REPORT NO. **R-20**

UNSTEADY FLOW SIMULATION OF THE DESIGNED PEHUR HIGH-LEVEL CANAL AND PROPOSED REMODELING OF MACHAI AND MAIRA BRANCH CANALS, NORTH WEST FRONTIER PROVINCE, PAKISTAN

Submitted to

Irrigation Department
North West Frontier Province
Government of Pakistan

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FOREWORD

The Pehur High-Level Canal (PHLC) Project was approved for loan funding by the Asian Development Bank (ADB) in late 1993. This U.S. $ 153 million project got underway in 1994. The executing agency is the federal Water and Power Development Authority (WAPDA) who work in collaboration with the Irrigation Department (ID) of the North West Frontier Province (NWFP). A consortium of European and Pakistani consulting firms, called Pehur High Level Canal Consultants, have been retained by WAPDA for the design and construction supervision of this project. The International Irrigation Management Institute (IIMI) has a contract with ID to provide support for operating this project.

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In September 1993, based on experiences at Chashma Right Bank Canal, the ADB Appraisal Mission decided that it would be advantageous to place the USC-PHLC system on the computer using the model, "Simulation of Irrigation Canals (SIC)", during the design stage rather than wait until PHLC was commissioned. Since this task had to be done anyway, there was an advantage in using an unsteady flow model to check the hydraulic design of PHLC and the remodelling of USC. Such a model is used to verify water surface elevations at design discharge rates for each reach to ensure adequate freeboard and for designing offtakes. Usually, a sensitivity analysis is done regarding hydraulic roughness, sediment deposition (however, SIC does not model sediment transport) and the stability of fairly rapid changes in discharge rates.

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The PHLC Consultants conducted a seminar, "Innovative Approach to Design of Irrigation and Drainage Works" on 06 October, 1996. After the formal presentation, there was considerable discussion about potential sedimentation. The Chief Guest, Mr. Khalid Mohtadullah, Member (Water) WAPDA, reflecting upon comments made by various participants, requested that IIMI also try to address the questions regarding sediment transport and deposition.

Regarding potential sedmimentation. initial discussions were held with Dr. Muhammad Siddique, Director General, international Sedimentation Research Institute, Pakistan (ISRIP) and Mr. John Ackers representing the PHLC Consultants. ISRIP provided sediment data previously collected for USC. This data was then analyzed by Mr. Alexandre Vabre and Mr. Gilles Belaud, two French Ph.D. students doing their research in Pakistan on sediment transport; they prepared the final section, "Potential Sedimentation" of this report

Finally, we would like *to* acknowledge the support provided by the PHLC Conultants. Both the Project Manager, Mr. Plamen Bozakov, and the Chief Designer, Mr. Adrian Laycock, met with us on numerous occasions. We were dependent upon them for the design data in order to conduct this study. We certainly appreciate their cooperation over the past two years.

Gaylord V. Skogerboe, Director
Pakistan National Program
International Irrigation Management Institute

EXECUTIVE SUMMARY

The Upper Swat Canal (USC) system is presently irrigating 111,700 ha (276,000 acres) of land in three districts; Charsada, Mardan and Swabi of North West Frontier Province of Pakistan. The existing system was designed in 1915 with a headworks on the Swat River at Amandara to divert 51 m^3/sec (1800 cfs) of water. A scheme to improve irrigation in the Swabi District was initiated in 1971 when the water rights to the area were provided from Terbella Reservoir. As a consequence, water was supplied to Pehur Branch Canal through a pump station from Terbella and provision for a high level canal was left by providing an irrigation tunnel portal in the reservoir.. The present scheme for remodelling of the old system and construction of Pehur High-Level Canal started in 1991. The major features of the scheme are: (1) a shift from supply-based to crop-based by increasing water duty of the command area from .30 Ilslha to .70 l/s/ha (4.5 to 10 cusecs per 1000 acres); and (2) an addition of 20,000 ha (49,400 acres) in command area.

The work presented in this report has been accomplished as a partial fulfillment of IIMI's Action Plan (1996) on "Operational Support for the Pehur High-Level Canal Project". The present study basically examines hydraulic and operational behavior of all three branch canals of the system; namely, Machai, PHLC and Maira for proposed physical parameters. Performance of the major hydraulic parameters for the branch canals like water levels (for canal capacities), velocities and Froude Numbers is studied using hydro-dynamic models. The delivery patterns of distributaries and direct outlets are computed to check the delivery efficiency. The responsiveness of different manual upstream control and automatic downstream control regulators has been evaluated under different sets of operations.

The reader must realize that any recommendations for alternative design options were outside the scope of this study; however, current analysis shows the flexibilities and restraints of the system. The simulation of the design parameters at this stage will provide a reference for optimization of operational options after construction.

Two hydrodynamic models have been used to evaluate the steady and unsteady state behavior of the system: (1) Simulation of Irrigation Canal (SIC), developed by a French Institute CEMAGREF; and **(2)** CANALMAN, developed by Utah State University. For purpose of analysis, the USC-PHLC system is divided into four subsystems: (1) Machai Branch Canal; (2) Pehur High-Level Canal; (3) Confluence reach consisting of the tail reaches of Machai and PHLC, and the head reach of Maira Branch Canal; and **(4)** Maira Branch Canal.

The first two chapters of the report give an overview of the project. Chapter three describes the salient features of the models, while Chapter Four gives the physical configuration of the system. Simulation results are presented in chapters five to nine.

The hydrodynamic model SIC is calibrated for the full supply conditions of Machai Branch, i.e inflow 66.67 m^3/sec and all of the distributaries their taking design discharge. The simulated water levels match fairly well with the design water levels for the base case. The model is then used to evaluate scenarios at low flows and rotation. The analysis presented in Chapter Five can be summarized as:

1. The canal capacities are sufficient to take 120% of the design discharge and sufficient freeboard is available to compensate small changes in roughness. The last reach of Machai Branch Canal can successfully dampen the water level fluctuations coming from upstream.

2. All of the distributaries can be fed with maximum discharge in kharif and with recommended discharge during rabi rotation by regulating downstream cross-structures;

3. The direct outlets seem to have some problems:

   (i) At full supply, a few outlets draw discharge (-10% to +15%) different from their design discharge;

   (ii) At low flows, outlets located at the downstream end of the reach draw more water as compared with the outlets at the upstream end because more working head is available for these outlets due to water ponding at the tail of the reach; and

   (iii) During rotation, there is no way to close the direct outlets in a reach where water is being conveyed, so these outlets continue to receive water.

