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Implementing SYMO in Phitsanulok Irrigation Project, Thailand

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Abstract
A number of mathematical models which can be used as decision-making tools in the management and operation of irrigation canal systems have been developed. The Irrigation System Management and Operation Model (SYMO) is one of them. It has the capability to simulate canal flow on a network with a branching configuration of any form. It includes a comprehensive database management system. It is interactive and is adequately supported by graphic capabilities. While its mathematical component had been adequately proven, its application to real-life irrigation situations is currently being evaluated. The Phitsanulok Irrigation Project in northern Thailand is being used as a case study. The results though preliminary indicate that the model is a potentially useful tool for improving performance of irrigation systems.

INTRODUCTION

Flow in open channels has been described in mathematical models for many years. Its use for practical purposes, however, started to receive significant impetus only recently with the advances in computing. The equivalent numerical model of an initial boundary value problem such as this demands the solution of a large number of simultaneous equations at the least but can now be handled rather easily. The direction, in fact, has been towards the packaging of these models into functionally-oriented user-friendly softwares that can be readily used for practical purposes with minimum effort required from the users.

The Irrigation System Management and Operation model, SYMO (Manguerra and Loof, 1992) is but one of the many models/softwares that can be used in the operation and management of irrigation canal systems. It has the capability to simulate canal flow on a network with a branching configuration of any form. It includes a comprehensive database management system; and is interactive and adequately supported by graphic capabilities. Its main computational component, the numerical solution of the governing unsteady flow equations had been proven mathematically correct (Manguerra et al., 1991).

In this paper, its application to a real irrigation system is presented. The Phitsanulok irrigation project

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located in the northern part of Thailand is currently used as a case study. The implementation is still on-going and the results are preliminary.

THE PHITSANULOK IRRIGATION PROJECT

The Phitsanulok Irrigation project consists of a concrete diversion dam (Naresuan dam) constructed across the Nan river and a distribution system which provides water to both banks of the river covering parts of three provinces of Thailand namely Phitsanulok, Pichit and Nakhon-sawan (Figure 1). The project involve two stages of development. Stage 1 which is already completed included the construction of an irrigation system on the right bank of the Nan river with an irrigable area of about 92,00 ha and on the upper left bank with an irrigable area of about 15,200 ha for a total area of about 107,200 ha. Stage 2 is expected to provide an additional irrigated area of over 54,000 ha on the lower left bank of the Nan river together with 66,400 ha in the extension area for a total area of 120,400 ha.

SYMO is currently implemented in the irrigated area at the right bank of the Nan river. The main distribution system (Figure 2) is composed of a 176 km long main canal, the head regulator of which is located 1600 m upstream of the Naresuan dam. The head regulator consists of three-6 meter wide radial sluice gates that has a combined discharge capacity of about 141 m^3/s. Moreover, the system includes a total of 107 laterals and sublaterals with a combined length of 721 km that includes 2,300 canal related structures. There are 25 cross-regulators (sluice gates) along the main canal which are about 10 km apart in the upstream reaches and about 7 km apart in the downstream reaches.

Irrigation activities in the area are controlled by three subprojects namely the Phlai-chumphon, Dong-setthi and Tha-bua subprojects. The Phlai-chumphon subproject is directly responsible for the area served by the main canal from km station 0+00 to km station 72+500 which includes the operation of 7 cross-regulators. The Dong-setthi subproject directly controls the operation of the main canal from km station 72+500 to km station 96+900 which includes the operation of three cross-regulators one of which controls the supply to an 82 km long lateral canal. The Tha-bua subproject controls the operation of the main canal from km station 96+900 to km station 175+940 which includes 15 cross regulators. It is evident the operation activities of a subproject are highly dependent on the activities of the other two. An operation and maintenance office coordinates the activities of the three subprojects to ensure that the whole project operates as one whole unit.

Operational problems

The main operational problem of the irrigation project can be described as two-fold, excess water during the wet season and limited water during the dry season. In both situations, the problem is always propagated from the upstream to the downstream reaches particularly to the Tha-bua area. It is further exacerbated because the downstream area is the least capable to withstand extreme conditions. This is normally true for any large distribution system where the quality of the downstream hydraulic system is significantly inferior than upstream.

In events of sudden excess water, the Tha-bua area usually finds itself on the receiving end of excessive loads of water. The situation is aggravated since the main canal downstream is much smaller in capacity. This often brings physical damages to the system downstream. Unless there exists regular repair and maintenance procedures, the hydraulic system downstream will continue to remain inferior. The drainage system which includes 96 drainage canals with a combined length of 512 km has been proven time and again to be inadequate. Moreover, the water level at the Pichit river where excess water is supposed to be finally disposed is higher than that of the draining canal thereby preventing evacuation of water by gravity. The mere closing of the head regulator will not solve the

