense in the model and a co-processor is very helpful to speed up the simulations. The model can, however, be configured for machines without the co-processor. In this case, the executable code is slightly larger, requiring a reduction in the maximum canal system dimensions that can be simulated simultaneously by the model. Thus, versions for machines with the co-processor can simulate systems with up to four branches, but versions for non-80287 machines can only accommodate a maximum of three branches. The HP 9836 version can simulate up to three branches.

The model results from hydraulic simulations can be displayed in both graphical and tabular form, and they can be printed on paper for hardcopy output. The hardcopy output can be used in the field when implementing model-generated operation schedules. Therefore, a printer should be available which can interface with the computer.

For the IBM XT and AT computers the model was developed to run on the DOS 3.1 operating system. DOS 3.2 or later versions are also acceptable. Some screen display features of the model require that the CONFIG.SYS file contain a line "DEVICE=ANSI.SYS". This means that the ANSI.SYS file should be in the directory from which the computer is booted. The DOS manual that accompanies the DOS operating system describes the use of the CONFIG.SYS and ANSI.SYS files.

Field Equipment

The calibration of control structures and determination of seepage losses requires flow rates to be measured in the canals. Some canal systems have fixed flow measurement structures such as broadcrested

weirs, but of course these must be calibrated too. And, flow control or measurement structures should be periodically checked, especially if physical deteriorization or other changes occur at or near the structures. Therefore, portable measurement devices such as mechanical and electronic current meters must be available. These devices measure flow velocities, which can be multiplied by flow cross-sectional areas to yield volumetric flow rates. Usually, only one portable flow measurement device is needed per irrigation system site for the purposes of obtaining hydraulic modeling calibration data. One device can also be shared among different sites since it is only needed when calibration work is being done.

A surveyor's level is necessary to measure canal bed slopes and pertinent control structure elevations. A flexible tape is needed to measure control structure dimensions and canal cross-sections, etc. Flow depths can usually be measured from the same rods that are used to place portable flow measurement devices in the water. Flow depths can also be measured from staff gauges placed along the canal banks.

DATA REQUIREMENTS

Application of the model in a main system requires site specific data collection. This data is important to enable the computer model to accurately simulate the true hydraulic behavior of the canal system. Much of the required data is usually available from design drawings and construction specifications for the canals and associated infrastructure. Data can also be collected from the field, both to verify design specifications and to complete missing values.

There are two general categories of data which are needed by the hydraulic model. The first of these two categories encompasses the system characteristics and can be sub-divided into the following two classes of data:

(1) Configuration data; and,
(2) Flow control and turnout structure data.

System characteristics data usually changes very slowly and it is only necessary to update the data when physical conditions change in the canals. For example, if canal lining deteriorates the roughness is affected, as may be the seepage rate.

The second of the two general categories includes system operational data which typically vary continuously as water demands and deliveries change each day (or each week) during the irrigation season. This type of data must be supplied to the model since it represents conditions which are external to the system being simulated. The inflow rate to the system from the water storage or collection facility must be specified as an input to the model. This system inflow rate is, then, presumed to be known and can change as often as needed during five-minute simulation intervals. The required inflow rate can be based on

expected turnout deliveries and seepage losses from the main system. It is the responsibility of the model user to decide upon an appropriate system inflow rate. Seepage losses can also be negative if a canal reach is in a cut section in which water seeps into the canal from the adjacent ground.

Other operational data needed are the turnout delivery schedules or water allocation schemes during simulation periods. This means that a proposed water distribution plan must be specified before the beginning of a simulation so that turnout demands are known. The model can then be used to help match actual deliveries to the turnout demand flows. Thus, the model does not determine what the turnout deliveries should be, but rather it determines appropriate control structure operation schedules based on the proposed water distribution plan.

System Characteristics

The hydraulic model is able to simulate the flow of water in a main system comprised of a maximum of four branches, nine reaches per branch, and nine turnouts per reach. Some versions of the model allow a maximum of only three branches. This reduces the memory requirements and allows the model to accommodate computers with less than 640K of RAM memory. Larger canal systems would need to be simulated as separate systems which are linked together. In the model a "branch" is defined to be a group of reaches connected in series. Branches can be connected in series or in parallel, depending on the configuration of the real canal system. Thus, if the main system at an irrigation project had twenty reaches connected in series, the model would simulate the system as three branches since there is a maximum of nine reaches per branch. The branches would be connected in series and the model would simulate all twenty reaches as a continuous canal system.

A reach is defined to be a length of canal that is bounded on both the upstream and downstream ends by a flow control structure such as a sluice gate or weir. The control structure at the downstream end of a reach is considered to belong to that reach, while the structure at the upstream end is considered to belong to the upstream reach. In the model there is no structure at the upstream end of the first reach of the first branch since this is the location of the system inflow, which is given as the inflow hydrograph. Turnout structures include farm turnouts and lateral offtakes where water is extracted in bulk from the main system. The definition of the main system is flexible for the purpose of performing hydraulic simulations, and it can extend beyond the main channel and out along lateral canals. Thus, laterals can be considered as part of the "main system".

The main system is divided as such into branches and reaches in order to organize the data entry and simulation results from the model, and to reduce a potentially complex system down to a tractable level which can be mathematically represented on a microcomputer. The division into branches and reaches also contributes toward model

generality for different canal systems. Reaches can be added or removed without requiring re-entry of existing data.

The configuration data is specified for each reach individually and includes the following items:

(1) Reach length (m);
(2) Canal longitudinal slope (m/m);
(3) Canal side slope (m/m);
(4) Canal base width (m);
(5) Canal depth (m);
(6) Canal material type (hydraulic roughness);
(7) Canal seepage rate (mm/day);
(8) Bottom elevation change upstream of the control (m);
(9) Bottom elevation change downstream of the control (m); and,
(10) Rectangular sluice gate setting correction (m).

The model is designed for use with trapezoidal canal cross-sections but can be changed to accommodate other section shapes. The canal material type (e.g. concrete, earth, etc.) and condition is specified in the form of a roughness coefficient as defined by the Manning equation (see the section on hydraulic roughness).

For structures operating under submerged flow conditions the difference in flow levels across the structure is relevant, while the actual flow depths upstream and downstream are not important. But, since the model computes flow depths it is necessary to reference these depths across a control structure to a common datum. For this reason it is necessary to know the bottom elevation change across the structure. This elevation change should not be measured locally at the structure because of the possibility that the canal bed has a high or a low spot in the vicinity. Instead, averaged reach slopes from "as-constructed" elevation surveys of the canal bed should be extrapolated to the control structures as illustrated in (Figure 1).

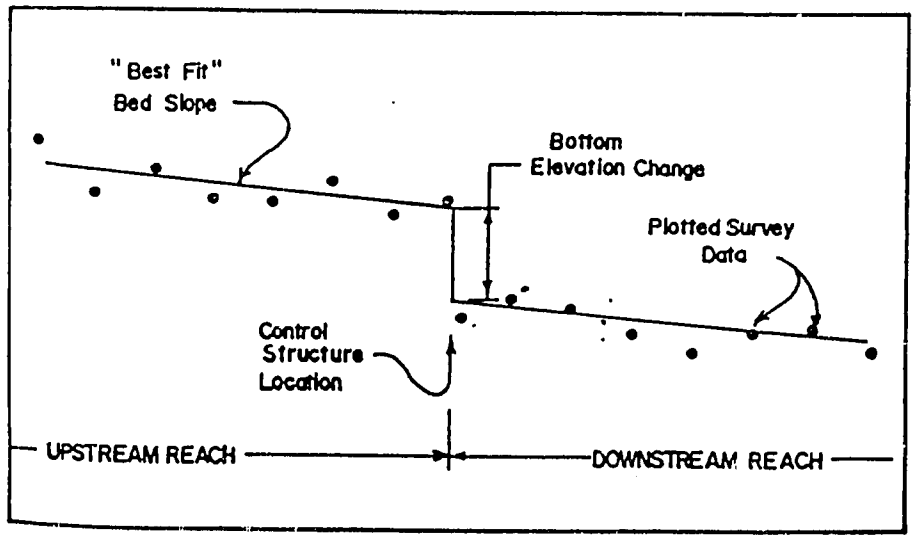


Figure 1. Illustrated Use of Slope Survey Data to Determine Bottom Elevation Changes Across Control Structures in Canals.

The "best-fit" bed slope can be determined graphically or by performing a linear regression on the survey data for each individual reach. If the structure is relatively long (e.g. culverts and inverted siphons) then the upstream and downstream slopes should be extrapolated to the center of the structure, as if the structure were only a single point. In this way the elevation change across the structure can be determined appropriately for the purpose of modeling the system, and errors due to local deviations from the average bed slope are eliminated.

In the model this bottom elevation change is separated into upstream and downstream changes which are referenced to the gate sill. Doing this makes control structure definitions more general since control structures operating under free flow conditions only require an upstream elevation change. By definition, downstream flow conditions do not affect the stage-discharge relationship through a structure operating under a free-flow regime. Thus, an upstream elevation change would correspond to an upstream channel transition in which the gate sill elevation is either above or below the canal bed. And, for submerged flow the individual values of upstream and downstream bottom elevation changes are not important, only the sum of the two is important.

The gate setting correction for rectangular sluice gates is a value that takes into account gate seating into the bottom of the gate frame. This value corrects the observed setting so that the gate "opens" as the bottom of the gate raises above the top of the gate seat. Similarly, the gate is "closed" from the time that the bottom of the gate enters the seat in the frame, to the time that the gate is fully seated. This setting correction is also used to provide a constant value of the discharge coefficient (see Skogerboe, et al, 1987).

Seepage Losses

The seepage loss rate can be determined by field measurements for individual canal reaches, or for groups of reaches. If the seepage rate is small, then it is more accurate to measure the rate over a long section of canal, and not just along a single reach. Three methods are described below for obtaining seepage loss rates from field measurements. It is assumed that the seepage loss rate is essentially constant with time during the test period. However, seasonal variations in the seepage rate may be significant if groundwater levels fluctuate near the canals, especially in cut sections where the seepage loss rate may be negative during periods of high precipitation. Seepage loss rates can often undergo long-term variations, such as when maintenance of canal lining is neglected and the seepage loss rate slowly increases. Therefore, seepage loss rates should be checked periodically to identify seasonal or long-term trends.

Additional information on measuring seepage losses can be found in the literature (Vlotman and Malano, 1986; United States Bureau of Reclamation, 1963).

Inflow-Outflow Method

For this method, it is preferable to select a canal section with minimum diversions, no appreciable bulk inflow (other than at the upper end) and where accurate water measurements can be made. Water measurement facilities are installed at the upstream and downstream ends of the test section and a steady flow is applied for a number of hours. If the canal section is large, then flow measurements with current meters will probably have to be made from a bridge or other cross structure.

It is important that a steady-state flow condition exist during the test. During this time, the inflow and outflow rates are measured, and the flow depths at various locations are determined. If the flow depths are fairly constant with distance along the section, then the depths only need to be measured near the upstream and downstream ends of reache(s) in the section. The flow cross-sectional data is used to estimate the average wetted perimeter in each reach and the channel loss is evaluated as follows:

$$Q_{SLR} = \frac{Q_{in} - Q_{out}}{L*W_p} * 86.4\ (10)^6 \qquad (1)$$

where,

Q_{SLR} = seepage loss rate (mm/day);
Q_{in} = section inflow rate (m3/s);
Q_{out} = section outflow rate (m3/s);
L = section length (m); and,
W_p = average wetted perimeter (m).

If any turnouts are discharging water along the test section, then these discharge rates must be measured and included in the Q_{out} value since they represent bulk ouflow and not seepage. If closed turnouts have leakage, then this flow can be considered as part of the seepage along the section, provided that the seepage rate is small and is nearly constant. Otherwise, any leakages from turnouts should be considered as turnout flow. If the outflow is more than the inflow, then the seepage loss rate is negative due to a net entry of water into the test section. This situation usually occurs when the irrigation channel is located in a deep cut and the surrounding groundwater levels are higher than the canal water level. This method is suited to large and medium-size canal sections.

Ponding Method

The ponding method involves first isolating a length of canal by constructing earthen dikes at both ends covered with plastic sheets to prevent seepage. Next, water is either added to maintain a constant flow depth, or the drop in water surface elevation is measured during the test period. Bulk lateral outflow from turnouts should be prevented during the measurement.

This method is suited to small canal sections where the test length can be blocked off at both ends to prevent any inflow or outflow other than that due to seepage. It is also appropriate for large canals that have very low seepage loss rates. The channel losses can be evaluated using either of the following two formulas:

(i) Adding water method:

$$Q_{SLR} = \frac{V}{L*W_p*t} * 1.44\ (10)^6 \qquad (2)$$

where,

V = volume of water added (m3); and,
t = test duration (min).

(ii) Changing flow depth:

$$Q_{SLR} = \frac{A_B - A_E}{W_p*t} * 1.44\ (10)^6 \qquad (3)$$

where,

A_B = average cross-section area at beginning (m2); and,
A_E = average cross-section area at end of test (m2).

Seepage meter

Whereby the previous two methods are used to determine average channel losses over the test section, seepage meters provide information on small areas of a canal. One type of seepage meter consists of a cylindrical bell that is connected to a measuring cylinder with a flexible hose. The cylindrical bell is pressed into the canal bed and the loss of water in the measuring cylinder is the seepage over the cross-sectional area of the test. The test should be repeated at several locations along the canal and the average seepage loss rate determined. The seepage loss rate is evaluated as:

$$Q_{SLR} = \frac{V}{A*t} * 1.44\ (10)^6 \qquad (4)$$

where,

V = volume of water seeped (m3);
A = cross-sectional test area (m2); and,

To date, the performance of seepage meters has been highly questionable, so they are generally not recommended for field use. The inflow-outflow and ponding methods described above have the advantage of automatically integrating the spatial seepage loss rate variations over the test section.

Hydraulic Roughness

The hydraulic roughness is a number that quantifies the effect of canal lining material, and conditions of the lining (i.e. new, old, weedy, etc.), on the flow characteristics. The roughness is not usually a constant value since it can change significantly due to aquatic and vegatative weed growth and removal, and due to changes in the canal cross-section from erosion or lining deterioration. The roughness can also change with flow rate or flow depth. These changes may be particularly important if the canals are operated within a wide range of flow rates.

The roughness must be known for hydraulic modeling since the shape of water surface profiles depends in part on the value of the hydraulic roughness. Changes in the hydraulic roughness can also be used as an index for determining when canal maintenance is necessary. It is not difficult to obtain values of the roughness from field measurements.

Manning's equation is used in the model to define the rate of energy loss as a function of roughness, flow rate, and water surface profile. The Manning equation can be expressed as follows:

$$Q = \frac{1}{n} A\ R_h^{0.67}\ S_f^{0.5} \qquad (5)$$

where,

Q = flow rate (m3/s);
n = hydraulic roughness;
A = flow cross-section area (m2);
R_h = hydraulic radius (m); and,
S_f = rate of energy loss (friction slope).

The hydraulic radius is equal to the cross-sectional flow area divided by the wetted perimeter, or

$$R_h = \frac{A}{W_p} \quad . \tag{6}$$

where,

W_p = wetted perimeter (m).

For trapezoidal cross-sections, the following relationships can be established (Figure 2):

$$A = y(b+my) \tag{7}$$

$$A = y(T+b)/2 \tag{8}$$

$$W_p = b + 2y(m^2 + 1)^{0.5} \tag{9}$$

where,

y = flow depth (m);
b = base width (m);
T = top width of flow; and,
m = inverse side slope (m/m).

For uniform flow, the longitudinal bed slope, S_o, can be substituted for the friction slope, S_f. Uniform flow occurs in prismatic channels when the flow is steady and the water surface is parallel to the bottom of the canal. Prismatic channels are channels that have constant cross-section and slope. In sloping irrigation canals uniform flow is most likely to occur near the upstream ends of relatively long reaches. On the downstream ends of sloping reaches, a gradually varied flow profile usually exists, meaning that the slope of the water surface is not constant, and not parallel to the bed slope.

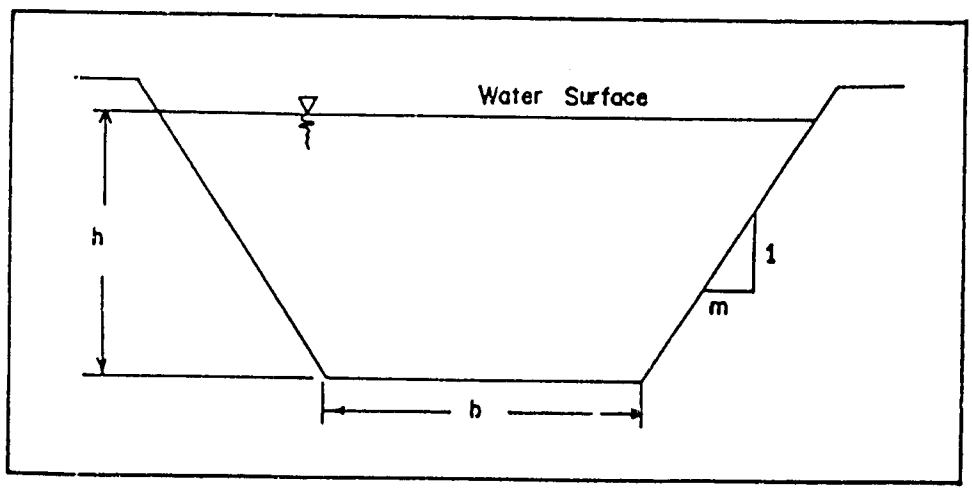


Figure 2. Cross-Section of a Trapezoidal Canal.