Chapter Six presents a summary analysis of the functioning of downstream control AVIO and AVIS gates proposed for Pehur High-Level Canal. This study is done using the CANALMAN model and has been presented in a separate consultancy report "Unsteady Flow Simulation of Pehur High-Level Canal including Automatic Downstream Water Level Control Gates". The results shown by six scenarios can be briefed as:

1) No problems are expected regarding canal capacities and operations of the gates when the Manning roughness coefficient is increased from 0.016 to 0.019;

2) All of the proposed gates show very stable behavior for 30% variation in discharge; and

3) The scenarios simulating extreme conditions indicate that water depths along the canal will not fluctuate very much even for large changes in water supply pattern. For example, ifthe water supply to the Machai tail reach is reduced from 12.41 m^3/sec to zero within six hours, the change in water levels will be a few centimeters.

The Machai-Pehur Confluence reach is considered as the most important section of the system to be checked for operational stability. The maximum tail reach discharges in Machai and Pehur could be 23 and 28 m^3/sec, while the Maira Branch canal carries a design discharge of 24 m^3/sec. The functioning of different components of the reach is checked using the CANALMAN model. A reference is made to a discussion paper by J.C.Ackers, "Canal (PHLC) self regulation at the Confluence with Machai". Two confluence systems (options) are simulated. The results briefly are:

(i) The simulation confirms the recommendation made in the abovementioned paper, which suggests an AVIS gate at the end of PHLC and the replacement of Machai tail cross-regulator by another AVIS gate, moved downstream of the escape; and

(ii) The results indicate that the escape plays an important role for the safe operations of the Confluence reach. In an extreme case, when the Maira head regulator is closed while the Machai tail is still being supplied **24** m^3/sec discharge, all of the extra water is spilled over the escape weir. Another extreme situation is simulated when the Maira Branch Head Regulator is closed while 23 m^3/sec from PHLC and 29 m^3/sec from Machai was arriving at the confluence. In this case, 29 m^3/sec is spilled over the escape weir, while water levels are raised, but within the permited level.

Chapter Eight presents the simulation results for Maira Branch Canal. The hydrodynamic responses of canal reaches and self-regulating downstream control structures are shown by selecting different operational scenarios.

(i) The performance of canal reaches and self regulating gates is checked for design discharge, design and 10-15 percent higher roughness coefficients, and for different operational levels.

(ii) With 30% increase or decrease in the discharge at the head of canal, there are modest perturbations in the system, which in the worst case takes about 12 hours to become stabilized.

The last chapter of the report provides an insight to the expected sediment trends in Machai and Maira Branch Canals. A sediment transport module is in the process of development through a collaborative research effort by IIMI-PAK, CEMAGREF (French Institute) and ISRIP (International Sediment Research Institute, Pakistan). This work uses the steady state module of the SIC hydraulic model. The first calibration of the module has been done using sediment data from Chashma Right Bank Canal. For this study, Machai and Maira data collected by ACOP (Alluvial Channels Observation Project, presently ISRIP) are used.

* The present system is in good equilibrium. The measured data and module's prediction confirm the well known concept about the sediment balance in alluvial channels of Pakistan: during kharif, the peak sediment concentration is higher than the canal sediment transport capacities, while in rabi,, the sediment transport is higher and scour occurs so that a balance is maintained.

* No sediment problems are expected at full supply operations. The PHLC-Maira link will be more sensitive to sediment inflows as compared to the Machai-Maira link; hence, a continuous monitoring of the sediment loads from Tarbella will be required.

* There is a strong link between operations of the cross-regulators and the sediment transport potential of a canal because velocity drops quickly with the development of backwater. The report suggests that a critical lower level for the velocity should be defined where problems are expected.

1. INTRODUCTION

The Pehur High-Level Canal (PHLC) Project was approved for loan funding by the Asian Development Bank (ADB) in late 1993. This US. $ 153 million project got underway in 1994. The executing agency is the federal Water and Power Development Authority (WAPDA) who work in collaboration with the Irrigation Department (ID) of the North West Frontier Province (NWFP). A consortium of European and Pakistani consulting firms, called Pehur High Level Canal Consultants, have been retained by WAPDA for the design and construction supervision of this project. The International Irrigation Management Institute (IIMI) has a contract with ID to provide support for operating this project.

An action plan (IIMI, 1996) on "Operations Support for the Pehur High-Level Canal Project" was developed using a strong collaborative mode with the Irrigation Department (ID) in implementing the operation of the Upper Swat Canal (USC) and PHLC system. In addition, there would be considerable interaction with the consulting firm responsible for designing and supervising the construction of irrigation facilities.

The water supply for the existing Upper Swat Canal (USC) comes from the Swat River, while the new Pehur High-Level Canal (PHLC) will divert water from the reservoir created by the construction of Tarbela Dam in the 1970s on the Indus River. The Swat River is a tributary to the Indus River, with their confluence being downstream of Tarbela Dam.

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Downstream Water Level Control Gates", which was shared with the design consultants. He was again brought to Pakistan for one week in early July and again the third week of October in 1996.

The design of the irrigation network (new PHLC and remodeling of the existing USC) has been placed in the SIC computer model that was developed by CEMAGREF, which is the French national research organization for water, agriculture and forests, as well as the CM model. The design of irrigation networks is commonly done using principles of steady-state hydraulics complemented with calculating backwater profiles. The advantage of both SIC and CM is that unsteady flow conditions are simulated, as well as steady-state flows. Surprisingly, many of the large canals in Pakistan are often operating under unsteady flow conditions. Thus, SIC and CM are valuable tools for simulating a variety of flow conditions that provides insights for design modifications.

This report summarizes the results from the unsteady flow simulations. For purposes of analysis, the USC-PHLC system was subdivided into four subsystems: (1) Machai Branch Canal; (2) Pehur High-Level Canal; (3) Confluence (consisting of the tail reaches of Machai and PHLC, as well as the head reach of Maira Branch Canal); and (4) Maira Branch Canal.

2. PROJECT DESCRIPTION

2.1 BACKGROUND[1]

The Upper Swat Canal (USC), which takes water from the Swat River at the Amandara Headworks, was originally designed for a discharge of 68.6 cumecs (2,420 cusecs) to irrigate an area of 127,500 ha (315,000 acres) of the Charsaddah - Mardan - Swabi Plain. The canal, after traversing the narrow ridge of Malakand hills through the unlined Benton Tunnel, eventually bifurcates at Dargai into the two branches of Machai and Abazai.