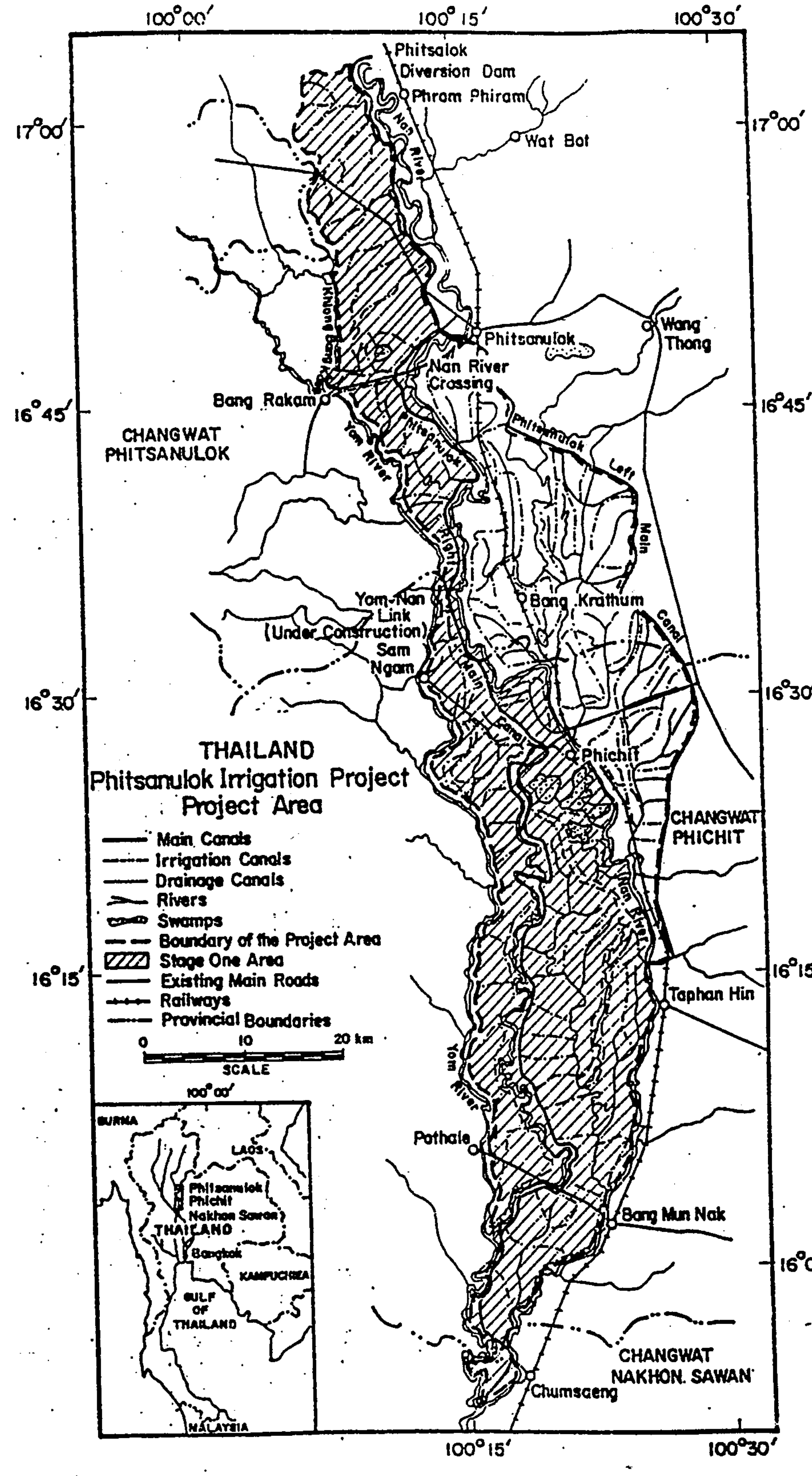


Fig. 1 General Layout of Phitsanulok Irrigation Project., Thailand

CANAL NETWORK

OF PHITSANULOK IRRIGATION PROJECT

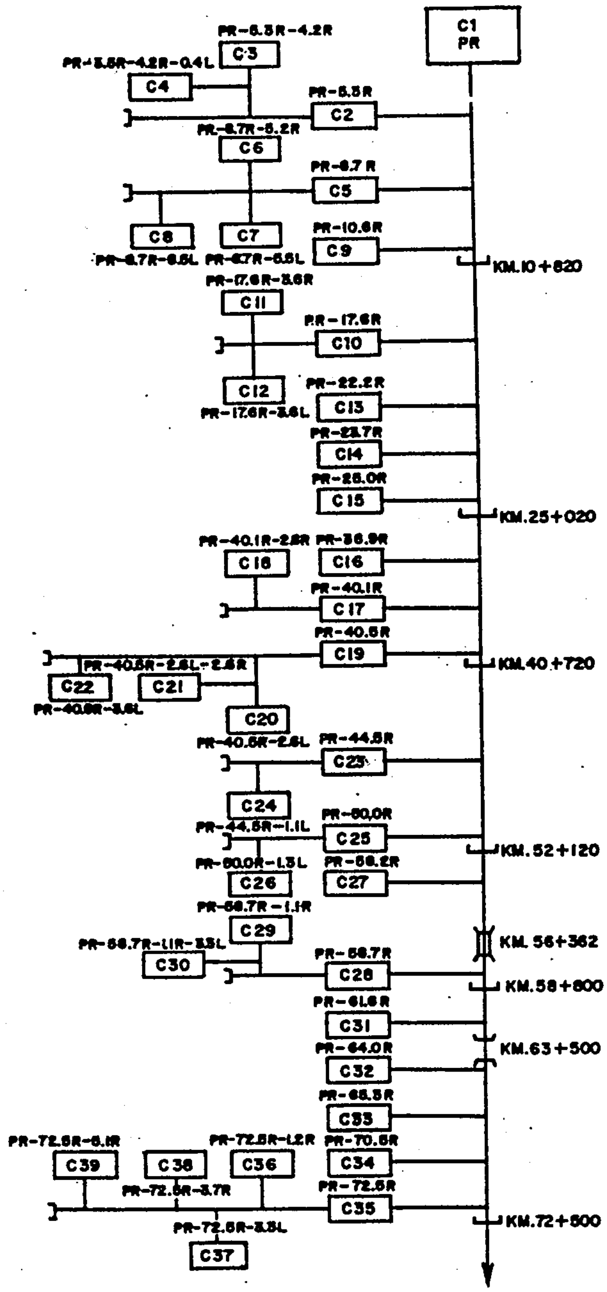


Figure 2. Canal network of Phitsanulok Irrigation Project

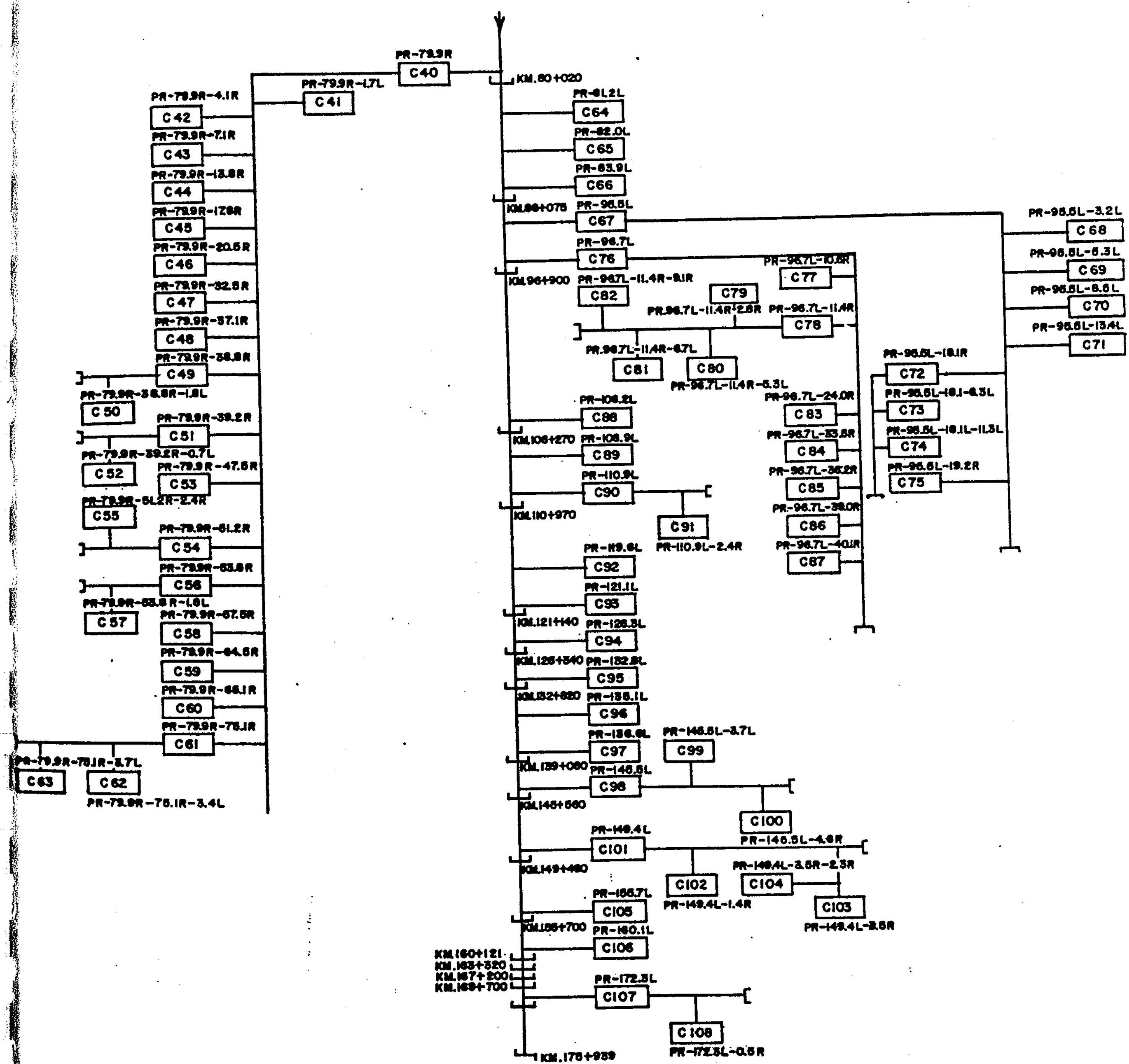


Figure 2 (cont.). Canal network of Phitsanulok Irrigation Project problem since excess water comes mainly from the whole irrigated area which serves as a large catchment basin particularly during heavy rains.

Communication and coordination is one of the keys to alleviate the problem. However, the lack of it among subprojects prevents them from making quick and coherent measures to minimize the unfavorable effects of flooding. While standard operating procedures are available in advance for extreme conditions like this, their implementation is usually uncoordinated and sporadic and lack the desired overall effect. The mode of communication among watermasters is mainly by radio which can be used for a mere distance of 7 km. Communication among zonemasters and gatetenders is facilitated by the use of bicycles and in some cases motorcycles. The system being understaffed makes the problem even worse. In one of the subprojects for example, only 3 watermasters, 16 zonemen and 45 gatetenders are assigned wherein 10 watermasters, 79 zonemen and 300 gatetenders are recommended during project appraisal.

When water is deficient, operation of the hydraulic system becomes more difficult and rigorous because of the extra activities brought about by water rotation procedures. While these procedures are available in advance, they are not strictly implemented and the conditions where they are supposed to be valid are non-existent. Illegal and unregulated diversions of flow particularly by direct pumping from the main canal are rampant and in some cases the gatetenders themselves make their own unscheduled diversions due to local pressures. It can be assumed that the gatetenders, being under continuous pressure from farmers, will from time to time succumbed to their irrigation requests even without the approval of his watermaster. It must be understood that once a part of the rotation procedure fails, a complete breakdown of the whole plan follows.

This assumes that the prescribed delivery schedule is appropriate which is not usually the case. It is ideal that rotation be limited to secondary and tertiary canals to minimize flow fluctuations in the mains. However, this is not possible due to the limited amount of water available. Because of the rotation procedures, the state of flow in the canal are more unstable for prolonged period of time. It has been mentioned though not verified that canal flow becomes stable again 10 days after the change assuming that there are no other gate operations after the change.