The inverse side slope can be measured in the field to check the design value. It can also be compared to a computed value from a flow cross-section area which is based on the flow depth and top width of flow. This is done by solving Equations 7 and 8 simultaneously with the inverse side slope, m, as the unknown. It is always good practice to perform cross checks like this on the field data.

Based on Uniform Flow

The hydraulic roughness can be calculated based on measurements from a region of uniform flow. The Manning equation can be re-arranged as follows:

$$n = \frac{1}{Q} A \, R_h^{0.67} \, S_o^{0.5} \qquad (10)$$

It can be noted that S_o replaces S_f in Equation 10. Thus, the above equation applies only for uniform flow conditions. All of the quantities on the right side of Equation 10 can be measured in the field to yield a single value for the hydraulic roughness, n. The procedure for doing this can be outlined as follows:

(1) Identify a section of canal with uniform flow conditions. The channel should be prismatic, the flow level unchanging, and the water surface parallel to the bed slope. If the flow depth is the same everywhere along the canal section, then the water surface is parallel to the bed;

(2) Measure the bed slope, base width, and inverse side slope of the canal section. If the bed slope is undular, then use an average value for the slope. Be aware that this data may differ from design specifications; therefore, field measurements are important to insure correct roughness values;

(3) Measure the flow depth and flow rate; and,

(4) Use Equation 10 to compute a value for the roughness.

Estimates of hydraulic roughness values can be obtained by consulting tables describing canal lining conditions and associated typical roughnesses. For example, Chow (1959) presents photographs and written descriptions of canal conditions with corresponding roughness values on pages 115 to 123. It is best, however, to compute the roughness value based on actual field measurements as described above, and only use tables of typical values to check the computed roughness. If the person conducting measurements already knows what the typical values are from field experience, the tables are not needed at all. However, in many cases engineers are quite familiar with the tables and have very rarely ever made field measurements of roughness, so it is quite easy to be in error by 100 percent or more.

Based on Gradually Varied Flow Profiles

In many cases the flow conditions are such that uniform flow does not occur at any point in the canals (even when the flow is steady). Usually in these cases the flow depth is higher than the normal depth everywhere along the canal reaches. The gradually varied flow profile does not develop fully within the length of a canal reach and the Manning equation cannot be used to directly calculate roughness values.

If uniform flow does not occur in a canal reach for which the roughness is to be determined, then gradually varied flow profiles can be computed based on the observed conditions. These profiles can be computed by selecting different values of roughness until the calculated upstream depth in the reach matches the field-measured value. When the depths match, the appropriate value of roughness is automatically determined since the shape of the water surface profile depends on the hydraulic roughness. For example, if an assumed value of roughness is too low, the computed upstream depth will be lower than the measured upstream depth. This type of analysis can be done very fast using a computer. A computer program to perform the analysis is available with the USU Main System Hydraulic Model software package.

The field data required for this type of analysis includes the following parameters (Figure 3):

(1) Reach length, L (m);
(2) Reach inflow rate, Q_{in} (m^3/s);
(3) Reach outflow rate, Q_{out} (m^3/s);
(4) Upstream flow depth, h_u (m);
(5) Downstream flow depth, h_d (m);
(6) Longitudinal slope, S_o (m/m);
(7) Canal base width, b (m); and,
(8) Canal inverse side slope, m (m/m).

The flow conditions in the reach should be steady, and the longitudinal slope should be taken from survey data of the canal bottom. A straight line can be graphically fitted to the plotted slope data, or a linear regression can be performed (or both), to compute a "best-fit" slope for the reach (see Figure 1). Water surface elevations should be referenced to this computed slope, rather than to the actual canal bottom, when determining "measured" flow depths on the upstream and downstream ends of the reach. This is because canal bottoms tend to have high and low spots, making local measurements of flow depth unreliable.

The computation of the gradually varied flow profile can be based on a flow rate that changes with distance along the reach. This is why it is necessary to know both the reach inflow and outflow rates. The downstream flow depth is always matched during the computations since this depth is affected by the downstream control (i.e control structure), and not by the hydraulic roughness. Thus, the flow profile is computed beginning at the downstream end and proceeding in the

upstream direction. This assumes that the flow is sub-critical, which is usually the case in irrigation canals.

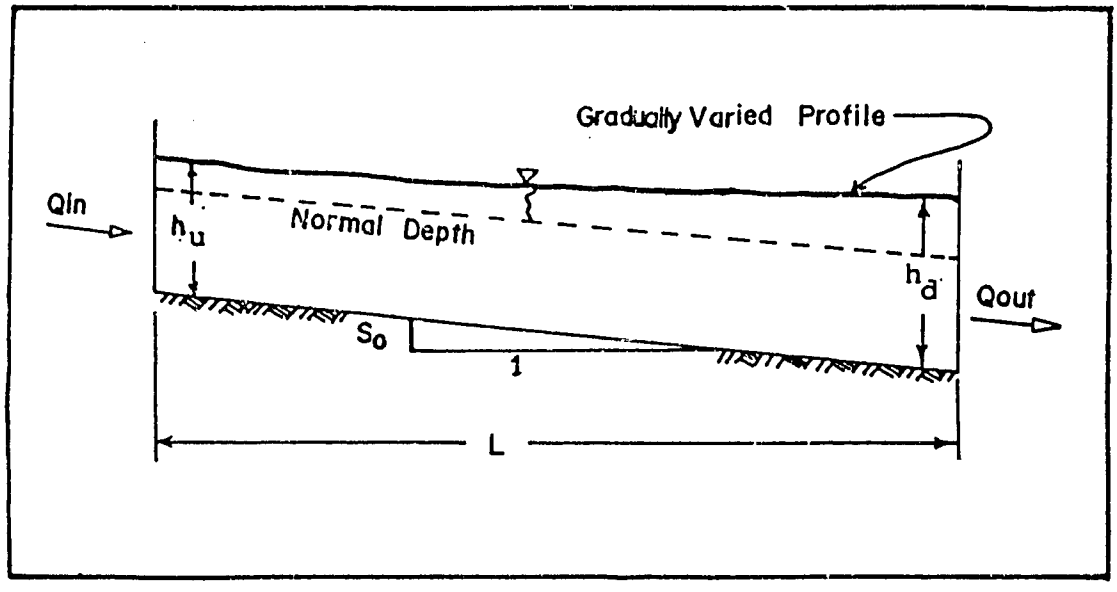


Figure 3. Side View of a Canal Reach Illustrating the Pertinent Parameters For Determining Manning Roughness Values Based on Gradually Varied Flow Profiles.

The field data collected for this can also be used for control structure calibrations if the structure setting is measured. Similarly, the field work for this could be combined with seepage loss measurements by the inflow-outflow method. Thus, many different parameters can be determined from the same pool of data collected in the field, and in this way field time can be spent very effectively.

Gate Calibration

The flow control and turnout structure data includes the type of structure, physical dimensions, and calibration constants. The calibration constants should be determined from field measurements under steady-state flow conditions. These measurements can be made for each and every structure; however, calibration results can be assumed to apply to other identical structures if the results are consistent for a representative sampling.

The calibration data should be collected as outlined in the following procedure:

(1) The system should be in a steady-state flow condition in the vicinity of the calibration measurements. Make sure that there are no weeds, debris, or sediment deposits in the water around or in the structure;

(2) Measure the dimensions (e.g. widths, heights, etc.) of the structure and record them along with a sketch of the structure and any pertinent comments;

(3) Check the water levels upstream of the structure for at least ten minutes to make sure that the flow is steady. If the structure is operating under submerged flow conditions then also check the downstream water level;

(4) Measure and record the water level on the upstream side of the structure. If submerged flow conditions exist then also measure and record the downstream water level. For submerged flow the difference in upstream and downstream water levels is important. For free flow the height of the upstream water level above the gate opening, or sill height, is important;

(5) If the structure is adjustable then measure the setting. It is preferable to measure control structure and turnout setting directly at the openings to reduce the possibility of errors;

(6) Using a current meter, calibrated weir, or flume, measure the flow rate at the structure. This rate can be measured either slightly upstream or slighly downstream of the structure. It is usually more accurate to measure the flow rate on the upstream side of a structure because there is less flow turbulence than there is on the downstream side. However, a measurement both upstream and downstream can give a more confident value since both values should be the same. It is important to take care in making discharge measurements since they are relatively difficult compared to dimensional measurements;

(7) Check the water level(s) again and record them. The levels should not have changed since the beginning of the calibration work; and,

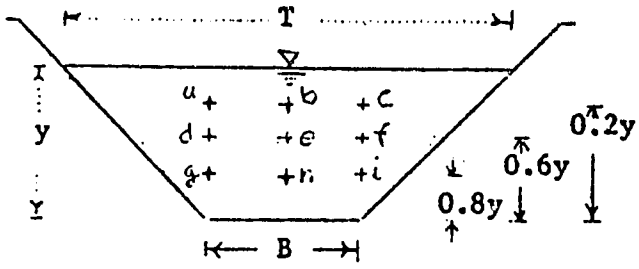
(8) Transfer the data from the field notes to a standard blank form (such as those provided herein in Figure 4).

The user's manual for the model contains detailed information on the types of control structures and turnouts which are currently available in the model. The exact flow equations are provided so that calibration measurements can be made to correspond with the control structure and turnout representations internal to the model. Sluice gates should be calibrated for both free flow and submerged flow conditions, unless the flow is always free flow or always submerged.

Sluice gate control structures in irrigation canals often have overflow weirs for passage of water during periods of unusually high flow rates, or when the gate structure is incorrectly operated. These weirs can prevent canal overflowing and they are safety devices if the sill heights are fixed. In the model, the sill height for such weirs is

TYPE OF STRUCTURE _____ K.M. _____ CANAL _____

RECORDER _____ DATE _____ TIME _____



BASE WIDTH (B) = _____ M.

TOP WIDTH (T) = _____ M.

FLOW DEPTH = _____ M.

FLOW VELOCITY : (M/SEC)

CONTROL STRUCTURE : _____

GATE OPENING : _____

U/S WATER ELE (Hu) : _____

D/S WATER ELE (Hd) : _____

Va =	Vb =	Vc =
Vd =	Ve =	Vf =
Vg =	Vh =	Vi =

Vaverage = _____ M/SEC

FLOW AREA : $A = y/2\ (T+B)$ =

FLOW RATE : $Q = A \times V$ =

SIDE SLOPE : $M = 1/y\ (A/y - B)$ =

WETTED PERIMETER : $WP = B+2y\ (m^2+1)^{1/2}$ =

Coeficient of Discharge (Cd) = $\dfrac{Q}{b \times W\ (2g\ (hu-hd))^{1/2}}$

=

Cd =

Figure 4. Sample Form for Recording Field Data from Control Structure Calibrations.

assumed to be fixed. For sluice gates without overflow weirs the sill height would simply be set to be equal to the canal depth, thus preventing the simulation of weir flow (unless the canal itself is overflowing near the structure). Weir flow control structures may have either fixed or adjustable sill heights. When there are more than one identical structural unit at one location, such as multiple weirs or sluice gates in parallel, the number of structures should be recorded along with the calibration data.

Orifice-type turnouts will also need field measurements referencing the downstream flow depth to the canal bottom and turnout discharge rate. These measurements require a typical turnout discharge and corresponding downstream depth to define a linear stage-discharge relationship. These relationships are used to approximate the flow conditions downstream of turnouts, which are external to the system as simulated by the model. This is described further in the user's manual. Datailed procedures and example computations are also show in the International Irrigation Center technical manual in "Field Calibration or Irrigation Structures for Discharge Measurement", dated December 1987.

MODEL INSTALLATION

Preparing for initial use of the model at an irrigation project site requires data collection and calibration work. This is described in the section on data requirements. After initial installation of the model only periodic data collection activities are needed to continue applying the model. The time required to complete the initial data collection depends on the number of people available to do the work, the quantity and type of field equipment, and size of the canal system. Much of the physical dimensions of the canals can be obtained from design drawings, provided that construction conformed to the design specifications and no significant changes have occured since completion of construction. This should be verified both by talking with construction personnel personnel, if possible and by making field checks.

Calibration work is usually the most time-consuming of the data collection activities. The calibration of control structures and turnouts can be performed over a period of time as different steady-state flow conditions occur in the system. This is because calibration data should be obtained for more than one single flow condition (i.e. flow rate, flow levels, and gate setting). This can be accomplished in a 3-week intensive training course or over a 2-3 year period by project field personnel. Participants in the course can be divided into groups to make the field measurements at the different locations along the canal system.

An outline of the procedure for installing the model follows:

(1) Review existing design and field data;

(2) Collect field data as needed to complete and correct the existing data;

(3) Calibrate the control structures and turnouts;

(4) Check the simulation results with actual transient and steady-state flow data from the field; and,

(5) Re-check calibrations, seepage loss rates, and hydraulic roughness values as needed so that model simulations agree with actual field conditions. The procedure for doing this is described in the following section.

Final Calibration Procedure

The final calibration procedure involves running simulations of the canal system for an observed steady-state flow condition. The simulations start out by using field-observed flow depths as operational supply levels (or full supply levels) on the downstream end of each canal reach, and the field-measured system inflow rate. Turnouts are "opened" during the simulations and the steady-state results are compared to the field-observed values of upstream flow depths and control structure settings in each reach. Seepage loss rates can also be verified by comparing field-measured flow rates for the same steady-state flow condition at different locations along the canals.

After being sure that all slope and dimensional data is correct, the roughness and control structure calibrations can be checked during actual simulations with the model using the following procedure:

(1) Collect data from a steady-state flow condition in the real canal system. Measure and record the discharge at each turnout in the system. Also, measure and record at each control structure:

a. discharge (m^3/s);
b. control structure setting (m);
c. upstream flow depth, or elevation (m); and,
d. downstream flow depth, or elevation (m).

(2) Use the CDAT program to set the operational supply levels to the actual flow levels (depths) upstream of each control structure (i.e. at the downstream end of each reach). Temporarily reduce the system dimensions to only the first reach. This will make the work go faster;

(4) Simulate the filling of the canal system with the model using the field-measured system inflow rate at the upstream end of the first reach. Use the measured turnout discharges to set the turnout demands and let the simulation continue until steady-state conditions are achieved. If it takes longer than twelve and one-half hours to reach steady-state conditions,

then the simulation will need to be continued in more than one "run";

(5) Observe and record the computed flow level at the upstream end of the reach. If this value is lower than the flow level measured in the field, the roughness must be increased; if it is higher, then the roughness must be decreased;

(6) After using the CDAT program to change the roughness, continue at step (4) again. Repeat this sub-procedure until the flow levels match at the upstream end of the reach. This will determine the correct value of the hydraulic roughness;

(7) Use the CDAT program to increase the system dimensions by one reach. Determine the roughness for this next reach by repeating steps (4) to (6) again;

(8) Observe the computed discharge through the control structure at the end of the upstream reach. If this is not the same as the field-measured value then:

a. the system inflow rate is incorrect; or,
b. the turnout discharge(s) are incorrect; or,
c. the seepage loss rate is incorrect; or,
d. a combination of the above.

Check the flow values and try to identify where the mistake is and correct the value(s);

(9) Now both the upstream and downstream flow levels at the control structure in the upstream reach are at the field-measured values, and the discharge is also correct. If the computed control structure setting is not the same as the field-measured setting then adjust the structure calibration as follows:

a. Rectangular Sluice Gates:

Both the discharge coefficient and the gate setting correction may be subject to adjustment. If the field-measured discharge coefficients are fairly constant in value then only the gate setting correction may need to be changed. In this case, the setting correction is equal to the difference between the computed setting and the field-measured setting for the steady-state flow condition.

b. Other Control Structures:

Use the control structure equation to compute the discharge coefficient value which will match the field-measured structure setting.

(10) Repeat at step (7) until the entire system has been checked.

FIELD PROGRAM FOR THE NORTHEAST SMALL SCALE IRRIGATION PROJECT IN THAILAND

Description

About 200 small scale irrigation projects were constructed in Northeast Thailand 15-20 years ago by the Royal Irrigation Department (RID). These systems generally consisted of small reservoirs, or "tanks", and main canals without on-farm delivery networks. Equitable water distribution was difficult to achieve because of the lack of an on-farm delivery system and water disputes among farmers was common. Most of the irrigation systems experienced rapid and extensive infrastructure deterioration, and the effectiveness of the projects to deliver water was deemed to be inadequate.

In the late 1970's the Asian Institute of Technology conducted feasibility studies for rehabilitation of a small number of these projects. This work was done under a United States Agency for International Development (USAID) contract. Seven project sites were subsequently selected for rehabilitation work under the Northeast Small Scale Irrigation Project (NESSI). NESSI has received funding from USAID grants and loans, and from the Royal Thai Government. The seven project sites are:

(1) Huai Aeng;
(2) Huai Kaeng;
(3) Phuttha Utthayan;
(4) Huai Khilek;
(5) Huai Chorakhe Mak;
(6) Huai Talat; and,
(7) Lam Chamuak.

