A RIVER BASIN NETWORK MODEL FOR CONJUNCTIVE USE OF SURFACE AND GROUNDWATER: PROGRAM CONSIM

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A RIVER BASIN NETWORK MODEL FOR CONJUNCTIVE USE OF SURFACE AND GROUNDWATER: PROGRAM CONSIM

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ABSTRACT

A computer model called CONSIM is presented for management of the conjunctive use of surface and subsurface storage in an interconnected stream-aquifer system. The model simultaneously simulates the hydrologic system and optimizes the water allocation period by period. The hydrologic model includes simulation of groundwater flow, subsurface storage, recharge and potential evapotranspiration. The methods selected for the hydrologic modeling are based on their simplicity, computational efficiency, and yet reasonable accuracy for planning and screening purposes. The optimization model uses a sequentially static approach which optimizes the water allocation one period at a time. The water resource system is set up as a network and solved by the out-of-kilter algorithm. For this particular study the model allocates water to various demands such as surface storage, irrigation, and other types of demands, according to relative priorities associates with each network link. These priorities are based primarily on water rights.

In order to demonstrate the usefulness of the model, a case study of the South Platte River section from the North Sterling inlet canal to the Julesburg gaging station in eastern Colorado was selected. The calibration results were satisfactory for the dry to average years but were less satisfactory for the wet years. A consistent underestimation of flows was noted during summer months which could be explained by unaccounted for direct reservoir inflows. The model as calibrated was deemed as a safe, conservative predictive tool. The priorities of vari-

ous demands obtained from the calibration were essentially the same as the relative ranking of water rights of those demands.

Two management alternatives were investigated. The first alternative considered artificial recharge to subsurface storage in comparison with current practice. With this alternative, water is spread over areas chosen for artificial recharge areas during non-irrigation season months. The results of the model analysis indicate that with proper artificial recharge, it is unlikely that subsurface storage would be severely depleted due to heavy groundwater pumping during drought periods.

The second groundwater management alternative studied the impact of changing downstream flow requirements governed by interstate compact agreements between Colorado and Nebraska. The results indicated that for the dry and average year conditions, any increase in the downstream flow would cause shortages in the State of Colorado.

The management model presented here successfully incorporates both aspects of conjunctive uses; i.e., the hydrologic simulation and the management analysis. Actual costs and benefits to the system components are not used in this study, but could easily be included in further work. Some advantages of this model over previously developed models include its ability to handle a large stream-aquifer system at reasonable cost, practicability and versatility for application and ease of use through conversational format type computer programming in data file creation.

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CHAPTER I

INTRODUCTION

A. CONFLICTS IN WATER USE

Available water supplies in the semi-arid west are under intensive pressure due to rapid growth in large population centers, and new energy industries could stimulate even greater rates of population influx. Irrigation is still the dominant use in most areas, but municipal and industrial expansion, in-stream flow needs and environmental concerns are creating serious conflicts for priority in water use. Economic pressures for transfer of water use to energy industries are particularly great in that they may be able to pay substantially more for water than agricultural interests.

Conflicts not only exist within States, but also between States since such rivers such as the Platte, Arkansas and Colorado cross several State boundaries. At the Federal level, law suits are being brought against certain States to reserve water for Federal lands and Indian reservations. International conflicts even arise over water quantity and quality for rivers such as the Colorado, which supplies water to Mexico.

As a further complication, energy concerns have stimulated interest in modifying the operating strategies of several large multi-purpose storage projects to accommodate increased hydropower production for peaking, and perhaps even base-load purposes. Hayes (1979) estimates there is potential for an additional 14 gigawatts production capacity from existing structures in the U.S. Also, certain existing multi-purpose projects are being reevaluated for possible add-on hydropower from low-head or run-of-the-river facilities. The Department of Energy's Small Scale Hydroelectric Program for facilities with 50 kilowatt to 50 megawatt generating capacity, and less than 20 meters head, resulted in 200 proposals for project development, with 32 of those coming from the west.

Enhancing one objective like hydropower will likely require the deemphasis of other water uses. For example, Culver and Millham (1981) estimate that losses in hydropower revenues in the Snake-Columbia River System in 1979 due to maintenance of commercial and recreational lockages amounted to $6.4 million. Trade-offs must be carefully evaluated, although the political and legal ramifications of these changes in operating policy are difficult to predict.

All of these conflicts are aggravated by drought, which is an ever present danger in the west. The severe drought of 1976-1977 clarified for many the fact that water resources in the west are limited and must be used wisely. It also called attention to the need for drought contingency planning, since many areas were unprepared for an extensive dry period. The need for contingency planning assumes greater significance as rapid growth continues. Since severity of drought is not only a function of available water supplies, but also the burden of demand on those supplies. Drier than normal periods in the past that were not acute enough to be classified as a drought may be so regarded in the future because of increasing demands.

Many streams in the west are already over-appropriated. That is, all normal flows in these streams have been claimed by someone, as well as high flows that occur only rarely. Actually, in many streams, total diversions greatly exceed total water supply due to downstream reuse of groundwater return flows to the stream from upstream water applications. For example, Evans (1977) claims that annual native surface flows in the South Platte River average around 2.3 million acre feet, whereas total diversions in the basin amount to over twice that. Many downstream users actually depend on these return flows so that improvement in upstream water use efficiency could actually harm downstream appropriators (i.e., one user's waste is another's supply). This suggests that conservation measures may not significantly reduce actual consumptive use. They can, however, affect the temporal distribution of streamflows, which may actually be detrimental in some cases. Groundwater return flows resulting from poor water use efficiency, particularly in irrigation, have a lagging effect due to slow groundwater movement. This increases the likelihood of a continuing supply to connected streams, even during the dry late summer and fall months. Conservation measures are important in the west, but their impacts on water availabilities in time and space must be carefully evaluated to determine the most cost-effective alternatives.

B. INTEGRATED BASIN MANAGEMENT

B.1 "Flexibility is the Key"

Farr (1977, page 21) states the "flexibility is the key to the future of managing Colorado's water; the ability to move water upstream or downstream, from one tributary to another, or from one high loss reservoir to another." This statement is valid for many other States in the west besides Colorado. Farr is proposing that integrated management of river basin water resources is of paramount importance in dealing with future conflicts in water use. Moving water "upstream or downstream" or "from one tributary to another," can be accomplished indirectly by innovative ways of regulating existing reservoirs. This must be achieved in such a way that individual water right holders in the basin are not injured by the integrated regulation scheme. Integrated basin management means "getting the most out of what we've got." Even in over-appropriated basins like the Cache la Poudre, which is tributary to the South Platte and a case study area for this project, large flows leave the basin each year that far exceed downstream legal requirements. Estimates as high as 40,000 acre feet per year have been put forth for uncaptured flows resulting from insufficient storage capacity.

B.2 Offstream and Onstream Storage

Development of irrigation projects in the west has resulted in construction of many small offstream reservoirs. They are often extremely "leaky," but this can enhance recharge of aquifers. A more serious problem is the capital needed for dam safety improvement and dredging operations for many of these reservoirs. In spite of these problems, it may be possible to more effectively utilize these reservoirs through operations in concert. Offstream storage is less effective in that large spring flows can be lost, even if there is storage space available, if there is insufficient capacity in canals diverting water to the reservoirs. Though additional onstream storage is needed, approval for such is becoming extremely difficult for political and financial reasons. This is also true for large transbasin diversion projects. Since only a few new projects will likely be authorized in the near future, it is important that these projects are carefully selected and then integrated into a total basin-wide management scheme. It may be possible to reduce investment in large onstream projects through better coordinated operation of existing reservoirs.

Integrated basin management need not be established by statute or changes in water law. In fact, several areas, with users in the Cache la Poudre Basin as an outstanding example, have for many years voluntarily instituted informal exchange agreements whereby users with junior water rights can be served with direct stream flows by releasing supplemental reservoir water to downstream senior water right holders.

B.3 Conjunctive Use of Surface and Groundwater

An important but often neglected resource in many basins is groundwater. Storage capacity in groundwater basins usually far exceeds what could ever be developed for surface systems and is an important hedge on drought. Groundwater is advantageous because evaporation is not a problem and investment in surface distribution systems is greatly reduced because groundwater can generally be pumped where needed. The major disadvantage is the expenditure of energy required in pumping. In addition, groundwater quality may not be acceptable in some cases.

Inclusion of groundwater as a significant portion of a total basin-wide management plan introduces an order of magnitude more complexity, particularly if major aquifers are hydraulically connected to surface waters. Within the last two decades, groundwater use has increased significantly, which has created conflict with surface water appropriators claiming that their rights are being damaged by stream depletions caused by pumping. An integrated, coordinated approach to this problem is needed, in spite of the complexity. Bittinger (1980) points out that Colorado is the only State that has passed legislation which encourages integrated management and use of surface and groundwater and recognizes the "intimate hydraulic relationships" between the two sources of water.

Even for large groundwater basins not directly connected with surface waters, such as the Ogallala aquifer system in the Great Plains region, heavy mining of groundwater reserves is creating grave concern and conflicts among groundwater users. An integrated approach is needed for managing this valuable and possibly unreplenishable resource.

Again, integrated basin-wide management involving groundwater is possible on a voluntary basis. As an example, Groundwater Appropriators of the South Platte (GASP) is a group of groundwater users that have decided that coordinating their efforts to work with surface water appropriators and consolidating their pumping operations is better than operating individually. Westen and Swain (1979) report on an innovative project involving GASP, the South Platte Ditch Company, and the Colorado Division of Water Resources to explore the feasibility of an integrated plan to divert and artificially recharge excess spring flows that would normally be wasted in order to minimize late season irrigation shortages.

B.4 Water Quality Impacts

Any integrated basin-wide management plan should include the prediction of long-term impacts on basin water quality. New energy industries are expected to create serious water quality problems. Additional water supplies will likely be needed for "diluting" the impacts of mining residuals and other contaminants.

As mentioned earlier, some areas may actually benefit from inefficient water use practices that augment streams with return flows during low-flow periods. However, there are other areas where inefficient irrigation practice actually contributes to the water quality problem. In Grand Valley, Colorado, for example, there exist certain shale formations with readily soluble or residual salts which are picked up in the irrigation flows and therefore contribute to degradation of connected surface waters. The groundwater resources in areas such as these are generally unusable due to high salt concentrations. In other areas where the groundwater is still usable, the basin management plan should include conjunctive use of surface and groundwater, while preserving and protecting groundwater quality. The slow movement of groundwater, combined with the fact that groundwater basins are the ultimate "sink" for salts left after consumptive use of water, makes them particularly vulnerable to salinization under improper management (Labadie and Khan, 1979).

C. PROGRAM CONSIM AS A MANAGEMENT TOOL

C.1 General Capabilities of CONSIM

Basin-wide integrated management has been suggested as a means of dealing with conflicts over priorities for water use and maximizing total beneficial uses of available water resources. It has also been proposed that this can be accomplished on a voluntary basis, which is attested by the past success of limited, voluntary water exchange agreements. Leadership and encouragement in this area should be provided first by the Office of the State Engineer, with help from various conservation districts and leading user organizations.

The increasing complexity of water management problems requires use of computer models as predictive tools for analyzing alternative schemes. A model is presented herein which is designed as just such a tool. Program CONSIM is not intended for use in day-to-day decisions in systems operations, but rather as a means of obtaining weekly or monthly management guidelines over an entire river basin or subbasin for a seasonal or multiyear planning horizon. The model can also be used as an initial hydrologic and economic screening tool for selecting and sizing new storage projects and analyzing tradeoffs between various conflicting water uses. The model is capable of assuring on a weekly or monthly basis that existing water right priorities are satisfied, as well as predicting the effects of transfer of water rights. Interactions between groundwater and surface water can be predicted as well as water quality impacts of alternative policies. The model input has been structured in an interactive, conversational format which encourages use by planners and managers with little computer experience.

C.2 Usage of CONSIM for Water Planners and Managers

CONSIM can be a valuable tool for State water planners, administrators, regional conservation districts, various user organizations, ditch companies, and municipalities. The model is designed to be a tool only. The results are only as good as the data input but the model can be used to help pinpoint data needs. Models can, and have been, abused, and model usage must be tempered with sound judgment and experience. The user should have a good understanding of the assumptions and approximations associated with the model. However, the interactive, conversational framework of CONSIM should greatly facilitate its usage.

At the State planning level, the model could be used for:

1. Screening various alternative proposed storage projects for cost-effectiveness.
2. Determining how much additional storage is needed in a river basin to meet new water demands and reservations.

3. Predicting the effects of proposed changes in water law (e.g., Farr (1977) has proposed that available water could be used more efficiently if water rights were based on a total monthly or seasonal volume, rather than on the basis of daily flow rates).
4. Predicting the effects of new water demands for energy development, and its (possibly adverse) impacts on other uses, particularly irrigated agriculture.
5. Comprehensive reevaluation of accepted water policy at the river basin level, including prediction of impacts of policies to enhance hydropower production.
6. Predicting impacts of minimum stream flows, as dictated by State and Federal instream use requirements and water quality regulations.
7. Analyzing the feasibility of new transbasin diversion schemes.
8. Predicting water quality impacts of alternative policies.

At the State administrative level, the river commissioners would benefit from the use of CONSIM for dealing with the following problems:

1. Determining innovative, integrated reservoir operation plans that maximize water use efficiency and minimize wasted outflows from the basin.
2. Planning real-time operations during drought periods (the model could be used to continually generate plans over a season as additional data become available).
3. Predicting the effects of transfer of water use.

4. Predicting the effects of conjunctive use of surface and groundwater, particularly artificial recharge and pumping schemes associated with "plans for augmentation."
5. Generating suggested exchange schemes for water users that would be implemented on a voluntary basis.
6. More effectively matching forecasts of seasonal water supply from snowpack data with estimated demand on a real-time basis.
7. Help document the experience of the administrator. This is particularly useful if an experienced river basin commissioner leaves his position for any reason. New, untrained personnel would find such a model invaluable.

Regional conservation districts would profit from use of CONSIM as a tool for analyzing the best ways of managing supplemental water resources, particularly during drought periods. Ditch companies could use CONSIM for designing voluntary exchange agreements among users and minimizing water shortages. Groundwater appropriators would find CONSIM valuable for coordinating with surface water appropriators, devising innovating pumping and recharge plans, and minimizing pumping costs. Optimal sizing and siting of artificial recharge and conveyance facilities could possibly be considered. CONSIM would be advantageous to municipalities for water supply and drought contingency planning and predicting effects of water conservation measures.

C.3 <u>Comparison with Other Models</u>

A summary of the capabilities of several existing models is given in Table 1.1. Program CONSIM has been synthesized from Programs MODSIM

Table 1.1. Summary of Capabilities of Several River Basin Models.

CAPABILITIES	MODELS A	B	C	D	E	F	G	H	I
1. optimizing capability?	No	Yes	No	No	Yes	No	No	Yes	Yes
2. accounts for groundwater-surface water interaction?	No	No	Yes	in part	in part	in part	in part	No	Yes
3. considers water rts.?	No	Yes	No	Yes	Yes	Yes	Yes	Yes	Yes
4. (a) easy to use?	Yes	Yes	?	No	?	Yes	No	Yes	Yes
(b) conversational format	No	No	No	No	No	No	No	Yes	Yes
5. suitable for planning?	Yes	Yes	Yes	Yes	Yes	No	?	Yes	Yes

A = HEC 3
B = SIMYLD
C = MITSIME
D = CORSIM
E = WBSM
F = WADIST
G = IPAWED
H = MODSIM
I = CONSIM

and MODSIME, developed by Labadie and Shafer (1979) and Shafer (1979), which are in turn based on Program SIMYLD, developed by the Texas Water Development Board (1972). Since SIMYLD is primarily a surface water model, major revisions have been required to modify SIMYLD to handle surface water-groundwater interactions. The conversational data management system has been added to facilitate model usability.

One of the most popular and well documented models available today is HEC 3: Reservoir System Analysis for Conservation, developed by the U.S. Army Corps of Engineers (1976). The primary disadvantages of HEC 3 are its inability to consider complex water rights and order of priority in diversions, lack of a groundwater component, and inability to perform optimizations. Reservoir operating rules must be established a priori, which may require a significant trial and error process in finding the best rules. The U.S. Army Corps of Engineers has also developed the SSARR model (1972), which suffers from the same disadvantages of HEC 3, and requires much more detail and diversity of data input.

Schreiber (1976) has extended the Massachusetts Institute of Technology Simulation Program MITSIM to consider groundwater interactions. The major disadvantages are: a detailed finite difference groundwater modeling approach with extensive data requirements; assumption of a constant percentage of applied water infiltrating to groundwater; water allocated to users on an upstream to downstream basis and not according to water rights, and no optimizing capabilities. Another model, CORSIM II (Fleming et al., 1975), considers water rights but inadequately includes groundwater interactions.

Perhaps the current model closest to the capabilities of CONSIM is Program WBSM (for Water Balance Simulation Model) (Bercha, 1981), developed for the Alberta Canada Environment Department. This model uses a network approach and has many of the same features as CONSIM. Program WBSM allows more complete operating criteria than CONSIM, but also requires a larger data base. It also does not seem to have been written for conversational, interactive usage. Though irrigation return flows are considered, WBSM lacks the capability of full consideration of stream-aquifer interactions, including pumping and artificial recharge.

A computerized model called WADIST has been developed which is based on the existing water rights structure in Colorado (Thaemert, 1976). Functionally, the model allocates surface water, including water held in storage, on a daily basis according to prespecified distribution criteria. Results indicate WADIST may be useful for daily management and record keeping, but is unsuitable for long range management studies. It may eventually prove useful as a submodule for CONSIM to obtain daily operational guidelines.

One of the more comprehensive models developed to date is IPAWED (Longenbaugh and Wymore, 1977), which is actually a series of individual modules requiring human interface, rather than a fully automatic program. The model includes groundwater interaction and basic water inputs, as well as an accounting of soil moisture changes and consumptive use for both irrigated and nonirrigated lands. The model was also directly linked to the Colorado Data Bank, which greatly facilitated data management.

The primary disadvantages of IPAWED are:

1. Use of HEC 3 as the reservoir system module, with associated disadvantages of that program.

2. Trial and error process of allocating and reallocating streamflows and reservoir releases to meet downstream water rights.

3. Groundwater return flows are assumed to occur in the same month that water is applied, when in actuality there may be considerable lagging of flows.

4. A sizable data requirement.

C.4 Unique Features of CONSIM

Program CONSIM has certain unique capabilities not found in these and other river basin models:

1. It is basically a simulation model with quasi-optimizing capability. That is, the model is designed to answer "what if" questions, but does perform a period-by-period optimization on system operations. The optimization is a sequentially static approach using network flow theory, rather than a fully dynamic optimization, which accounts for labeling it as a quasi-optimizing model. This is actually an advantage for the types of problems CONSIM is designed to analyze. Fully dynamic, deterministic optimization assumes perfect foreknowledge of future inflows, which is obviously an impossibility in practical management problems. There are, however, indirect methods for incorporating forecast information into management guidelines. Most other available river basin models are of the simulation type only, which means that optimal operating policies must be found by trial and error procedures that cannot guarantee that the final

solution is optimum. CONSIM effectively blends simulation and optimization together to accentuate the advantages of each approach.