The problem is again not only due to the operational nature of the delivery schedule but also due to the limitations imposed by the hydraulic characteristics of the system. Plusquellec and Wickham (1985) noted that while RID canal designs are acceptable in situation of abundant water during the wet season, it poses several operational problems during the dry season. There are cases where the prescribed full supply level (FSL) is not enough to push water completely to subsequently-ordered canals. It was noted that since canal capacities were originally designed for supplemental irrigation during the wet season (0.8-1.0 lps/ha), the higher discharge requirements (1.5 lps/ha) during the dry season are basically more difficult to sustain. The situation becomes worse during the middle of the season where a complete breakdown in irrigation water plans results due to the servicing of unprogrammed areas. Farmers at the start of the dry season are able to cultivate unprogrammed areas by using shallow tube wells. However, during the later stages, these tubewells dry up and irrigation officials are then forced to supply water to sustain the growth of the crops in these unprogrammed areas.

In general, the problem can be summed up as coming from the higher level of management where rules and procedures are formulated and their verbatim implementation desired; and from the field operation level where these rules are actually manipulated or ignored in response to changing conditions. With a mathematical model opportunities for reconciling the diverging objectives between the two levels of management become available.

Opportunities for a mathematical model as a tool.

For large irrigation systems, the operation should be guided by a set of appropriate what-if scenario procedures. This necessitates adequate understanding of the flow phenomenon. However, because of the enormous database and the infinite possible scenarios required, and the rather involved mathematics that goes along with it, irrigation managers in the past have resorted to much simplified methods for describing the flow for a limited number of idealized scenarios. With the available models, the computational effort required from irrigation managers will be much lesser while their decision making capabilities are much more exercised.

As in any large irrigation system, the potential for improving the overall performance of the Phitsanulok Irrigation system is great. During periods of excess water for example, a combination of gate settings of the cross regulators can be obtained to provide maximum use of storage capacity of the main canal itself and delay propagation of flood downstream. When water supply is limited, a much more appropriate rotation plan can be formulated in advance.

The advantages are not limited to having access to practically infinite number of decision alternatives. A comprehensive and more accurate database of the project will be available paving way for more appropriate research and extension. Data collection activities will be more meaningful and directed. Morale of the irrigation community will be improved and irrigation drills or trainings can be done to improve response behavior in case of emergencies. As delivery schedules become more reliable, there will be lesser illegal water diversions and water-related conflicts making the system more manageable. Problem areas such as canals with excessive seepage, insufficient control structures and insufficient freeboard can be isolated and rehabilitated. Design can be evaluated and appropriate maintenance procedures implemented. Other opportunities such as night irrigation, volume-based assessment of irrigation fees and automatic data collection can be easily pursued.

MODEL DESCRIPTION

In modeling, emphasis should be placed on the modeling results and not on the model itself and its formulation. The main objective of the whole modeling exercise is not to achieve a comprehensively formulated model nor a database that is fully detailed but to produce results that are consistently representative of the actual field situation. This constitutes a modeling tradeoff between how far the model can be refined and how accurate the results should be to be acceptable.

The model is described by its computational and software attributes. Some of the computational attributes desired are that the analysis should be based on unsteady flow; and analysis of hydraulic structures should be included. The model should be valid for a wide range of irrigation scenarios in terms of heterogeneity of the problem domain (cross-section, hydraulic structures), temporal and spatial variation of excitations (seepage, recharge) and different shapes of boundary and initial conditions.

The Saint-Venant continuity and complete hydrodynamic equations are used to mathematically describe the unsteady behavior of flow in irrigation canals. An implicit finite difference scheme is used to obtain a numerically equivalent set of equations with user-specified weighting coefficients. Boundary conditions which make the numerical model determinate can be provided in the form of either discharge or stage hydrographs. The double sweep algorithm is employed to solve the resulting system of simultaneous equations. Steady state values are computed using the unsteady module. Keeping the boundary conditions equal to the steady values, the computation is allowed to iterate until such time when the variations in the state of flow satisfy a specified tolerance level. The analysis of flow through the hydraulic structures is based on existing discharge rating equations. The structures considered are vertical and radial sluice gates with or without side weirs, constant head orifices (CHO's), single orifice turnouts with or without downstream controlling weirs and wasteway weirs.

These are typically found in Phitsanulok irrigation system.

The software attributes should be enhanced to transform the model into a functional user-friendly tool. However, the accuracy and reliability of the computational part should not be compromised while enhancing the functionality of the software. SYMO's user-interface adopts established control and display techniques employed by commercially available packages. However, emphasis was given to the special interests of the target users by including familiar graphic displays such as network schematic; canal plans and profiles; and results in the form of discharge and stage hydrographs.

The user-system interface provides the user with complete control of the simulation. It is believed that only when there is control, the user will perceive the model an acceptable tool. Keeping this in mind, the user is provided with two interface levels that allow quick global and local access to the three main modules namely input, output and process (math) modules.

FIELD IMPLEMENTATION

The study is part of a larger research project "Improved operation and management of large scale irrigation systems" which is jointly sponsored by the Asian Institute of Technology in Thailand and GTZ (Germany). The project started in January 1989 and one of the immediately selected field sites is the Phitsanulok Irrigation system which is managed by the Royal Irrigation Department (RID) of Thailand. While SYMO was still in the early stages of development, numerous field visits were made to introduce the concept of using a mathematical model for operating the main systems. The feasibility of installing a telemetry system was obviously discussed but was realized to be not urgent. Data collection was started and a general feel of the operation and maintenance activities of the project was obtained. An RID staff closely involved in the study was invited to do graduate study in AIT which furthered the close collaboration between the two agencies. The significance of this action is that the student has competently took-over the task of implementing SYMO to his project as part of his graduate program (Chuenchooklin, 1992). Prior to this, a number of students have done preliminary modeling studies on localized parts of the irrigation project.

While the ultimate objective is to be able to implement the model over the entire distribution system, the presently concluded results were limited to the implementation of the model to the main canal particularly from km sta. 0+00 to km sta. 96+900. The system includes 10 reaches where a reach is defined in this paper as the shortest distance bounded by control structures both at the upstream and downstream end. The results are preliminary but are indicative of the potential application of the model.