Soon after construction of the Benton Tunnel in 1914, it was noticed that, though constructed to the full design section, its discharge capacity was not more than 51 cumecs (1800 cusecs) due to its unlined nature and surface roughness. Due to this constraint, the authorized full supply discharge of the canal was fixed at 51 cumecs (1,800 cusecs) and the culturable commanded area (CCA) reduced to 111,700 ha (276,000 acres)

[1] The material in this section has been extracted from the Inception Report of the Pehur High Level Consultants, March 1995. p 2-1

With progressive development of agriculture in the area, a wide disparity has been noticed within the command of USC. Whereas the areas served by Abazai Branch Canal and the upper reaches of the Machai Branch Canal have attained irrigation intensities as high as 170 percent, the intensities attained so far in the lower reaches situated in the Swabi and Nowshera districts are as low as 100 percent.

A scheme for drawing irrigation water from Tarbela reservoir to improve and extend irrigation in Swabi District was initiated in 1971. Under the scheme, water, to supplement the water supplies of Machai Branch was to be conveyed via a tunnel through the right abutment of the Tarbela Reservoir and then via Pehur High Level Canal (PHLC). Later, it was also considered appropriate to bring new areas under irrigation.

Until 1991, PHLC was the subject to PC-I proformas by WAPDA, based on a feasibility report prepared in 1975. With the signing of the inter-provincial Water Apportionment Accord in March 1991, the main impediment to project implementation was removed. This Accord allows 0.52 million acre-feet per year from Tarbela Reservoir via the Gandaf Tunnel into PHLC. Consequently, GoP approached ADB initially for technical assistance for feasibility studies, and then for financial assistance for implementation. Consultants completed a project feasibility study in June 1993, which was accepted. Loan No. 1294-PAK for project implementation was then sanctioned. Implementation of the PHLC Project began in December 1994.

2.2 OBJECTIVE[2]

The Pehur High-Level Canal (PHLC) Project (Figure 1) aims to realize the full agriculture potential of about 100,000 acres (40,300 ha) of agriculture land. The Project will also increase agricultural production in the adjacent Swabi SCARP Project (SSP) area by supplementing its water resources, allowing additional irrigation development in about 10,000 acres (4,000 ha) of mainly rainfed land. In providing the physical facilities and developing system management procedures to manage the increased irrigation supplies efficiently and control the groundwater table, the PHLC Project will preserve the resource base for agriculture in the Project area. Specific measures have been covenanted to prevent the Project from aggravating waterlogging in the adjacent command area of the existing Pehur Main Canal. In addition, the Project will prepare a detailed program for land resource conservation for the catchment area of the Naranji Hill Torrent. The program will provide a long-term solution to the current problems in maintaining surface drains in part of the Project area and SSP.

[2] The material in these sections has been extracted from the Asian Development Bank (ADB) Report RRP:PAK 19141 dated November 1993 for the Pehur High-Level Canal (PHLC) Project, pages 11-13.

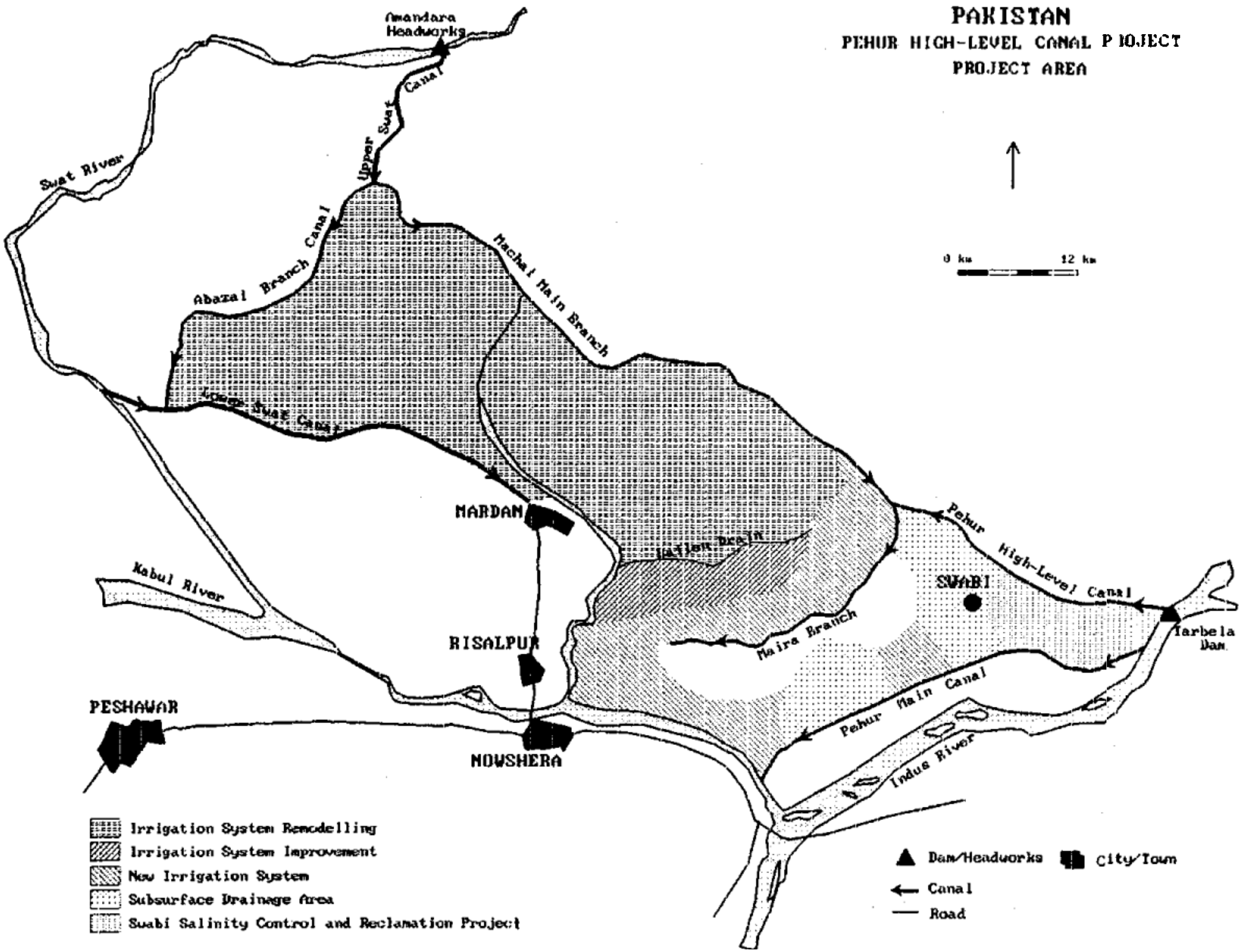

Figure 1. Pehur High-Level Canal Project Area.