2. Few other models adequately consider stream-aquifer interactions and channel seepage in conjunction with surface water regulation systems. Models are usually either designed for regulating surface systems alone, or managing groundwater aquifers and stream depletions. Rarely is there the capability of considering reservoir operations, pumping and artificial recharge decisions together. CONSIM has been specifically designed to evaluate the conjunctive operation of surface water and groundwater systems.

3. The optimizing feature in CONSIM is particularly advantageous in including formal water right priorities and informal exchange agreements in the model. Most other models are not capable of considering water rights, or attempt to satisfy them on a trial and error basis.

4. As alluded to previously, most models are unfortunately written by modelers for other modelers. Development of CONSIM has been carried out with the user in mind. It is believed that the conversational format will encourage model usage by water planners and managers with little computer background.

5. An attempt has been made to design CONSIM to be compatible with data that would normally be readily available to a water planner or manager and is consistent with the

degree of accuracy needed in water planning and management studies. Though the model is not suitable for day-to-day decisions, it also does not require the enormous data base needed for such models. CONSIM could be used to develop weekly or monthly guidelines that could be input to more detailed models.

D. PROJECT OBJECTIVES

The following is a listing of the original objectives of this research and a discussion of degree of achievement. This will identify the work that has been accomplished and suggestions for future work:

1. Construct a planning and management model which simulates the dynamics of a complex river basin system under desired levels of aggregation in time and space. The following modules should be included:

   (a) virgin streamflow

   (b) groundwater-surface water interaction

   (c) system storage, transport, and distribution morphology

   (d) consumptive water use

   (e) nonbeneficial consumptive losses

   (f) inclusion of water rights and informal institutional structures

   (g) concentrations of important water quality constituents

   Achievement: All of these aspects have been included in Program CONSIM, with the exception of the water quality component. A water quality module is available, but needs further modification to consider water quality aspects of stream-aquifer interaction. It is believed that the proper balance between model sophistication and suitability for planning and broad management on a weekly or monthly basis has been achieved in Program CONSIM. As a part of this project, a real-time streamflow forecasting model has been developed by Lazaro et al. (1981) which utilizes the Kalman filter. The second order statistical information provided with the forecasts generated by this model allow the possibility of explicitly including risk in management guidelines.

2. Design the model input format to facilitate use by the planner and manager in asking important policy questions: Achievement: The interactive, conversational input format greatly facilitates use by planners and managers. Hyrologic input data are stored on separate files, which makes it easy to change, say, streamflow inputs for critical period analysis. A special editing capability is included for examining several sites and sizes of proposed reservoirs without redesigning the system network or drastically modifying the input. Other options are available, which are discussed in this report.

3. Design the model output format in such a way that it is usable and understandable to the planner and manager. The output should include:

   (a) available water in storage

(b) river flow at selected points

(c) diversion quantities

(d) possible water shortages

(e) quality of river flows at selected points

Achievement: All of this information is available for output in a complete or summarized form. High speed printer plots are available for water storage and flows, and concentrations of important water quality constituents at selected locations.

4. Develop guidelines for decomposing a basin into a number of interlinked subbasins which can be analyzed independently and then reconnected.

   Achievement: A decomposition procedure has been designed and is presented in a report by Labadie, et al. (1980). More computational experience with the algorithm is needed.

5. Design the model to interact with the Colorado Water Data Bank.

   Achievement: Direct interaction has not been accomplished to date, but the Data Bank has been heavily used in obtaining data for this project.

6. Incorporate an automatic calibration capability into the model.

   Achievement: This has been accomplished using an optimizing algorithm which interacts with Program CONSIM, and is presented in reports by Phamwon (1982) and Ault (1981).

7. Design the calibrated model to perform sensitivity and impact analyses.

   Achievement: Certain conversational editing capabilities are included in CONSIM which facilitate sensitivity analysis once basic data files are set up.

8. Include an optimizing capability in the model to the degree that at least some flexibility exists in operation of the system.

   Achievement: The model uses the out-of-kilter algorithm for performing cost minimization of a flow network.

9. Provide guidelines as to appropriate degrees of aggregation of the components listed in Objective #1.

   Achievement: Aggregation of system components has been used in the case studies for this project, but more work is needed to generalize guidelines.

10. Select one of the river basins in Colorado to serve as a case study both to demonstrate the capabilities of the model and provide valuable information for ongoing water policy studies.

    Achievement: The Lower South Platte in Colorado and its tributary the Cache la Poudre basin have been used as case studies.

E. TECHNOLOGY TRANSFER

E.1 Briefings and Presentations

Extensive technology transfer activities have been carried out in conjunction with this project. Several special briefings on the practical usefulness of Programs MODSIM and CONSIM in addressing many of the pressing water problems in Colorado have been given to a number of important State legislators, local and state governmental agencies, conservation districts, various user organizations, and a number of engineering practitioners and water attorneys.

1. Presentation to the Interim Committee on Agriculture of the General Assembly, State of Colorado. [July 21, 1980]

2. Briefing before the Legislative Council Committee on Energy, Colorado General Assembly. [August 1980]

3. Briefing before the Colorado State Legislative Council. [1980]

4. Special briefing to State Senator Ted Strickland.

5. Special presentation to the Office of the State Engineer, Colorado Division of Water Resources. [Oct., 1980]. Also attended by representatives from:

   Colorado Water Conservation Board
   Denver Water Department
   U.S. Geological Survey
   U.S. Soil Conservation Service

6. Special presentation at Colorado State University [May 12, 1981] attended by representatives from:

   Colorado Water Conservation Board
   Poudre River Commissioner, Colorado Division of Water Resources
   Northern Colorado Water Conservation District
   City of Ft. Collins, Department of Public Works
   Laramie Country Farm Bureau
   Cache la Poudre Irrigation Co.
   North Poudre Irrigation Co.
   Cache la Poudre Water Users Assoc.
   Larimer and Weld Irrigation Co.
   New Mercer Ditch Co.
   Windsor Reservoir Co.

7. Presentation at the Urban Water Management and Conservation Workshop, sponsored by the Colorado Water Resources

Research Institute, Colorado State University, Feb. 16, 1981; attended by public works officials from several cities in Colorado (discussed use of MODSIM and CONSIM for drought contingency planning).

8. Presentation at the Voluntary Basin-Wide Management Seminar for Water Attorneys, sponsored by the Colorado Water Resources Research Institute, Colorado State University, April 20, 1982.

9. Presentation at the Computer Assisted Water Management Seminar and Workshop for Engineers, sponsored by the Colorado Water Resources Research Institute, November 17 and 18, 1982.

The emphasis in these briefings has been on presenting the model in a clear, nontechnical fashion. Feedback has been positive on these presentations, and opportunities for use of CONSIM for a wide range of water planning and management problems in Colorado seem to be expanding. As a result of these interactions, copies of the CONSIM Program and documentation have been sent to a number of agencies, firms, and individuals requesting them. It is hoped that CONSIM will become an accepted and oft-used planning and management tool in Colorado, and possibly other States. Work will continue, as available funding allows, in updating, modifying and improving CONSIM as more experience with its usage is gained.

E.2 <u>Publications</u>

In the early stages of this project, the synthesized model MODSIM from which CONSIM was subsequently developed, was used to evaluate the impacts of alternative management schemes for three case studies within the Cache la Poudre River basin:

1. Determining if recreation opportunities could be provided in selected high mountain reservoirs by maintaining satisfactory monthly storage levels without injury to downstream water users.
2. Determining if sufficient reusable effluent from the City of Fort Collins, Colorado is available (given an assumed hydrological sequence) to meet monthly water demands for a proposed coal-fired power generating plant.
3. Integration of a streamflow forecasting model with MODSIM for finding real-time, multireservoir control strategies in the Cache la Poudre basin that minimize wasted basin outflows and maximize water use.

These were real problems and not simply hypothetical exercises. The key to each problem was a coordinated approach to operating a multi-reservoir system. Close interaction was maintained between public agencies and private consultants involved in these projects. Publications resulting from these previous studies are listed as follows:

1. Labadie, J. W. and J. M. Shafer, "Water Management Model for Front Range River Basins," Technical Report No. 16, Colorado Water Resources Research Institute, Colorado State University, Ft. Collins, Colo., April 1979.
2. Shafer, J., "An Interactive River Basin Water Management Model: Synthesis and Application," Technical Report No. 18, Colorado Water Resources Research Institute, Colorado State University, Ft. Collins, Colo., August, 1979.

3. Labadie, J. W., J. M. Shafer, and R. Aukerman, "Recreational Enhancement of High Country Reservoirs," Water Resources Bulletin, Vol. 16, No. 3, June 1980.

4. Shafer, J. M., J. W. Labadie and E. B. Jones, "Firm Water Supply for a Coal-Fired Power Plant in North Central Colorado through Integrated Multireservoir Management," Proceedings of the Symposium on Surface-Water Impoundments, ASCE, Minneapolis, Minnesota, June 1-5, 1980.

5. Shafer, J. M., J. W. Labadie, and E. B. Jones, "Analysis of Firm Water Supply Under Complex Institutional Constraints," Water Resources Bulletin, Vol. 17, No. 3, June 1981.

6. Lazaro, R., J. W. Labadie and J. Salas, "State-Space Streamflow Forecasting Model for Optimal River Basin Management," Proceedings of the International Symposium on Real-Time Operation of Hydrosystems, University of Waterloo, Waterloo, Ontario, Canada, June 24-26, 1981.

7. Lazaro, R. C., "Adaptive Real-Time Streamflow Forecasting Model for Hydrosystem Operational Planning," Ph.D. Dissertation, Colorado State University, Ft. Collins, Colorado, 1981.

8. Lazaro, R., J. Labadie, and J. D. Salas, "Optimal Management of Multireservoir Systems Using Streamflow Forecasts," Proceedings of the International Conference on Time Series Methods in Hydrosciences, Burlington, Ontario, Canada, October 6-8, 1981.

F. SCOPE OF REPORT

It has been emphasized that CONSIM can be used for planning, sizing and locating facilities, and for obtaining long term management guidelines. It can also be used for real-time operational planning through input of forecasted flow sequences expected to occur over a certain lead time, using the model developed by Lazaro (1981). The model can be used in real-time to obtain an optimal set of weekly or monthly average flow and storage targets. Only the guidelines for the current period need be implemented since data can be observed and processed and this information used to update forecast model parameters for issuing a new set of forecasted flows. Subsequently, an updated operational plan of water allocation for implementation is generated.

Chapter II of this report presents the hydrologic basis of CONSIM and the assumptions associated with stream-aquifer interaction, runoff estimation, return flows, pumpage, and evapotranspiration predictions. Chapter III then incorporates the models of Chapter II into the network flow structure of CONSIM for river basin management decisions. A case study for Program CONSIM is presented in Chapter IV which uses the lower South Platte River basin in Colorado to demonstrate the ability of CONSIM to adequately predict stream-aquifer interaction and return flows, and to provide an indication of the kinds of river basin conjunctive use alternatives that can be examined. Appendices have been added to this report to provide detailed programmer documentation of CONSIM, including input data requirements and output format and options.

CHAPTER II

PROGRAM CONSIM: HYDROLOGIC COMPONENTS

A. OVERVIEW

Stream-aquifer systems are usually described by one or two-dimensional groundwater flow equations, detailed mathematical derivation of which can be found elsewhere, such as Glover (1974). These equations may be applied in a simple lumped parameter form or in a discrete finite difference (or finite element) form. The finite difference approach has been widely used to simulate stream-aquifer systems.

The application of linear system theory to the groundwater flow equations has been extensively explored by several investigators such as Maddock (1972), Morel-Seytoux and Daly (1975), and Illangasekare (1978). Though various names are given to this approach, such as the technological function, discrete kernel, influence coefficient, or response function approach, the main idea is to find the response of the groundwater system (e.g., groundwater level at a particular location) to an external excitation such as groundwater pumping, river stage, or recharge. This approach has the advantage that the response coefficients need be calculated only once for any system configuration and time period and then stored for further use. This makes detailed simulation of the stream-aquifer system operation more manageable. All of this is contingent upon the ability to reasonably model the system in a linear fashion.

The approach adopted for Program CONSIM is similar to the response function approach. The system is decomposed into smaller subsystems or subareas which include one or more surface water diversions, surface water reservoirs, and groups of pumping wells. That is, some aggregation of system elements is necessary in order to couple the groundwater hydrologic model to the management model without exhorbitant computational cost. It is expected that defining a large number of smaller sized subareas will result in greater accuracy, but data collection and computational costs will increase accordingly.

The hydrologic system components included in CONSIM are summarized in Table 2.1. Due to computational costs, some of the less important components are not considered, as noted in the remark column of Table 2.1 The following assumptions are used to further reduce the complexity of the subsystem components.

1. Effective precipitation is uniformly distributed over a subarea. Since the model does not perform a soil moisture accounting, an indirect way is to provide threshold values of precipitation above which surface runoff occurs and to adjust recharge fraction model parameters. These can be varied to account for expected soil moisture conditions.

2. Groundwater pumping is uniformly withdrawn over a subarea. This also holds for groundwater pumpage applied over the subarea for irrigation. This is considered a valid assumption, particularly if there are several wells distributed over the subarea.

3. Reservoir seepage is applied as a point source to the groundwater reservoir. This assumption is valid only if the reservoir is small compared to the subarea.

Table 2.1 Summary of Major Hydrologic System Components (from Illangasekare, 1978).

No.	Description	Remark*
1	Precipitation	*
2	Canal seepage (canal not in hydraulic connection with water table)	x
3	Reservoir evaporation	*
4	Reservoir seepage	*
5	Evapotranspiration from irrigated lands	*
6	Deep percolation from irrigated lands	*
7	Aquifer withdrawal by pumping	*
8	Tributary seepage (stream not in hydraulic connection with water table)	x
9	Phreatophyte losses	*
10	Aquifer return flow to stream	*
11	Surface return flow to stream	*
12	Effective deep percolation from precipitation	*
13	Upstream inflow	*
14	Diversion from stream to reservoirs	*
15	Reservoir releases	*
16	Diversion to ditches supplying irrigated land	*
17	Tributary inflows to stream	*
18	Downstream outflow	*
19	Canal seepage (canal in hydraulic connection with water table)	*
20	Aquifer return flow to tributary	x
21	Groundwater inflow	*
22	Groundwater outflow	*

Note: * included in CONSIM

x not included in CONSIM

4. Canal seepage is applied as a line source for the groundwater reservoir, assuming an average parallel distance from the canal to the stream is reasonably representative.
5. Surface water irrigation is uniformly applied over the subarea. As long as the subareas are not too large, this is generally a reasonable assumption.
6. Surface water seepage is assumed to totally reach the groundwater table during the time period considered. If weekly time increments are used, it may be necessary to lag these flows.
7. Aquifer and stream response to water application or water withdrawal is described by linear processes and superposition is applicable. The groundwater basin can therefore be separately modeled for uniformly applied or withdrawn water, for line source canal water, and for point source reservoir seepage. These amounts are algebraically added in the groundwater basin accounting.

Again, assuming an appropriate level of physical decomposition is selected for the system, the above assumptions would appear to be reasonable.

B. STREAM-AQUIFER MODEL

B.1. Uniformly Distributed Water Application

The interaction of a water table aquifer receiving recharge from irrigation and precipitation, and an interconnected stream, can be modeled utilizing the method developed by Maasland (1959). This method was developed for a parallel drain system and can be applied to a stream-aquifer system as well. The idealized parallel drain system is shown in Figure 2.1.

The nonlinear partial differential equation for one-dimensional groundwater flow is

$$K\frac{\partial}{\partial x}(d+h)\frac{\partial h}{\partial x} = S\frac{\partial h}{\partial t} \qquad (2.1)$$

where K = permeability of the aquifer

d = original saturated thickness of the aquifer

S = specific yield of the aquifer

h = height of the water table measured upward from the assumed original stable water table level

x = distance measured along the path of flow

t = time.

By assuming that h is small compared to d, the linearized form of equation 2.1 is obtained as

$$\alpha\frac{\partial^2 h}{\partial x^2} = \frac{\partial h}{\partial t} \qquad (2.2)$$

where $\alpha = \frac{T}{S}$

T = transmissivity, which is equal to K·d

$h = 0$ when $x = 0$ for $t > 0$

$h = 0$ when $x = L$ for $t > 0$

$h = H$ when $t = 0$ for $0 < x < L$

Maasland obtained the solution as

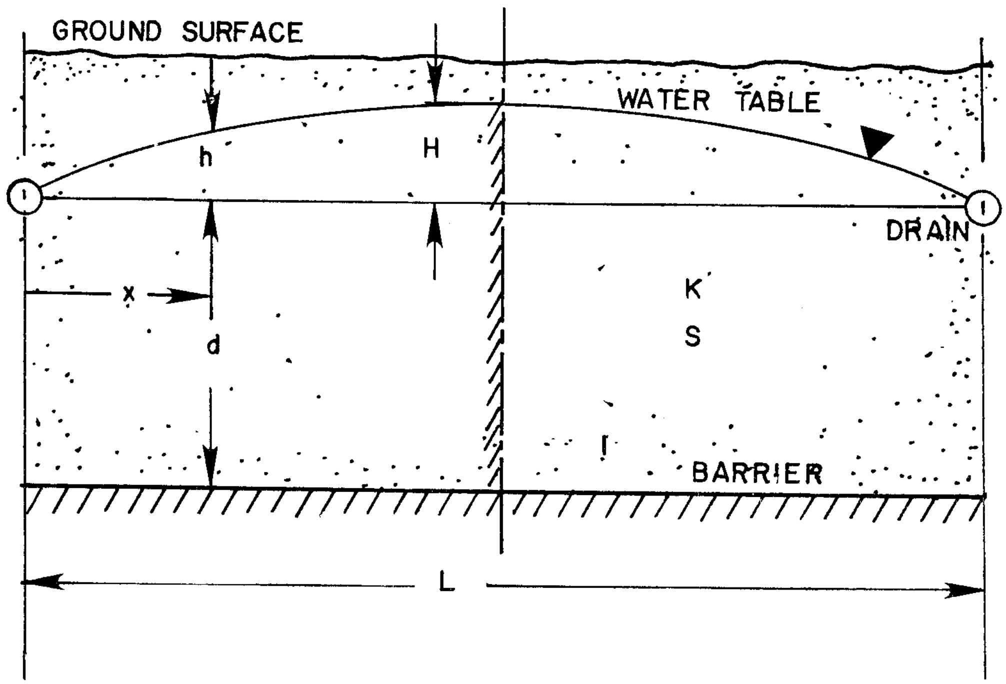

Figure 2.1. Idealization of Parallel Drain System (from Glover, 1974).

$$h = \frac{4H}{\pi} \sum_{n=1,3,5..}^{\infty} \frac{1}{n} \exp(\frac{-n^2\pi^2\alpha t}{L^2}) \sin(\frac{n\pi x}{L}) \tag{2.3}$$

where H = initial uniform height of recharge water

L = spacing of the parallel drains

The volume of water remaining to be drained is

$$V_d = S \int_0^L h \, dx \tag{2.4}$$

and the fraction remaining to be drained is

$$F = \frac{V_d}{V} \tag{2.5}$$

where V = initial drainable volume

= S·H·L

Therefore

$$F = \frac{S \int_0^L h \, dx}{S \cdot H \cdot L} \tag{2.6}$$

With the substitution of h from equation 2.3 and integration, we have

$$F = \frac{8}{\pi^2} \sum_{n=1,3,5..}^{\infty} (\frac{1}{n^2} \exp(-n^2\pi^2\frac{\alpha t}{L^2}) \tag{2.7}$$

This represents the fraction of the total initially drainable volume in the aquifer at the end of time t that is available for flow to the drains. For any time t from the beginning of recharge, F can be predetermined. The differencing of successive F values over two adjacent time periods represents the flow fraction to the drains during that time interval.