Database preparation

Canal elements. The main canal is trapezoidal and is made of earthen material. The cross-sectional dimensions available were those produced after the construction i.e., as-built specifications. In this study, the cross-sections of selected points along the main canal were remeasured. Figure 3 shows how the present cross-section in reach 1 (km 0+00 - 10+620) differs from the original specifications. It was determined whether these deviations produce significant effect on the flow characteristic at a particular point. Assuming all other conditions the same, the model was used to simulate two identical flow scenarios over the first reach using the two sets of cross-sections. It can be noted from Figure 4 that water surface elevation is more sensitive to changes than the discharge. The mean absolute deviation in water surface elevations is equal to 3.3 cm which from the operational point of view i.e., available supply head at a particular offtake structure, could be significant. However, it should be noted that the figure does not indicate which hydrograph represents the real state of flow since the results were not compared to actual field data. This implies that one can get away with small deviations in the actual cross-section if the results provide a good representation of the real-life phenomenon. In contrast, the discharge hydrograph is not at all sensitive to the present deviations of the cross-section from the as-built specifications. The results in Figure 4 also include the effects of the deviations in canal bed slope.

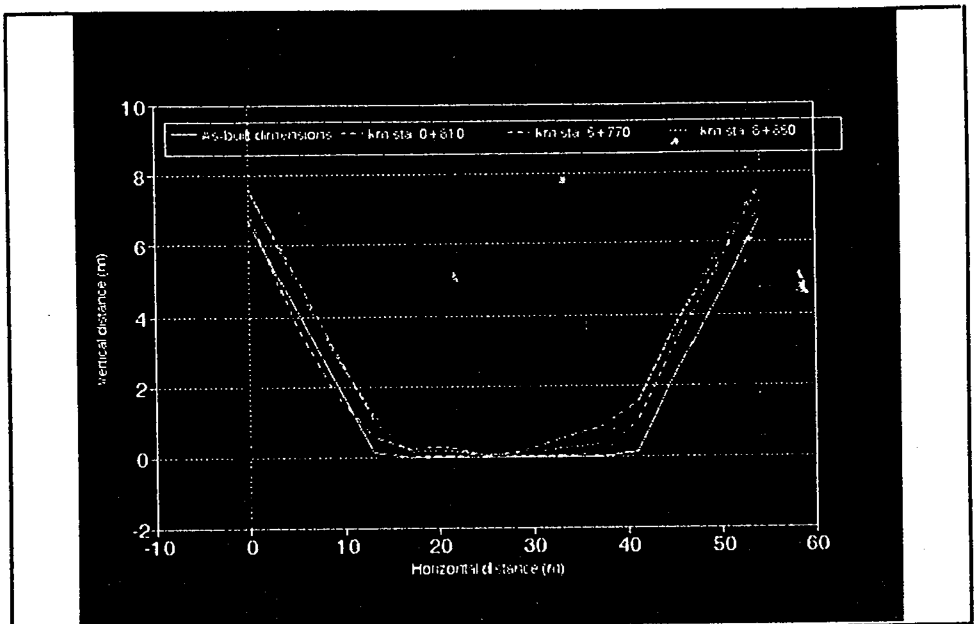


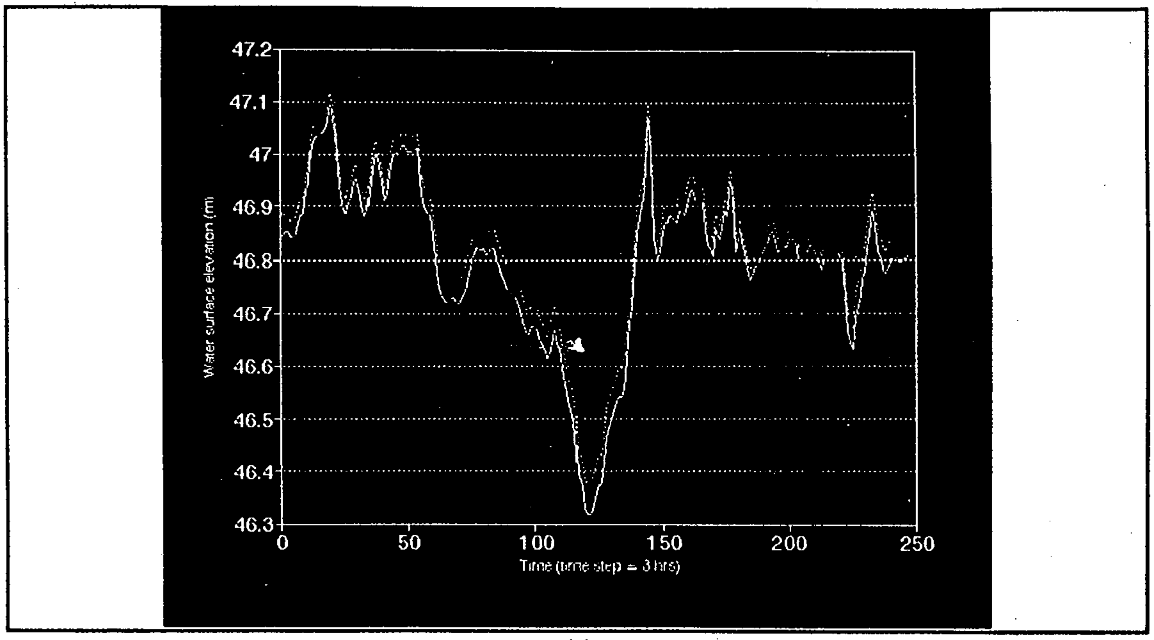
Figure 3. Comparison of actual cross-section of selected locations along the main canal from their as built specifications.

Hydraulic structures. The system that was considered includes 11 cross regulators, 22 lateral head regulators and numerous turnout structures. Except for the turnout structures, each was individually calibrated to obtain the discharge coefficient. A limitation of the model is that it presently considers discharge coefficient of a particular structure as constant. Thus, while the actual calibration curves provide the coefficient as a function of some combination of gate opening and immediate water levels upstream and downstream of the structure, the average values provided in Table 1 were used. The discharge levels obtained through the sluice gates when using these values were compared to the values obtained from equations developed by Swamee (1992). It can be noted in Figure 5 that the degree of correspondence is good particularly at intermediate flow regimes. It follows that at high flow regimes the actual flow is underestimated while at low flow regimes, it is overestimated.