2.3 SCOPE[1]

To achieve its objectives, the proposed Project provides for: (i) irrigation and drainage improvement and development; (ii) measures for accelerating agriculture development capitalizing on improved irrigation and drainage conditions; (iii) a land resource conservation study; and (iv) environmental and benefit monitoring and evaluation. The Project will provide the necessary support for implementing these components, including consulting services, as well as staff, vehicles and equipment for Project supervision and administration.

2.4 IRRIGATION AND DRAINAGE IMPROVEMENT AND DEVELOPMENT[2]

This component incorporates: (i) construction, remodelling and improvement of irrigation and drainage infrastructure; and (ii) optimization of operations and maintenance (O&M).

2.4.1 Construction, Remodelling and Improvement

New irrigation facilities will be constructed, including: (i) the PHLC and the Gandaf and Baja Tunnels serving the entire Project area; and (ii) a distribution system for 4,500 ha in the Topi area. Most of the existing irrigation system below canal kilometer (km) 73.8 of the Upper Swat Canal's (USC's) Machai Main Branch Canal will be remodelled (i.e. its present canal capacity will be substantially increased). However, canals serving about 19,000 acres (7,600 ha) will not be remodelled but improved (i.e. their capacity will not be significantly enlarged) because present irrigation supplies are adequate and waterlogging in the adjacent command area of the Pehur Main Canal could be aggravated by further increasing present supplies.

The canals will be designed for a peak irrigation requirements of about 0.7 liters per second (l/sec) per ha (10 cusecs per 1,000 acres) and crop-based system management requiring the operation of the canal system at supply regimes ranging between 40 per cent and full design capacity. Existing watercourses with capacities above 4 cusecs (140 l/sec) will be converted into minors as discharges of this magnitude are difficult for WUAs to manage.

The existing Gandaf Tunnel will be extended through the right abutment of Tarbela Dam to supply water from the existing reservoir to the PHLC. The tunnel will be 3.9 km long with a 3.8-meter (m) internal diameter, and will have a conveyance capacity of 28.3 cubic meters per second (cumecs), which is 1,000 cubic feet per second (cusec). the Baja Tunnel will be 1.2 km long with a 3.5-m internal diameter. The PHLC, to be lined over its entire 26.2 km length, will be constructed for a capacity consistent with a capacity of 28.3 cumecs at the outlet of the Gandaf Tunnel. Four new distributaries will be constructed to irrigate the Topi area.

2.4.2 Optimization of Operation and Maintenance

The Project will provide for the development of operational procedures for crop-based operation of the linked USC and PHLC (USC-PHLC) main canal system and cost-effective maintenance procedures and methods for the Project's irrigation and drainage facilities. To optimize the use of the Indus and Swat Rivers water resources, and minimize the loss of hydro-power energy at Tarbela Dam, a comprehensive computer model of the USC-PHLC main canal system will be developed. [This report represents the first step in developing this comprehensive computer model, which represents the designed system, that will be later refined based on any construction changes, then after commissioning of the PHLC based on actual field calibrations jointly undertaken by the NWFP Irrigation Department (ID), the International Sedimentation Research Institute, Pakistan (ISRIP) and IIMI.]

3. HYDRAULIC UNSTEADY FLOW MODELS

As alluded to earlier, there are two hydraulic models used in this study that simulate unsteady flows: (1) Simulation of Irrigation Canals (SIC); and (2) CanalMan (CM). SIC was developed in France by CEMAGREF and CM was developed by Utah State University in the U.S.A. Both models have been employed in other countries. Each model solves the Saint-Venant equations, so their difference is related to computer programming and the variety of physical conditions that can be accommodated. For example, CanalMan has an algorithm for automatic gates.

3.1 SIMULATION OF IRRIGATION CANALS (SIC) MODEL

The French hydraulic model SIC is being used for the simulation of Machai Branch. IIMI has already used this model in different countries. In Pakistan SIC has been used at the main canal level for the operational and delivery performance studies of Chashma Right Bank Canal (CRBC) in NWFP (Ref 1) and Fordwah Branch Canal, Forswah Eastren Saddiqia irrigation system in Punjab (Ref 2). At the secondary canal level SIC has been used to study water distribution, canal capacities and maintenance for a couple of distributaries in different study areas. In 1992, the preliminary hydraulic studies of two distributaries (#3 and **#4)** of CRBC were conducted. In the Chishtian Subdivision of Punjab, SIC has been used at the main canal and distributary levels in a number of studies addressing the above-mentioned issues. Recently, it has been applied as an integrated tool along with other models, to address the salinity, sodicity and water market issues.

These studies indicate that SIC is a powerful tool to simulate the hydraulic behavior of the alluvial and manually controled irrigation system.

3.1.1 Structure of the Model

The model is built around three main computer programs TALWEG, FLUVIA and SIRENE, that, respectively, carry out topography generation, steady flow computation and unsteady flow computation. These units can be run either separately or in sequence.

Unit I generates the topography files used by the computation programs of units II and III. Access to this unit is restricted through a hardware lock. If this lock is not connected with the printer port, the menu options to access Unit I are not displayed.

Unit II performs the steady state computations and generates the water surface profiles for any given combination of offtake discharges and cross-regulator gate openings. These water surface profiles may be used as initial conditions for the unsteady flow computation in Unit III. Unit II also allows the determination of offtake gate openings and adjustable-regulator gate settings required to satisfy a given water distribution plan, while simultaneously maintaining a set of target water levels in the main canal.

Unit III processes the unsteady flow computations. This allows the user to test various scenarios of water demand schedules and operations at the head works and control structures. Starting from an initial steady flow regime, it will help the user to identify the best way to attain a new water distribution plan. The efficiency of the operational strategy may be evaluated via a set of water delivery indicators computed at the offtakes.