Consider the idealized stream-aquifer system as shown in Figure 2.2. The river is assumed to be located at the center of the valley. The solution described above can be applied directly with L equal to the valley width. The analogy is applicable since the middle section of the parallel drains is a no-flow boundary and is analogous to either the left boundary or the right boundary of the stream-aquifer system. If the parallel drain system is divided in half at the no flow boundary and rearranged to bring the drains into coincidence, we have a direct analogy with the stream-aquifer system. The drains are replaced by the river and the flow to the drains represents return flow to the river.

In cases where the river is not located in the center of the valley, the above solution is still applicable with L equal to twice the width of each side of the valley (i.e., $L^2 = 4W^2$). The fraction F can be determined for each side of the valley and return flows from each side computed separately.

Let N be the total number of time intervals of length Δt and I_k the recharge rate during the k-th time interval, where $k < N$, as shown in Figure 2.3. The fraction of return flow to the river during time interval k is

$$\delta_k = F_{k-1} - F_k \qquad (2.8)$$

where δ_k is a unit response for a recharge rate I of unity and F_k is computed with $t = k\Delta t$ in equation 2.7. Therefore,

$$\delta_{k-\tau+1} = F_{k-\tau} - F_{k-\tau+1} \qquad (2.9)$$

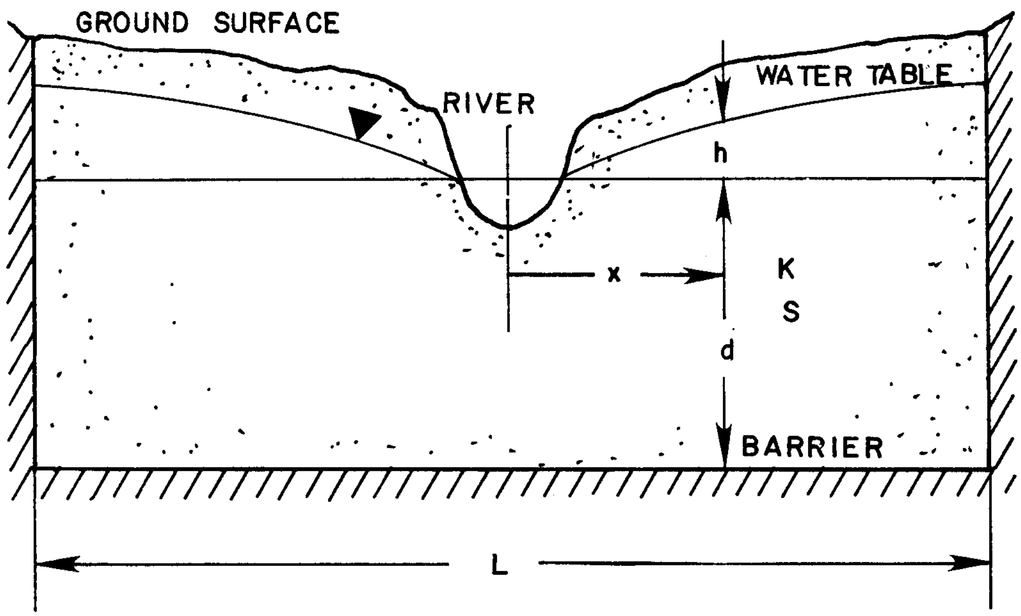

Figure 2.2. Idealization of Stream-Aquifer System (from Glover, 1974).

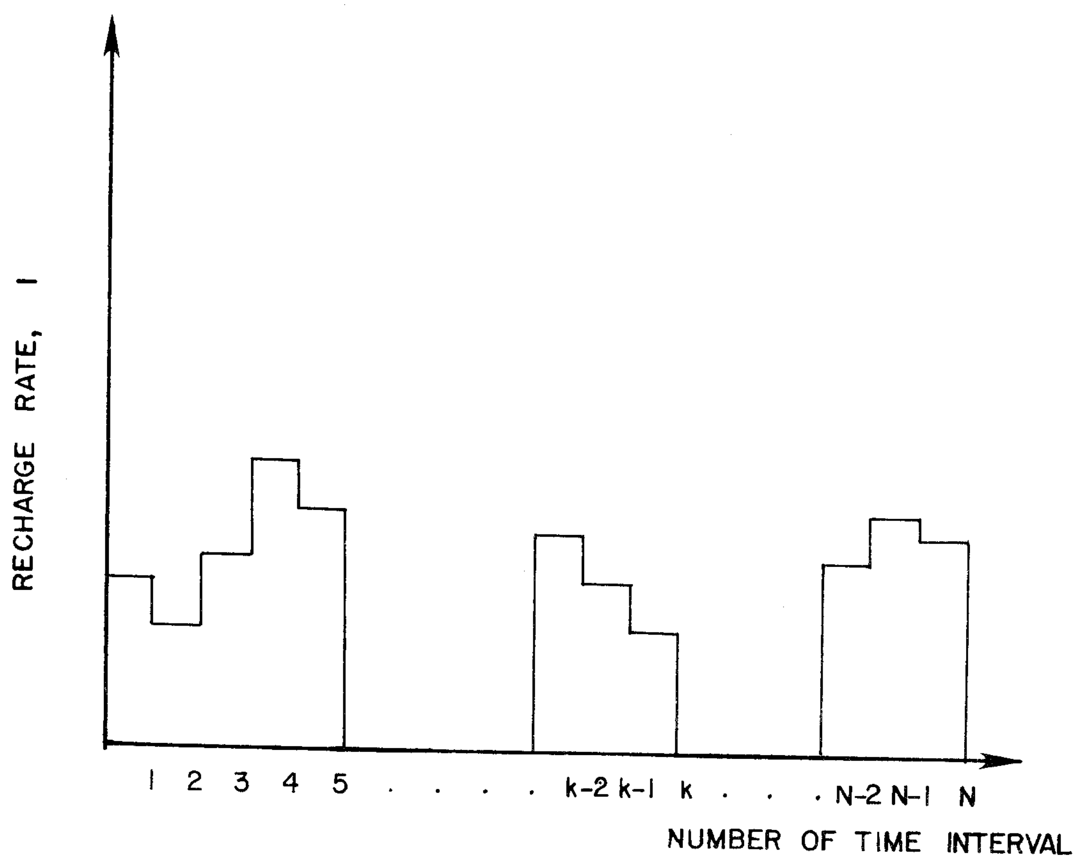

Figure 2.3. Series of Recharge Events.

The total return flow IRF_k during time interval k is the sum of the return flows contributed from the recharge events $I_1, I_2, \ldots, I_k$. That is

$$IRF_k = \sum_{\tau=1}^{k} I_\tau \cdot \delta_{k-\tau+1} \; ; \; \delta_{k-\tau+1} = 0 \quad \text{for } k-\tau+1>N \qquad (2.10)$$

The response δ_k is essentially a discrete kernel, and can be predetermined for any chosen operational horizon consisting of N time intervals. Kernels need be determined only once and then subsequently used to calculate return flows for any time interval as required.

In the case of groundwater withdrawal, the same principle described above is applicable. Here, however, it is river depletion that is considered rather than return flows to the river. The total stream depletion PSD_k during time interval k from groundwater pumping stresses $P_1, P_2, \ldots, P_k$ is

$$PSD_k = \sum_{\tau=1}^{k} P_\tau \cdot \alpha_{k-\tau+1} = 0 \quad \text{for } k-\tau+1>N \qquad (2.11)$$

where P_τ = groundwater pumping during the time interval τ.

B.2. Canal or Stream Seepage

Seepage from a canal or a stream is assumed to correspond to a line source of recharge water. For a one-dimensional line source in an infinite aquifer, as shown in Figure 2.4, the governing flow equation is (McWhorter, 1972)

$$\alpha \frac{\partial^2 q}{\partial x^2} = \frac{\partial q}{\partial t} \qquad (2.12)$$

where q = the flow rate or Darcy velocity; where

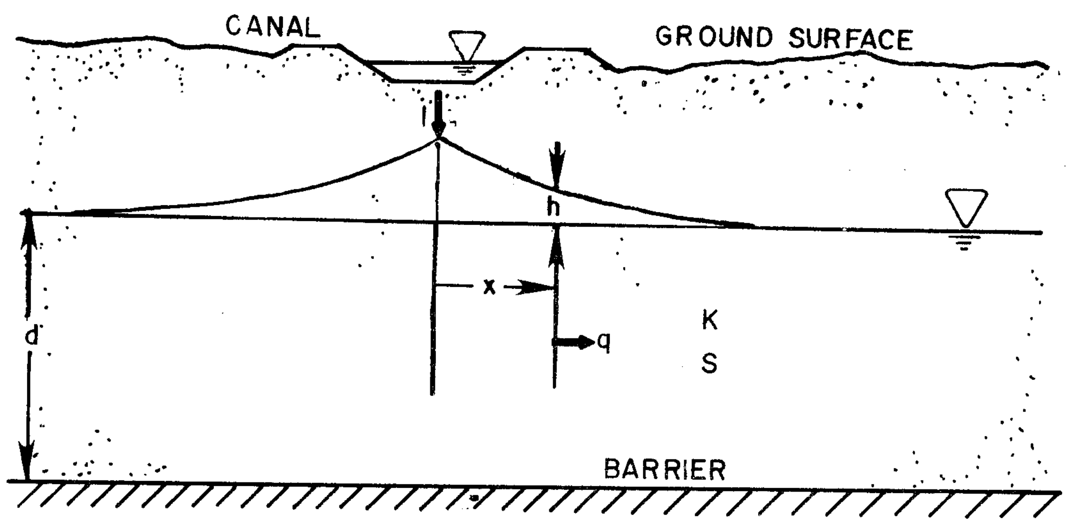

Figure 2.4. Idealization of Line Source.

$$q = -K \frac{\partial h}{\partial x} \tag{2.13}$$

x = Cartesian coordinate in the horizontal plane

For the boundary and initial conditions:

$$q = \frac{I}{2} \text{ at } x = 0$$

$$q = 0 \text{ as } x \rightarrow \infty \tag{2.14}$$

$$q = 0 \text{ at } t = 0 \text{ for all } x$$

where I = one dimensional magnitude of the source in units of length per unit time.

The solution is (McWhorter, 1972)

$$q = \frac{I}{2} \operatorname{erfc} \frac{x}{\sqrt{4\alpha t}} \tag{2.15}$$

where erfc = the complementary error function

$$= (2/\sqrt{\pi}) \int_{\frac{x}{\sqrt{4\alpha t}}}^{\infty} e^{-u^2} du \tag{2.16}$$

Define $q_0 = \frac{I}{2}$ as the applied line source flow rate in the aquifer at the line source location. Note that the denominator of two is necessary since q can flow in two horizontal directions. By integrating equation 2.15 from zero to t, the ratio of the volume of flow to the volume applied up to time t is

$$\frac{v}{q_0 \cdot t} = (\frac{x^2}{2\alpha t} + 1) \text{ erfc } [\frac{x}{\sqrt{4\alpha t}}] - [\frac{x}{\sqrt{4\alpha t}}] \frac{2}{\sqrt{\pi}} \exp(-\frac{x^2}{4\alpha t}) \qquad (2.17)$$

The above solution is for a continuous application of a line source. After termination of the source, the residual effect still contributes flow to the stream. The residual is taken into account by assuming an imaginary pumping source, at the same location, initiating pumpage at the same rate as the recharge source from the time recharge terminates. The volume ratio at any time after recharge ceases is the difference between the volume ratio obtained if recharge had continued and the volume ratio obtained from pumping of the imaginary pumping source. For a discrete time interval, if the applied line source volume $q_0 \cdot t$ equals one, the volume ratio is in essence the unit response of line source or canal seepage.

Let ϕ represent the unit response of canal seepage. Then, the total return flow CRF_k from canal seepage C_1, $C_2, \ldots, C_k$ during each time interval k is

$$CRF_k = \sum_{\tau=1}^{k} C_\tau \cdot \phi_{k-\tau+1} \; ; \; \phi_{k-\tau+1} = 0 \quad \text{for } k-\tau+1>N \qquad (2.18)$$

B.3. Point Source Water Application

Reservoir seepage RS is defined as a point source application. From a modeling viewpoint, the effect on the stream corresponds to the effect of a recharge well, which in turn has the same absolute flow magnitude as the effect of a pumping well, with the flow direction reversed. This solution turns out to be exactly the same as that for the line source solution (Glover, 1974). Therefore, we simply replace C_τ with RS_τ in equation 2.18, and define the resulting return flow as

RRF_k. Again, there is little error in assuming reservoir seepage as a point source, as long as the reservoir surface area is small in comparison with the area of the subsystem containing it.

B.4. Bank Storage

In a stream-aquifer system, changes in the stream stage can affect the return flow. When the stream stage is high, part of the streamflow will flow into the aquifer as bank storage water. This water returns to the stream as the stage lowers. The flow in or out of the aquifer can be obtained using the same principles as the other water sources.

The governing equation is the same as that of the parallel drain system (Glover, 1974). The volume of flow in or out of stream bank VB, for any time t, is

$$VB = 2\ HO \cdot TB \sqrt{\frac{t}{\pi\alpha}} \tag{2.19}$$

where HO = stream stage change

TB = transmissivity of the adjacent aquifer for bank storage computation

The volume of flow VB_k in or out of the stream bank during time interval k, due to stream stage changes of $HO_1, HO_2, \ldots HO_k$, is (McWhorter, 1978)

$$VB_k = \sum_{\tau=1}^{k} (HO_\tau - HO_{\tau-1}) \cdot \beta_{k-\tau+1} \tag{2.20}$$

where

$$\beta_{k-\tau+1} = 0 \quad \text{for } k-\tau+1>N$$

$$= -\frac{4}{3}\frac{TB}{\sqrt{\pi\alpha}}[(k-\tau+1)^{3/2} - (k-\tau)^{3/2}] \qquad (2.21)$$

otherwise

This is the discrete kernel for a unit change of stream stage, with N as the total number of time intervals considered.

The stream stage can be determined from a stage-discharge relationship tabulated for a stream section with a flow gaging station, or from a simple power equation relating stage and discharge:

$$HB = aQ^b \qquad (2.22)$$

where HB = stream stage

Q = discharge

$HO_t = HB_t - HB_{t-1}$

a and b = empirical constants obtained from curve fitting procedures

Program CONSIM employs this latter approach.

C. SURFACE RUNOFF MODEL

There are two sources of surface water runoff considered. The first is runoff from effective precipitation which exceeds the current water holding capacity of the top soil layer. The second type of surface water runoff occurs from irrigated areas and includes effective precipitation plus irrigation application water exceeding crop potential evapotranspiration and groundwater recharge rates. This latter portion is usually known as the "tailwater". A detailed soil moisture accounting is not included in this model because of the large data requirements. The use of effective precipitation and a parameter to be discussed subsequently is a simple way of considering soil moisture effects by allowing time variance of assumed water holding capacity of the soil.

The first type of surface water runoff for time interval k and area i is computed by the following equation:

$$RO_{ik} = (PR_{ik} - AVPR_{ik})\ AREA_i \tag{2.23}$$

where RO_{ik} = surface runoff

PR_{ik} = effective precipitation

$AVPR_{ik}$ = threshold value of precipitation above which surface runoff occurs; this value must be determined by trial and error calibration procedures

$AREA_i$ = land area

The tailwater is determined as a certain portion of the water in excess of the crop potential evapotranspiration. This fraction is also determined by a trial and error calibration procedure. Again, if the threshold value of precipitation is allowed to vary over each time period, then this becomes a rough approximation for taking into account varying soil moisture over each period, along with use of a parameter discussed in the following section.

D. EVAPOTRANSPIRATION MODEL

There are several methods for estimating potential evapotranspiration. The combination methods such as the Penman model utilize an energy equation which includes net radiation from extraterrestrial radiation, percent sunshine and humidity. The radiation method, such as the

Jensen method, utilizes only the net radiation and empirical constants. The evaporation methods utilize a simple empirical relationship correlating potential evapotranspiration to pan evaporation. The temperature methods, such as the Blaney-Criddle and Thornthwaite methods, correlate mean monthly temperature with evapotranspiration. Other methods include the humidity methods, multiple regression methods, and a variety of other miscellaneous procedures. Interested readers are referred to Jensen (1980) for more details on these methods. The method currently used for this model is the modified Blaney-Criddle. This is suitable for seasonal and monthly time intervals. For weekly intervals, however, the modified Penman method is a better choice. Potential evapotranspiration estimates from this approach can be computed outside the model and then input as given demands, if desired.

The modified Blaney-Criddle method (USDA, SCS Technical Release No. 21, 1970) is often chosen for modeling crop consumptive use because the method is simple and can be applied utilizing basic climatic data such as average air temperature, daylight hours and crop data. Again, this method yields good results when applied over longer time periods such as a season, or perhaps monthly, but is unreliable for shorter periods. The crop consumptive use determined from this method is the potential evapotranspiration rate used in the subsequent recharge water computations.

Let U represent the consumptive use of the crop in inches, over a certain seasonal period, where

$$U = kf \tag{2.24}$$

where k = empirical consumptive use crop coefficient for each period

f = monthly consumptive use factor

$$= \frac{T \cdot p}{100} \qquad (2.25)$$

where T = mean air temperature in degrees Fahrenheit over the period

p = percentage of daylight hours over the period

Also, k can be computed by

$$k = k_t \cdot k_c \qquad (2.26)$$

where k_t = a climatic coefficient related to mean air temperature

$$= 0.0173t-0.314 \qquad (2.27)$$

k_c = coefficient reflecting growth stage of the crop

Values of p and k_c can be obtained from the reference cited above or any other related references. However, if the actual type of crop grown in each particular section or subarea is not known, k_c must be estimated.

The estimation of k_c is based on data for the entire basin as related to types of crops grown and percentage of the area planted in each crop. The value of k_c for each month is the weighted average of k_c for all crops grown in that month.

E. GROUNDWATER RECHARGE MODEL

Simulating groundwater recharge is complex because of the many causative factors involved. Among these factors are top soil evaporation, crop evapotranspiration and water holding capacity of the soil. The following method to determine recharge water was originally developed by R. R. Luckey and was successfully applied by Konikow and Bredehoeft (1974) in the Arkansas River basin.

The following assumptions apply:

1. The ratio of an incremental amount of recharge water to an incremental amount of applied water equals one when the total applied water exceeds the potential evapotranspiration. In other words, when total applied water exceeds potential evapotranspiration, any excess water is recharge water.
2. The above ratio is less than one when the total applied water is less than the potential evapotranspiration. In this case, recharge water increases as total applied water increases and approaches the potential evapotranspiration.