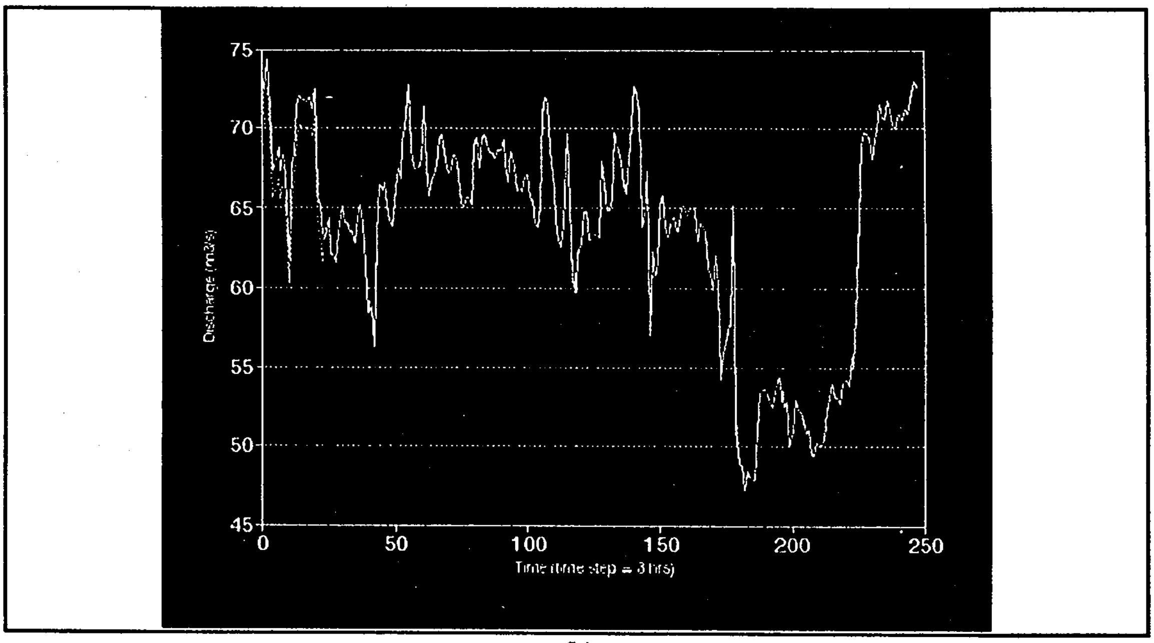
Canal seepage. The canal seepage is measured using the input-output method. This was carried out during the midseason where water elevations are maintained at FSLs. Table 2 shows the results.

Calibration of roughness coefficient

Calibration of roughness coefficient can be done in the field under steady uniform flow. However, accuracy of measurement is difficult to obtain and the flow conditions required for measurement are hard to maintain. Besides, the coefficient is believed to be varying with flow depth. With all other field parameters known, an alternative method is to use the model to calculate for the roughness coefficient. Since the inverse of the problem is required, a number of simulation runs can be done where in each run an arbitrarily selected set of roughness coefficients can be used. The run that provides the best fit between observed and simulated flow state will give the appropriate values for the roughness coefficient (Figure 6). To simplify the calibration process, it was assumed that



(a)



(b)

Figure 4. Comparison of (a) water surface elevation and (b) discharge hydrographs **obtained from** simulating flow over reach 1 as affected by cross-section.

Table 1. **Calibrated discharge coefficients (Cd) of hydraulic structures.**

Station (km)	Type of Structure	Total Width (m)	Sill Elevation (m)	Cd Average	R Square
a) Cross regulators					
0.0750	head regulator	18.0	42.739	0.780	0.979
10.620	sluice gate	18.0	40.961	0.709	0.968
25.020	sluice gate	18.0	40.548	0.639	0.961
40.720	sluice gate	18.0	39.295	0.729	0.986
52.120	sluice gate	18.0	38.581	0.817	0.997
58.800	sluice gate	18.0	38.055	0.714	0.999
63.511	sluice gate	18.0	38.790	0.466	0.998
72.500	sluice gate	18.0	34.708	1.035	0.939
80.020	sluice gate	12.0	35.304	0.691	0.997
88.075	sluice gate	12.0	34.503	0.936	0.921
96.900	sluice gate	6.0	33.974	0.682	0.992
b) Head regulators					
5.340	C-002	3.0	44.850	0.646	0.835
8.715	C-005	1.5	44.100	0.608	0.943
10.600	C-009	2.0	44.867	0.285	0.950
17.000	C-010	1.5	43.546	0.640	0.999
22.200	C-013	2.0	42.803	0.190	0.992
23.700	C-014	1.25	44.010	0.243	0.910
25.000	C-015	1.5	43.500	1.162	0.997
36.900	C-016	1.0	41.749	0.357	0.984
40.104	C-017	2.0	41.749	0.524	0.991
40.500	C-019	1.5	41.771	0.346	0.910
44.500	C-023	2.0	40.765	0.561	0.998
50.000	C-025	1.25	41.159	0.507	0.932
58.189	C-027	1.0	40.360	0.590	0.920
58.661	C-028	2.0	40.050	0.527	0.974
61.556	C-031	1.0	38.554	0.447	0.971
63.956	C-032	0.8	39.650	0.040	0.787
65.256	C-033	0.8	39.650	0.048	0.787
70.456	C-034	0.8	39.200	0.029	0.787
76.920	C-040	4.8	36.859	1.182	0.822
81.156	C-064	1.0	37.280	1.027	
95.538	C-067	4.0	34.438	0.640	0.938
96.702	C-076	3.6	34.397	0.898	0.980
c) CHOs Group					
		0.4		0.872	0.741
		0.5		0.633	0.815
		0.6		0.689	0.408