The irrigated areas of the seven NESSI sites range from 9,000 to 21,000 rai (1,440 to 3,360 ha). Rehabilitation of existing infrastructure and construction of new secondary and tertiary canals has been completed at Huai Aeng, Huai Kaeng, Phuttha Utthayan, and Huai Khilek. At these first four sites work is currently being undertaken in agricultural extension, land development, and improvement of operation and maintenance practices at all levels of the canal systems. The last three NESSI sites (Huai Chorakhe Mak, Huai Talat, and Lam Chamuak) are as yet under rehabilitation and construction. The overall construction status of the seven NESSI sites is that about 80% of the work is completed as of Fiscal Year 1988.

Significant improvements have been made at the first three project sites in terms of wet season rice yield and dry season planted area since the beginning of NESSI in 1981. These improvements can be attributed to improved system infrastructure, agricultural extension, water users organization, and water user participation in operation and maintenance of the tertiary system.

Hydraulic Modeling

In order to improve water management at the seven NESSI sites it was decided to apply computer hydraulic modeling to assist in the operation of the irrigation main system at each of the project sites. The main system was defined as being the primary water conveyance and distribution network of canals. This hydraulic modeling work was initiated at Huai Aeng, the first NESSI site, during the final stages of rehabilitation and construction work. This work was performed by the Agricultural and Irrigation Engineering Department of Utah State University under the Water Management Synthesis II Project (WMS II) of USAID/Washington in which both WMS II and NESSI grant funds were utilized.

Data Collection

A two-week training course was held at the Huai Aeng NESSI site by the International Irrigation Center (IIC) of Utah State University during June of 1986. The course involved both classroom and field work in which the participants were instructed on how to calibrate flow control structures and determine seepage losses in canals. The participants consisted of RID irrigation engineers from various Northeast provinces who had graduated from the RID School of Irrigation in Bangkok. They were divided into several working groups of 3-4 people each before beginning the field work. The field work was conducted along the Right Main Canal and its principle lateral 2L-R. No work was done on the Left Main Canal because calibration and seepage loss data had already been collected there by RID irrigation engineer Kanching Kawsard during 1983. The course was useful for training the participants, and it also benefitted the hydraulic modeling efforts by providing a large amount of useful data in a short period of time. A distinct advantage in collecting field data during such a training course is that cross-checks can be easily made by the trainers as data is analyzed, and measurement or computational errors can be identified more readily than if data is collected on only one canal reach at a time by a single working group.

Further data collection activities for the application of hydraulic modeling have been undertaken since June 1986 at Huai Aeng, Huai Kaeng, and Phuttha Utthayan. These activities were directed by Kanching Kawsard and Charey Ekdamrong. The data is useful by itself, even without hydraulic modeling, because it represents a quantification of actual flow distributions and canal system behavior. The availability of the data contributes to an increased understanding of the true operational characteristics of the system, thereby providing one important basis upon which water measurement improvements can be made. The NESSI budget has funds allocated for the completion of data collection at the first four of the seven project sites.

Current Status

The data collected from the Huai Aeng, Huai Kaeng, and Phuttha Utthayan sites has been steadily increasing since the June 1986 training course at Huai Aeng. Nevertheless, data is not yet complete at any of the sites. One of the difficulties is that much of the "as-constructed" dimensions and slopes at the Huai Aeng site apparently differ from the design values. It is not known where these differences occur in the infrastructure and this requires that systematic verification of actual dimensions and slopes be performed in the field. Also, some of the recorded data from Huai Aeng is definately incorrect, and other data is of questionable validity. The support staff for doing the field work in collecting the required data are usually the zonemen who work at the site. These people are trained to do calibration and surveying work, but they are not engineers or technicians and they often lack the ability to perform consistant data collection work unless constant supervision is available. Furthermore, they aparently have limited incentive to perform the data collection activities, which are above and beyond the normal daily tasks for canal operation at the project site.

The model has been used at NESSI sites only for analysis and testing, but not yet applied to the actual operation of the canals. This is partly because the data collection is not yet complete, but also because the computer used to implement the model is at NESSI project headquarters near Muang, Khon Kaen. The closest of the seven NESSI sites to the project headquarters is Huai Aeng, which is more than an hour away by car. Also, the computer for the hydraulic modeling is heavily used for administrative and other work.

Recommendations

Data collection should be completed at the first four NESSI sites. This includes configuration data, flow control structure calibration data, seepage loss rates, and hydraulic roughnesses. The remaining data collection and field measurement activities should be scheduled with a target date for completion, otherwise they may be subject to frequent and prolonged delays. The model should be applied to the Huai Aeng site first since it has the most data already and is the closed site to the project headquarters. During initial testing of the calibration data it would be very preferable to locate the computer at the project site so that the work would go faster and more smoothly. Initial on-site calibration means resolving differences between field-observed flow conditions (flow rates and flow levels) and model-generated values. This is important to make the final adjustments in the collected data. After the initial on-site calibration the computer could be returned to the project headquarters and modeling results could be implemented through radio transmissions. This may be feasible since flow changes are generally made in intervals of several days, and the modeling demands would not be very intense.

After completion of the construction work at the last three NESSI sites field data should be collected and the model applied. Also, the recent aquisition of identical IBM AT computers at Department of Agricultural Extension (DOAE) offices in Khon Kaen, Nakorn Ratchasima, Sakol Nakorn, Roi Et, Ubon Ratchatani, and Buri Ram could be taken advantage of for hydraulic modeling. A version of the USU Main System Hydraulic Model can be easily adapted to the DOAE computers and the model may be applied through them when and if feasible, depending on the availability of the machines.

If difficulties arise in applying the model due to computer unavailability, the field data collected for the application of the model can be used to schedule steady-state canal operation. This can be done with hand-held calculators and can lead to significant improvements in the canal operations in terms of meeting intended water distribution schedules. Thus, regardless of whether the model is ever actually used or not at any particular NESSI site, the data collection can be usefully applied to canal operational improvements.

FIELD IMPLEMENTATION AT THE LAM NAM OON IRRIGATION PROJECT IN THAILAND

Physical Description of Project

The Lam Nam Oon (Lam is a Thai word for river) is the major tributary of the Lam Nam Songkam, a major drainage area north of the Phu Phan mountain range located on the Korat Plateau in Northeast Thailand. The Nam Oon Dam is located roughly 60 km west of the provincial capital city of Sakol Nakorn and 120 km east of the city of Udon Thani as shown in Figure 5.

The Nam Oon Dam is an earthfill structure having a crest length of 3,300 m, height above the river bed of 29.5 m, and bedrock 9.5 m below the river bed. The reservoir has a total storage capacity at 185 m above mean sea level (msl) of 520 million cubic meters, and a dead storage capacity at 175 m above msl of 45 million cubic meters, leaving an active reservoir capacity of 475 million cubic meters. The average annual reservoir inflow is 365 million cubic meters, with the estimated mean annual reservoir evaporation loss being 60 million cubic meters. The reservoir area at 185 m above msl is 85 square kilometers, while the watershed area is 1,100 square kilometers. The average annual rainfall is 1,400 mm, most of which occurs during the monsoon season of May-October.

The irrigated area of approximately 185,000 rai (1 rai is 40 m x 40 m, or 0.16 hectares, or 0.4 acres) is served by two major canals -- the Right Main Canal and the Left Main Canal. The Right Main Canal is 45.7 km in length and serves 122,545 rai, where Table 1 shows the irrigable land and discharge capacity of each Farm Turnout (FTO) structure along the Right Main Canal (e.g., FTO 1L is the control regulator serving 121 rai, whereas R-1L is the FTO regulator for lateral

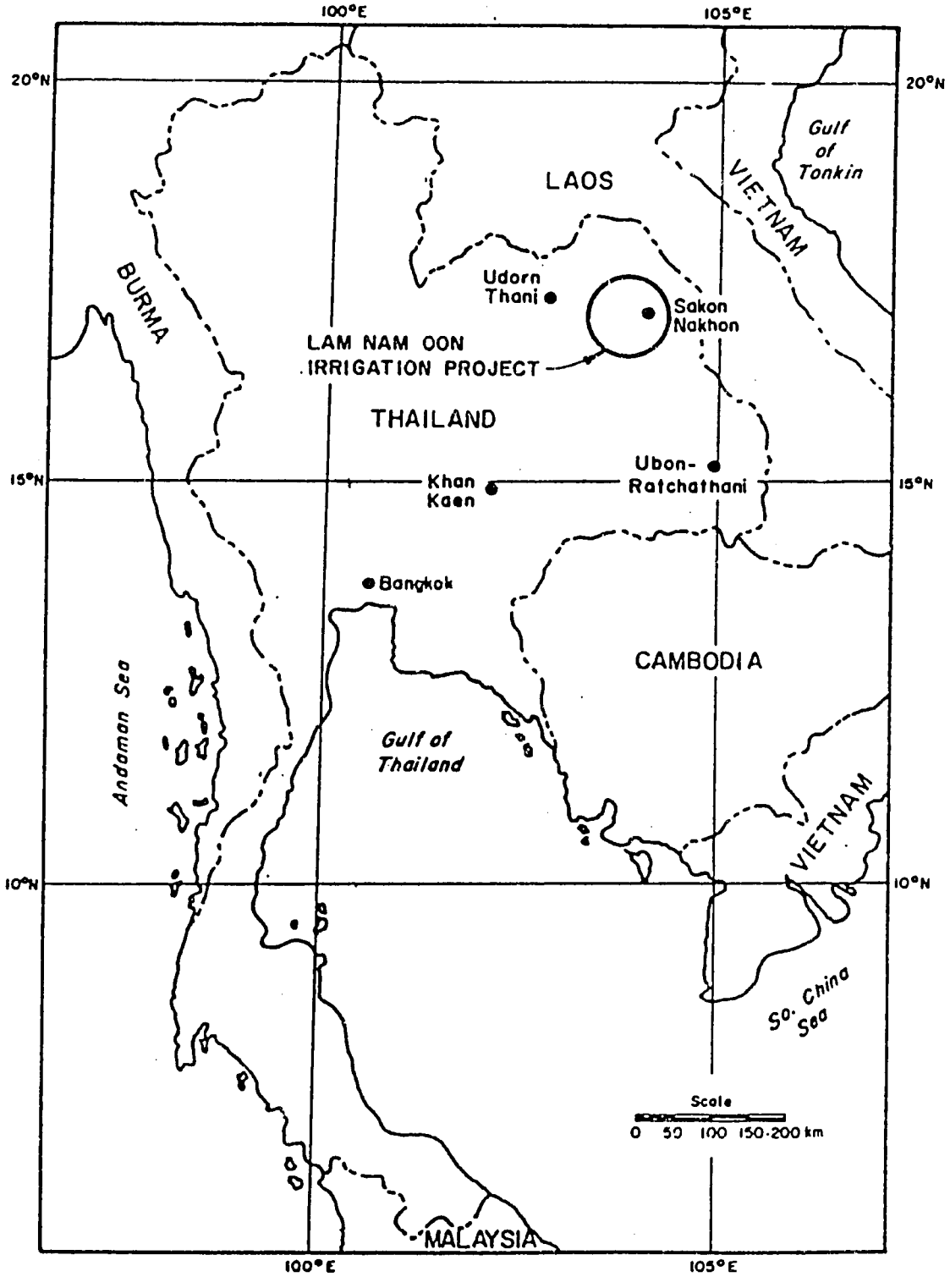


Figure 5. Location of the Lam Nam Oon Irrigation System, Sakon Nakhon Province, Northeast Thailand

Table 1. Number, Location, Irrigable Area, and Discharge Capacity of Farm Turnouts (FTO's) Along the Right Main Canal

FTO Number	Location, meters	Irrigable area, rai	Discharge capacity, m^3/s(cms)
1L	1+850	121.0	0.048
R-1L	2+170	12,356.7	2.424
2L	3+390	114.7	0.048
3L	3+820	152.0	0.061
R-2L	4+195	664.5	0.2658
4L	4+252	143.5	0.057
5L	4+800	87.5	0.035
6L	5+350	151.0	0.060
7L	5+650	207.0	0.083
R-3L	5+760	741.2	0.2965
8L	5+800	97.5	0.039
9L	6+200	151.0	0.060
10L	6+850	122.0	0.049
11L	7+600	72.0	0.0288
12L	8+600	56.0	0.0224
R-4L	8+822	5,096.5	1.2899
13L	9+700	50.0	0.0200
14L	11+100	150.5	0.0602
15L	11+550	69.0	0.0276
R-6L	11+935	17,660.4	3.240
16L	13+000	298.8	0.1195
17L	13+600	118.7	0.04748
18L	14+960	126.0	0.0504
19L	16+800	248.5	0.0994
20L	17+400	142.0	0.0568
21L	17+950	231.5	0.0926
R-1L	18+895	6,152.2	1.456
22L	19+305	88.0	0.0352
R-8L	19+495	1,599.3	0.6397
23L	20+220	212.5	0.085
24L	21+200	118.5	0.0474

Table 1. continued

FTO Number	Location, meters	Irrigable area, rai	Discharge capacity, m^3/s(cms)
R-9L	21+515	5,183.3	1.313
25L	22+800	105.0	0.042
26L	23+350	159.0	0.0636
R-10L	23+516	284.1	0.1136
27L	23+550	192.0	0.0768
28L	24+000	218.0	0.0872
R-11L	24+470	8,663.5	1.880
29L	25+350	166.5	0.0666
30L	26+300	55.5	0.00222
R-12L	26+403	2,642.6	0.907
31L	26+550	208.0	0.0832
32L	27+550	163.0	0.0652
33L	28+396	160.0	0.064
34L	28+796	217.5	0.087
35L	29+645	178.5	0.0714
36L	29+835	207.5	0.083
37L	30+195	171.0	0.0684
38L	31+540	416.0	0.1664
39L	31+820	350.5	0.1402
R-13L	31+860	4,767.9	1.237
40L	32+400	188.8	0.0755
41L	32+940	230.8	0.0923
42L	34+030	155.0	0.0620
43L	34+500	232.5	0.0930
44L	35+660	243.3	0.0973
R-14L	35+733	642.1	0.2568
45L	35+800	284.7	0.1139
R-15L	36+123	2,474.9	0.880
46L	36+200	136.1	0.0544
47L	36+650	318.7	0.1275
48L	36+650	432.2	0.1729

Table 1. continued

FTO Number	Location, meters	Irrigable area, rai	Discharge capacity, m^3/s(cms)
49L	37+000	382.9	0.1532
50L	37+500	343.9	0.1376
51L	38+000	476.4	0.1906
R-16L	38+100	13,552.0	3.5200
52L	38+500	479.8	0.1919
53L	38+850	329.5	0.1318
R-17L	38+900	16,199.7	3.520
54L	39+250	728.8	0.2915
55L	40+500	995.3	0.3981
56L	41+400	521.5	0.2086
57L	42+000	500.0	0.200
58L	42+350	250.0	0.100
59L	43+700	565.2	0.2261
60L	44+700	652.9	0.2612
61R	45+230	493.4	0.1974
62	45+700	481.0	0.1924
	Total	114,400.9	29.5045

Photo 1. The Lam Nam Oon Dam and Service Spillway.

Photo 2. The Upstream End of the Left Main Canal at the Lam Nam Oon Irrigation Project.

Photo 3. The Operation and Maintenance Headquarters at the Lam Nam Oon Irrigation Project.

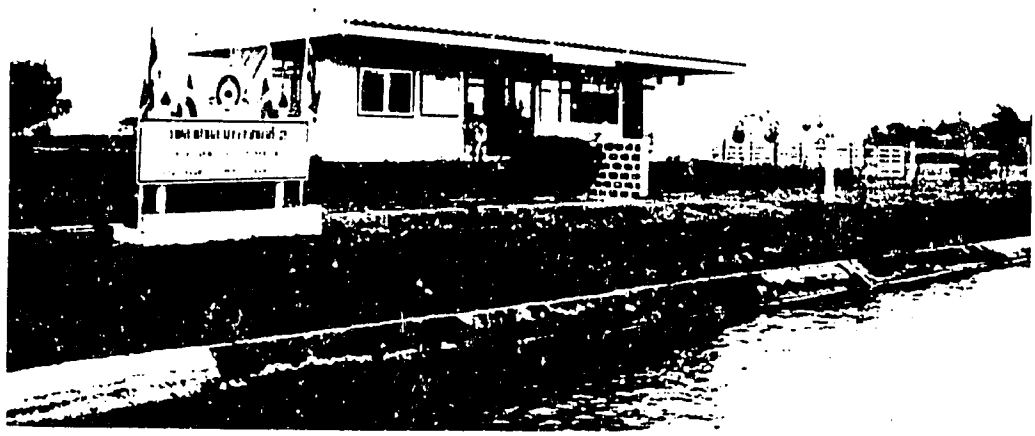


Photo 4. An Operation and Maintenance Section Office on the Left Main Canal.

R-1L, which has many Farm Turnouts along its length). The Right Main Canal has a design discharge capacity of 21.8 cubic meters per second (m^3/s or cms) and an initial concrete trapezoidal cross-section having a bottom width of 5 m, side slopes of 1½ horizontal to 1 vertical, and a vertical depth of 2.85 m.

The Left Main Canal is 28 km in length and serves 63,177 rai, where Table 2 shows the irrigable land and discharge capacity of each FTO structure. The Left Main Canal has a design discharge capacity of 9.1 cms with an initial concrete trapezoidal cross section having a bottom width of 3 m, side slopes of 1½ horizontal to 1 vertical, and a vertical depth of 1.85 m.