With the above assumptions, the recharge water during a particular time interval, over a given area, is expressed as

$$\frac{dR}{dA} = 1 \quad \text{for} \quad A \geq U \tag{2.28}$$

$$\frac{dR}{dA} = \left(\frac{A}{U}\right)^c \quad \text{for} \quad A \leq U \tag{2.29}$$

where R = total recharge water

A = total applied water

U = potential evapotranspiration

c = recharge parameter

Integrating equations (2.28) and (2.29) between zero and A yields

$$R = U\left(\frac{1}{c+1} - 1\right) + A \quad \text{for} \quad A \geq U \tag{2.30}$$

$$R = \frac{A^{c+1}}{U^c(c+1)} \quad \text{for} \quad A \leq U \tag{2.31}$$

After dividing equations (2.30) and (2.31) by A, the recharge fraction R/A of the total applied water is obtained as

$$R_f = \frac{1}{W}\left(\frac{1}{c+1} - 1\right) + 1 \qquad \text{for} \qquad W \geq 1 \tag{2.32}$$

and

$$R_f = \frac{W^c}{c+1} \qquad \text{for} \qquad W < 1 \tag{2.33}$$

where $R_f = \frac{R}{A}$ = the recharge fraction

$W = \frac{A}{U}$ = the normalized applied water

The relationship between R_f and W for various values of the parameter c is shown in Figure 2.5. It can be seen that as c increases, the recharge fraction approaches the potential evapotranspiration limit at which the total applied water can be consumed. The parameter c is rather subjective and needs to be calibrated. A larger c value indicates a soil with a more substantial water holding capacity.

This method assumes that the recharge rate does not exceed the infiltration capacity of the soil. Konikow and Bredehoeft (1974) found from field inspections that some flooding of fields occurred occasionally when infiltration capacity was exceeded. Most of this water was observed to return to the river as surface runoff or tailwater. As mentioned previously, division of the calculated recharge rate into recharge and tailwater components is accomplished by multiplying it by a subjective coefficient between zero and one that must be calibrated.

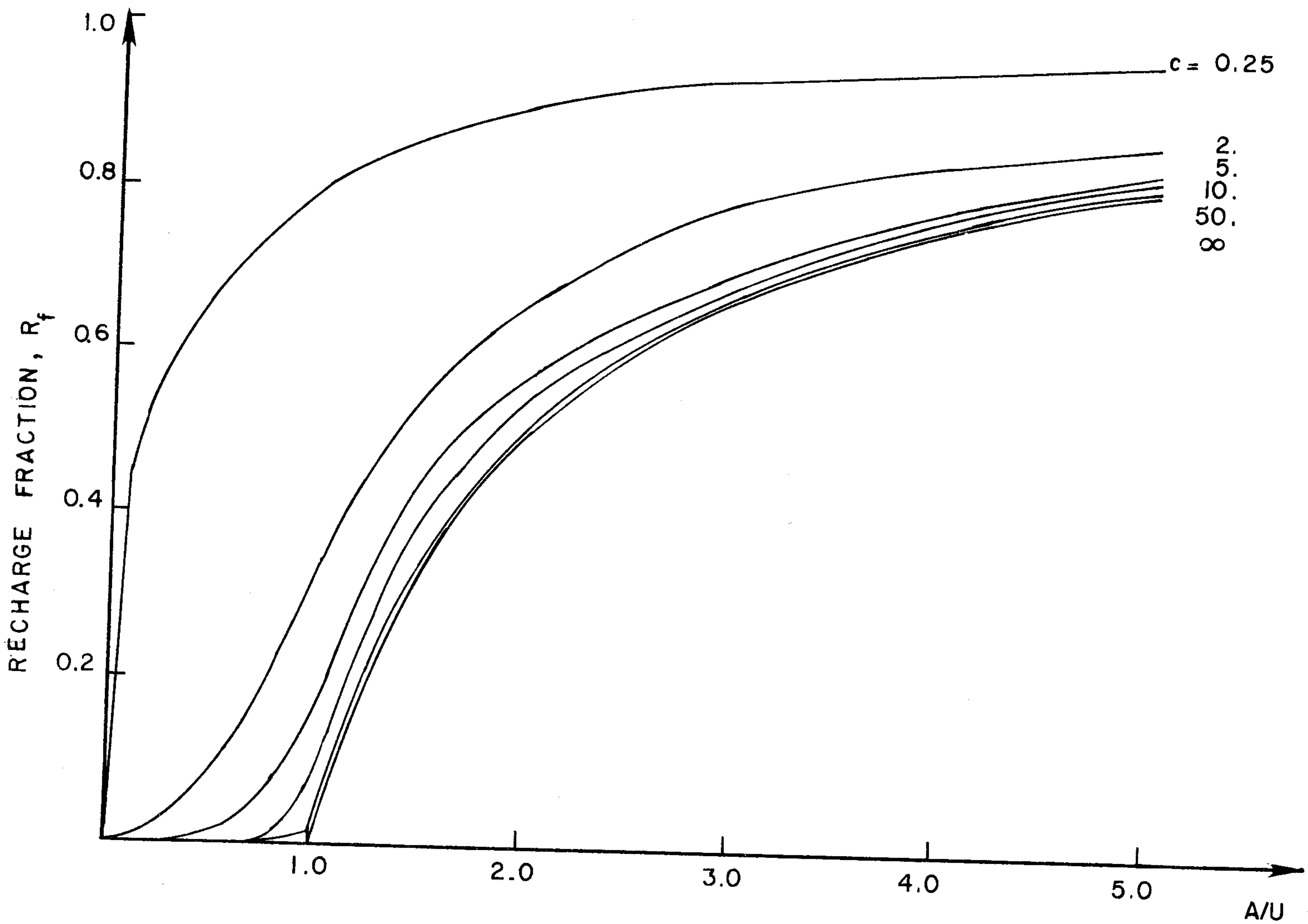

Figure 2.5. Relationship between Recharge Fraction and A/U Ratio (from Konikow and Bredehoeft, 1974).

F. GROUNDWATER STORAGE

The groundwater storage at the end of any period is determined using basic mass balance calculations. For each section i, the mass balance equation for period k is

$$G_{i,k+1} = G_{ik} + C_{ik} + RS_{ik} + I_{ik} - P_{ik} + GI_{ik} - GO_{ik} \tag{2.34}$$

where G_{ik} = groundwater storage at the beginning of period k

C_{ik} = channel seepage to groundwater during period k

RS_{ik} = reservoir seepage to groundwater during period k

I_{ik} = deep percolation to groundwater from irrigation plus effective precipitation during period k

GI_{ik} = groundwater inflow into section or subarea i during period k

GO_{ik} = groundwater outflow out of this section during period k

P_{ik} = groundwater pumpage during period k

Previous work in the South Platte River basin has indicated that phreatophyte consumptive use is significant only if the groundwater table is less than ten feet from the ground surface; hence it is not considered in this model.

Once the groundwater storage is calculated from mass balance, an average groundwater level over each section or subarea is easily obtained from tabulated storage-groundwater level tables for each section.

The groundwater outflow GO_{ik} is actually the return flow to the stream for reach i during time period k:

$$GO_{ik} = IRF_{ik} + CRF_{ik} + RRF_{ik} + TRO_{ik} \qquad (2.35)$$

where IRF_{ik} = total groundwater return flow from applied irrigation plus precipitation into reach i during time period k

CRF_{ik} = total groundwater return flow from canal seepage into reach i during time period k

RRF_{ik} = total groundwater return flow from reservoir seepage

TRO_{ik} = total surface return flow precipitation runoff and tailwater into reach i during time period k.

G. MODEL CALIBRATION

The actual hydrologic system is obviously much more complex than is suggested by the simple relations presented previously. Some of the coefficients in the models may require considerable adjustment. For the aquifer parameters S and T, Labadie (1975) has presented a calibration technique that recognizes that when simple models are used, the parameters must be regarded as "surrogates" of the true parameter values.

The subsurface inflow GI_{ik} represents stream depletions due to pumping, or PSD_{ik} as computed by equation 2.11. Lateral terms between sections are currently not included in this term, though future work should consider this. Bank storage is considered separately and not integrated into total groundwater storage.

G.1. Selection of Parameters

Hydrologic parameters presented in this Chapter can be catagorized into three classes as follows:

Class one includes parameters for which model users assume reasonable values from available information or data. These parameters generally include canal loss coefficients, reservoir seepage coefficients, average widths of subareas, average canal distance from the river and average reservoir distance from the river. Generally, models such as these have too many degrees of freedom; i.e., there may not be a unique set of model parameters that produce the best fit to observed streamflows and other data. Therefore, there is generally little danger in fixing some of the parameters to reasonable values and calibrating over the remaining parameters.

Class two includes parameters which model users assume take on values within a reasonable range, and are then adjusted by trial and error until satisfactory results are achieved. These parameters include monthly effective precipitation coefficients, tailwater coefficients, potential evapotranspiration for nonirrigated land coefficients, transmissivity multiplier for bank storage computation and recharge parameters.

Class three includes parameters with a wider range of adjustment which can possibly be automatically identified by optimization techniques. These parameters include surrogate aquifer parameters such as transmissivity and specific yield. Phamwon (1982) describes such an optimization procedure for transmissivity using this model.

G.2 Automatic Calibration

The simplest method for parameter calibration is trial and error but much repetitious computation is often required and the final result may not be the best answer.

There are several optimization algorithms available that can be used for calibration purposes. The Powell conjugate direction method (Powell, 1964) was chosen for parameter calibration by Phamwon (1982). The Powell method essentially finds the minimum value of a function of several variables. This method has been proven to be extremely efficient when linked with simulation models (Wurbs, 1978).

A computer program called PMIN, utilizing the Powell method for searching for the minimum value of a function, was coupled with this hydrologic model for automatically calibrating the aquifer parameters. The results are presented by Phamwon (1982).

The function in this case is a least-squares error criterion relating simulated return flow to estimated return flow. The optimization problem in this case was:

$$\min_{\underline{T}} \sum_{k} \sum_{i} [GO_{ik} - RE_{ik}]^2 \tag{2.36}$$

where RE_{ik} = total return flows (surface and groundwater) into reach i during time period k as estimated by available observed data

$$= IN_{ik} - DIV_{ik} - OUT_{ik} \tag{2.37}$$

IN_{ik} = total observed or estimated surface inflow into reach i during time period k

DIV_{ik} = total documented or observed surface diversions from reach i during time period k

OUT_{ik} = total observed outflow from reach i during time period k

$\underline{T} = (T_1, \ldots T_n)$ = vector of unknown transmissivities for each of n subareas.

The optimization was restricted to transmissivities since for the case study area, storage coefficients were relatively invariant. Phamwon (1982) found that the optimization results were somewhat inconclusive and obtained at a high computation cost. This is an area requiring further study and research.

CHAPTER III

PROGRAM CONSIM: MANAGEMENT MODEL

A. INTRODUCTION

The MODSIM model, as described in detail by Shafer (1979) and Labadie et al. (1980), builds upon a surface water model developed by the Texas Water Development Board (1972) called SIMYLD. The main disadvantage of the MODSIM model is that it does not fully incorporate groundwater-surface water interactions. It is primarily a surface water model with limited capability of predicting return flows to a stream due to irrigation application or other consumptive uses, and is not capable of predicting stream depletion from pumping. Multiple regression analysis is used for predicting return flows rather than physically-based models describing stream-aquifer interaction. Since the main impetus for modifying MODSIM is to expand the capabilities of the model to consider conjunctive use of groundwater and surface water, this modified model is called CONSIM for CONjunctive use SIMulation.

The CONSIM model is a revised form of an experimental version of the MODSIM model called MODSIME that was developed by Shafer and Labadie (1980). MODSIME does include some physically-based modeling of stream aquifer systems, and provides a strong basis for the improved version CONSIM.

A.1 Concept

The basic concept of CONSIM is that any physical surface water and groundwater system can be conceptualized as a capacitated network flow problem. Any complex, interconnected system of river reaches, tributaries, reservoirs, diversions, well fields, pipelines, canals, and demands can be considered. The physical elements of the system are represented by nodes and links as shown in Figure 3.1. Surface reservoirs and groundwater basins are represented as storage nodes. Non-storage nodes would be points of surface water diversion, river intersection, irrigation demand areas, or other types of demands. A link is analogous to a river reach, canal, or closed conduit with specified direction of flow and specified maximum and minimum capacities. The word "capacitated" is used since the nodes and links must have finite capacities. Any water quantities entering or points of export from the system must occur at a nodal point. That is, losses or gains to a stream are assumed to accumulate at a downstream node rather than being distributed over a link. Higher orders of accuracy in defining distributed gains or losses can be obtained by simply defining more nodes. Each link has associated with it a unit "cost" of transferring one unit of flow from one node to another. The link cost term may be an actual cost, such as pumping cost for a pipeline, or a fictitious cost representing a relative priority for flow in that link, perhaps based on water rights. It may also be a negative cost, which represents a benefit or other quantity which should be maximized rather than minimized, such as hydropower.

The river basin network is constructed in a fully "circulating" manner by adding certain artificial nodes and links to the actual network system. This guarantees that mass balance is satisfied at all nodes throughout the entire network, as shown in Figure 3.1. Details on these artificial nodes and links are given subsequently. It should be noted that the model user only supplies the actual system nodes and

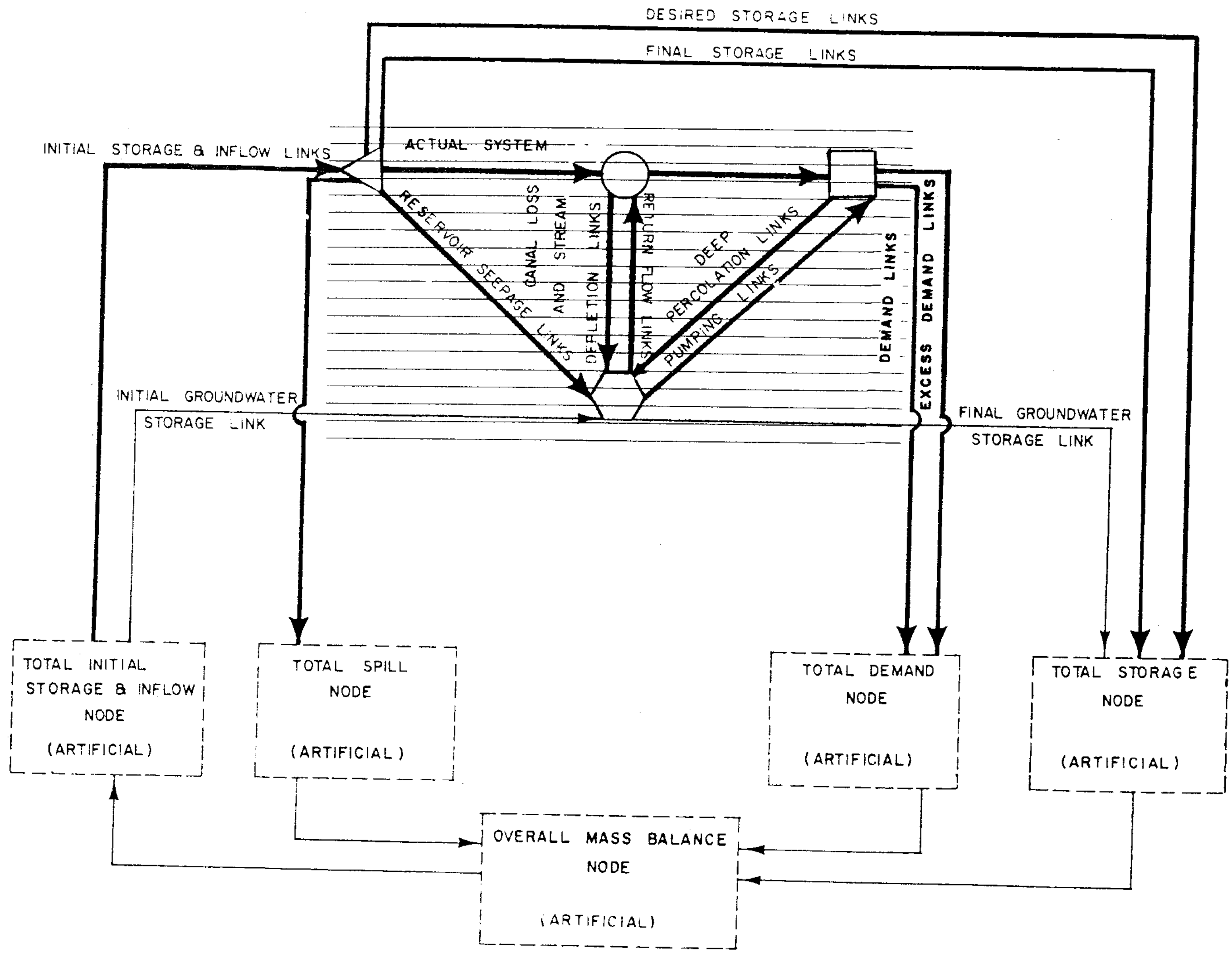

Figure 3.1. Configuration of Network Flow Problem in CONSIM.

links. The artificial nodes and links are automatically added by the model.

With a selected time interval and a given operational or management horizon, the system is formulated as a network flow problem. The set of solutions of this network flow problem provides the sequential operation of the system over the horizon. The current model uses a monthly or weekly time interval, at the user's option. The model solves the system month by month or week by week over the horizon by minimizing the total "costs" over each period. Since these costs may actually be priority rankings, this optimization procedure serves to guarantee that flows are allocated according to priorities or rankings specified by the user. This process of satisfying demands and flow allocations is accomplished by the efficieut "out-of-kilter" algorithm, which is a significant improvement over other conjunctive use river basin simulation models that employ trial and error procedures to satisfy water rights and flow priorities. Note that this is a static sequential optimization rather than a fully dynamic one. Therefore, the model should be called a quasi-optimization model since it optimizes one time interval at a time. This is actually an advantage since it does not presume to have perfect forecasts of future inflows, climatic variables, etc., in making decisions for the current period. However, the availability of forecasts to effect current operations can be handled in an indirect way by appropriate adjustment of weighting factors.

A.2 Capabilities

The management model is capable of simulating the operation of surface water systems which are hydraulically interconnected to underlying and adjacent water table aquifers. Therefore, it can be used for both planning and operational purposes. The model can simulate the operation of conjunctive surface and groundwater systems utilizing historical records on a monthly or weekly time interval over several years. After proper calibration, the model then can be used for a conjunctive water use management plan. During the operational phase, the model can be used to check the system response to any change in the operation scheme.

Reservoir operation: Both MODSIM and CONSIM utilize reservoir target storage levels or guide curves for all reservoirs as specified by the user. As an alternative, an operating rule can be specified that sets target end-of-period storages for each time interval as a function of up to three levels of known storage at the beginning of the time interval plus expected or forecasted inflows during the period.

That is, define

$$R = \sum_{i=1}^{N} S_{it} + \sum_{i=1}^{N} I_{it}$$

$$W = \sum_{i=1}^{N} S_{imax}$$

where N = number of reservoirs in the system

t = current month of operation

I_{it} = unregulated inflow to reservoir i during period t

S_{it} = storage at the beginning of month t in reservoir i

S_{imax} = actual conservation storage capacity for reservoir i

The user then specifies fractions x_1 and x_2 where

$$0 < x_1 \leq x_2 < 1$$

and conditions storage guide curves on the following hydrologic "states":

Dry: $R < x_1 W$

Average: $x_1 W \leq R \leq x_2 W$

Wet: $R > x_2 W$

Notice that since the hydrologic states are based on an accumulation of storages in each reservoir, it is assumed that all the reservoirs are subject to similar climatic and hydrologic conditions.