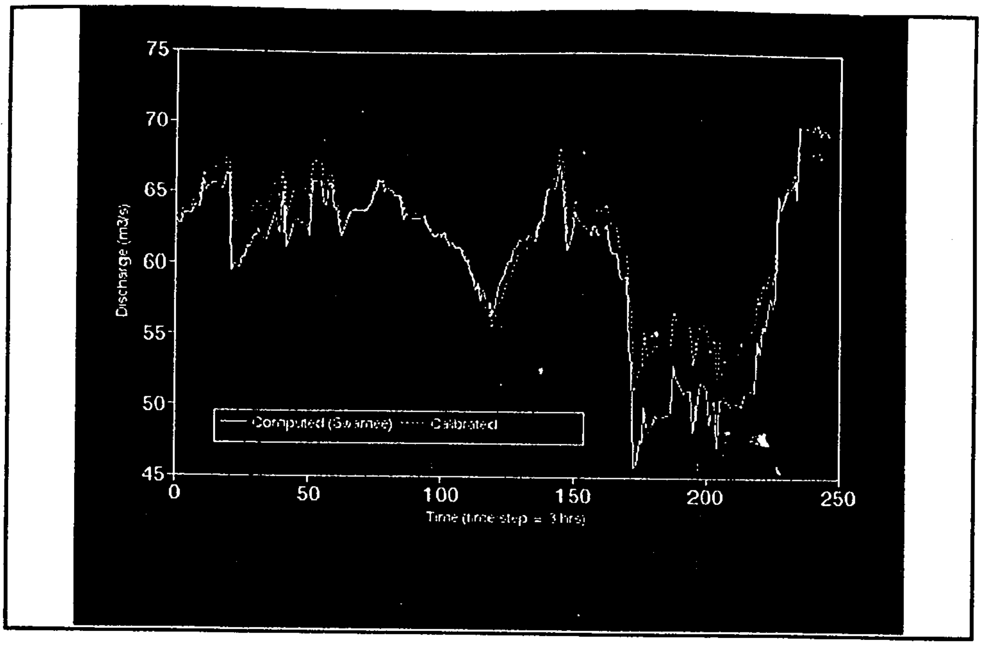


Figure 5. Comparison of discharge hydrograph through the cross regulator at km sta. 10+620 using discharge coefficient values measured in the field and values based on the equation of Swamee, 1992.

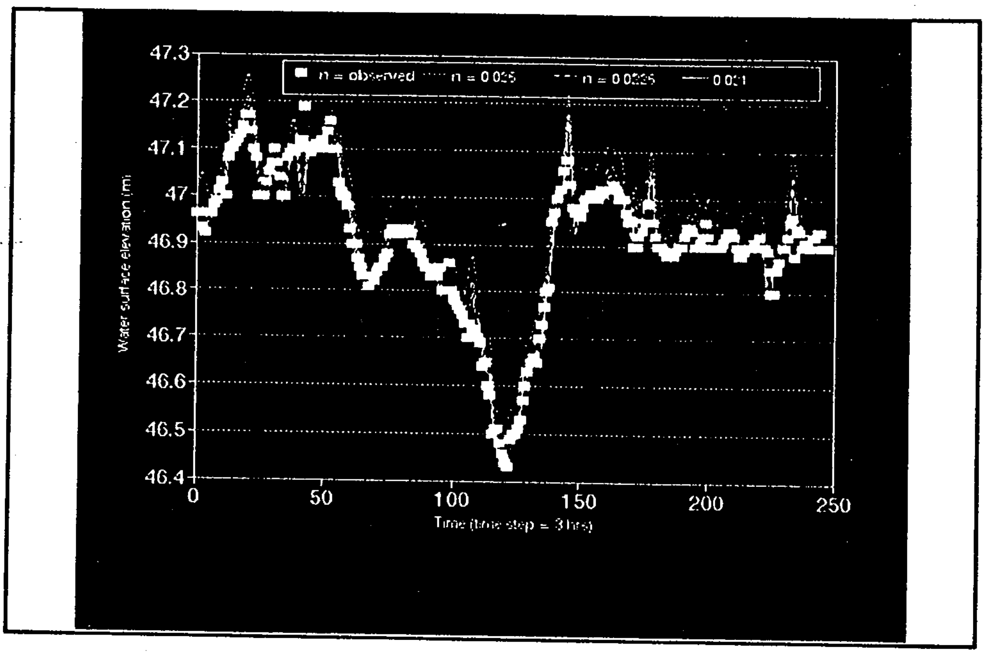


Figure 6. Comparison of water surface elevation hydrographs obtained at km 5.770 using different values for roughness.

roughness is uniform along a given reach.

The design roughness coefficient of the entire main canal is 0.0225. With the model, the roughness coefficients were found to be 0.021 for reach 1 and 2 (km 0+00 - 25+020); 0.016 for reach 3 (km 25+020 - 40+720); 0.024 for reach 4 and 5 (km 40+720 - 58+800); 0.020 for reach 6 (km 58+800 - 63+511); 0.033 for reach 7 and 8 (km 63+511 - 80+020); 0.023 for reach 9 (km 80+020 - 88+075); and 0.033 for reach 10 (km 80+020 - 96+900). These values can be considered representative of the entire flow regime. They depend on the magnitude of the other measurable parameters, thus, the values may vary from system to system. The significance of this is that the roughness coefficient is now transformed from being merely another physical parameter of the system to an important model coefficient the values of which can be customized for calibration purposes.

Table 2. Measurement of seepage loss from main canal.