3.1.1.1 Hydraulic Network of Unit 1

For Unit **I,** the canal typology is defined in terms of nodes reaches and branches while the geometric description is given in terms of cross-sections. Each offtaking structure is defined as a node, a reach is defined as a set of cross-sections along the longitudinal distance between two nodes, and a branch is defined as a group of reaches.

The reach geometry is determined by the cross-section profile characteristics of the shape and the volume of the canal. The cross sections can be entered in three different ways:

i. abscissa-elevation description;

ii. width-elevation description; and

iii. parametric form as circle, culvert, power relationship, trapezoid or triangle

The cross regulating structures are defined by marking the corresponding cross-section as singular sections. In these sections. the general hydraulic laws for computing water surface profile are replaced by the discharge formulas of the structures. Based upon the topographic information, Unit I computes the physical parameters 'like cross-sectional area, hydraulic radius, wetted perimeter, etc.

3.11.2 Computational Procedure of Unit II

Information about structures, channel roughness and seepage rates are input into Unit II. The initial state of the system is generated by this unit.

Definition of structures. In the model, a distinction has been made between high and low sill elevation devices ant then between gated and ungated structures. In the case of offtake structures, downstream boundary conditions are very important and need to be defined through a rating curve, weir type structure or fixed water levels. The general equations for free flow over a weir or through an orifice are defined as follows;

For a weir structure,

$$Q = \mu_f L\sqrt{2g}h^{3/2} \tag{1}$$

For an orifice structure,

$$Q = \mu L\sqrt{2g}(h_1{}^{3/2} - (h_1 - W)^{3/2} \tag{2}$$

Where the default values of , and are 0.4 and 0.6 respectively, L is the width of the structure and W is the gate opening (or y in the case of APM outlets).

For submerged flow conditions, the equations are modified as shown below,

For a submerged weir structure,

$$Q = \mu_s Lh_2\sqrt{2g}(h_1 - h_2)^{1/2} \tag{3}$$

For a submerged orifice structure,

$$Q = \mu' LW\sqrt{2g}(h_1 - h_2)^{1/2} \tag{4}$$

The coefficients for submerged flow conditions have been calculated as a function of the free flow coefficient, submergence ratio and gate opening.

Graduallv varied flow in a reach. With the canal being divided into homogeneous zones (reaches), the problem is reduced to calculating the water surface profile under subcritical, steady flow conditions in a reach. The classic hypotheses of unidimensional hydraulics in canals are considered to apply when:

* The flow direction is sufficiently rectilinear, so that the free surface could be considered to be horizontal in a cross section;

* The transversal velocities are negligible and the pressure distribution is hydrostatic; nad

* The friction forces are taken into account through the Manning-Strickler coefficients.

Therefore, only monodimensional steady flow is studied, and only subcritical flow is considered.

The differential equation used to compute the water surface profile in a reach can be written as follows:

$$\frac{dH}{dx} = -S_f + k\frac{qQ}{gA^2} \tag{5}$$

with

$$s_f = \frac{n^2Q^2}{A^2R^{4/3}} \tag{6}$$

To solve this equation, an upstream boundary condition in terms of discharge and a downstream boundary condition in terms of water surface elevation are imposed. **As** the equation does not have an analytical solution in the general case, it is discretized in order to obtain a numerical solution. The water surface profile is integrated step-by-step, starting from the downstream end.

3.1.1.3 Unsteady Flow Computations of Unit III

The canal is divided into homogeneous zones, called reaches. In computing the unsteady flow water surface profile in a reach, the hypotheses of unidimentional hydraulics is applied, as in Unit **1.**

Two equations are used to describe unsteady flow, the continuity equation and the momentum equation. The continuity equation which accounts for the conservation of the mass of water is, expressed as (Ven Te Chow 1988):

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{7}$$

The momentum equation, or dynamic equation, is expressed as:

$$\frac{\partial Q}{\partial t} + \frac{\partial Q^2/A}{\partial x} + g.A\frac{\partial z}{\partial x} = -gAS_f + kqV \tag{8}$$

The partial differential equation must be completed by setting initial and boundary conditions in order to be solved. The initial condition is the water surface profile resulting from the steady flow computation. The boundary conditions are the hydrographs at the upstream nodes of the reaches and a rating curve at the downstream node of the model.

The Saint-Venant equations have no known analytical solutions in real geometry. They are solved numerically by discretizing the equations. Various schemes may be used to provide solution to these equations. The discretization chosen in the SIC model is a four-point implicit scheme known as Preissmann's scheme. This scheme is implicit because the values *of* the variables at the unknown time step also appear in the expression containing the spatial partial derivatives. The double sweep method is then used to solve the linear systems obtained when discretizing the Saint Venant's equations. The singularities and the offlakes have to be introduced in the double sweep process.

3.1.2 Input Data

The input data files are prepared through a user friendly interface and the data are primarily saved in three ASCII text files for topography, hydraulic and structural information, and time dependent sets of operations. The output information can be retrieved in graphical and numerical format. A list of input is given in the following;

- cross-sectional data in absolute levels;
- exact'location of structures and offlakes;
- dimensions and crest levels of structures and offtakes;
- seepage losses;
- estimated Manning coefficient;
- calibration coefficient and users defined rating curves for offlakes; and
- downstream boundary conditions.

3.1.3 Output Information

3.1.3.1 Numeric and Graphic Output

The results of steady and unsteady state computations are saved as numeric and graphical output. These results include water levels along with velocity and discharge at computational points. The gate opening is computed for the given targeted discharge and coefficient of discharge, or discharge is computed for the given gate opeing. For unsteady state flow hydrograph at the computational points are stored for the time interval set by the user.

3.1.3.2 Performance Indicators

A few simple performance indicator have been incorporated with a view to evaluating the water delivery efficiency at the offtakes. This allow the integration of the information on water delivery, either at a single offtake or at all of the offtakes. There are two kinds of indicators: volume indicators and time indicators.

The volume indicators refer to three kinds of volumes:

* The demand volume (V_d), which is the target volume at each offtake
* The supply volume (V_s), which is the volume supplied at each offtake
* The effective volume (V_{ef}), which is really the useable part of the supply volume.