Reservoirs with higher priorities are filled to their target storage levels first. Water is stored above these levels only to prevent unnecessary spillage or wasted outflows. The latter can, however, be given a lower priority during flood season so as to prevent storage above the guide curve levels. As another option, users can input separate target storage levels for each reservoir for each month throughout the entire period of analysis. This is particularly useful for calibration purposes when observed historical storages are input to check if the model can match these, as well as match computed downstream flows to observed data. It is also useful when certain reservoirs have inviolate guide curves for storage, such as run-of-the-river projects.

Program MODSIME, the extended version of MODSIM, and CONSIM have the capability of computing average monthly or weekly hydropower generated from reservoir releases or run-of-the-river power projects. Programs MODSIM and CONSIM also analyze firm yield for any reservoir or group of reservoirs as specified by by the user. Users can input area-capacity-head data for up to 18 discrete points for each reservoir. Evaporation is computed as a function of average surface area over the time interval (week or month) and seepage is calculated as a function of average storage. An iterative, successive approximation procedure is used in which average surface area is first based on initial storage levels and then adjusted based on releases during the period.

Demand satisfaction: Programs MODSIM and CONSIM compute amounts of water allocated to each demand according to demand priorities. Users can input a separate priority for each demand node and each year of analysis. Demand nodes can be terminal demand nodes or instream use ("flow-through") demand nodes. At terminal demand nodes, water is consumed and lost from the network. In Program CONSIM, recharge and tailwater are additionally computed. On the other hand, water that flows through instream or flow-through demand nodes remains in the network for subsequent diversion. These can be prioritized much like any demand at various points in the basin.

Nodes and links: Several import nodes (i.e., flows which originate outside of the network) are included in both MODSIM and CONSIM. All physical link upper bounds can be input as one value, or monthly or weekly values, for up to 12 periods. Variable link lower bounds should be handled by specifying the downstream node as a flow-through demand.

Channel losses: Programs MODSIM and CONSIM compute river reach and diversion canal losses as specified by user input of channel loss coefficients representing the fraction of total flow lost to seepage. Channel losses are completely lost from the network in MODSIM whereas they can return to groundwater storage in CONSIM. If a channel or canal receives groundwater return flow, it should be regarded as a river reach in the model.

Return flow: Programs MODSIM and CONSIM compute return flows from irrigation application at particular demand nodes specified by users. Return flows computed by MODSIM are based on canal diversions using a multiple regression technique, whereas CONSIM uses a more physically-based approach.

Additional capabilities of CONSIM over and above the attributes of MODSIM include consideration of potential evapotranspiration, surface runoff from irrigated and nonirrigated lands, groundwater recharge and storage, improved stream-aquifer interaction calculations, and direct consideration of precipitation for estimating recharge, runoff and demands.

Potential evapotranspiration: Program CONSIM computes potential evapotranspiration for each irrigation demand node and for each discrete period throughout the entire time horizon of analysis using the modified Blaney-Criddle method. As mentioned previously, other methods can be used, such as the modified Penman method, by computing potential evapotranspiration outside the model and entering these values in the model as demands.

For demand node i, during time period k, potential evapotranspiration for irrigated area, PEI_{ik} in acre-feet is

$$PEI_{ik} = AI_i \cdot U_{ik}/12 \tag{3.1}$$

where consumptive use U_{ik} is calculated by equation 2.34, and

AI_i = irrigated area of demand node i

The denominator 12 converts U_{ik} to units in feet.

For nonirrigated areas, the potential evapotranspiration PEN_{ik} is

$$PEN_{ik} = AN_i \cdot U_{ik} \cdot ce/12 \tag{3.2}$$

where AN_i = nonirrigated area of demand node i ce = empirical coefficient, with value between zero and one.

<u>Surface runoff</u>: Surface runoff from each node is computed for each time period and accumulated at river reach nodes specified by users.

For demand i, during time period k, effective precipitation PR_{ik} over the entire node area is

$$PR_{ik} = cp_k \cdot PC_{ik} \tag{3.3}$$

where PC_{ik} = actual precipitation

cp_k = effective precipitation coefficient

Surface runoff RO_{ik} for node i is

$$RO_{ik} = (PR_{ik} - AVPR_{ik}) \cdot AREA_i \tag{3.4}$$

as given previously in Chapter II. Users must input the river node number to which each particular surface runoff subarea accrues.

<u>Groundwater recharge and storage</u>: For irrigation demand node i, during time period k, the total applied water A_{ik} for irrigation is

$$A_{ik} = (Q_{ik} - C_{ik}) + P_{ik} + PR_{ik} \tag{3.5}$$

with channel loss calculated as a fraction of canal diversions Q_{ik}, where Q_{ik} is a decision variable in the model.

$$C_{ik} = Q_{ik} \cdot cc_i \tag{3.6}$$

where cc_i = canal loss coefficient for canal which diverts water to demand node i

Recharge fraction can then be computed using the method described in Chapter II. The groundwater recharge I_{ik} is

$$I_{ik} = (1 - ct) \cdot A_{ik} \cdot R_f \tag{3.7}$$

where ct = tailwater coefficient

with tailwater TW_{ik} computed as

$$TW_{ik} = ct \cdot A_{ik} \cdot R_f \tag{3.8}$$

For nonirrigated areas the total applied water comes from effective precipitation so it is assumed that no tailwater calculation is necessary.

The groundwater storage for each aquifer section is computed by equation 2.34 given in Chapter II. Note that each aquifer section can have more than one demand node. Model users must specify the aquifer section number to which each demand node accrues.

<u>Stream-aquifer interaction</u>: CONSIM computes return flows from canal losses, reservoir seepages, groundwater recharge from irrigation and precipitation, and stream depletion from groundwater pumping.

For demand node i and any current time period considered, the total return flow IRF_{ik} from previous and current time periods due to groundwater recharge is

$$IRF_{ik} = \sum_{\tau=1}^{k} I_{i\tau} \cdot \delta_{i,k-\tau+1};\ \delta_{i,k-\tau+1} = 0 \quad \text{for } k-\tau+1 > N \tag{3.9}$$

and the total stream depletion due to pumping PSD_{ik} is

$$PSD_{ik} = \sum_{\tau=1}^{k} P_{i\tau} \cdot \alpha_{i,k-\tau+1};\ \alpha_{i,k-\tau+1} = 0 \quad \text{for } k-\tau+1>N \qquad (3.10)$$

The CONSIM model is currently dimensioned for up to N=36 previous time periods. Since the computation is sequentially carried out period by period in CONSIM, the current period stream-aquifer interactions are contingent upon stresses during previous periods. Therefore, for $k<N$, it is assumed that current period interactions are based on known conditions prior to the start of the simulation period, which are input by the model user.

Similarly, for canal i during time period k, the total return flow CRF_{ik} from canal losses during the current and previous time periods is

$$CRF_{ik} = \sum_{\tau=1}^{k} C_{i\tau} \cdot \phi_{i,k-\tau+1};\ \phi_{i,k-\tau+1} = 0 \quad \text{for } k-\tau+1>N \qquad (3.11)$$

For reservoir i during time period k, the total return flow RRF_{ik} from reservoir seepage, based on current and previous period seepages, is

$$RRF_{ik} = \sum_{\tau=1}^{k} RS_{i\tau} \cdot \phi_{i,k-\tau+1};\ \phi_{i,k-\tau+1} = 0 \quad \text{for } k-\tau+1>N \qquad (3.12)$$

For bank storage, it is assumed that it occurs only during high flow periods preceded by a low flow period. Also, it is assumed that any previous stream stage changes prior to that period have no effect on the current bank storage computation. These assumptions are valid only for an effluent stream-aquifer system. For an ephemeral stream-aquifer system, bank storage can be a major water source and should be fully considered.

The bank storage for a river reach i, during time period k is

$$VB_{ik} = \sum_{\tau=1}^{k} (HO_{i\tau} - HO_{i,\tau-1}) \cdot \beta_{i,k-\tau+1} \tag{3.13}$$

B. DESCRIPTION OF THE MODEL

It was stated previously that the network must be fully circulating in order to use the out-of-kilter solution method. This is accomplished by adding artificial nodes and links to the actual system, which is automatically carried out internally in the model. The fully circulating network with inclusion of artificial nodes and arcs was shown in Figure 3.1. Note that in this figure, the groundwater node is assumed to be a single node sink with large storage capacity. Groundwater storage in each of several aquifer sections can, however, still be determined and traced by specifying node numbers and link numbers to each aquifer section. Detailed descriptions for Figure 3.1 are as follows.

B.1 Physical System of Nodes and Links

This block represents the network representation of the actual surface water system. Here, NJ is the total number of the actual surface water system nodes and NL is the total number of actual links. The link lower bounds are zero and the link upper bounds are equal to the actual link capacities (e.g., channel capacities).

B.2 Groundwater Node

This node represents groundwater storage reservoir which is hydraulically connected with the river. This node is connected with the surface water system nodes by two sets of links; one set eliminates from the node and another set flows to the node. The links from the groundwater nodes represent return flow links to river section nodes on the river, as well as pumping links for demand nodes. The links to the groundwater nodes represent reservoir seepages , channel losses, deep percolation from irrigated lands and river depletions from groundwater pumping. The total number of links from this node to the real nodes is 2NJ. The link lower bounds are zero and the upper bounds, except the pumping links, are calculated directly from the hydrologic part of the model, as presented in the previous section on model capabilities. For the pumping links, the lower bounds are zero, and the upper bounds are set equal to the actual pumping of the corresponding demand areas in calibration runs. For management runs, upper bounds on pumping links are set equal to shortages that would have occurred if only surface water were available for diversions for meeting demands.

Return flow calculation: In CONSIM, upper bounds on return flow links are adjusted iteratively. The iteration procedure is as follows:

1. In the first iteration, all upper bounds are set equal to return flows computed from previous development activities which are read in as input data. The return flow from current activities are yet unknown. The total return flow from all links is computed.
2. CONSIM is now run for the current period using these bounds. Return flows from all sources are recomputed using available link flows obtained from this solution. The total return flow is computed and compared to the previous one. If the difference of the total return flow is within specified tolerance limits, the solution is

assumed to have been found, otherwise step two is repeated until convergence is achieved.

Computation of required groundwater pumping: Upper bounds for pumping links are also adjusted iteratively for management purposes:

1. In the first iteration, all upper bounds are set equal to zero or any low initial values specified by users.
2. The network is solved the first time. Shortages are computed for all demands. Upper bounds on pumping links are reset at computed shortages and the total shortage is computed.
3. The network is solved again within the new bounds on pumping links. Total shortages for all demands are recomputed and compared to see if deviation from the previous total shortage estimate is within the specified tolerance limit. If convergence is not achieved, upper bounds on pumping links are reset at previous iteration values, plus any additional shortages, and the procedure repeats until covergence is achieved.

Computation of channel losses: The iteration procedure for return flow and pumping link upper bounds is carried out within another iteration loop; the channel loss iteration loop. The channel loss iteration procedure is similar to the above procedure. Here, the convergence check is based upon the total channel loss for the present iteration, as compared with that of the previous iteration. Computational experience thus far suggests that return flow upper bounds require only three to four iterations with the default tolerance limits specified in the model. Pumping link upper bound adjustments require around six or seven iterations with the default tolerance limit, and the channel loss iterative procedure requires eight or nine iterations with the default tolerance limit. The accuracy obtained with smaller tolerance limits for channel loss iterations does not justify the additional computation time, in the opinion of the authors. The present CONSIM model includes only channel loss convergence checks due to the above explanation.

B.3 Artificial Total Initial Storage and Inflow Node

This node is added to serve as a basic water source node. The node is connected to each real node via a link which serves to carry initial storage to the storage nodes, or as an inflow link for streamflow input, or both. It is also connected to the groundwater node by a link which provides initial storage for the groundwater node. Therefore, the total number of links connected to this node is NJ+1. The link lower and upper bounds are set equal to each other, with the values being the initial storages for each reservoir and streamflow at each inflow point.

B.4 Artificial Total Storage Node

This node is added to account for total storage in the system, including surface water reservoirs and groundwater storage. There are two types of links that join each reservoir to the total storage node; desired storage and final storage. The lower bounds for the desired storage links are the specified minimum storage levels. The upper bounds on desired storage links are the reservoir target storages and the lower bounds on the final storage links are zero.

An additional link to this node is the storage link from the groundwater reservoir node. The lower bound is zero and its upper bound is the groundwater initial storage plus the upper bounds on all links from real nodes to the groundwater node.

B.5 Artificial Total Demand Node

This node is connected to the real nodes by artificial demand links and excess demand links. The demand link lower bounds are zero and upper bounds are set equal to the demand of that corresponding demand node. The excess demand links are introduced to carry excess water, such as extra water diverted for artificial recharge or maximum canal capacity, whichever is larger.

There are two types of demand nodes; the terminal demand nodes and instream use or "flow-through" demand nodes. The terminal demand nodes consumptively use the water diverted to that node. Water specified for instream use, or flow-through demand, bypass the node and are available for downstream diversion. This is handled in the model in an iterative manner. First the model allocates the water to all demand areas. The allocated water for the flow-through demand node is then treated as an unregulated inflow into a downstream node, as specified by the user, for the next iteration. This iteration procedure is also carried out within the channel loss iteration loop.

B.6 Artificial Total Spill Node

This node is connected to all surface water reservoirs by spill links. Any water spilled over any reservoir is considered to be lost from the system. In order to minimize spills, the cost in spill links are set to be excessively high such that there will be minimum flows in these links. Their lower bounds are zero and their upper bounds are set equal to ten times the sum of all reservoir capacities. It is best that all reservoirs be specified as spill nodes to avoid infeasibilities in the network if inflows are excessive.

B.7 Artificial Overall Mass Balance Node

This node is added to serve overall mass balance purposes. The flows into this node are from the artificial storage node, the artificial demand node and the artificial spill node. The flows out of the node are to the artificial total storage and initial inflow node. Node inflows must equal the flows out of the node. The lower bounds and upper bounds for these links have to be set up in keeping with the mass balance requirement.

The various types of links and their corresponding bounds are summarized in Table 3.1.

B.8 Network Link Priorities (User Defined)

There are two types of link priorities. The first type is the preset priority which is built into the computer program. The second type is the priority which is read in by the user. These priorities, associated with demands and storage, should generally correspond to water right priorities, but may need adjustment, as discussed in the next chapter.

For river reaches and canals: The user should generally read in a weighting value of zero or one. No modification of this number is carried out by the computer program. If pumping is required in a pipeline, then actual pumping cost can be used.

For desired storage links: The user reads in reservoir priorities $OPRP_i$ for carryover storage of water relative to other demands. These priorities are subsequently modified in the program by the following

Table 3.1. Link Types and Their Corresponding Lower and Upper Bounds.

No.	Link Type	Lower Bound, LO	Upper Bound, HI
1	Physical system link		
	a. river reach link	Minimum river capacity* (U)[1]	Maximum river capacity (U)
	b. canal link	Minimum canal capacity (U)	Maximum canal capacity (U)
2	Initial storage and inflow link	Previous end-of-month storage plus current monthly or weekly inflow, plus current monthly or weekly return flows (B)	Same as LO (B)[2]
3	Final desired storage link	Reservoir minimum pool plus minimum estimated seepage (U)	Percent of maximum capacity desired (monthly or weekly operating rule) plus maximum estimated seepage (U)
4	Final storage balance link	Zero (B)	Zero (B)
5	Demand link	Zero (B)	Demand at node (U)
6	Excess demand link	Zero (B)	Amount required for artificial recharge (U), or canal capacity and demand link difference
7	Spill link	Zero (B)	Total of all surface water reservoir capacities multiplied by ten (B)
8	Mass balance links		
	a. total initial storage plus inflow links	Σ initial storage plus initial groundwater storage (B)	Same as LO (B)
	b. total final storage links	Σ final desired storage (B)	All surface water reservoir capacities plus their maximum evaporation and seepages plus HI of groundwater final storage (B)

Table 3.1. (Continued)

No.	Link Type	Lower Bound, LO	Upper Bound, HI
	c. total demand link	Zero (B)	Σ Demands (B)
	d. total spill link	Zero (B)	Σ Spills (B)
9	Node to groundwater links**		
	a. reservoir seepage link	Zero (B)	Actual seepage (B)
	b. deep percolation link	Zero (B)	Actual deep percolation at node (demand node in particular) (B)
	c. river depletion link	Zero (B)	Actual river depletion at depletion node (B)
	d. channel loss link	Zero (B)	Actual channel loss (B)
	e. initial groundwater storage link	Initial groundwater storage (B)	Same as LO (B)
10	Groundwater to node links		
	a. return flow links	Zero (B)	Actual return flows (B)
	b. pumping links	Zero (B)	Pumping capacities (B)
	c. groundwater final storage links	Zero (B)	Σ upper bounds of links in No. 9 plus initial groundwater storage (B)

Note: * Should be zero unless minimum flow is required. However, a minimum flow requirement may cause infeasible solutions.

** The river depletion and channel loss may flow through the same link. In this case, the upper bound is the summation of both flow.

[1] U is for user supplied parameters.

[2] B is for program default parameters.

equation:

$$w_{ij} = -(1000 - OPRP_i \cdot 10) \qquad (3.14)$$

where w_{ij} = the modified priority for link from reservoir node i to artificial storage node j

$OPRP_i$ = the user specified operating rule priority between 0 and 100 for reservoir i

Notice that w_{ij} now represents a negative cost, which is treated as an artificial net benefit in the program. Therefore, a larger value of $OPRP_i$ indicates a lower priority.

<u>For demand links</u>: The user reads in relative priorities for all demands, $DEMR_i$. The modified priority is given by

$$w_{ij} = -(1000 - DEMR_i \cdot 10) \qquad (3.15)$$

where $DEMR_i$ = the priority for demand node i between zero and 100 (larger value represents lower priority)

<u>For excess demand links</u>: Artificial recharge can be specified as a separate demand node or as flow in excess of demand at a particular node. Priorities for the areas to be artificially recharged relative to each other are specified by the user. The modified priority for the flows in excess of demand carried in the link between demand node i and artificial demand node j is

$$w_{ij} = -(3000 + KRANK_i) \qquad (3.16)$$

where $KRANK_i$ is the priority of demand node i for artificial recharge relative to other nodes. Ideally, $KRANK_i$ should be selected such that the w_{ij} for artificial recharge has a slightly lower absolute value than that of the corresponding demand link. That is, set $KRANK_i < -(2000 + DEMR \cdot 10)$.

For spill links: The user inputs priorities for surface reservoirs in order of preference for spilling of water. The modified priority is given by

$$w_{ij} = 2000 \cdot (1 + KS_i) \tag{3.17}$$

Note that these modified link priorities are high positive values (therefore, low priority). These are in fact the highest cost links and therefore have the lowest priority so as to minimize unnecessary spillage.

Again, the w_{ij} values for demand and storage are negative numbers as computed by the program. These are analogous to benefits since the network algorithm performs a minimization. It allocates as much water as available through these links and, at the same time, allocates as little water as possible through the links with positive w_{ij} values. The optimization problem is formulated in the following section.

B.9 Preset Link Priorities:

For links from groundwater node: These links include return flow links to the river reach nodes and pumping links to the demand nodes. The preset limit priorities are high negative numbers since it is considered that benefits are incurred from flow through these links. For the pumping links, this simply assures that pumpage is at the upper bound for that link, which assumes that shortages are minimized. These bounds are adjusted iteratively, as explained earlier.