A map showing the location of canals, laterals, and sublaterals in the Lam Nam Oon irrigation system is shown in Figure 6. Tables 3 and 4 give the length of concrete lining for each lateral and sublateral. Sandy soils are predominant throughout the Northeast, including the Lam Nam Oon project lands. Irrigation channels generally require concrete lining to prevent serious degradation due to erosion, as well as to avoid excessive seepage losses.

The Lam Nam Oon Irrigation Project is divided into six Operation and Maintenance (O & M) sections. The first section is located at the project headquarters. The second and the third sections are located at km 6+000 and 13+300 along the Left Main Canal, respectively. O & M Sections 4, 5 and 6 are located at km 10+100, 25+200 and 38+500 along the Right Main Canal. The Water Master is the head of the O & M section, which answers to the Project Engineer, who is in-charge of the entire project. Each O & M Section is divided into 3 units: (a) the Administrative unit; (b) Operation unit; and (c) Maintenance unit. There are a total of 21 Zonemen, who work in the operation units and are responsible for 185,800 rai (1 ha = 6.25 rai). The gate tenders, canal tenders and laborers work in the Maintenance unit. The radio operator, the administrative accountant and Warehouse personnel work in the Administrative unit.

Field Data Collection

The required data for implementing the hydraulic model at the Lam Nam Oon Irrigation Project was similar for both the Left Main Canal (LMC) and the Right Main Canal (RMC).

Discharge Ratings

The RMC head regulator, all of the control structures, and all of the lateral head regulators have been calibrated. Some of the different sizes of the farm turnouts (outlets to tertiary systems) were also calibrated. Along the Left Main Canal discharge ratings have been completed for the head regulator, lateral regulators and control structures. Current metering was used to measure flow velocities. The discharge coefficient can be directly calculated for each control structure using field measured flow rate, water surface elevations, and

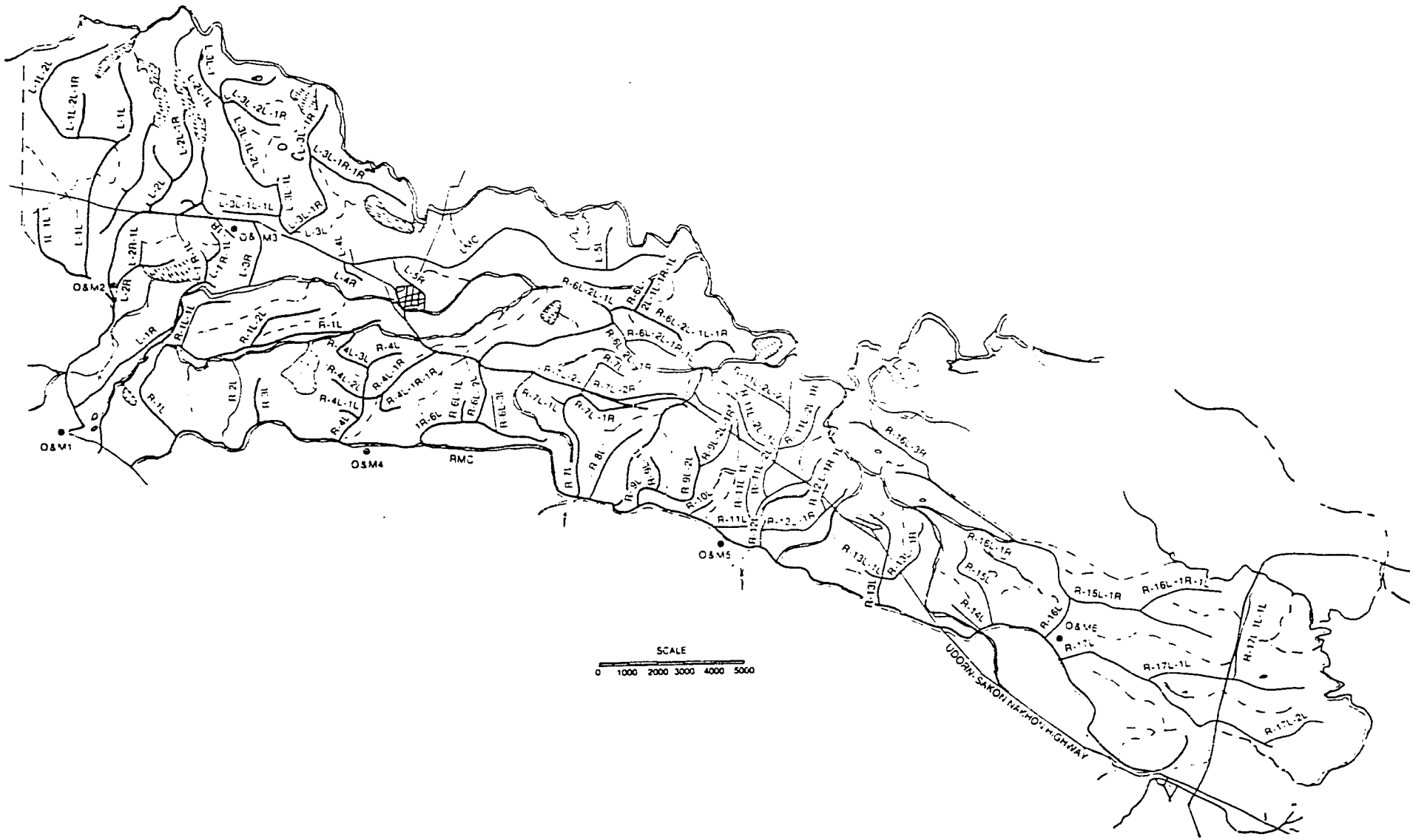


Figure 6. Map of Lam Nam Oon Irrigation System Showing the Location of Canals, Laterals and Sub-Laterals.

Table 2. Number, Location, Irrigable Area, and Discharge Capacity of Farm Turnouts (FTO's) Along the Left Main Canal

FTO Number	Location, meters	Irrigable area, rai	Discharge capacity, m^3/s(cms)
L-1R	1+420	4,440.40	1.1608
1R	1+850	286.50	0.1146
2L	2+900	87.80	0.0351
3R	3+200	319.50	0.1278
4R	3+600	217.00	0.0868
5R	3+800	52.50	0.0210
6R	4+400	183.00	0.0732
7R	4+800	489.00	0.1956
L-1L	6+000	17,492.20	3.2117
L-2R	6+000	2,369.50	0.8483
L-2L	9+724	3,619.30	1.035
L-3R	13+400	1,759.50	0.7038
8L	13+880	253.85	0.1015
L-3L	14+560	15,527.60	2.8526
9L	16+000	341.50	0.1366
10R	16+500	573.852	0.2151
L-4R	16+600	510.500	0.2040
11L	16+860	341.00	0.1364
L-4L	16+975	560.00	0.1839
12L	17+290	352.00	0.1408
13L	17+650	271.150	0.1085
14R	17+675	367.00	0.1468
15R	17+960	358.00	0.1432
L-5R	18+400	1,244.50	0.4978
16R	18+650	275.50	0.1162
17L	19+400	585.50	0.2324
18L	19+980	383.50	0.1534
19R	20+036	459.50	0.1838
20L+20R	20+200	242.50	0.0970
21R	20+500	139.00	0.0556
22L	21+500	507.50	0.2030

Table 2. continued

FTO Number	Location, meters	Irrigable area, rai	Discharge capacity, m^3/s(cms)
23R	21+700	229.00	0.0916
24R	22+200	435.00	0.1740
25L	22+200	679.00	0.2716
26R	22+800	77.00	0.030
27L	22+800	307.50	0.1230
28L	22+950	305.00	0.1220
29L	23+260	277.00	0.1108
30R	23+260	249.00	0.0996
31L	23+950	309.50	0.1238
32R	24+100	540.00	0.2160
33L	24+520	537.00	0.2148
34R	24+520	509.00	0.2036
35L	25+500	342.50	0.1370
36R	25+500	167.50	0.0670
37L	25+920	307.50	0.1230
38R	25+980	351.00	0.1404
L-5L	26+800	1,029.20	0.4117
39L	26+900	312.00	0.1248
40R	26+900	413.00	0.1652
41R	27+550	200.0	0.0800
42R	28+000	180.0	0.0720
43L	28+000	175.0	0.0700
44	28+040	161.50	0.0646
	Total	63,153.352	16.7373

Table 3. Length of Concrete-Lined Laterals and Sub-Laterals Served by the Right Main Canal

Canal	Lateral	Sub-Lateral	Length, km	Year Constructed
RMC			45.700	1975-1979
	R-1L		11.415	1975
		R-1L-1L	2.500	1975
		R-1L-2L	2.800	1975
	TOTAL FOR LATERAL R-1L		16.715	
	R-2L		3.515	1976
	R-3L		1.600	1976
	R-4L		4.600	1977
		R-4L-1L	0.800	1977
		R-4L-2L	1.120	1977
		R-4L-3L	2.020	1977
		R-4L-1L	3.200	1977
		R-4L-1R-1R	1.150	1978
	TOTAL FOR LATERAL R-4L		12.890	
	R-6L		4.900	1976
		R-6L-1L	0.900	1976
		R-6L-2L	7.790	1976-1979
		R-6L-3L	1.300	1976
		R-6L-2L-1L	1.120	1979
		R-6L-2L-1L-1R	5.400	1979
		R-6L-2L-1L-1R-1L	1.550	1979
		R-6L-2L-1R	1.200	1979
		R-6L-2L-1R-1L	1.810	1979
	TOTAL FOR LATERAL R-6L		25.970	

Table 3. continued

Canal	Lateral	Sub-Lateral	Length, km	Year Constructed
	R-7L		6.120	1976-1978
		R-7L-1L	0.620	1976
		R-7L-1R	2.500	1976
		R-7L-2L	1.020	1976-1978
		R-7L-2R	3.420	1976-1978
	TOTAL FOR LATERAL R-7L		13.680	
	R-8L		3.050	1977
	R-9L		2.655	1977-1978
		R-9L-1L	1.180	1977
		R-9L-2L	2.160	1977-1978
		R-9L-2L-1R	3.450	1979
	TOTAL FOR LATERAL R-9L		9.445	
	R-10L		1.100	1977
	R-11L		4.800	1978
		R-11L-1L	1.500	1978
		R-11L-2L	4.500	1977-1978
		R-11L-2L-1R	2.300	1977-1978
		R-11L-2L-1L	2.900	1979
		R-11L-2L-2L	2.100	1979
	TOTAL FOR LATERAL R-11L		18.100	
	R-12L		0.800	1979
		R-12L-1R	3.700	1979
		R-12L-1R-1L	1.230	1979
	TOTAL FOR LATERAL R-12L		5.730	
	R-13L		3.720	1979
		R-13L-1L	0.760	1979
		R-13L-1R	3.050	1979
	TOTAL FOR LATERAL R-13L		7.530	

Table 3. continued

Canal	Lateral	Sub-Lateral	Length, km	Year Constructed
	R-14L		1.300	1979
	R-15L		3.700	1979
	R-16L		9.500	1980
		R-16L-1L	2.068	1980
		R-16L-1R	5.000	1980
		R-16L-1R-1L	2.943	1980
		R-16L-3R	3.430	1980
	TOTAL FOR LATERAL R-16L		22.941	
	R-17L		8.370	1980
		R-17L-1L	5.580	1980
		R-17L-1L-1L	3.670	1980
		R-17L-2L	2.460	1980
	TOTAL FOR LATERAL R-17L		20.080	
		GRAND TOTAL	213.046	

Table 4. Length of Concrete-Lined Laterals and Sub-Laterals Served by the Left Main Canal

Canal	Lateral	Sub-Lateral	Length, km	Year Constructed
LMC			28.040	1973-1975
	L-1L		9.310	1973-1977
		L-1L-1L	3.400	1973
		L-1L-2L	6.500	1978
		L-1L-2L-1R	1.500	1978
	TOTAL FOR LATERAL L-1L		20.710	
	L-1R		6.624	1973
		L-1R-1L	1.170	1974
		L-1R-1L-1R	1.569	1974
	TOTAL FOR LATERAL L-1R		9.363	
	L-2L		3.500	1974
		L-2L-1R	2.880	1974
	TOTAL FOR LATERAL L-2L		6.380	
	L-3L		0.835	1978
		L-3L-1R	3.173	1978
		L-3L-1R-1R	3.500	1979
		L-3L-1R-1L	1.920	1978
		L-3L-1L	2.020	1978
		L-3L-1L-1L	1.900	1978
		L-3L-1L-2L	6.320	1979
		L-3L-1L-2L-1R	2.155	1979
		L-3L-1L-2L-1L	2.000	1979
	TOTAL FOR LATERAL L-3L		23.823	

Table 4. continued

Canal	Lateral	Sub-Lateral	Length, km	Year Constructed
	L-4L		0.530	1974
	L-5L		1.600	1975
	L-2R		3.010	1973
		L-2R-1L	1.250	1973
	TOTAL FOR LATERAL L-2R		4.260	
	L-3R		2.500	1974
	L-4R		1.500	1974
	L-5R		1.650	1974
		GRAND TOTAL	100.356	

structure gate setting. For each structure, a number of discharge measurements have been made. The results from each discharge measurement were compared in order to determine the gate opening correction that would provide a constant discharge coefficient for the entire operational range of the structure.

Seepage Loss Rates

The inflow-outflow method was used to determine the seepage loss rates. Seepage loss rates for the RMC were determined by flow rate and depth measurements made in each reach. Seepage loss rates for the LMC have been measured for all reaches along the main canal, in each lateral, and in each sub-lateral. The distance between two control structures define the length of the reach. The flow measurement at each control structure is the outflow from one reach, and the inflow to the next reach downstream. The wetted perimeter at different points along each reach can be calculated from measurement of depth. Seepage loss rates were determined by using Equation 1. The seepage loss rates have not yet been determined in the laterals and sub-laterals of the RMC. The seepage loss rate will vary according to the depth of water in the canal; therefore, the seepage loss rates are measured for various discharge releases from the outlet works at the dam.

Hydraulic Roughnesses

The hydraulic roughnesses were determined from the RMC head regulator to the check structure at km 22+662 using the "ROUGH" utility program of the hydraulic model package. The reach length, inflow and outflow discharge rates, upstream and downstream flow depths, and irrigation channel geometry (longitudinal slope, side slopes, and the bed width) of each reach were measured. The computer program solves the gradually varied flow equation iteratively until the computed upstream flow depth matches the observed depth. This approach is required because of the extensive backwater effects in most reaches, so that non-uniform flow conditions exist along the full length of each reach. In the LMC, the determination of variation in hydraulic roughness along each reach has not yet been done.

Time Lags

This is defined as the time for water to flow from the RMC head regulator to each lateral head regulator or turnout along the main canal. To observe lag time, a person should be placed at the staff gage of the laterals or turnouts along the canal. The RMC head regulator gate opening will be increased. The increasing amount of water will travel along the canal causing the water levels to increase with time. The person will record the water level and time until there is no change in water level, which is then steady state flow. The lag time for each

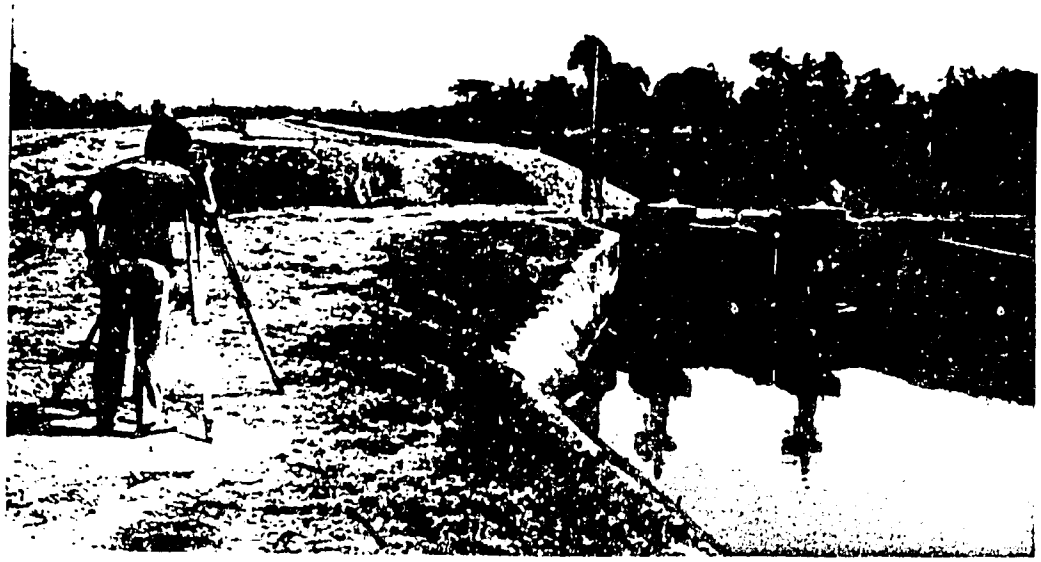


Photo 5. Field Survey of Canal Elevations.