The definition of the effective volume depends on two coefficients. W and **X** (in percentage):

$$(1 - X/100).\ Q_d \le Q_s \le (1 + W/100).Q_d \Rightarrow Q_e \qquad (9)$$

Only the supply discharge close to the water demand is taken into account

Indicator IND1 = V_s / V_d

Indicator IND2 = $V_{ef} / V,$

Indicator IND3 = V_{ef} / V_s

These indicators can be defined for a single offtake or for a set of offtakes

3.1.4 Calibration, Sensitivity and Limitations

Calibration of the model is accomplished by matching the computed and measured water levels. The seepage losses are usually measured and the roughness coefficient are adjusted to achieve the required levels. The discharge coefficients of the structures are measured in the field and then further adjusted to have the same discharge through the structures.

The sensitivity of the model with reference to different parameters is recently checked in Fordwah area, analysis indicates that the model is sensitive to;

- volume conservation;
- slope & roughness combination;
- flow conditions and calibration of outlets; and
- downstream boundary conditions.

There are the following limitations:

- supercritical flow and negative slope is not acceptable to the model;
- some of the structures like tunnels could not be defined directly;
- calibration & flow conditions of offtake structures ;
- For unsteady state, optimization of operation when the consecutive steady states are achieved for relatively small time periods.

3.2 CANALMAN (CM) MODEL

The ***CanalMan*** model was developed by the Department *of* Biological and Irrigation Engineering, Utah State University. (This part of report is adopted from the ***CanalMan*** manual.) This software application performs hydraulic simulations of unsteady flow in branching canal networks. The model is intended primarily for use in operational and training activities, and it can also be applied to design and analysis studies of canal systems. The model can be used to simulate canal operations in manual mode, and it can generate proposed operating schedules through a centralized automatic mode. Several common types of local gate automation schemes are also included in the model, and these can be easily selected and calibrated through the model interface.

CanalMan implicitly solves an integrated form of the Saint-Venant equations of continuity and motion for one-dimensional unsteady open-channel flow. Computation nodes are used internally by the model, and they are automatically inserted along the length of the canal reach, between the system layout nodes that the user creates. Simulations can be started by filling an empty canal system, continuing a previous simulation, or from a specified steady or unsteady flow condition.

The model will directly simulate the layout of most canal systems, including branching canals. Canal reaches are separated by in-line control structures such as gates, weirs, and pumps. Several in-line structures can be independently simulated in parallel at the downstream end of a canal reach. Turnouts can be used to remove water from the simulated canal system, or divert water into laterals (distributaries) or sub-laterals (minors) within the system. Turnout operation can be simulated by specifying a setting (the model calculates the flow rate), specifying a "demand" flow rate (the model then calculates the setting), or given an "actual" flow rate in which the model simply assumes the flow is correct. Time graphs of flow rate and setting can also be created for individual turnouts and applied to these three operational modes.

A variety of in-line and turnout structures can be selected from the editor for inclusion in a system layout. including weirs, underflow gates, pumps, "non-structure" section changes, and uniform flow boundaries. The inflow rate at the system source can be "manually" specified, calculated according to local conditions, or calculated according to system-wide delivery requirements.

Results from ***CanalMan*** include flow depths in the canal reaches, volumetric flow rates, and control structure (gate) settings -- all as a function of time. Simulation results can be viewed in numerical **and** graphical formats on-screen, and tabular results can be written to text files. Direct printouts of the tabular results can also be obtained from the model. Changes in selected reach depth profiles, downstream target levels, and gate settings can be monitored during a simulation through graphical flow profile windows. Reach inflow and outflow rates, modes and status can also be viewed during a simulation.

3.2.1 AVIS and AVIO in ***CanalMan***

The AVIS and AVIO gates are radial gates operated by one or several floats rigidly fitted to an arm opposite to the gate leaf. The radial gates are in equilibrium whenever the downstream water levels are at the gate hinge elevation. The hydraulic thrust on the leaf passes through the gate hinge axis and has no effect on the equilibrium. Therefore, only the torque due to the weight of the gate and the buoyancy of the float affect the gate opening. A decrease in the downstream water surface causes the buoyancy torque to become smaller than the weight-acting torque, causing the gate to open, and approximately restoring the downstream water level to the reference water level. The reverse process is applied for raising the upstream water surface.

An equilibrium condition of the AVIS and AVIO gates can be expressed as follows:

$$Q = KR^2 \frac{\Delta z}{\Delta z_{max}} \sqrt{H_u - H_d} \tag{10}$$

where Q is the flow rate (m^3/s); K is a constant depending on the gate type; R is the float radius (m); Δ_Z and Δ_{Zmax} are the decrement and maximum decrement, respectively (m); and H_u and H_d are upstream and downstream flow depths, respectively. The values of K are 4.1, 1.0 and 2.0 for the AVIS. high-head AVIO, and low-head AVIO, respectively. The value of Δ_{Zmax} is equal to five percent of the float radius.

Since both AVIS and AVIO gates are orifice gates and typically operate under submerged conditions, the flow rate can also be expressed as

$$Q = C_d \cdot b\, H_d \cdot \sqrt{2g\,(H_u - H_d)} \tag{11}$$

and,

$$C_d\ \xi = s1\ .\left(\frac{A_o}{H_d}\right) \tag{12}$$

where Q is the flow rate (m^3/s); C_d **is** a coefficient of discharge for submerged orifice flow; A_o is the area of gate opening (m^2); and ξ_{S1} and ξ_{S2} are coefficients. All other parameters are as defined above.

As a rule, ΔZ_{max} in the AVIS and AVIO gates corresponds to the maximum gate setting, and a decrement of zero corresponds to a closed gate. Assuming the gate setting and decrement are linearly related, the following can be stated:

$$\frac{Go}{Go_{max}} = \frac{\Delta_Z}{\Delta Z_{max}} \tag{13}$$

Substituting A_o which is the product of the gate opening, G_o, and gate width, b, into Eq. 12 and rearranging:

$$C_d = \xi_{S_1}\left[\frac{bGo_{max}}{0.05R}\right]^{\xi_{s2}}\left[\frac{\Delta_z}{H_d}\right]^{\xi_{s2}} \tag{14}$$

where b is the gate width (m); and Go_{max} is the maximum gate setting (m). Since Eqs. 10 and 11 are equivalent; therefore, the equation for C_d can also be rewritten by setting Eq. 10 equal to Eq. 11, substituting $_{z\ max}$ = 0.05 R, and rearranging.

$$C_d = \left[\frac{20KR}{\sqrt{2_g}}\right] \cdot \frac{\Delta Z}{H_d} \tag{15}$$

Rearranging Eqs 14 and 15, the following relationships are obtained:

$$\xi_{S_2} = 1.0,\ \xi_{S_1} = \frac{KR^2}{Go_{max} \cdot b\sqrt{2g}} \tag{16}$$

CanalMan requires the maximum gate setting, allowable maximum of gate changing in one step, and gate float radius for simulation of AVIO and AVIS gates. The maximum gate setting and gate float radius can be obtained from the manufacturer. The allowable maximum gate change is specified as a percentage of the maximum gate setting. This restriction is for simulation purposes, and is believed to be automatically taken care of in actual installations. Without a limit on the amount of change in the gate setting from one time step to the next, the model will often manifest fluctuations of the gate, possibly leading to failure of the simulation.