For links to groundwater node: These are depletion links for the links from river reach nodes and deep percolation links for the links from demand nodes. The preset link priorities are also high negative numbers.

For all remaining links: The link priorities are preset to zero.

C. SOLUTION ALGORITHM

Having specified the "costs" w_{ij} for all links in the stream-aquifer system network, we now formulate the following optimization problem for some current time period k and iteration loop n:

$$\underset{Q_{ij}^{(n)}}{\text{minimize}} \sum_{i=j}^{N} \sum_{i=1}^{N} w_{ij}\, Q_{ij}^{(n)} \tag{3.18}$$

subject to mass balance constraints as shown in Figure 3.2:

$$\sum_{i=1}^{N} Q_{ij}^{(n)} - \sum_{i=1}^{N} Q_{ji}^{(n)} = 0 \qquad \text{for all nodes } j=1,\ldots,N \tag{3.19}$$

These constraints simply specify that the sum of all flows entering a mode must equal the total flow leaving. This includes prespecified inflows and outflows as well as bound constraints:

$$Q_{ij}^{(n)} \leq u_{ij}(Q_{ij}^{(n-1)}) \qquad \text{for all nodes } i,j=1,\ldots,N \tag{3.20}$$

$$x_{ij} \geq l_{ij} \qquad \text{for all nodes } i,j=1,\ldots,N \tag{3.21}$$

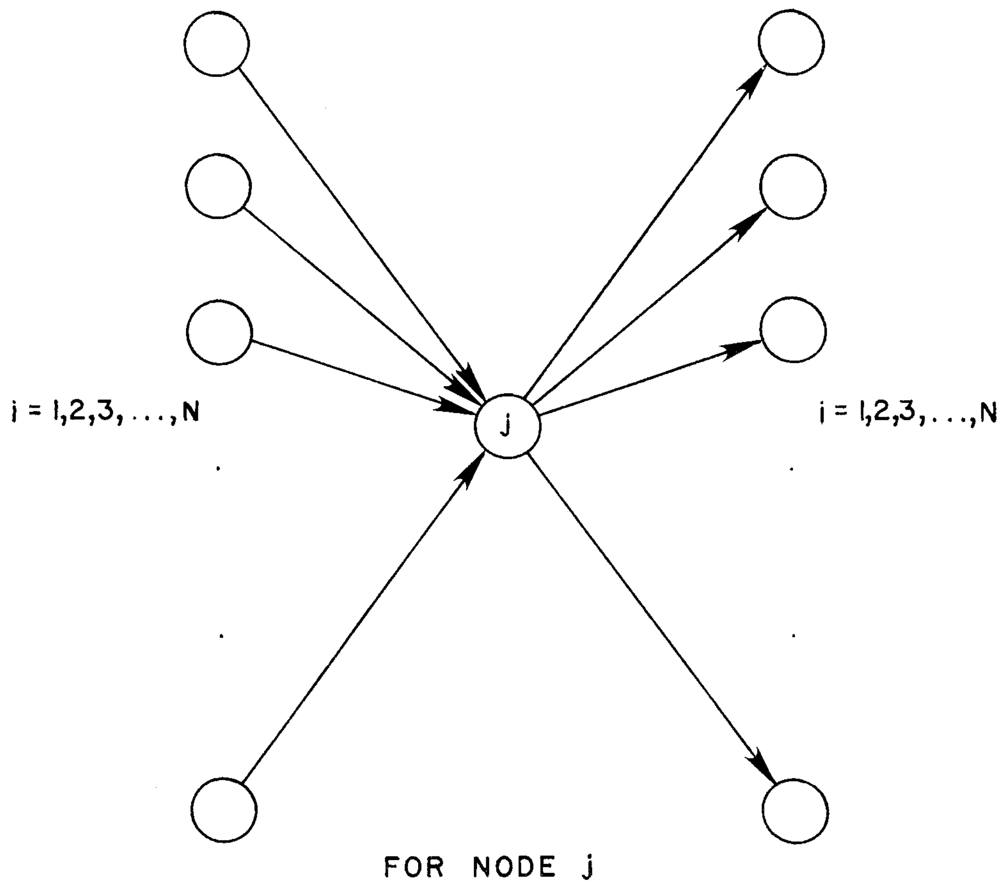

Figure 3.2. Mass Balance Constraint.

where w_{ij} = "cost" associated with one unit of flow in link (i,j) between nodes i and j

$Q_{ij}^{(n)}$ = flow in link (i,j) for current iteration n (these are the problem decision variables)

$Q_{ij}^{(n-1)}$ = given flows from previous iteration n-1

$u_{ij}(Q_{ij}^{(n-1)})$ = maximum capacity for link (i,j)

l_{ij} = minimum flow requirement for link (i,j)

Note that any prespecified inflows or outflows are handled by representing the flow as a decision variable, but setting l=u = the desired value. All link flows are regarded as decision variables.

Though the standard simplex algorithm of linear programming can be used to solve this problem, a more efficient algorithm is the out-of-kilter technique developed by Fulkerson (1961). The algorithm solves this problem via an efficient primal-dual simplex technique. This algorithm is used in the management model for allocating both surface and groundwater to various demands. A simple explanation of how the out-of-kilter algorithm works can be found in Shafer (1979).

Notice that in this general formulation, the upper bounds for links representing return flows, pumping and channel losses are adjusted iteratively for n=1,2,3..., until $Q_{ij}^{(n-1)} \simeq Q_{ij}^{(n)}$. Constraints 3.20 actually create a nonlinearity in this problem, which is handled by this iterative procedure. Computational experience indicates that this is a contraction mapping and that a fixed point solution always occurs. A general convergence proof is not yet available.

CHAPTER IV

APPLICATION TO SOUTH PLATTE RIVER BASIN

A. DESCRIPTION OF STUDY AREA

The case study area selected is the South Platte River basin, which is tributary to the Missouri River. Total drainage area of the basin is 24,300 square miles, of which around 80 percent is in the State of Colorado. The subarea selected for study is the section of the river valley from the North Sterling inlet to Julesburg, as shown in Figure 4.1. This section has a length of approximately 90 miles and a drainage area of about 500 square miles. Detailed description of the South Platte basin can be found in Gerlek (1977).

A.1. Hydrogeology

The South Platte plains consist of rolling hills and valleys. The climate is arid and precipitation is in the form of snow in winter and rainfall in spring and summer. Annual precipitation varies from 12 inches near Kersey to 16 inches near Julesburg, with 70-80 percent of it occurring in the summer.

The South Platte River Valley consists of alluvium of Pleistocene to recent ages and bedrock of Cretaceous time such as the Pierre Shale, the Fox Hills and the Laramie formations. The river traverses the Denver formation near Denver and the igneous rockbed near the South Platte gaging station. There is also some contact with the Ogallala near Julesburg. The river valley widths range from two to ten miles. The valley trough consists of alluvium of heterogeneous mixture of clay, sand and gravel, or lenses of these materials. This alluvial aquifer serves as a large groundwater reservoir; storing and releasing water for

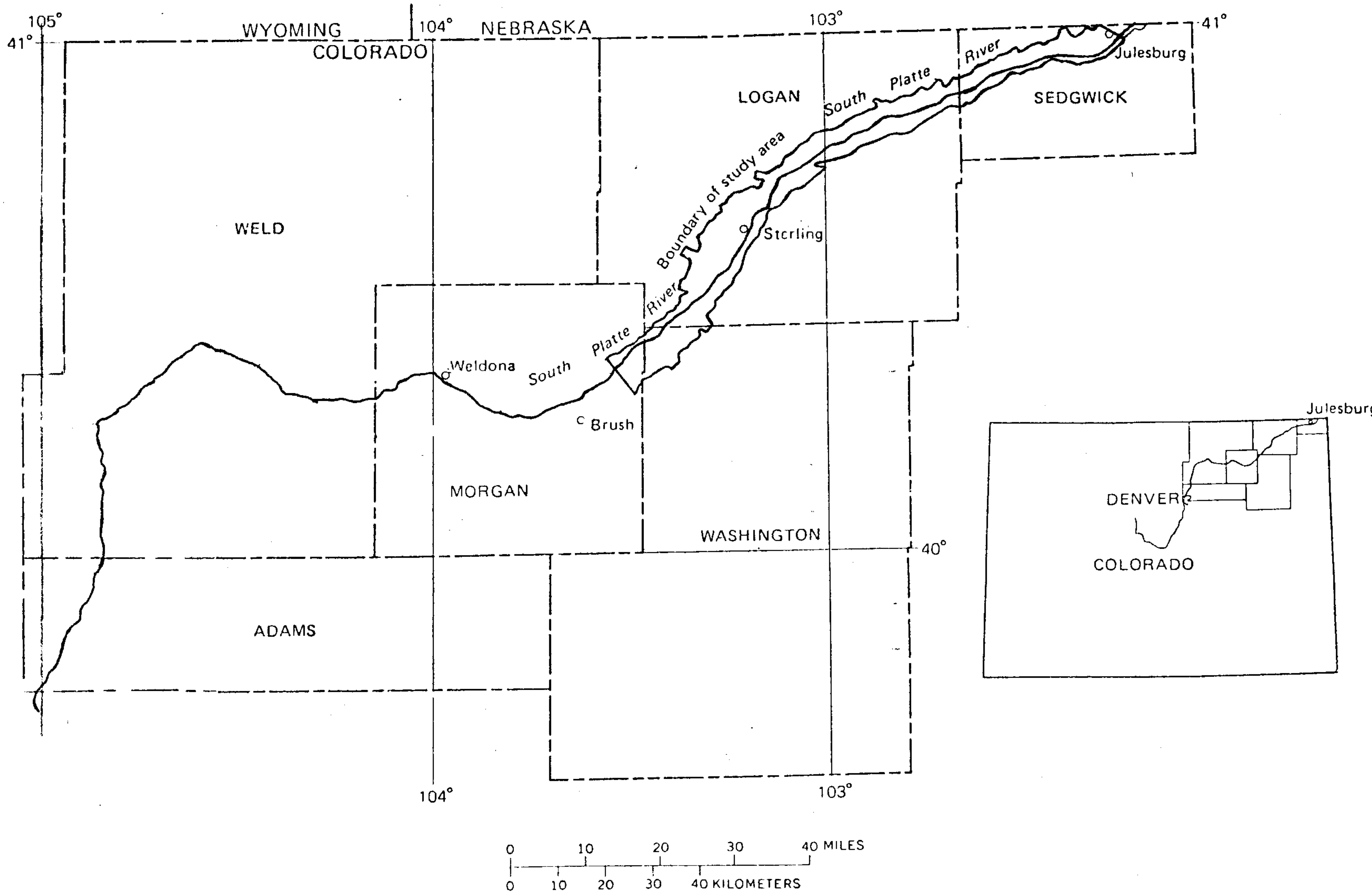

Figure 4.1. Index Map Showing Location of the Case Study Area (Hurr et al., 1975).

all beneficial uses, with the agricultural user as the most important sector.

The natural streamflow in this basin is mostly snowmelt from the mountain watershed. About 70 to 80 percent of the total annual streamflow occurs during the period April-July. Before water resources were developed in the State, the river carried most of the flow in the spring flood to the Missouri River. The plains position of the river was an ephemeral stream, with late summer, fall and winter low flow from the tributaries sinking into the sands of the main stem.

During the major water development period of 1870 to 1924, and continued development to the present, numerous diversion structures, storage reservoirs and irrigation systems have been developed. The South Platte is no longer an ephemeral stream. Augmented groundwater levels have changed the river to an effluent stream. The river and its adjacent aquifer comprise a complexly interacted stream-aquifer system.

A.2. Water Resources

The primary surface water sources are from tributary subbasins and from the main stem of the South Platte River. Native surface runoff is assumed to be negligible, except during heavy rainfall periods, since most of the water infiltrates to the groundwater reservoir or is lost through evaporation. The average annual recorded inflow is 440,330 acre-ft at the entry point of the case study area. The recorded average annual outflow at the gaging station near Julesburg is 355,340 acre-ft.

An important source of water for this area is the alluvial aquifer along the river valley. From the United States Geological Survey estimates, the alluvial aquifer transmissivity can be as high as 1,200,000 gallon per day per foot near the river and the average specific yield is about 0.2. The deep aquifer water has not yet been utilized because of smaller yield and high development cost.

Surface water is supplied to agricultural lands and other demands by networks of canals and ditches from storage reservoirs and direct diversion from the South Platte River. Groundwater is pumped through wells for irrigation and other uses. Figure 4.2 shows the locations of storage reservoirs and canals within the study area.

B. MODEL CALIBRATION

B.1. Disaggregation of the Study Area

The case study area is disaggregated into subareas for modeling purposes. The main criteria used for the disaggregation is that each subarea is associated with one diversion canal or one releasing canal, if possible. The subarea configuration may be irregular but each must have the river as one boundary. The disaggregation of the case study area is shown in Figure 4.2 and Table 4.1 summarizes the subarea characteristics.

B.2. Estimation of Parameters

The suitable values of class one and class two parameters obtained from various sources are shown in Tables 4.1 and 4.2.

Class 3 parameters can include aquifer parameters such as transmissivity T and specific yield S. Transmissivity is chosen here as the primary parameter requiring calibration, with storage coefficient assumed as a class 2 parameter, as given in Table 4.2. U.S. Geological Survey maps, such as those given in Hurr et al. (1972), present T values as contour lines ranging from low values at the aquifer outer boundary to order of magnitude or higher values at the river. For the

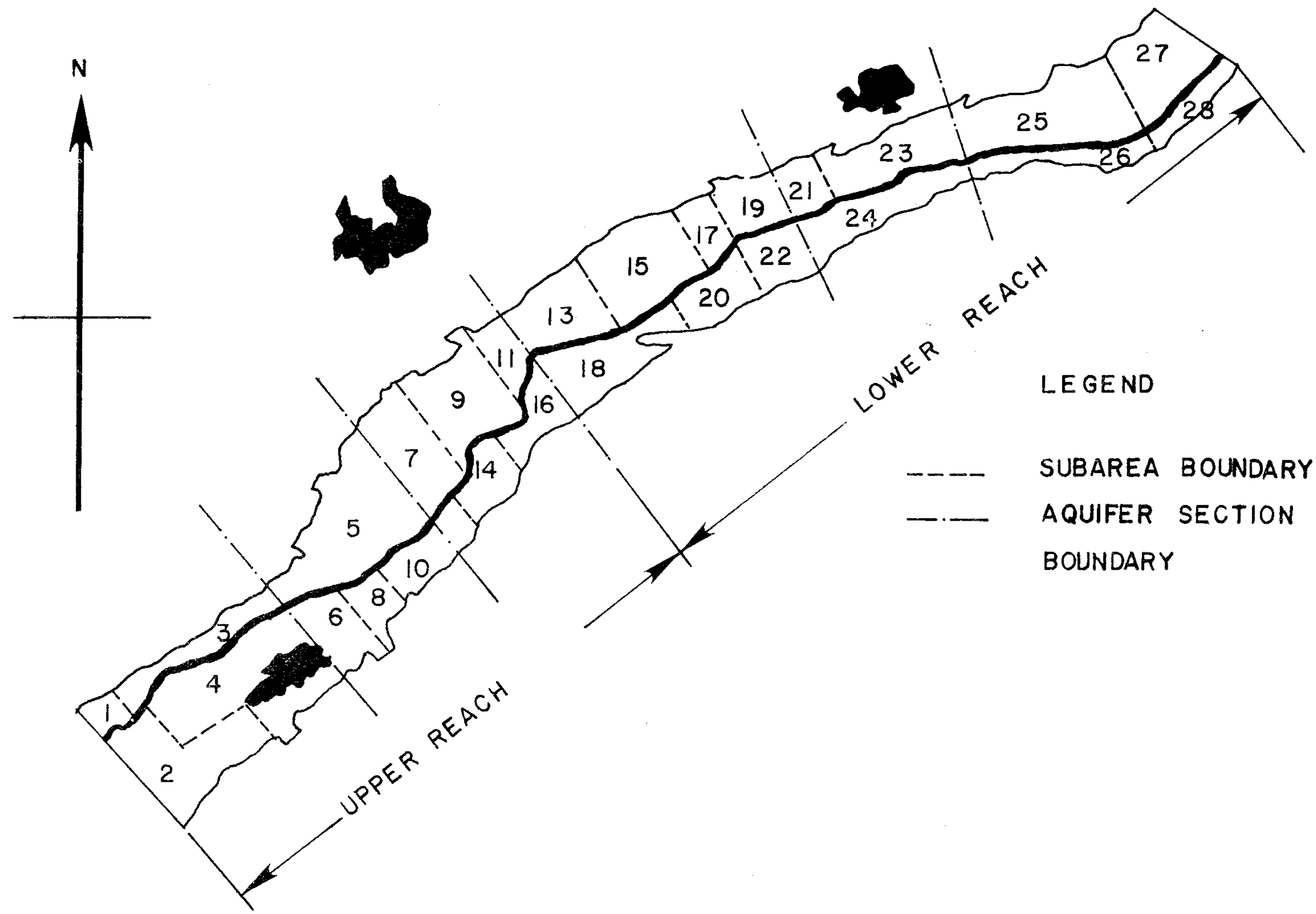

Figure 4.2. Approximate Spatial Decomposition of the Case Study Area.

Table 4.1. Summary of Physical Data of the Case Study Area.

Area No.	Average Width (ft)	Canal Name	Average Canal Length (ft)	Canal Loss Coeff. (cc)	Total Area (acres)	Irrigated Area (acres)
1	8295	Synder; Smith	5280	0.21	4163	1237
3	8295	Tetsel	4224	0.21	4026	1085
3		North Sterling Intake	3696	0.10		
2	19500	Lower Platte; Beaver	11616	0.21	17824	7360
4	9800	Johnson; Edwards	3696	0.21	5723	1722
4		Prewitt Intake	4800	0.21		
6	9800	South Platte	6000	0.28	11175	5579
5	17709	Pawnee	10912	0.29	31780	9408
8	9800	Davis Brothers	4000	0.21	5029	2323
10	9504	Schneider	3696	0.21	8241	2180
7	18480	Springdale	9500	0.33	7325	4486
9	21120	Sterling No. 1	7900	0.21	10442	6733
12	7656	Henderson; Smith	1250	0.21	4320	367
11	12144	Lowline	3696	0.175	4640	2151
14	9208	Bravo	7393	0.25	7991	1519
16	8100	Farmer	2112	0.21	970	367
13	9800	Iliff; Platte Valley	6336	0.175	8966	5461
18	8800	Lone Tree	2640	0.21	5930	183
15	7946	Powell	6336	0.175	5892	2464
17	9800	Ramsey	2112	0.21	2156	393
20	9300	Chamber	2640	0.21	25193	5901
19	9800	Harmony No. 2	6834	0.21	10549	3636
21	9800	Settler; High Line	5016	0.21	10975	5694
22	7940	Tamarack	2112	0.21	6540	1505
24	5300	Hemming House	2112	0.21	3470	414

Table 4.1. Continued.