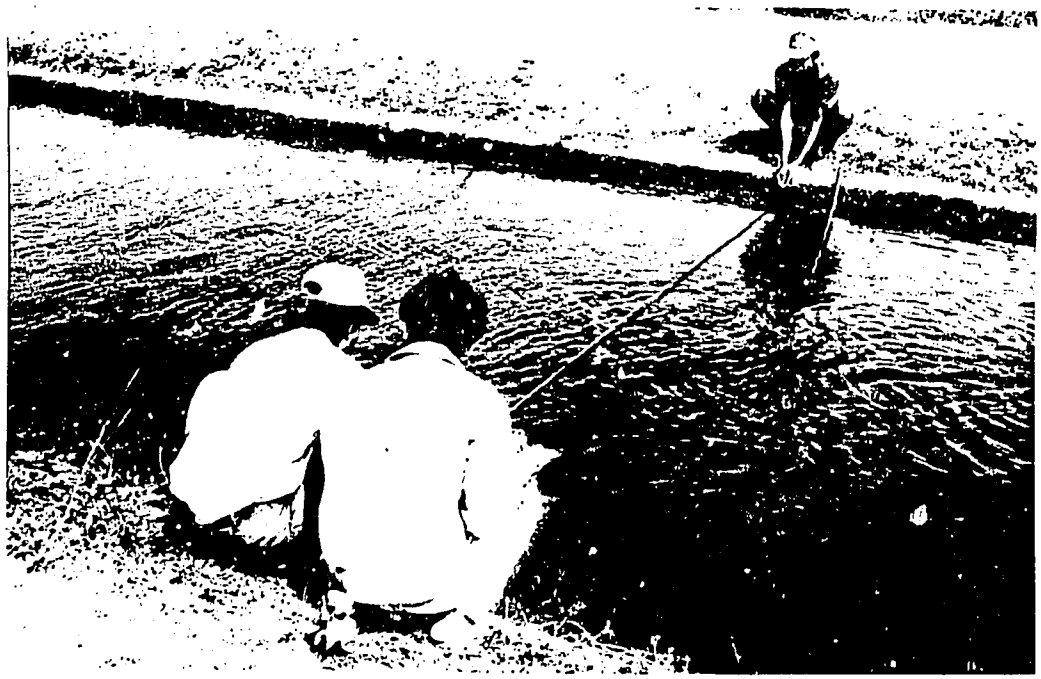


Photo 6. Measuring the Top width of Flow in a Canal Reach.

Photo 7. Calibration Activities for the Head Regulator of Right Main Canal.

Photo 8. Turnout Structure Calibration Activities.

lateral head regulator along the RMC was measured in 1984. For the LMC system, the time lags have not been measured yet.

Model Calibration

Only the first six reaches of the RMC have been completely calibrated. The hydraulic data is still needed for the laterals and sub-laterals served by the RMC. All of the hydraulic data for the LMC, including laterals and sub-laterals, are in the computer and only survey data is needed in order to calibrate the entire LMC system. But before conducting the calibration work, the data for model installation should be collected as follows:

1. Collect configuration data (Table 5). The side slopes, bottom widths, canal depths and reach lengths for each reach should be measured. These data can be obtained from the design drawings, but they should be checked in the field.

2. Bed elevations survey (Table 6). The steps in doing the bed elevations survey are as follows:

   a. Survey the bed elevations at intervals of 100 meters;
   b. Control structure sill elevations must be surveyed with an engineers level;
   c. Control structure lengths should be measured with a tape;
   d. Perform linear regression to find the appropriate bed elevation slope;
   e. The bed elevation should be plotted on rectangular graph paper, and the average slope for each reach based on the linear regression should be drawn on the graph paper; and
   f. Determine the bottom elevation change at each control structure using the plotted data.

3. Collect control structure data (Table 7). The type of control structure, the dimensions, and the number of the structures should be identified. The location from the water source and any other pertinent observations should be recorded.

4. Collect turnout structure data (Table 8). The type of turnout (also called main system outlet) structure and the dimensions should be identified. The sill height and the location from the beginning of the reach to the turnout structure should also be identified, as well as the outlet bed elevation.

5. Calibration of control structures. The discharge coefficient for the head regulator and each control structure must be calibrated and recorded by using a consistant format. The seepage loss rates and hydraulic roughnesses should also be measured.

TABLE 5. Configuration Data for the Right Main Canal of the Lam Nam Oon Irrigation Project in Thailand.

Brnch No.	Rch No.	Lngth (m)	Base (m)	Side (m/m)	Depth (m)	Slope (m/100m)	Rough (n)	Seep (mm/day)	OSL (%)	Q_Des (cms)
1	1	1400	5.0	1.50	2.85	0.0155	0.020	750	82	14.4
1	2	3340	5.0	1.48	2.85	0.0125	0.020	750	82	14.4
1	3	5326	4.0	1.51	2.85	0.0152	0.023	750	84	12.9
1	4	41[illegible]	4.0	1.29	2.85	0.0129	0.032	750	84	12.9
1	5	4498	4.0	1.50	2.65	0.0125	0.019	750	85	11.5
1	6	3957	4.0	1.50	2.60	0.0128	0.049	750	85	11.0
2	1	3469	4.0	1.50	2.45	0.0125	0.020	750	84	9.6
2	2	5446	4.0	1.50	2.40	0.0125	0.020	750	83	9.0
2	3	2175	3.5	1.50	2.15	0.0125	0.020	750	84	6.8
2	4	1845	3.0	1.50	2.15	0.0125	0.020	750	84	6.1
2	5	3088	3.0	1.50	2.05	0.0125	0.020	750	83	5.4
2	6	565	2.0	1.50	1.70	0.0125	0.020	750	85	3.0

Table 6. Canal Bed Elevations
Right Main Canal Lam Nam Oon Project
(Based on Regression of As-Constructed Data)

Station (m)	Structure	Upstream Elevation (msl)	Dwnstream Elevation (msl)	Elevation Difference (m)
0 + 000	Head Reg	-------	175.210	-----
1 + 400	Check #1	174.993	174.676	+0.317
4 + 740	Check #4	174.257	174.227	+0.030
10 + 066	Check #10	173.418	173.242	+0.176
14 + 207	Check #14	172.702	172.739	-0.037
18 + 705	Check #18	172.179	171.960	+0.219
22 + 662	Check #22	171.456	-------	-----
0 + 400	Bridge	175.148	-------	-----
2 + 170	R - 1L	174.579	-------	-----
4 + 195	R - 2L	174.325	-------	-----
5 + 760	R - 3L	174.072	-------	-----
8 + 882	R - 4L	173.607	-------	-----
11 + 935	R - 6L	172.998	-------	-----
18 + 895	R - 7L	171.936	-------	-----
19 + 495	R - 8L	171.859	-------	-----
21 + 515	R - 9L	171.602	-------	-----

TABLE 7. Control Structure Data for the Right Main Canal of the Lam Nam Oon Irrigation Project in Thailand.

Brnch No.	Rch No.	Dist (m)	Reach Name	No.	Width (m)	Disch Coeff	Delta "b" (m)	Drop (m)
1	1	1+400	Check 1	2	2.30	0.40	-----	+0.317
1	2	4+740	Check 4	3	1.75	0.68	-0.020	+0.030
1	3	10+066	Check 10	3	1.75	0.82	-0.030	+0.176
1	4	14+207	Check 14	3	1.75	0.76	-0.045	-0.037
1	5	18+705	Check 18	2	2.00	0.78	-0.042	+0.219
1	6	22+662	Check 22	2	2.00	0.68	+0.000	-0.030
2	1	26+131	Check 26	2	2.00	0.61	+0.000	+0.290
2	2	31+577	Check 31	2	1.75	0.72	+0.000	+0.000
2	3	33+752	Check 33	3	1.50	0.45	+0.000	+0.270
2	4	35+597	Check 35	2	1.75	0.81	+0.000	+0.370
2	5	38+685	Check 38	1	1.75	0.55	+0.000	-0.140
2	6	39+250	Check 39	1	1.00	0.67	+0.000	-0.550

TABLE 8 . Turnout Structure Data for the Right Main Canal of the Lam Nam Oon Irrigation Project in Thailand.

Brnch No.	Rch No.	Tnout No.	Name	Cd	Width (m)	Height (m)	Posn (m)	Distn (m)	DS Depth	DS Slope
1	1	1	WW 1	1.83	3.00	----	2.40	400	----	----
1	1	2	1	0.55	0.60	0.60	1.51	500	0.00	0.00
1	1	3	2	0.55	0.60	0.60	1.60	545	0.00	0.00
1	1	4	3	0.59	0.60	1.25	1.57	1310	0.00	0.00
1	2	1	4	0.61	0.60	0.60	1.81	470	0.00	0.00
1	2	2	R-1L	0.61	2.50	1.25	0.49	770	0.49	0.32
1	2	3	5	0.61	0.60	0.60	1.56	1790	0.00	0.00
1	2	4	6	0.61	0.60	0.60	1.74	2410	0.00	0.00
1	2	5	R-2L	0.60	0.80	0.80	1.13	2795	1.04	0.10
1	2	6	7	0.61	0.60	0.60	1.73	2852	0.00	0.00
1	2	7	WW 2	1.83	9.65	----	2.65	3040	----	----
1	2	8	8	0.61	0.60	0.60	1.60	3270	0.00	0.00
1	3	1	9	0.61	0.60	0.60	1.84	610	0.00	0.00
1	3	2	R-3L	0.55	0.60	0.60	1.12	1020	1.03	0.10
1	3	3	10-11	0.61	1.20	0.60	1.50	1835	0.00	0.00
1	3	4	12-13	0.61	1.20	0.60	1.33	3245	0.00	0.00
1	3	5	14	0.57	0.60	0.60	1.89	3860	0.00	0.00
1	3	6	R-4L	0.67	2.00	1.00	0.79	4082	0.33	0.56
1	3	7	WW 3	1.83	9.62	----	2.69	4660	----	----
1	3	8	15	0.57	0.60	0.60	0.07	4960	0.00	0.00
1	3	9	16	0.57	0.60	0.60	1.38	5285	0.00	0.00
1	4	1	17	0.57	0.60	0.60	0.06	1034	0.00	0.00
1	4	2	18	0.57	0.60	0.60	-0.02	1484	0.00	0.00
1	4	3	R-6L	0.71	2.50	1.25	0.58	1869	0.55	0.08
1	4	4	19	0.57	0.60	0.60	0.02	2934	0.00	0.00
1	4	5	20	0.57	0.60	0.60	0.00	3534	0.00	0.00
1	4	6	WW 4	1.83	9.66	----	2.60	3774	----	----
1	5	1	21	0.57	0.60	0.60	1.67	735	0.00	0.00
1	5	2	22	0.61	0.60	0.60	1.62	2593	0.00	0.00
1	5	3	23	0.57	0.60	0.60	1.68	3193	0.00	0.00
1	5	4	24	0.61	0.60	0.60	0.00	3743	0.00	0.00
1	5	5	WW 5	1.83	8.62	----	2.44	4393	----	----
1	6	1	R-7L	0.64	2.00	1.00	0.64	190	0.07	0.72
1	6	2	25	0.61	0.60	0.60	1.16	645	0.00	0.00
1	6	3	R-8L	0.48	1.00	1.00	0.63	790	0.09	0.22
1	6	4	26	0.61	0.60	0.60	1.30	1515	0.00	0.00
1	6	5	27	0.57	0.60	0.60	1.72	2495	0.00	0.00
1	6	6	R-9L	0.54	2.00	1.00	0.87	2810	0.11	0.88
1	6	7	28	0.57	0.60	0.60	1.54	3895	0.00	0.00

TABLE 8 . (cont) Turnout Structure Data for the Right Main Canal of the Lam Nam Oon Irrigation Project in Thailand.

Brnch No.	Rch No.	Tnout No.	Name	Cd	Width (m)	Height (m)	Posn (m)	Distn (m)	DS Depth	DS Slope
2	1	1	WW 6	1.83	10.44	----	2.05	538	----	----
2	1	2	29	0.61	0.60	0.60	1.32	688	0.00	0.00
2	1	3	R-10L	0.61	0.60	0.60	1.01	854	0.50	0.10
2	1	4	30	0.61	0.60	0.60	1.39	888	0.00	0.00
2	1	5	31	0.61	0.60	0.60	1.39	1338	0.00	0.00
2	1	6	R-11L	0.39	2.00	1.00	0.48	1814	-0.12	0.46
2	1	7	32	0.61	0.60	0.60	1.41	2688	0.00	0.00
2	1	8	WW 7	1.83	9.20	----	2.22	3318	----	----
2	2	1	R-12L	0.61	0.90	0.90	0.09	271	0.01	1.41
2	2	2	34	0.61	0.60	0.60	1.35	419	0.00	0.00
2	2	3	35-36	0.61	1.20	0.60	1.36	1119	0.00	0.00
2	2	4	37-38	0.61	1.20	0.60	1.33	2465	0.00	0.00
2	2	5	WW 8	1.83	9.26	----	2.19	3469	----	----
2	2	6	39-40	0.61	1.20	0.60	1.34	3609	0.00	0.00
2	2	7	41	0.61	0.60	0.60	0.02	4064	0.00	0.00
2	2	8	42	0.48	0.60	0.60	0.09	5409	0.00	0.00
2	3	1	43	0.48	0.60	0.60	0.99	243	0.00	0.00
2	3	2	R-13L	0.37	0.90	0.90	-0.30	285	-0.30	0.10
2	3	3	44	0.61	0.60	0.60	1.10	823	0.00	0.00
2	3	4	45	0.61	0.60	0.60	1.17	1363	0.00	0.00
2	3	5	WW 9	1.83	6.54	----	1.97	1743	----	----
2	4	1	46	0.61	0.60	0.60	1.12	278	0.00	0.00
2	4	2	47	0.61	0.60	0.60	1.09	748	0.00	0.00
2	4	3	48	0.61	0.60	0.60	1.22	1298	0.00	0.00
2	5	1	49	0.61	0.60	0.60	1.65	63	0.00	0.00
2	5	2	R-14L	0.58	0.60	0.60	0.92	135	0.20	0.10
2	5	3	50-51	0.55	1.20	0.60	1.02	403	0.00	0.00
2	5	4	R-15L	0.69	1.00	1.00	0.16	526	0.00	0.10
2	5	5	52-53	0.55	1.20	0.60	0.10	1228	0.00	0.00
2	5	6	WW 10	1.83	5.03	----	1.83	1743	----	----
2	5	7	54-55	0.48	1.20	0.60	0.88	2153	0.00	0.00
2	5	8	R-16L	0.63	2.50	1.25	0.35	2503	0.29	0.43
2	5	9	56-57	0.53	1.20	0.60	0.93	2706	0.00	0.00
2	6	1	58	0.48	0.60	0.60	0.69	165	0.00	0.00
2	6	2	R-17L	0.61	2.40	1.20	0.54	222	0.28	0.15
2	6	3	59	0.48	0.60	0.60	0.70	515	0.00	0.00

Notes: Turnout names beginning with "R-" are lateral turnouts. Turnout names beginning with "WW" are wasteway weirs, and all others are farm turnouts. Lateral R-5L does not exist in the real system although it was included in the original canal system design.

6. Calibration of farm turnout structures (FTO's). Each FTO structure must be calibrated in the field to determine the discharge coefficient. Due to the tremendous number of turnouts, representative structures should be selected for calibration, and the results can be extrapolated to other structures. The same discharge coefficients may be used for the same size turnout, and can be checked later as time permits.

7. Check structure calibrations, seepage loss rates and hydraulic roughnesses under steady-state conditions with either uniform or non-uniform flow.

8. Update the data on structure calibrations, seepage loss rates and hydraulic roughnesses regularly as needed.

The calibration data for the RMC was obtained from the head regulator at km 0+000 to the check structure at km 22+662. Table 6 shows the canal bed elevations upstream and downstream of the control structure based on the regression analysis of the as-constructed data. Table 9 shows the flow depths, which were calculated from the canal bed elevations, and the water surface elevation field data for the control structures and laterals. The data was collected on January 23, 1988 for non-uniform steady-state flow conditions. The computed Manning roughness, n, based on the computer program ROUGH is shown in Table 10. The comparison between the hydraulic model computed discharge and the daily actual reported discharge for the lateral head regulators and farm turnouts (FTO's)are listed in Table 11. The Manning roughness and gate correction determination by comparing the measured and computed upstream flow depths and gate settings are listed in Table 12.

Project Field Communication

At the Lam Nam Oon Irrigation Project there is a good communication network. The central radio station is located at the headquarters. Walky-talkies and the magneto-telephone system are used to communicate inside the project area. If the telephone system is out of order, then the walky-talky system can be used. And, problems usually occur with the telephone system during the rainy season, when it is very necessary to communicate because of flooding. At such times, the water levels along the canal, or the amount of rainfall, should be reported to the headquarters staff, so that appropriate action can be taken according to the situation.