3.2.2 AOSM in ***CanalMan***

The Adjustable Orifice Semi Modules (AOSIM) are of traditional design in PHLC, Machai and Maira branches. The ***CanalMan*** does not provide for this particular type of control structure. Fortunately, orifice structures can be simulated, so that the AOSM can be accommodated as explained below.

The discharge for an AOSM is given by the following formula:

$$q = C_{oe} \ . \ B_t \ . \ y \ . \ h_s^{0.5} \tag{17}$$

where q *is* the discharge (ft^3/s); C_{oe} is the coefficient of discharge fixed at 7.5; B_t is the throat width (ft); y is the orifice height (ft); and h_s is the height of canal water level above the soffit of the roof block (ft).

Substituting B_t = 3.28 G_w; y = 3.28 G_o h_s = 3.28 (h_u - G_o), and q = 35.315 Q, where Gw is the width of the opening' (m), G_o is the maximum and fixed gate opening (m), hu is the upstream depth (m), and Q is the discharge (m^3/s), then Eq (8) becomes:

$$Q = 4.136 \cdot G_w \cdot G_o \cdot (h_u - G_o)^{0.5} \tag{18}$$

The rectangular gate is a type of water control structures provided in the ***CanalMan.*** This structure type includes vertical sluice gates (slide gates) and radial (or tainted) gates. Slide gates and radial gates are grouped together under this structure type because they both are assumed to have rectangular openings. The difference between them is manifested in the calibration parameters which define the discharge coefficients. The flow rate through a radial gate will usually be higher than for a slide gate with the same area of opening, and the same upstream and downstream flow depths.

Rectangular gates can operate as orifices or as free-surface channel constrictions (when the gate is raised above the water surface). Either free or submerged flow may occur at the structure at any given time. Orifice flow is assumed when the bottom of the gate is below the upstream water surface elevation; otherwise, energy balance equations are used to relate flow depth to flow rate.

The discharge of a rectangular gate is assumed to be submerged orifice flow when the downstream water surface **is** more than 0.61 G_o where G_o is the vertical gate opening. Under these conditions, the rating equation is:

$$Q_s = C_{ds} \cdot (h_d - E_l + Z_l) \cdot \sqrt{2g \cdot (h_u - Z_l - h_d)} \tag{19}$$

where C_{ds} is the discharge coefficient; and Z, is the reach invert rise (m).

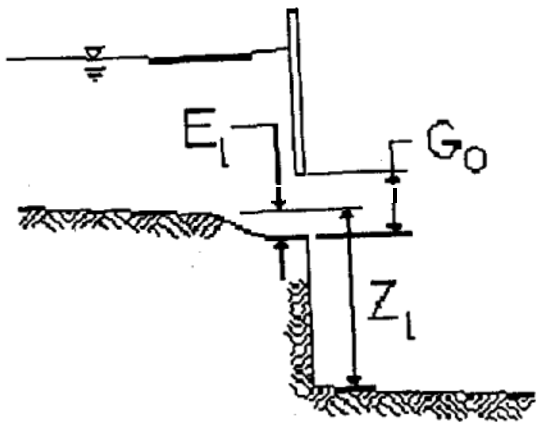

Figure 2. Definition sketch for a gate structure.

The reach invert rise, Z,, is negative for a *drop* in elevation, as illustrated in the example configuration above (Figure **2).** The above figure also shows an upstream decrease in the bed elevation at the gate, which would also cause the **E,** value to be negative. In most canals, Z, is usually either zero, or is a negative value, indicating a drop in the bed elevation in the downstream direction.

The discharge coefficient is a function of the downstream depth, the bed elevation change, and the gate opening:

$$C_{ds} = \xi_{s1} \left(\frac{A_o}{h_d - E_1 + Z_1} \right) \tag{20}$$

where ξ_{s1} is a calibration coefficient; ξ_{s2} is a calibration exponent; and all other parameters are as previously defined. As with the modular flow equation, if the exponent, ξ_{s2} ,is equal to unity, the discharge coefficient becomes a constant equal to ξ_{s1}.

In this study, the downstream depth will be fixed at the value of G_o, so that h_d is always equal to G_o. For **Z,** equal to zero, the **Eqs *19*** and 20 are combined and rewritten as;

$$Q_s = \xi_{s1}\left(\frac{G_w \cdot G_o}{h_d - E_1 + Z_1}\right)^{\xi_{s2}} \cdot (h_d - E_1 = Z_1)\ \sqrt{2g\cdot(h_u - Z_1 - h_d)} \qquad \textbf{(21)}$$

The value off,, can oflen be entered as unity; therefore, the equation can be rewritten as;

$$Q = 4.429 \cdot \xi_{s1} \cdot G_w \cdot G_o \cdot (h_u - G_o)^{0.5} \qquad \textbf{(22)}$$

Since Eqs.18 and 22 are equivalent, therefore, the values of ξ_{s1} is equal to 0.934.

4.2.3 Section Chanae

A section change boundary condition represents a "non-structure" location at the downstream end of a canal reach, like a uniform flow boundary, except in this case the specific energy equation is applied. Any of the following can change abruptly at a section change location: cross-sectional channel shape or size; longitudinal bed slope; bed elevation; hydraulic roughness; or seepage loss rate. Thus, a "section change" could involve only a change in the bed slope or roughness, for example, without any difference in the section shape or size. This boundary condition can be used to effectively divide a single reach into multiple reaches with different bed slopes, side slopes, etc.

If the section change boundary involves a downstream drop in the bed elevation (i.e. a negative invert rise for the reach), the flow regime may be free (modular). The model assumes free-flow conditions if the depth just downstream of the section change (at the upstream end of the next reach, if any) is less than or equal to the absolute value of the invert rise. This condition is illustrated in the accompanying figure (Figure 3).