Area No.	Average Width (ft)	Canal Name	Average Canal Length (ft)	Canal Loss Coeff. (cc)	Total Area (acres)	Irrigated Area (acres)
23	8100	Red Lion	1584	0.21	2221	592
25	8700	Peterson	7392	0.21	14852	7360
26	3839	South Reservation	3168	0.21	5771	927
27	8000	Liddle	2640	0.21	1085	533
28	3839	Carlson	2112	0.21	1093	208

Notes:

1. Only one surface reservoir is located within the stream-aquifer boundaries; i.e., Prewitt Reservoir. Average distance to the river is 11680 feet and the seepage loss coefficient is 0.14.

2. Data sources:

   Burns, Alan W. and Theodore Hurr, U.S. Geological Survey, Denver. By personal communication.

   Toups Co., (1978).

Table 4.2. Parameters for Hydrologic Model.*

Month	Threshold Precip. (AVRAIN') (in.)	Effective Precip. Coef (cp)	Crop Coef (k_c)	Daylight Hours (p) (%)
January	3.36	0.95	0.00	6.76
February	3.36	0.95	0.00	6.72
March	3.36	0.90	0.09	8.32
April	3.36	0.87	0.33	8.93
May	3.36	0.82	1.02	10.00
June	3.12	0.80	1.25	10.03
July	1.20	0.87	1.24	10.21
August	0.96	0.90	1.12	9.54
September	1.20	0.90	1.06	8.39
October	1.20	0.95	0.43	7.75
November	2.40	0.95	0.00	6.74
December	3.12	0.95	0.00	6.55

Calibration Parameters

1. Runoff (calibration factor)
   Used for adjusting AVRAIN in Equation 3.4, where

   $AVRAIN_{ik} = AVRAIN'_{ik} \cdot cr$ cr = 2.00

2. Parameter for estimating potential evapotranspiration from nonirrigated lands (Equation 3.2) ce = 0.30

3. Tailwater coefficient (Equations 3.7 and 3.8) ct = 0.22

4. Bank storage transmissivity adjustment factor (Equation 2.21)

   $TB = JTRAN \cdot T$ JTRAN = 3

5. Recharge parameter (Equation 2.29) c = 99

6. Aquifer specific yield (Equation 2.1) S = 0.20

*Data source: Burns, Alan W. and Theodore Hurr, U.S. Geological Survey, Denver, personal communication; also Hurr et al. (1975).

case study area, the river valley is quite uniform in width throughout the entire length. with transmissivity contour maps also showing a gradual change in the downstream direction (Hurr et al., 1972). For the purpose of this study, it was deemed reasonable to divide the groundwater basin into two sections; an upper reach and a lower reach, with constant T values assumed for each section. The two major sections are in turn each divided into six subsections for flow prediction purposes, with three subsections on each side of the river as shown in Figure 4.2.

B.3. Prestress Period Surrogate Unit Responses

It is important that an appropriate length of historical development period be chosen to account for time lag effects on the system prior to the period under study. This is called the prestress period. To illustrate this point, the unit responses of a uniform water application and line source water application are shown as solid lines in Figures 4.3 and 4.4. Their corresponding accumulated unit responses are shown in Figures 4.5 and 4.6. The prestress period selected for this study is 36 months. Note that the unit responses extend longer than the 36 month period, especially for the line source case. However, the limitations of data available for this study does not allow the period chosen to be longer than 36 months or 36 weeks.

Aquifer boundary conditions should also be considered when deriving unit responses. Since the aquifer is obviously not of infinite extent. Therefore, all complexities affecting the stream-aquifer response are lumped into "surrogate" transmissivity parameters. Labadie (1975) introduced the concept of surrogate parameters in hydrologic modeling as a means of including complex system features into approximate models.

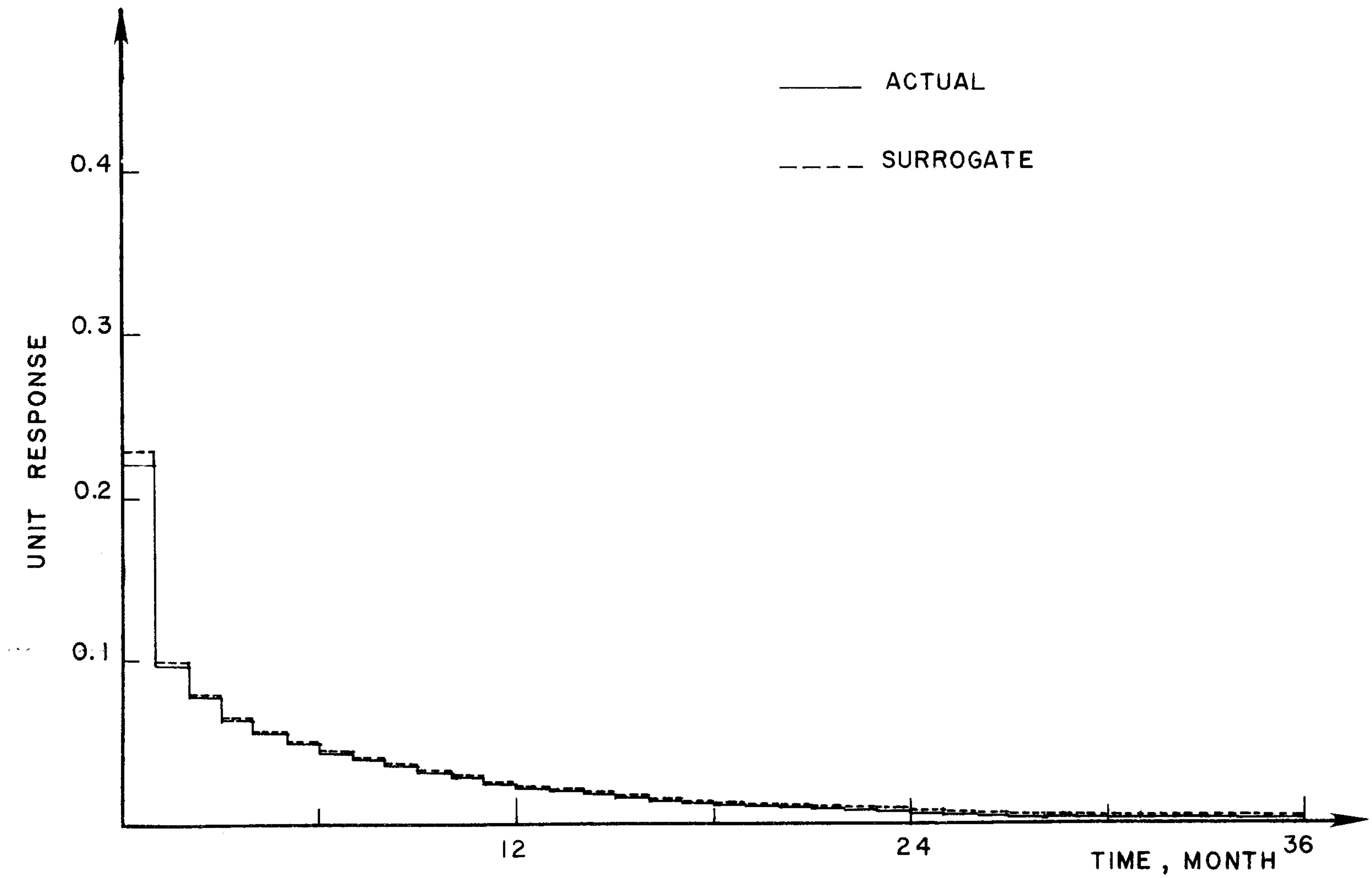

Figure 4.3. Example of Unit Responses for Uniformly Distributed Water Application; Subarea 23, Red Lion, Average Width = 8100 Feet, T=180,000 Gallon per Day per Foot.

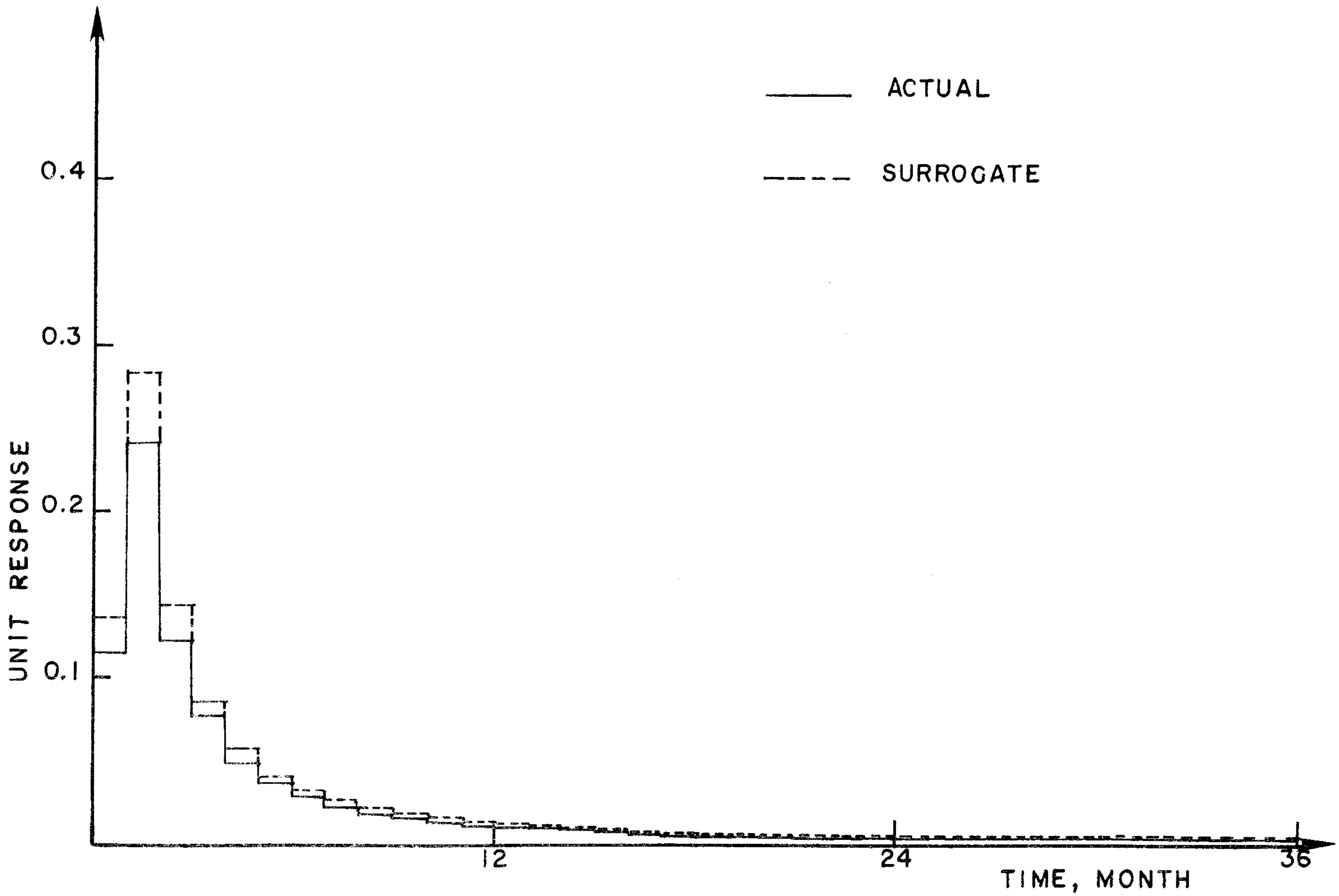

Figure 4.4. Example of Unit Response for Line Source Water Application; Johnson and Edwards Canal, Average Distance = 3966 Feet, T= 160,000 Gallon per Day per Foot.

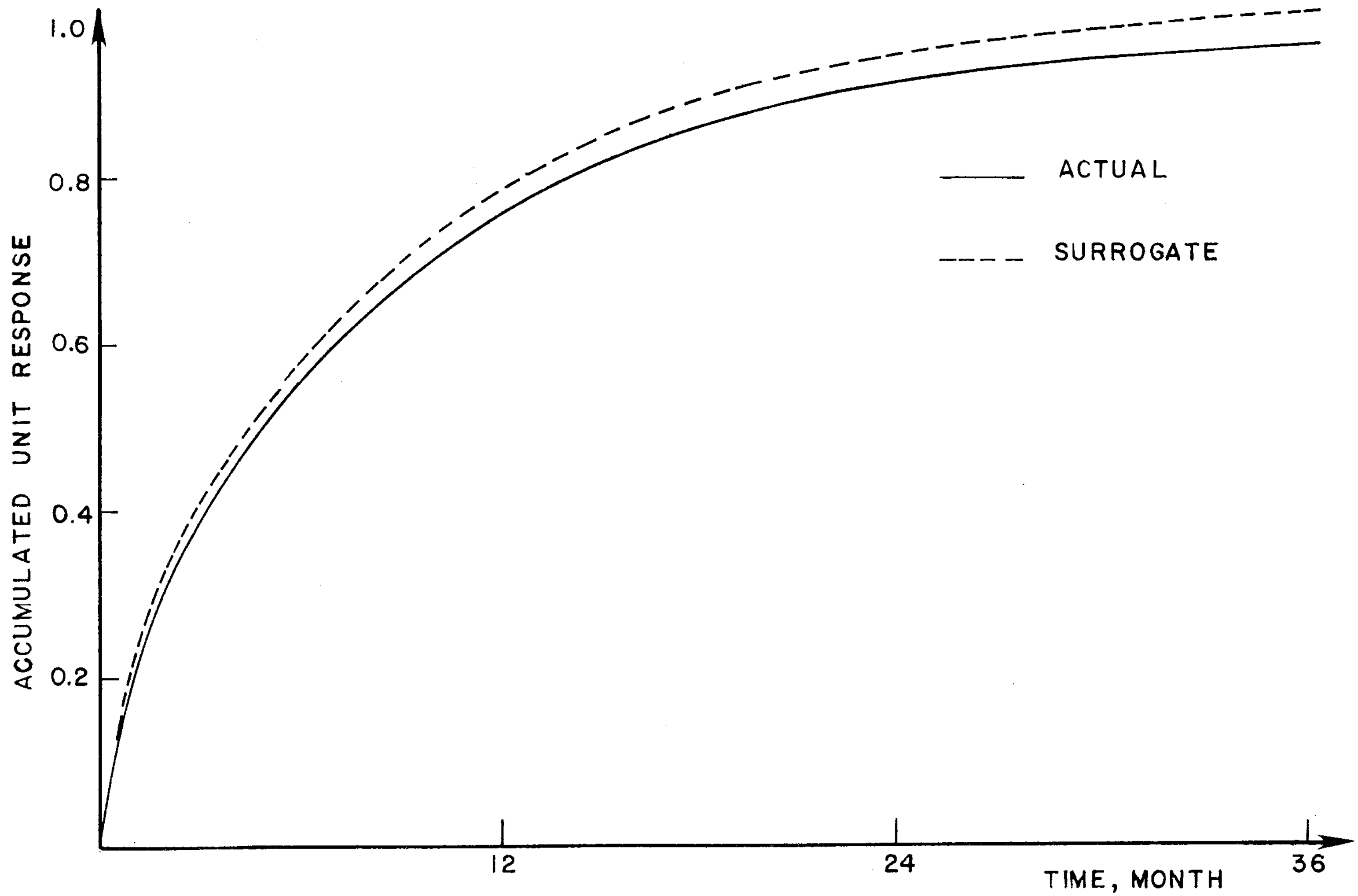

Figure 4.5. Example of Accumulated Unit Responses for Uniformly Distributed Water Application.

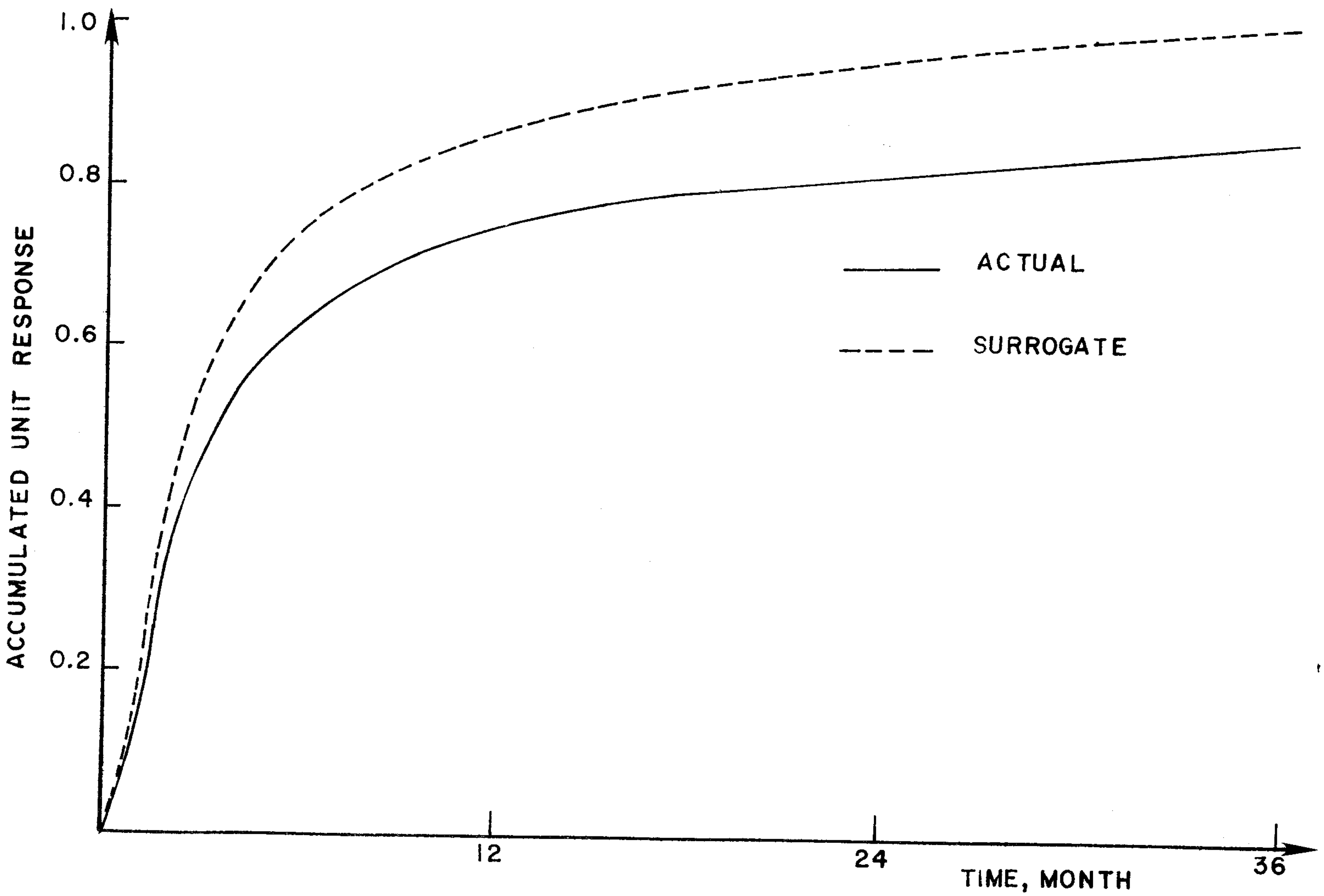

Figure 4.6. Example of Accumulated Unit Responses for Line Source Water Application.