Currently, communication using the walky-talky is widely used. In addition, a big antenna is located at the radio station office. In case the O & M Sections are far apart and would like to get in touch, the message will pass through the central radio station, which means that in every O & M Section the message can be sent forward and returned easily. Also, a side-band radio is located at the central radio station. Messages can be sent to, or received from, the Royal Irrigation

Table 9. 23 January 1988
Steady-State Flow Depths
Right Main Canal Lam Nam Oon Project

Station (m)	Name of Structure	Canal Bed Elevation (msl)	Water Surface Elevation (msl)	Flow Depth (m)
0 + 400	First Bridge	175.148	177.120	1.972
4 + 740	U/S Check #4	174.257	176.700	2.443
	D/S Check #4	174.227	176.070	1.843
10 + 066	U/S Check #10	173.418	175.770	2.352
	D/S Check #10	173.242	175.300	2.058
14 + 207	U/S Check #14	172.702	175.080	2.378
	D/S Check #14	172.739	174.400	1.661
18 + 705	U/S Check #18	172.179	174.300	2.121
	D/S Check #18	171.960	173.800	1.840
22 + 662	U/S Check #22	171.456	173.450	1.994
	D/S Check #22	-------	-------	-----
2 + 170	Lateral R-1L	174.579	176.870	2.291
4 + 195	Lateral R-2L	174.325	176.710	2.385
5 + 760	Lateral R-3L	174.072	176.030	1.958
8 + 882	Lateral R-4L	173.607	175.800	2.193
11 + 935	Lateral R-6L	172.998	175.170	2.172
18 + 895	Lateral R-7L	171.936	173.810	1.874
19 + 495	Lateral R-8L	171.859	173.720	1.861
21 + 515	Lateral R-9L	171.602	173.560	1.958

Table 10. **Computed Manning Hydraulic Roughness Right Main Canal Lam Nam Oon Project**

Reach No.	Reach Inflow (m3/s)	Reach Outflow (m3/s)	US Depth (m)	DS Depth (m)	Reach Length (m)	Bed Slope (m/m)	Manning Roughness "n"
3	7.98	6.91	2.003	2.565	5326	0.000152	0.0185
4	6.91	6.31	2.098	2.308	4141	0.000129	0.0260
5	6.31	5.79	1.651	1.971	4498	0.000125	0.0160
6	5.79	4.32	1.780	1.984	3957	0.000128	0.0245

Note: Based on data taken on September 19, 1987. Roughness values were calculated using a program which computed gradually varied flow profiles to match "M1" curves to the measured upstream depth in each canal reach. The roughness could not be computed directly since normal depth did not exist (i.e. backwater curves did not fully develop upstream of the checks).

Table 11. 23 January 88
Lam Nam Oon Right Main Canal
Steady-State Lateral and Turnout Discharges

Brnch	Rch	Turnout	Turnout Name	Turnout Setting (m)	Head Differ (m)	Gate Width (m)	Disch Coeffic	Computed Discharge (cms)	Reported Discharge (cms)
1	2	1	04	----	----	----	----	----	0.03
1	2	2	R-1L	0.15	0.59	2.50	0.61	0.78	0.77
1	2	3	05	----	----	----	----	----	0.03
1	2	4	06	----	----	----	----	----	0.03
1	2	5	R-2L	0.06	0.21	0.80	0.60	0.06	0.06
1	3	2	R-3L	0.11	0.28	0.60	0.55	0.09	0.09
1	3	3	11	----	----	----	----	----	0.03
1	3	4	13	----	----	----	----	----	0.03
1	3	5	14	----	----	----	----	----	0.03
1	3	6	R-4L	0.15	0.28	2.00	0.67	0.47	0.47
1	3	8	15	----	----	----	----	----	0.03
1	4	3	R-6L	0.22	0.70	1.25	0.71	0.72	0.77
1	4	5	20	----	----	----	----	----	0.03
1	5	4	24	----	----	----	----	----	0.03
1	6	1	R-7L	0.08	0.38	2.00	0.64	0.28	0.28
1	6	2	25	----	----	----	----	----	0.03
1	6	3	R-8L	0.10	0.19	1.00	0.48	0.09	0.09
1	6	4	26	----	----	----	----	----	0.03
1	6	5	27	----	----	----	----	----	0.03
1	6	6	R-9L	0.08	0.36	2.00	0.54	0.23	0.23
1	6	7	28	----	----	----	----	----	0.03
2	1	2	29	----	----	----	----	----	0.03
2	1	5	31	----	----	----	----	----	0.03
2	1	7	32	----	----	----	----	----	0.03
2	2	4	37	----	----	----	----	----	0.03
2	2	6	39	----	----	----	----	----	0.03

Note: CHO's opened discharge = 0.030 m^3/s

Table 12. **FINAL CALIBRATION RESULTS**

System Inflow = 7.23 m3/s
Lam Nam Oon Right Main Canal
Measured Values from January 23, 1988

Trial Number	Manning Roughness Reach#3	Measured US Depth Reach #3	Computed US Depth Reach #3	Measured Setting Reach #2	Computed Setting Reach #2	Gate Correction Reach #2
1	0.0195	1.843	1.770	0.41	0.386	-0.024
2	0.0210	1.843	1.800	0.41	0.396	-0.014
3	0.0220	1.843	1.820	0.41	0.403	-0.007
4	0.0225	1.843	1.832	0.41	0.465	+0.055
5	0.0230	1.843	1.844	0.41	0.470	+0.060

Trial Number	Manning Roughness Reach#4	Measured US Depth Reach #4	Computed US Depth Reach #4	Measured Setting Reach #3	Computed Setting Reach #3	Gate Correction Reach #3
1	0.0260	2.058	1.989	0.40	0.344	-0.056
2	0.0265	2.058	1.995	0.40	0.348	-0.052
3	0.0270	2.058	1.999	0.40	0.349	-0.051
4	0.0285	2.058	2.016	0.40	0.355	-0.045
5	0.0295	2.058	2.026	0.40	0.359	-0.041
6	0.0310	2.058	2.043	0.40	0.365	-0.035
7	0.0320	2.058	2.054	0.40	0.369	-0.031
8	0.0322	2.058	2.056	0.40	0.370	-0.030
9	0.0323	2.058	2.057	0.40	0.370	-0.030

Trial Number	Manning Roughness Reach#5	Measured US Depth Reach #5	Computed US Depth Reach #5	Measured Setting Reach #4	Computed Setting Reach #4	Gate Correction Reach #4
1	0.0160	1.661	1.631	0.30	0.250	-0.050
2	0.0200	1.661	1.670	0.30	0.257	-0.043
3	0.0180	1.661	1.650	0.30	0.253	-0.047
4	0.0190	1.661	1.660	0.30	0.255	-0.045

Trial Number	Manning Roughness Reach#6	Measured US Depth Reach #6	Computed US Depth Reach #6	Measured Setting Reach #5	Computed Setting Reach #5	Gate Correction Reach #5
1	0.0244	1.840	1.602	0.39	0.286	-0.104
2	0.0350	1.840	1.701	0.39	0.307	-0.083
3	0.0450	1.840	1.802	0.39	0.335	-0.055

Photo 9. The Central Radio Station at the Project Headquarters .

Photo 10. Telephone Communication Within the Central Radio Station.

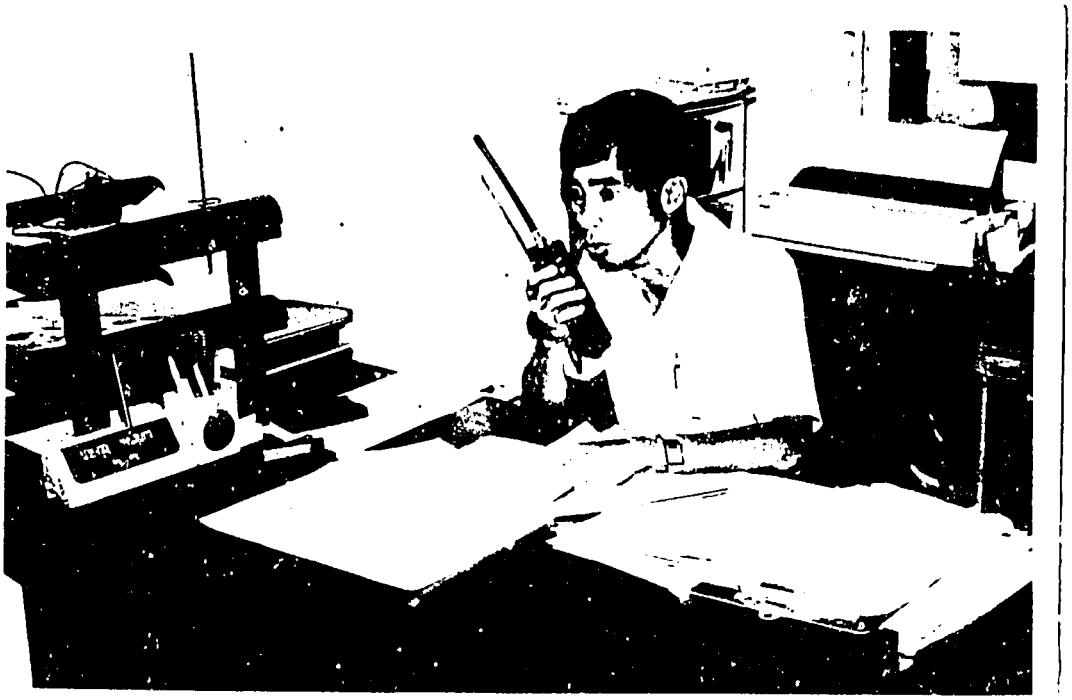


Photo 11. Walky-Talky Communication Between Project Management and Field Personnel.

Photo 12. Telephone Communication Within the Project Headquarters.

Department Office in Bangkok, or to any other irrigation project in Thailand. If the distance is too great, then the radio message must be passed through the central radio station in Bangkok. Communication in the Operation and Maintenance Section at a project headquarters is usually done by telephone; the speaker must call the operator at the project headquarters radio station first, then the operator will make the necessary connection. The messages that are sent in this office are mostly regarding administrative work.

Communication within the O & M Section, the Project Engineer, and the Chief of the Water Management Section (who supervises the main system operations on behalf of the Project Engineer) will be by walky-talky, which each of them carry at all times in case of an urgent message. The message will be sent from them through the central radio station to the appropriate field O & M Section, or directly to the field O & M Section; therefore, immediate action can be undertaken. There are three O & M Sections along the RMC and two O & M Sections along the LMC, each of them having a radio station. Communicating between two field O & M Sections is very easy if they are not very far apart. In case the two O & M Sections are far apart, the message must be sent through the project headquarters radio station. Sometimes, a message is communicated to all of the field O & M Sections when there is some change in the operations, which may affect all of the field O & M Sections.

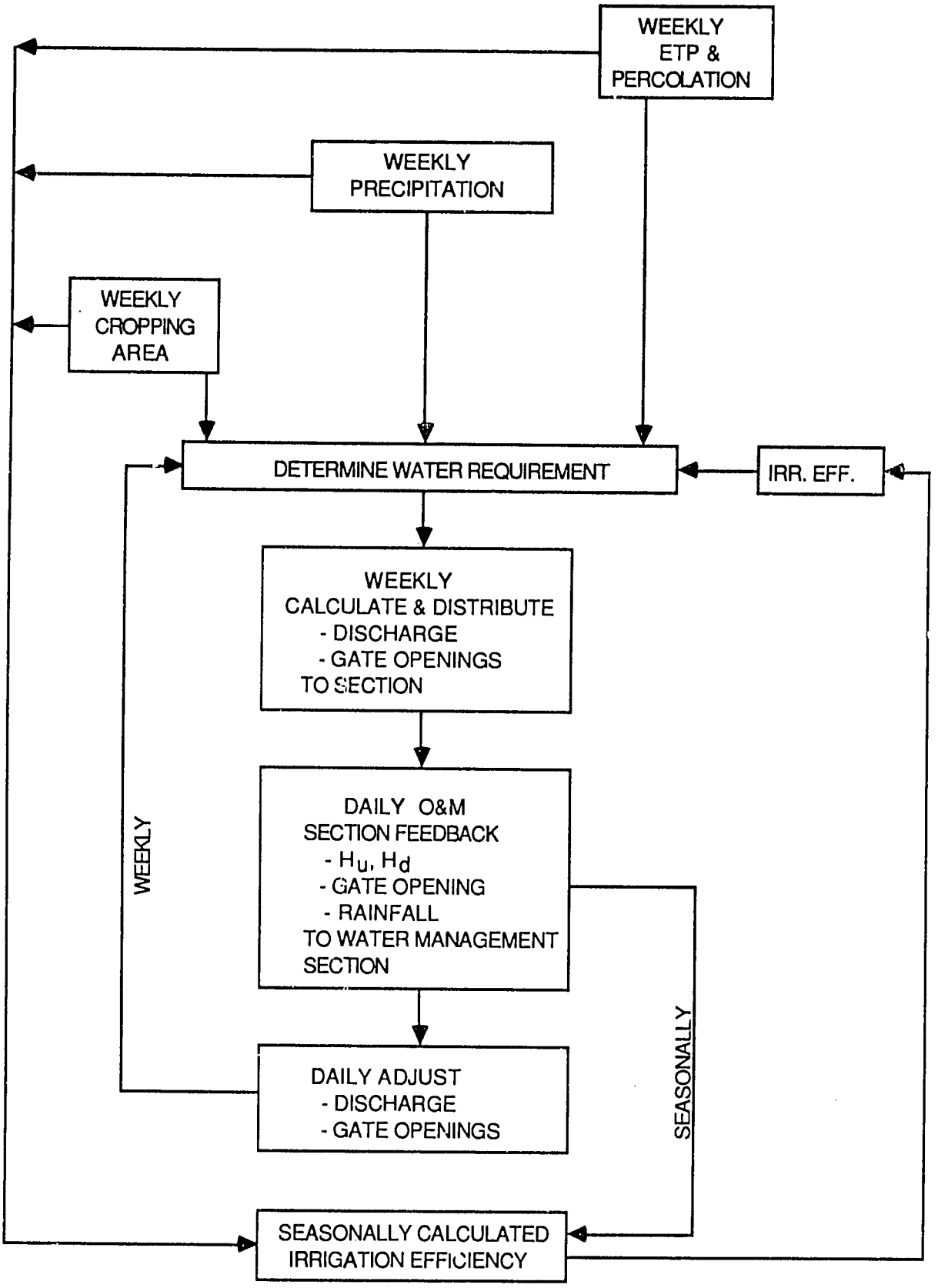
Weekly and Daily Computer Operations

Wet Season Operations

Rice cultivation is predominant during the wet season. Before the wet season, the irrigation schedule is predicted by using information from the previous year. The curves from the previous year for land preparation area, rice nursery area, and transplanting area versus time will be used. In addition, the average precipitation and the assumed irrigation efficiency are also used to predict the crop water requirements.

Dry Season Operations

In the dry season, rice and upland crops are cultivated. The crop type and cropping area should be surveyed in advance before the dry season, so that the irrigation schedule can be planned. The water requirement determination will be made in the same way as for the wet season (Figure 7).



$$\text{IRRIGATION EFFICIENCY} = \frac{\text{WATER REQUIREMENT - EFFECTIVE RAINFALL}}{\text{ACTUAL WATER DELIVERY}}$$

Figure 7. Weekly Operation of the Right Main Canal at the Lam Nam Oon Irrigation Project.

Weekly Wet and Dry Season Operation of the RMC

For the first week of the dry season, the water requirement will be applied according to the previous year's information as explained above. For the following week, the actual land preparation area, type of crop and land area under each outlet, the nursery area, and transplanting area will be determined in the field to update the cropping area. This information will be inputed to the micro-computer to update the weekly water requirement at the RMC head regulator and the lateral head regulators. The gate openings for the head regulators and control structures can be determined by using the hydraulic model, and the results can be sent to the O & M Section office by walky-talky. The head of the section (Water Master) will inform each Zoneman of the gate openings in his area of responsibility. The Zoneman will go out to the field and adjust the gate openings accordingly. The water levels and discharge rates in the irrigation channels will remain constant for about one week.

The gate opening must be adjusted again only in the case of rainfall. The new discharge will be inputed to the hydraulic model to determine the new gate openings. The new gate openings will be sent by radio to the O & M Section offices again. The irrigation schedule will be repeated as a cycle throughout the season according to the cropping area, stage of growth, and rainfall.

Daily Wet and Dry Season Operation of the LMC

The system operations for the Left Main Canal are the same as for the Right Main Canal. But the irrigation schedule will be different because the Left Main Canal is operated by pumps in some reaches. The pumping stations are located at Lateral L-1L (km 5+900), Lateral L-3L (km 0+211) and along the LMC at km 16+000. Water is not distributed during the night time. When the gate i closed at the dam during the afternoon, then the gates throughout the channel network are operated to maintain high water levels. By operating in this manner, the time required to fill the channels is minimized the next morning when the LMC head regulator is opened. The water levels and gate openings are reported to the water management office every day. After the end of each season, the irrigation efficiency will be calculated and used for the next season.

Photo 13. A Zoneman Collecting Weekly Cropping Area.

Photo 14. Zonemen Summarizing the Weekly Cropping Area in the Section Office.

Photo 15. A Zoneman Submitting the Weekly Cropping Area to the Water Master at the Operation and Maintenance Section.

Photo 16. The Operator at the O & M Section Sending Cropping Area Information Through Walky-Talky to the Central Radio Station.

Photo 17. An Operator at the Central Radio Station Receiving and Distributing Messages.