Reach No.	Station (km)		Discharge(m3/s)		Wetted Perim. (m)	Lateral outflow(m3/s)	Seepage Loss (mm/min)
	u/s	d/s	u/s	d/s			
R1-2	0.780	13.084	61.387	47.318	36.194	8.196	0.791
R1-2	0.075	10.620	50.207	38.902	36.900	8.196	0.479
	0.780	13.084	64.031	58.147	34.281	3.364	0.358
R1-2	0.780	13.084	69.983	56.574	37.169	9.483	0.487
R1-2							
						average	0.529
R2-4	15.749	36.807	50.310	48.150	36.473	0.850	0.102
R3-4	27.500	41.694	30.240	23.618	30.000	3.527	0.436
R3-4	41.694	52.950	43.754	40.325	31.721	2.579	0.143
R3-4	27.500	42.848	57.464	51.866	32.255	5.396	0.024
R3-4	42.848	52.950	57.522	54.150	35.093	2.733	0.108
						average	0.178
R4-5	52.950	59.930	40.355	39.047	31.909	0.000	0.352
R6-7	68.250	74.000	41.821	40.232	31.879	0.210	0.451
R6-8	59.930	76.600	40.355	38.000	27.143	0.000	0.312
R7-8	74.000	80.900	12.186	11.271	20.283	0.000	0.392
R8-9	80.900	87.370	11.271	10.950	23.240	0.000	0.128
R8-10	76.600	98.240	38.000	7.696	20.414	27.630	0.363

Initial and Boundary conditions

Initial conditions are automatically calculated and are set equivalent to steady state values. This is based on the assumption that flow is steady before any gate adjustment is made. Moreover, these values are rarely available in most cases because of the difficulty in observing water surface and discharge levels at different locations at the same time.

A boundary condition can be in the form of a discharge or a water surface hydrograph. When simulation is set to network mode (several reaches are simulated simultaneously), then the only boundary conditions required are those at the head regulator of the main canal and at the upstream vicinity of every control structure. For practical purposes, the boundary required can be equivalent to a discharge hydrograph for the head regulator and the FSL's at the upstream vicinity of every cross regulator. Since the simulation was limited only to the main canal, downstream boundary conditions in terms of water surface elevations were arbitrarily provided for the offtake structures. When simulation is set to unit mode (simulation of a single reach) boundary conditions are required at the upstream and downstream vicinity of both control structures. This mode can be used for calibration purposes and when trying to isolate problem areas for simulation.

Automatic and manual mode of gate operation

The model provides the user two modes of gate operation. The automatic mode is used for computing the opening required to pass a particular discharge through a gate. Conversely, the manual mode is used for computing the actual flow diverted through a gate with a given opening. In field situations, the offtake structures which are more difficult to regulate can be set to manual mode while the head regulators of laterals can be set to automatic mode. While good simulation of flow across a manually set gate can be expected, it is extremely difficult to get the same result for gates set in automatic mode. This is because the model implicitly assumed that the gate movements are instantaneous. The error can be minimized by representing the intended gate movement with several incremental movements with time. It follows that the time step should be reduced during the gate movements.

While the simulation of the state of flow (discharge and water surface levels) at a particular point of the system at a given time is good, the results for the gate movements are inconsistent. As mentioned, this is due mainly to the inadequacy of the present model to describe the physics of the gate movement. In field situations, gate movements are minimized as much as possible, i.e., the gate settings are maintained constant for a specified length of time. Contrary to this, the model will most likely prescribe a gate movement that is erratic with time (Figure 7). However, it is noted that the prescribed gate settings need not be literally implemented. At the moment, its can be used as an indicator of what the general gate movement with time should be. The oscillations can be smoothed out, thus, average values over arbitrarily selected periods can be obtained.

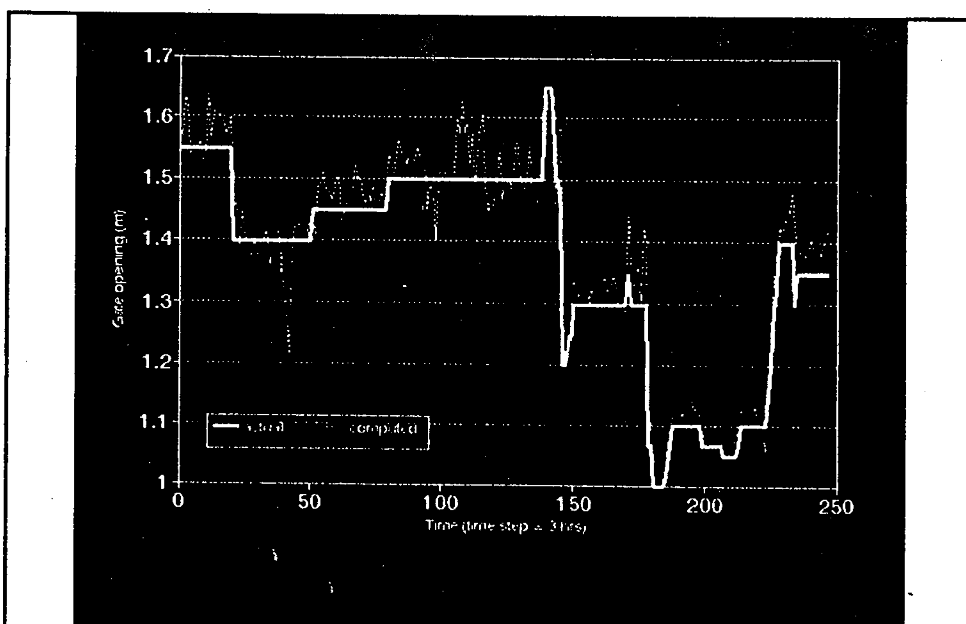


Figure 7. Comparison of actual and simulated gate opening of the main canal head regulator with time.

CONCLUSION

The Irrigation System Management and Operation Model (SYMO) in its present form can be used as a decision-making tool in the operation and management of irrigation canal system. Difficulties and problems due to the limitations of the present model formulation and the calibration and implementation part are presented. The results of the study though preliminary indicate that the model can bring about significant improvements in the performance of the system.

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