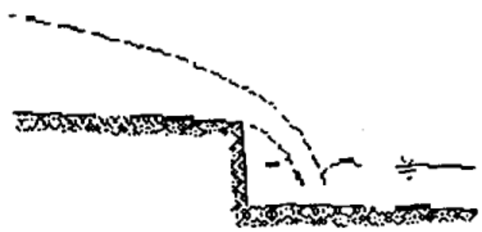

Figure **3.** Definition sketch for a change in canal cross-sections.

Under free-flow conditions, the depth at the brink of the overfall is taken to be the critical depth for the upstream section, and the flow rate is calculated as:

$$Q_f = \sqrt{\frac{gA^3}{T}} \quad \textbf{(23)}$$

where Q_f is the free-flow discharge (m^3/s); A is the upstream flow area at the section change (m^2); and T is the top width of flow (m). The downstream flow depth does not affect the discharge.

In the absence of a significant drop in the bed elevation at the section change, as described above, the flow regime is assumed to be submerged. In this case, the specific energy is balanced at the section change as follows:

$$h_u + \frac{V_u^2}{2g} = h_f + Z_f + h_d + \frac{V_d^2}{2g} \quad \textbf{(24)}$$

where h_u is the upstream depth (m); h_d is the downstream depth (m); h_f is the head loss (m); Z_f is the reach invert rise (m); V_u is the mean upstream flow velocity (m/s); and V_d is the mean downstream flow velocity (m/s). Note that Z_f takes a negative value for a drop in elevation. During a simulation, the model will not allow the head loss, h_f, to exceed 50% of the upstream specific energy at any given time step.

4. PHYSICAL CONFIGURATION FOR MODELING OF USC-PHLC SYSTEM

The physical layout of the USC-PHLC system was shown in Figure 1. Under this ARB-funded project, the Pehur High-Level Canal Consultants have prepared the design of the new PHLC that includes automatic downstream water-level control gates. They have also prepared the design for the remodeling of the existing Machai Branch Canal, which will not have automatic gates. Finally, the design consultants have designed the remodeling for the existing Maira Branch Canal that also includes automatic downstream water-level control gates.

A natural modeling configuration was to have three canals placed separately on the computer: (1) Machai Branch Canal; (2) Pehur High-Level Canal; and (3) Maira Branch Canal. In addition, there was a need to conduct unsteady flow simulations of the confluence where the PHLC discharges into the lower portion of the Machai Branch Canal. A schematic of this configuration containing four parcel is shown in Figure 4.

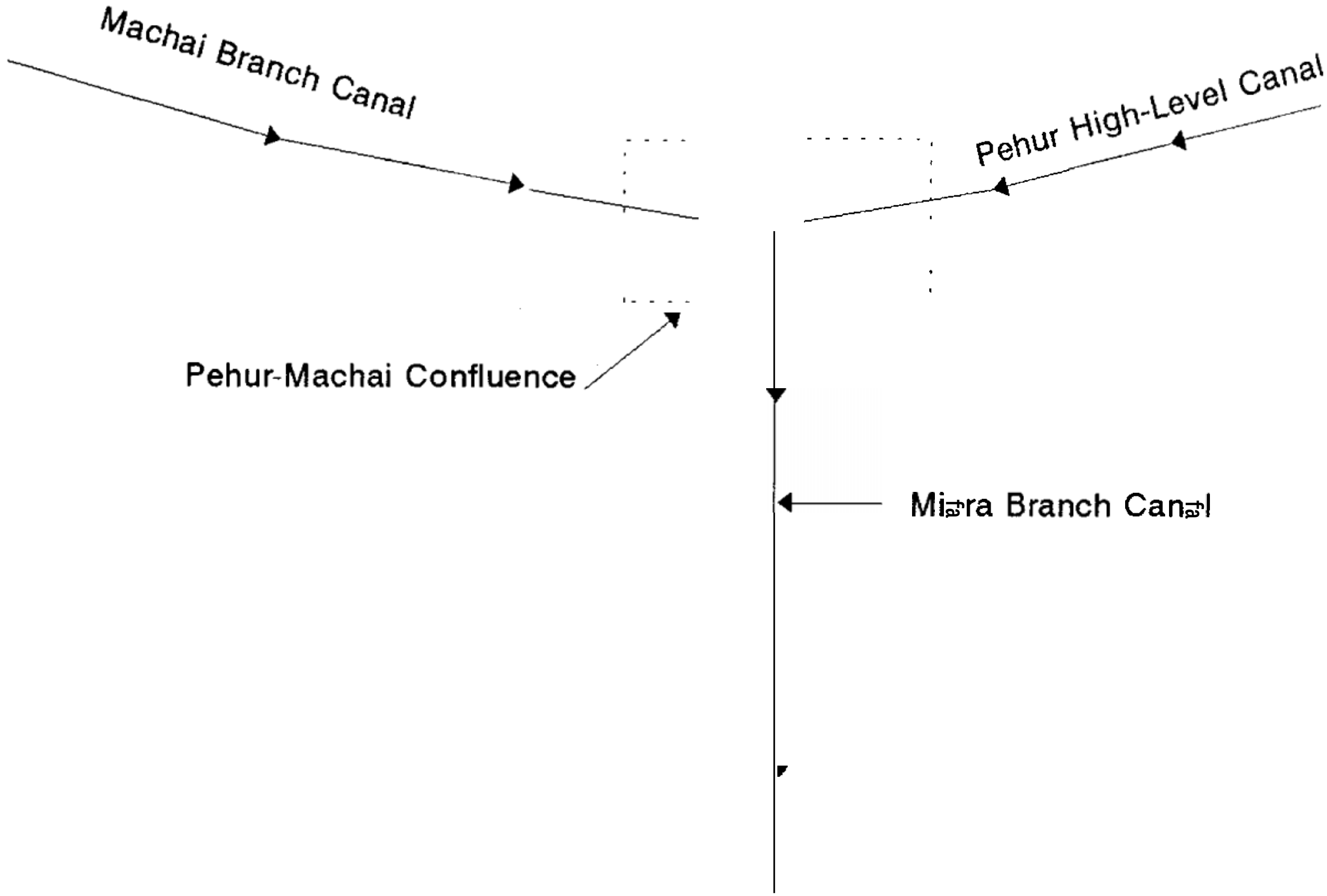

Figure 4. Schematic configuration for modeling of USC-PHLC system.

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