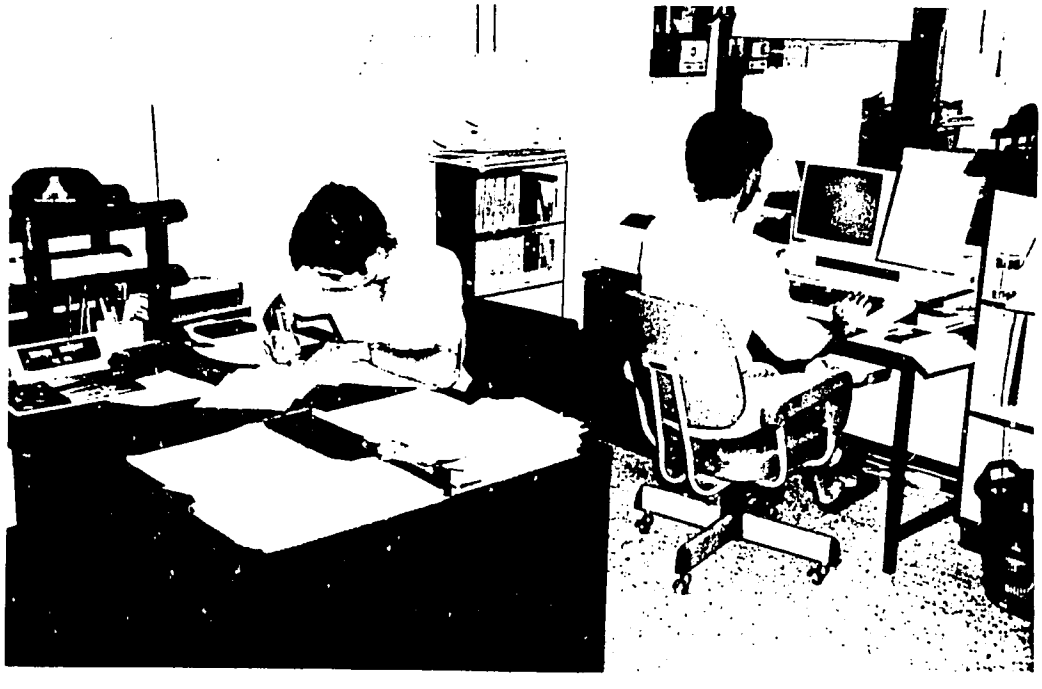


Photo 18. Using the Predicted Cropping Area for Next Week to Predict the Water Demand.

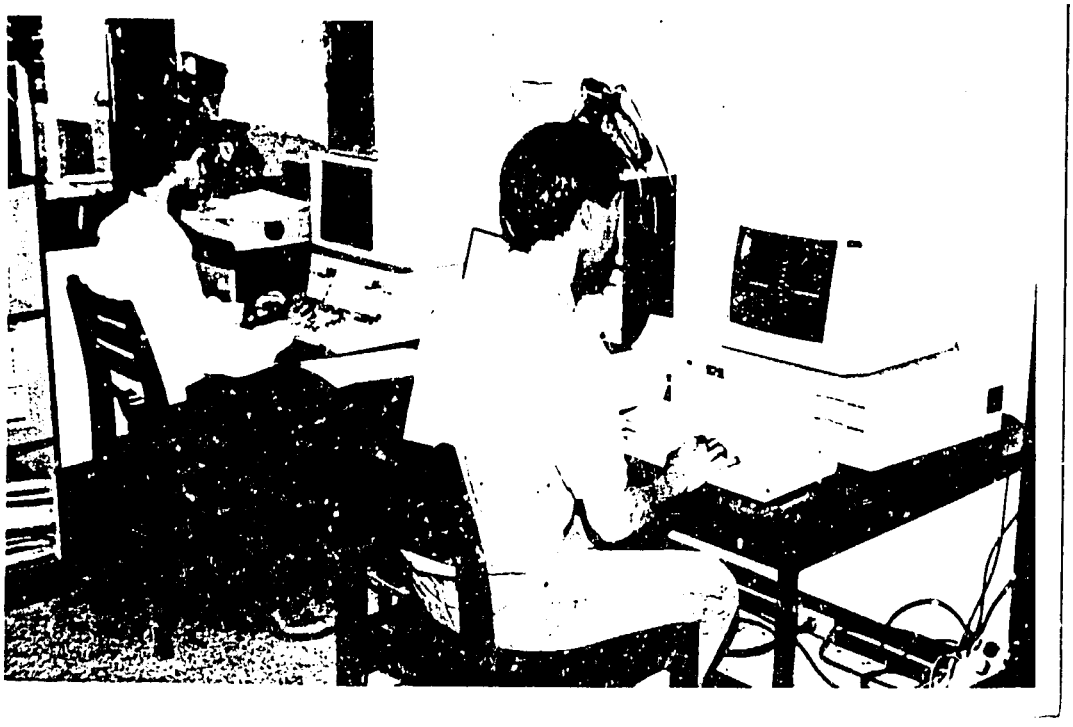


Photo 19. Inputting the Water Demand for the Main Canal, Laterals and Turnouts to the Hydraulic Model.

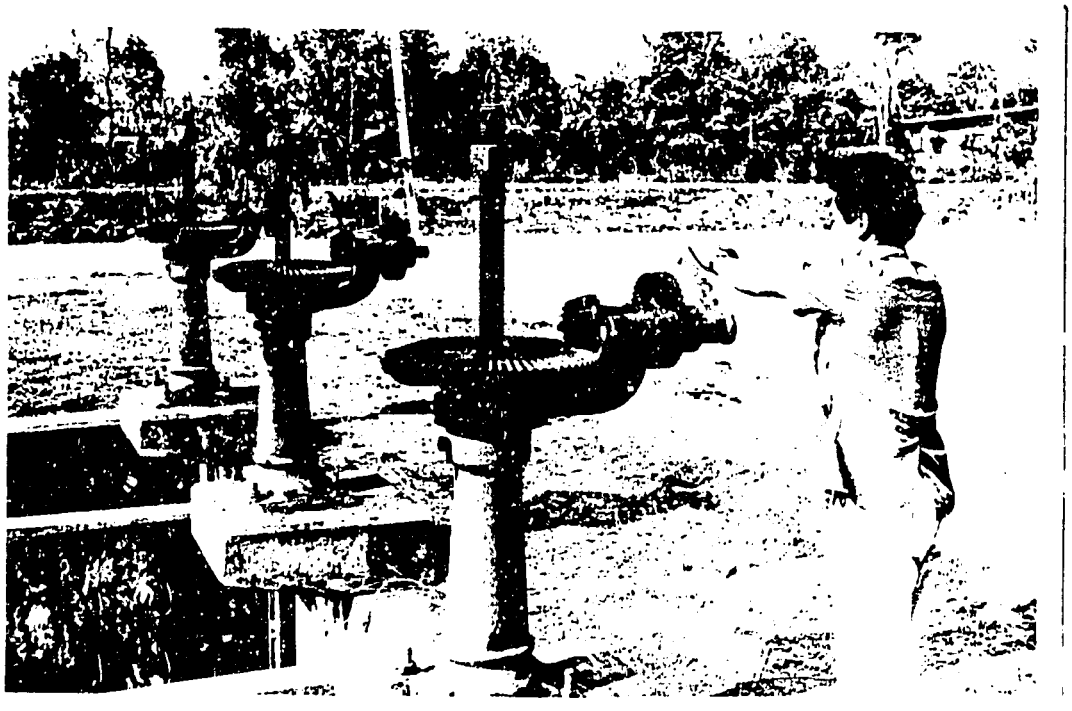


Photo 20. A Zoneman Opening a Control Structure According to Simulation Results from the Model.

Photo 21. Zoneman Adjusting a Turnout Structure.

Photo 22. Zoneman Recording the Depth of Water at a Control Structure.

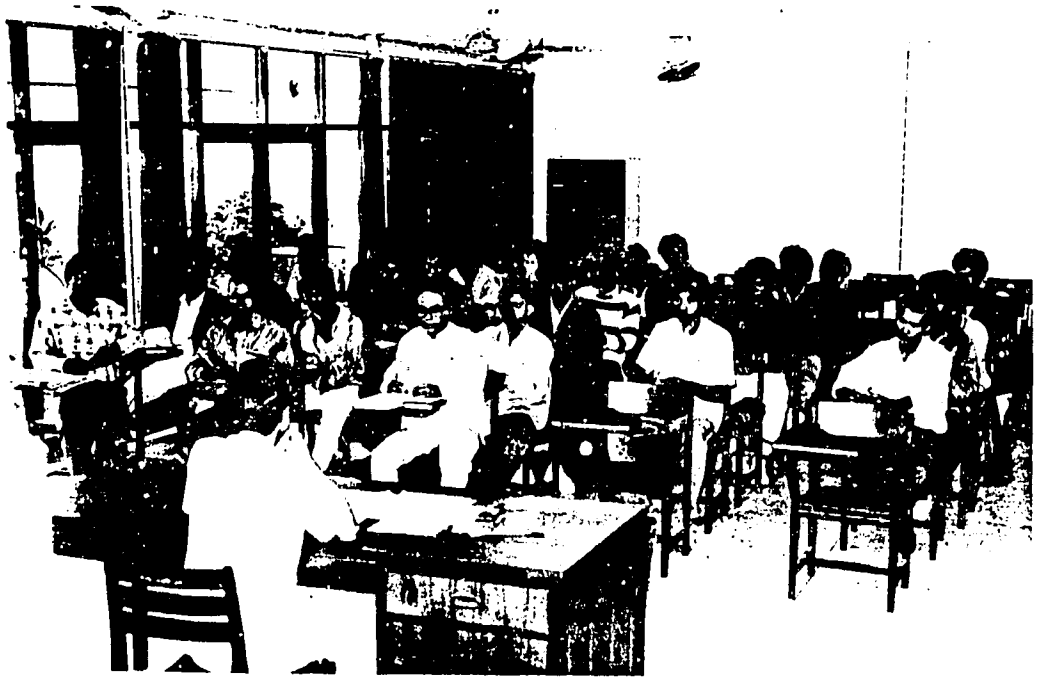


Photo 23. The Weekly Meeting for Water Masters and Zonemen at the Project Headquarters.

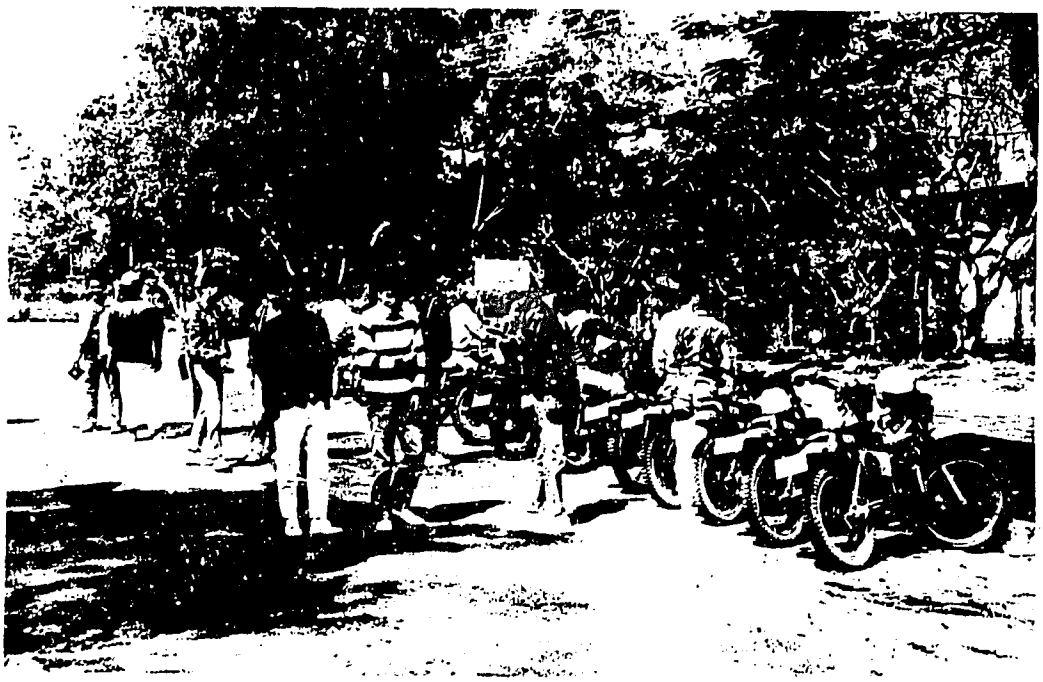


Photo 24. Zonemen Returning to the Field After a Meeting.

IMPLEMENTING COMPUTER-ASSISTED IRRIGATION SYSTEM MANAGEMENT IN OTHER COUNTRIES

Background

Under USAID's centrally funded project, "Water Management Synthesis II," the Department of Agricultural and Irrigation Engineering at Utah State University has been developing computer software to assist in the planning, design, and operations of irrigation projects. Four submodels have been under development, with two of them completed. This report describes only one of the models. Parallel to the USU studies, several agencies and consulting companies have been moving toward computerized irrigation system management in one respect or another.

The two completed USU models, the unit command area model and the main systems hydraulic management model, are being implemented on a trial basis on two irrigation projects in the Northeast of Thailand, as described in the previous two sections of this report. Along with the models, the project developed six short video programs outlining the concepts related to the models' functionality and their use. Project leaders felt the development had reached a stage where it was appropriate and timely to demonstrate the package to selected USAID Missions and host country personnel. Presentations were made in Thailand, India, Pakistan, Sri Lanka, and Egypt during the time period June 6 to July 17, 1986.

These seminar presentations encompassed three items:

1. A strategy for identifying and implementing institutional changes necessary to provide manpower levels and stability necessary to put computerized practices into place;

2. A strategy for selecting personnel for specialized training and afterwhich who would be located in implementation positions; and

3. A strategy for the necessary training activities.

Each of the seminar objectives were discussed and followed by a detailed analysis of a process for achieving computer-assisted irrigation system management practices. The writers believe that successful computer implementation is highly dependent on incorporating the computer-based analysis into the regular operation and maintenance regime of the irrigation agency (Irrigation Maintenance and Operations Learning Process) as well as that of the water users below the agency controlled turnout (tertiary system management and on-farm water management). Thus, a three tier program is proposed. The first is the implementation of the computer system itself, including equipment acquisition, personnel training, software adaptation, field installation, and technical backup. The second is the implementation of a main system operations and maintenance program. Finally, the writers

propose a program for implementing improved unit command area (tertiary system) water management practices. The attached flowchart shows the elements of this program and their relationship to each component. This report summarizes this as a strategy with particular emphasis on the computer system implementation and only general focus on the other two aspects. One will also note that the process outlined in the flowchart sketches our view of the entire scheme for irrigation system management of which the training pre-requisites are our primary interest and capability.

To reference these suggestions to a cost framework, the writers have made a crude budget covering the training aspects, both in the field and in the U.S.A. An actual budget can be developed in cooperation with any agency that wants to implement Computer-Assisted Irrigation System Management.

Potential Computer Uses

At the outset, the reader is alerted that reference to computers really refers to what are called microcomputers. Machine capability in terms of memory, execution speed, language, operating systems, and peripheral capacity are now sufficient to handle nearly every water management need, even at the highest levels of theoretical detail and structural complexity. One can hypothesize the list of tasks computers might be used to perform, the personnel skills needed, the field data collection and communication involved, and the costs. It is also possible to identify software already available, under development, or needed to make the computerization a practical reality. What cannot be determined, except on a case-by-case basis, is the best use given the unique features of the irrigation agency and the individual irrigation project.

Both short and long term computer options are needed. Short term options include: (1) those immediate uses in irrigation offices such as word processing and personnel records; (2) programs for real time irrigation systems operations, particularly in the conveyance system; (3) water balance forecasts in the river-canal network; and (4) flood forecasting and river management. Their long term uses would include: (1) crop water requirements and irrigation scheduling; (2) optional allocation of water to farms during periods of shortage; and (3) economic analyses of cropping patterns under water stress. In essence, the division of short and long term interests is very decidedly a top down approach. The upper end modeling is, therefore, to be developed and implemented without any realistic estimates of the agricultural water demands. The USU models are bottom-up in nature and, therefore, may allow the long term objectives above to become short term opportunities since the necessary software is already available.

The writers believe the list of computer uses listed above can be developed and implemented. It does not seem overly ambitious in terms of scope; in fact, there are a multitude of additional possibilities

such as data base management. However, there is an obvious problem in the writers' opinion with regard to the requirements for incorporating microcomputers into an institution's operational framework. For instance, simply installing a microcomputer in an irrigation office would not lead to any of the uses noted above in any meaningful way. During the advent of microcomputers in the U.S.A., the lag between introduction of the hardware and its productive use was, in some cases, as much as five years. By then, equipment became obsolete. Looking back, mocrocomputers were used in very limited ways until software was written and users became accustomed to the microcomputer environment.

Irrigation departments in other countries need not repeat this expensive experience if they will take two important steps. first, selected personnel need to be sent abroad for about one year of detailed training. Secondly, the irrigation project staff will need to establish an operation and maintenance regime that naturally and continually feeds the data needed to the computers to be of assistance in water management. The latter also requires U.S. and in-field training exercises. Both of these training components were outlined to personnel present at the seminars and are summarized in the following subsections.

Development of Microcomputer Expertise

An effective microcomputer application in today's irrigation system requires a small cadre of trained specialists who understand the machine and its operating system as well as the concepts of irrigation water management. The microcomputer specialists need to use several programming languages, the use of special purpose software which convert irrigation concepts into functional analyses, and the remedy for numerous problems, which occur within the machine, its operating system, and the software it uses. Why such a broad specialization? Because, first, irrigation systems management spans several disciplines like agricultural engineering, agronomy, economics, civil and irrigation engineering, and institutional administration and management, and secondly, because until the manpower levels in an institution are upgraded to utilize computers in special tasks, these first specialists must be able to provide a vital assistance to numerous people. This is not to imply that the training proposed by the writers will allow the specialist to become fully trained in each discipline, but that he or she will be capable of bringing the technical personnel together with the capacities and opportunities of the machine, and then be available to resolve their particular differences and problems.

Table 13 shows a training program developed by the writers as a result of the seminars. It was at first simply a "what-we-would-do-if" exercise, but after visiting other countries, it has evolved into a program of 49 weeks duration offered by the International Irrigation Center at Utah State University. It integrates six existing Center short courses:

Table 13. **TRAINING COURSE (49 WEEKS) ON COMPUTER-ASSISTED IRRIGATION SYSTEM MANAGEMENT INTERNATIONAL IRRIGATION CENTER UTAH STATE UNIVERSITY, LOGAN, UTAH**

JANUARY 8 - DECEMBER 16, 1989

Revised March 1988

Topic	Dates	Place	Cost
* Applied Microcomputer Use in Irrigation & Drainage	Jan. 8-Feb. 18 (6 weeks)	Logan, UT	3,575
*Computerized Operation of Irrigation Delivery Systems Language	Feb. 19-Mar. 4 (2 weeks)	Logan, UT	1,175
Microcomputer Operating Systems and Programming Language	Mar. 5-May 6 (9 weeks)	Logan, UT	4,575
*On-Farm Irrigation Design and Evaluation	May 7-June 17 (6 weeks)	Logan, Delta, UT Grand Jct. CO	3,775
*On-Farm Irrigation Scheduling	June 18-July 8 (3 weeks)	Alamosa, CO	1,975
*Main System Irrigation Scheduling	July 9-July 29 (3 weeks)	Logan & Delta UT	1,975
Microcomputer Applications in Hydrology, Water Resources and Statistics	July 30-Sept. 2 (5 weeks)	Logan, UT	2,625
Administrative Uses of Microcomputers	Sept. 3-Sept. 30 (4 weeks)	Logan, UT	2,100
*Operation, Maintenance & Management of Irrigation	Oct. 1-Nov. 11 (6 weeks)	Logan, Delta UT. Phoenix AZ. Imperial & San Joaquin Valleys, CA	3,825
Microcomputer Maintenance Diagnostics, and Field Care	Nov. 12-Dec. 16 (5 weeks)	Logan, UT	2,625
		Course Total	$28,225

*Prescheduled IIC short courses: all others are additional activities especially designed for this course.

1. Applied Microcomputer Use in Irrigation and Drainage;
2. Computerized Operation of Irrigation Delivery Systems;
3. On-Farm Irrigation Design and Evaluation;
4. On-Farm Irrigation Scheduling;
5. Main System Irrigation Scheduling; and
6. Operation, Maintenance and Management of Irrigation Delivery Systems.

These short courses cover 26 weeks of specialized training in irrigation engineering on the conveyance and distribution network as well as the on-farm utilization of water. This constitutes the principal conceptual training needed to support the analyses of the irrigation engineer, agronomist, and agricultural economist.

The first course is intended to immediately submerge the trainee in the computer environment by learning to use microcomputers in a basic way to solve traditional, but relatively simple, irrigation problems. Unlike other Center participants taking only one course, the specialists under this program will be able to apply microcomputer capability to each of the other segments of the irrigation system in following courses.

After the first course of 6 weeks, followed by 2 weeks of specialized training on "Computerized Operation of Irrigation Delivery Systems", the trainee will feel comfortable in the microcomputer environment and should visualize its applications. He or she will then perceive that the full capacity to use the computer will require a detailed knowledge of how it is actually operated. To deal with this, the second phase of the training, 9 weeks, will cover the operating system and programming languages. The operating system controls machine and peripheral functions like reading and writing from disks, etc. It is the array of instructions the computer needs to respond to the users intentions. The programming languages of BASIC, PASCAL, and FORTRAN would be reviewed and practiced until a proficiency is reached to develop programs of varying complexity. The trainee may well forget certain programming aspects soon afterward, but will be able to quickly refresh his or her memory from technical materials provided and a review of the course exercises.

Then over the next 12 weeks, the trainee would look into on-farm irrigation in detail with the second, third, and fourth regular Center courses noted above. However, as mentioned, the microcomputer will be the tool used in these courses instead of the hand-held programmable calculator. The trainee will see for the first time the field work which generates the input data and will gain a sensitivity for how the physical system looks, feels, and responds.

Following the Main System Irrigation Scheduling short course, a special training phase will deal with microcomputer applications in watershed hydrology, water resources engineering, reservoir and river management modeling, and their supporting software in statistics, operations research, and numerical analyses. This 5 weeks is not

offered by the International Irrigation Center except as part of this program of extended training.

In a similar fashion, a course on administrative microcomputer use for 4 weeks will be given. This would consist of word processing, data base management, accounting, personnel records, financial management, etc. The trainee will find this course very useful to the professional in the irrigation agency because it will greatly simplify many office functions. It may be interesting to the trainee also in that it will primarily involve administrative staff at USU rather than the professional staff, and the writers would expect the trainee to gain substantial additional insight into the operation and management of an engineering department.

The next 6 weeks in the Computer-Assisted Irrigation System Management course is the existing IIC short course entitled, "Operation, Maintenance and Management of Irrigation Delivery Systems." This is the USU International Irrigation Center's course for people interested in the irrigation project above the farmer turnout and gives particular emphasis to main systems planning and management. The trainee will not only receive information useful to computer adaptation, but also see irrigation projects in the western U.S. where this is being done.

Finally, the 49-week course ends with 5 weeks of training in microcomputer maintenance, diagnostics, and field care. Maintenance in the field environment is expected to be a major problem in developing countries until the microcomputer becomes widely enough utilized for vendors to locate service centers in the individual country. The participants in this course will not be expected to repair microcomputers or their peripherals, but he or she should be able to routinely care for the equipment and cables, clean certain parts and replace others, isolate malfunctions to some extent, transport and install equipment, and interface the microcomputer to others like itself to central main frame facilities, or data acquisition systems.

Field Implementation

Concurrent with the development of human resources specialized in microcomputer technology, is the development of trained personnel for field operations. An earlier comment emphasized the need to incorporate the computer into on-going operations and maintenance tasks. This is critical to successful implementation. If the basic input data as well as the feedback and control loop are outputs of normal operations, then computer-assisted management will not require additional field personnel. In fact, what limited evidence as now exists tends to indicate that when the input and output of the computer are not naturally formed and used during the course of a normal operation, the computer quickly becomes an additional work load and is subsequently abandoned. Consequently, the writers suggest that Computer-Assisted Irrigation System Management be viewed in the larger context of irrigation systems operations and maintenance.

Because O & M is generally less emphasized than design, construction, and rehabilitation, there is often acute weaknesses in these functions with respect to procedure. The writers' view of O & M is shown in the attached flowchart. To upgrade and revise O & M practices, the first step is the development of a cadre of O & M training specialists who can in turn train O & M personnel within the irrigation agency. The data which come from the revised O & M procedures which are needed by the computer models include: (1) the discharge ratings of control, division, and turnout structures; (2) channel sections, slopes, lengths, and branching; (3) operational rules for water distribution; and (4) the dynamics of items (1)-(3) relative to time and space. Some of this information can be collected at a project site as part of O & M training programs and then monitored periodically. Thus, while this data may be collected specially for computerization, it need not represent an added burden to the schedule of day-to-day work. Additional data are necessary to describe water levels, gate settings, actual diversions, and water losses. These data are used by the models to calibrate and refine, but they are data that should normally be collected during routine operations. In fact, it is the judgment of the writers that computerization streamlines and redirects field data gathering in such a way that less effort is needed, not more.

A similar O & M procedure for the irrigation system below the canal turnout is also suggested by the attached flowchart. This component does require more special effort and has rarely been included in the management of irrigation systems. The weakness of many irrigation systems is that very little information is exchanged across the user-agency interface. The irrigation agency does not know what the real demand/yield function is in the command areas. The data needed to forecast water demands include: (1) cropping patterns; (2) climate variations; (3) soil characteristics; (4) planting, harvesting, pest management, fertilization, and other cultural practices; and (5) the dynamics of change within each of these subjects. Thus, the Unit Command Area monitoring, evaluation, and improvement system is intended to improve water management practices within the command area, improve the interface between the water user and the irrigation agency, and to provide the data needed to forecast water demands and crop yields for a rational basis for making water allocation decisions and policies within the irrigation project.

The final aspect of field implementation concerns the interaction between the computer and the field operations. Once personnel have been trained and operations and maintenance practices are revised and implemented, both in the project delivery system and in the command area, computer-assisted management can begin. This is in essence a two-way communication. In the first direction, the input data flows from the field to the models for calibration and set up. Then, the computer evaluates the water management situation and generates suggestions for operating the irrigation system, which must then be communicated to and acted upon by field personnel. Finally, the feedback control loop is completed by monitoring and evaluation data from the field reflecting

the response of the system. To the extent this two-way communication either breaks down or is ineffective, the microcomputer plays less and less of a role in water management. It should be noted that the implementation of the microcomputer may take two to three years to achieve and follow up service in the form of revised software and/or newer versions of the hardware is certain to be needed.

Summary and Conclusions

Microcomputers now have sufficient capacity and versatility to be of significant assistance to personnel administering, operating, designing, or planning irrigation projects. Perhaps within a decade, microcomputer use as a water management tool will be widely accepted and commonly applied. In the interim, special programs are needed to develop manpower to take the first critical steps in this direction. These first efforts are probably going to be expensive, and in some cases ineffective. They must begin with unified support from donor agencies, irrigation department administrators, and field and project personnel.

There is a general hesitancy to proceed. Questions concern the type of equipment to buy, how it can be used, does it really help improve water management, what data are needed, and many others. It is important, in fact imperative, to realize that implementing computer assisted water management procedures will probably require major revision in some O & M practices. It will require new monitoring and evaluation procedures below the main system turnout and improved communications networks between the public and private sectors involved in irrigation. If the computer analysis cannot be structured as a logical and integrated part of "normal" operations, it will not significantly improve water management practices.

The most immediate opportunity is to develop the "prototype" project. In other words, design and implement computer-assisted irrigation system management at one or two sites where enough control and expertise is provided to ensure that the software is adequately adapted, the computer is integrated into the daily routine, and the cost-effectiveness is established. Every effort should be made to create a "visible success story" from which this technology can radiate to other projects. It is, therefore, necessary for the host country irrigation agency, in concert with external donors when appropriate, to identify the target irrigation system and the specialists who will receive the training. These specialists will fall into three groups: (1) those responsible for control center computer applications; (2) those responsible for command area water management; and (3) those responsible for management of storage and conveyance facilities. Because of the larger manpower requirements associated with field operations, the latter two groups are really trained to be the "trainers" of field personnel responsible for data acquisition and implementation of water management practices at the field or canal level.

The cadre of operations and maintenance trainers attend a 6-week course in the U.S. on "Operation, Maintenance and Management of Irrigation Delivery Systems" and remain afterwards for four weeks to adapt the training materials to the physical conditions in the region of their country where this process is to be implemented. Likewise, the Unit Command Area water management trainers attend a 6-week course for "On-Farm Water Management" and remain afterwards for adapting the appropriate training materials to the physical situation in their country. Often, upon return to their country, these materials should be translated into their native language.

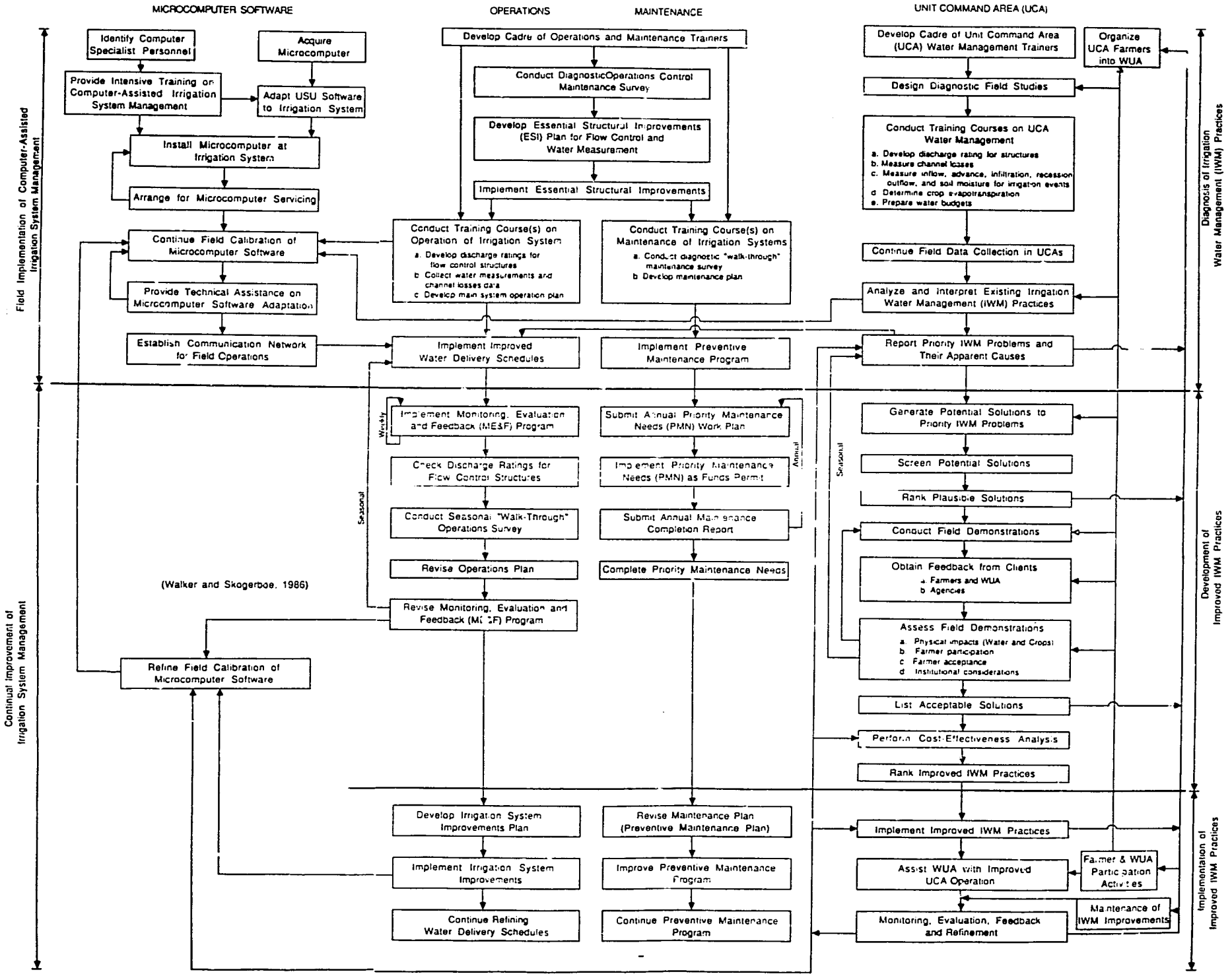
An important aspect of this process is conducting training courses at the project site. This serves a dual purpose of training project personnel for the exact situation where they will be working and also generates much of the field data necessary for calibrating the microcomputer software. In fact, all of the field training is identical whether or not computer operation is to be implemented. It is directed towards generating skills and information about the internal functioning of the irrigation system to provide adequate maintenance, improved operation, and equitable distribution of the available water supplies to farmers. The introduction of the microcomputer allows all of this information on to be more efficiently processed and greatly facilitates daily operations and higher levels of water management practices.

In order to give the reader an idea of the U.S. dollar costs of implementing a computer based management scheme in an irrigation project, the crude budget listed below (Table 14) is given as a guide. The time frame is three years. These figures are based on two people participating in the microcomputer course, and ten in the operations and maintenance, and then in the command area courses. The budget reflects costs of field installation and on-site training for either one or two 20,000 to 40,000 hectare projects. These costs are only the U.S. dollar costs and do not reflect local currency costs within the country where this program is being implemented.

Table 14. Estimated U.S. Dollar Costs For a Three-Year Time Period to Implement Computer-Assisted Irrigation System Management at One or Two Project Sites in Another Country.

U.S.A. Training	First Site	Second Site	Total
Computer Specialists	(2)$ 79,800	(2)$ 79,800	$159,600
O&M and UCA Trainers	(10) 95,000	(10) 95,000	190,000
Subtotal	$174,800	$174,800	$349,600
Equipment			
Microcomputers	$ 20,000	$ 20,000	$ 40,000
Communications	30,000	30,000	60,000
	$ 50,000	$ 50,000	$100,000
In-Country Support			
Computer Operation	$ 97,500	$ 48,750	$146,250
O&M and UCA Training	138,700	69,350	208,050
Subtotal	$236,200	$118,100	$354,300
Campus Support			
Project Director	$ 45,500	$ 22,750	$ 68,250
Secretarial	14,500	7,250	21,750
Training Materials	16,000	4,000	20,000
Shipping	4,500	1,500	6,000
Subtotal	$ 80,500	$ 35,500	$116,000
USU Overhead			
Equipment	$ 20,000	$ 20,000	$ 40,000
In-Country Support	94,480	47,240	141,720
Campus Support	32,200	14,200	46,400
Subtotal	$146,680	$ 81,440	$228,120
TOTAL	$688,180	$459,840	$1,148,020

PROCESS FOR COMPUTER-ASSISTED IRRIGATION SYSTEM MANAGEMENT



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74	Hydraulic Modeling Applications in Main System Management	G. P. Merkley
75	User's Manual for the Pascal Version of the USU Main System Hydraulic Model	G. P. Merkley
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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

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

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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

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

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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

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

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

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