To compensate for the long history of aquifer development, a surrogate unit response is derived which assumes that the activities prior to the 36 month prestress period can be considered by adjusting the prestressed period accumulated unit responses to unity during the selected 36 month period. The monthly unit responses are adjusted accordingly by the following equation

$$\delta_k' = \frac{\delta_k}{\sum_{k=1}^{36} \delta_k} \tag{4.1}$$

for uniform water application, and

$$\phi_k' = \frac{\phi_k}{\sum_{k=1}^{36} \phi_k} \tag{4.2}$$

for line source and point source water application,

where δ_k' = surrogate unit response of month k for δ_k

ϕ_k' = surrogate unit response for month k for ϕ_k

The surrogate unit responses and their corresponding accumulated unit responses for the uniform water application case and the line source case are shown as dashed lines in Figures 4.3 to 4.6. Though this method accounts for the total flow reaching the river, the timing of those flows is obviously different. This is true primarily for the first two or three months of the study period, which should be taken into consideration during calibration.

In the computation of bank storage, it is assumed that it is significant only during the high flow months. The transmissivity value used for bank storage computations is generally assumed to be larger than the aquifer transmissivity. This multiplier must be determined by trial and error calibration procedures.

The concepts of surrogate parameters and surrogate unit responses are incorporated into the river basin hydrologic model and applied to the case study area to calibrate transmissivity values and other class two parameters as explained previously. Values of class two calibrated parameters are given in Table 4.2. The values of calibrated transmissivity are 160,000 and 180,000 gallon per day per foot for the upper river section and the lower river section, respectively. It was judged in this case that use of the optimization algorithm PMIN yielded little improvement since final values differed little from initial estimates. Details on this calibration can be found in Phamwon (1982).

C. NETWORK SETUP

The case study area is set up as a network as shown in Figure 4.7. The network consists of 53 nodes and 53 links. With the addition of all artificial nodes and links, the totals become 59 and 434, respectively.

C.1. Data Input

The data needed for utilizing the model include:

1. Model control parameters
2. Physical description of the surface water system and the groundwater system
3. Historical operational criteria
4. Priority of each demand node; both irrigation demands and storage targets
5. Monthly or weekly historical unregulated inflows

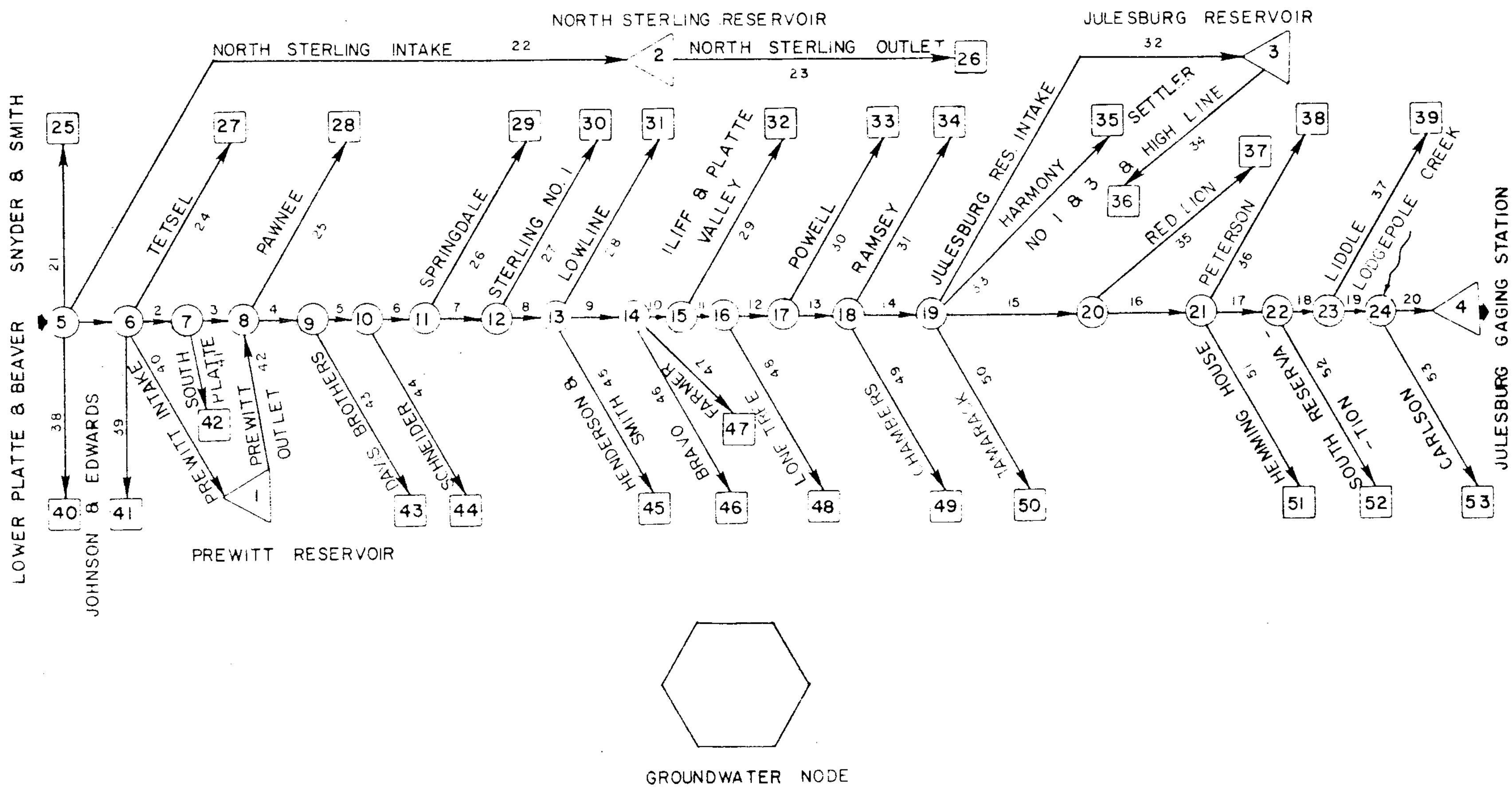

Figure 4.7. Network Representation of the Case Study Area.

6. Monthly or weekly demands
7. Monthly or weekly evaporation rates
8. Monthly or weekly precipitation rates
9. Monthly or weekly pumping rates

A monthly time increment is used for this case study.

These data are contained in two separate data files for convenience. The first file is the coded data file which includes all fixed data such as those of categories 1-4. The second file is the binary file which includes varied data such as those of categories 5-9. Examples for creation of these two files are shown in the Appendices.

Besides these two data files, two additional files are needed. The first file contains precomputed recharge rates, reservoir seepages, channel losses and pumping up to 36 months or weeks previous to the period of interest. The second file contains all precomputed unit responses up to 36 months or weeks. These two files are introduced in order to save computer time and storage.

C.2. Procedure for Network Setup

The following steps are used for network setup.

1. The river is divided into subreaches. Generally, a canal diversion point is used as a point of diversion. However, several small canals in close proximity are assumed to divert from the same point in order to reduce the number of subreaches. There are 20 river subreaches for this case study area.
2. Each irrigation demand area (node) corresponds to a subarea, as mentioned previously, with one area generally corresponding to one diversion canal. There are 28

irrigation demand nodes within the groundwater basin boundaries, and one irrigation demand node outside the groundwater basin boundaries. Small areas can be aggregated into larger adjacent areas.

3. All links are numbered starting at the upstream river reach link as number one, two and so on. Then all other links are numbered.
4. The groundwater basin is divided into sections or cells. Each groundwater cell can underlie several irrigation demand areas and include several river subreaches. All groundwater cells are numbered using odd numbers for the left river bank and even numbers for the right river bank, starting from upstream and continuing downstream. There are six groundwater cells on each side of the river for this case study area.
5. River reach nodes are selected as depletion nodes. Again, as explained previously, all flows must be accumulated at nodal points. Stream depletions are removed from river reach nodes and added to specified groundwater cells. The canal diversion node for each irrigation demand node is also used for depletions from pumping.
6. River reach nodes are selected as return flow nodes. Usually, the next downstream node from the canal diversion node of each irrigation demand area is its return flow node. A topographic map of the area is useful for providing some basis for selection. Note that the same node can be used for return flow node from a certain

demand area and a depletion node for the next downstream irrigation demand area.

D. SELECTION OF MODEL PRIORITIES

The period from 1952 to 1956 was selected for calibration and verification of priorities. This period is used because it contains two average years, 1952 and 1956, and three consecutive drought years, 1953 to 1955. The calibration was carried out for the year 1952, with verification checks over the next four years using results obtained from the year 1952. It was found that calibrated priorities obtained from 1952 can be applied to the next four years with similar results. The calibrated priorities are shown in Table 4.3 together with the actual water right priorities. It can be seen that they change little from actual water right priorities in this case because exchanges are not prevalent in this area.

The last downstream node is treated as a fictitious storage reservoir node with zero capacity. It is given the lowest priority such that all upstream demands would be satisfied first before remaining water flows out of the stream via spilling from this fictitious reservoir.

The historical operating storages of the three surface reservoirs in comparison with simulated storages are shown in Figures 4.8 to 4.13. The historical monthly outflows and simulated outflows are shown in Figures 4.14 and 4.15.

The output obtained from the model includes summary information for all surface reservoirs, demands, link flows, return flows, pumping, seepage loss, and river depletions, as well as groundwater storage and groundwater level summaries. Examples of output from the model are shown in the Appendices.

Table 4.3. Result of Priority Calibration.[1]

Node No.	Name[2]	Type of Water Use	Actual[3] User Priority	Calibrated User Priority
1	Prewitt Reservoir	Storage and suppl.release	27	27
2	North Sterling	Storage and release for irrigation	26	26
3	Julesburg	Storage and release for irrigation	25	25
4	Fictitious Resv.	Downstream Outflow	99	99
25	Snyder and Smith*	Irrigation	1**	1
26	North Sterling	Irrigation	26***	1
27	Tetsel	Irrigation	7	7
28	Pawnee	Irrigation	5	5
29	Springdale	Irrigation	11	11
30	Sterling No. 1	Irrigation	4	4
31	Lowline	Irrigation	9	9
32	Iliffe and Platte Valley	Irrigation	10	10
33	Powell	Irrigation	15	15
34	Ramsey	Irrigation	17	17
35	Harmony No. 1 and 3	Irrigation	18	18
36	Settler and Highline	Irrigation	25***	1
37	Red Lion	Irrigation	12	12
38	Peterson	Irrigation	23	23
39	Liddle	Irrigation	21	21
40	Lower Platte and Beaver*	Irrigation	1**	1
41	Johnson and Edwards	Irrigation	2	2
42	South Platte	Irrigation	1	1

Table 4.3. Continued.

Node No.	Name[2]	Type of Water Use	Actual[3] User Priority	Calibrated User Priority
43	Davis Brothers	Irrigation	6	6
44	Schneider	Irrigation	3	3
45	Henderson and Smith	Irrigation	8	8
46	Bravo	Irrigation	14	14
47	Farmer	Irrigation	20	20
48	Lone Tree	Irrigation	16	16
49	Chamber	Irrigation	19	19
50	Tamarack	Irrigation	24	24
51	Hemming House	Irrigation	28	28
52	South Reservation	Irrigation	13	13
53	Carlson	Irrigation	22	22

Notes: [1]The lower the priority number, the higher the user priority.

[2]For irrigation demand areas, the name denotes the canal name conveying water to the area.

[3]Water right priority rankings.

*The diversion points for these canals are located above the inflow point of the study area, but some of the water is used to irrigate land in the study area. For Snyder and Smith canal, 21% is assumed to be used in the study area. For the lower Platte and Beaver canal, 45% is assumed to be used in the study area.

**Since these canals are located above the inflow point of the study area, their priorities are always the highest, to ensure full demand satisfaction.

***Since these demands obtain water from their reservoir release, their priorities are set equal to the corresponding reservoir.

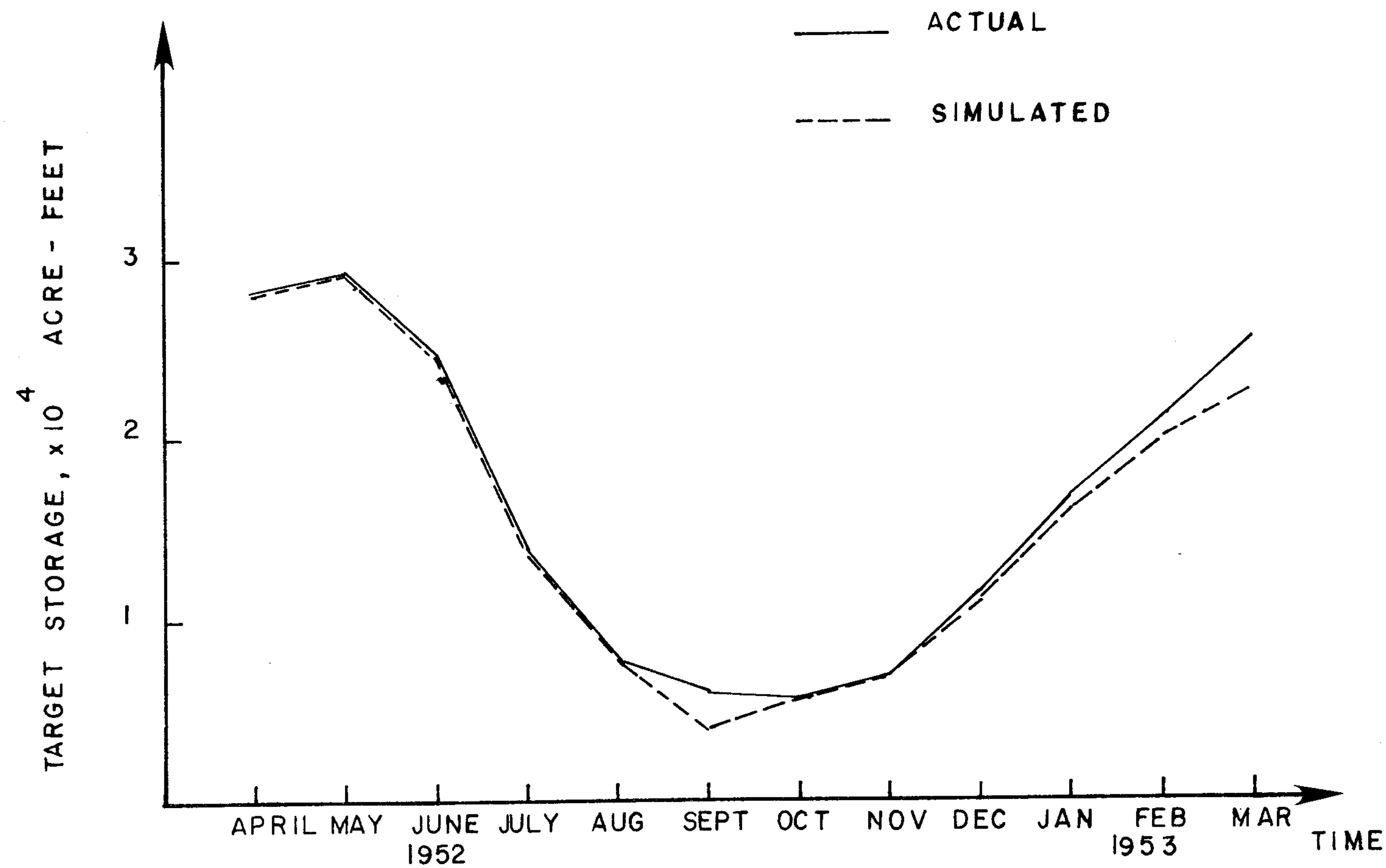

Figure 4.8. Comparison of Actual and Simulated Target Storages for Prewitt Reservoir, 1952-53.

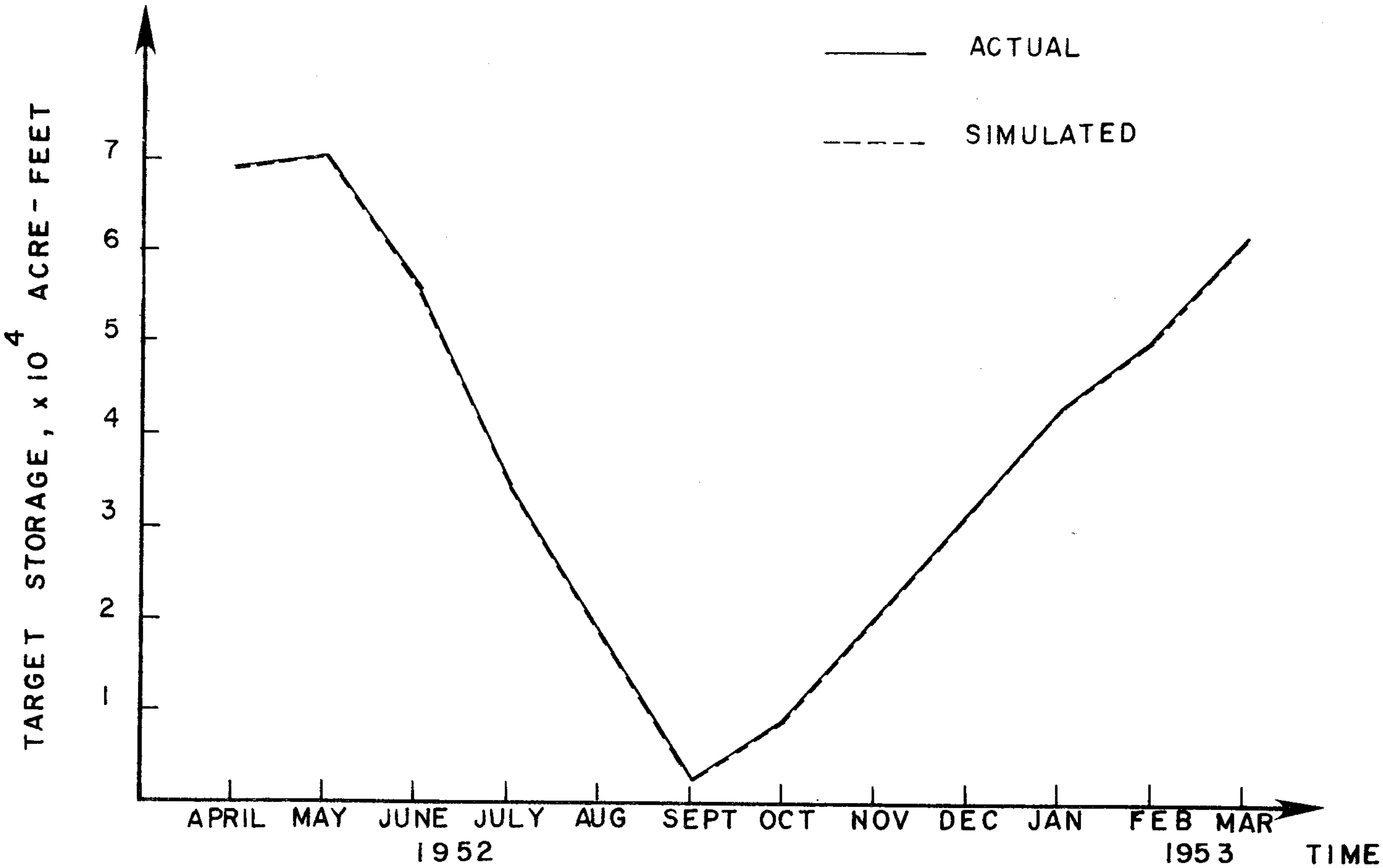

Figure 4.9. Comparison of Actual and Simulated Target Storages for North Sterling Reservoir, 1952-53.

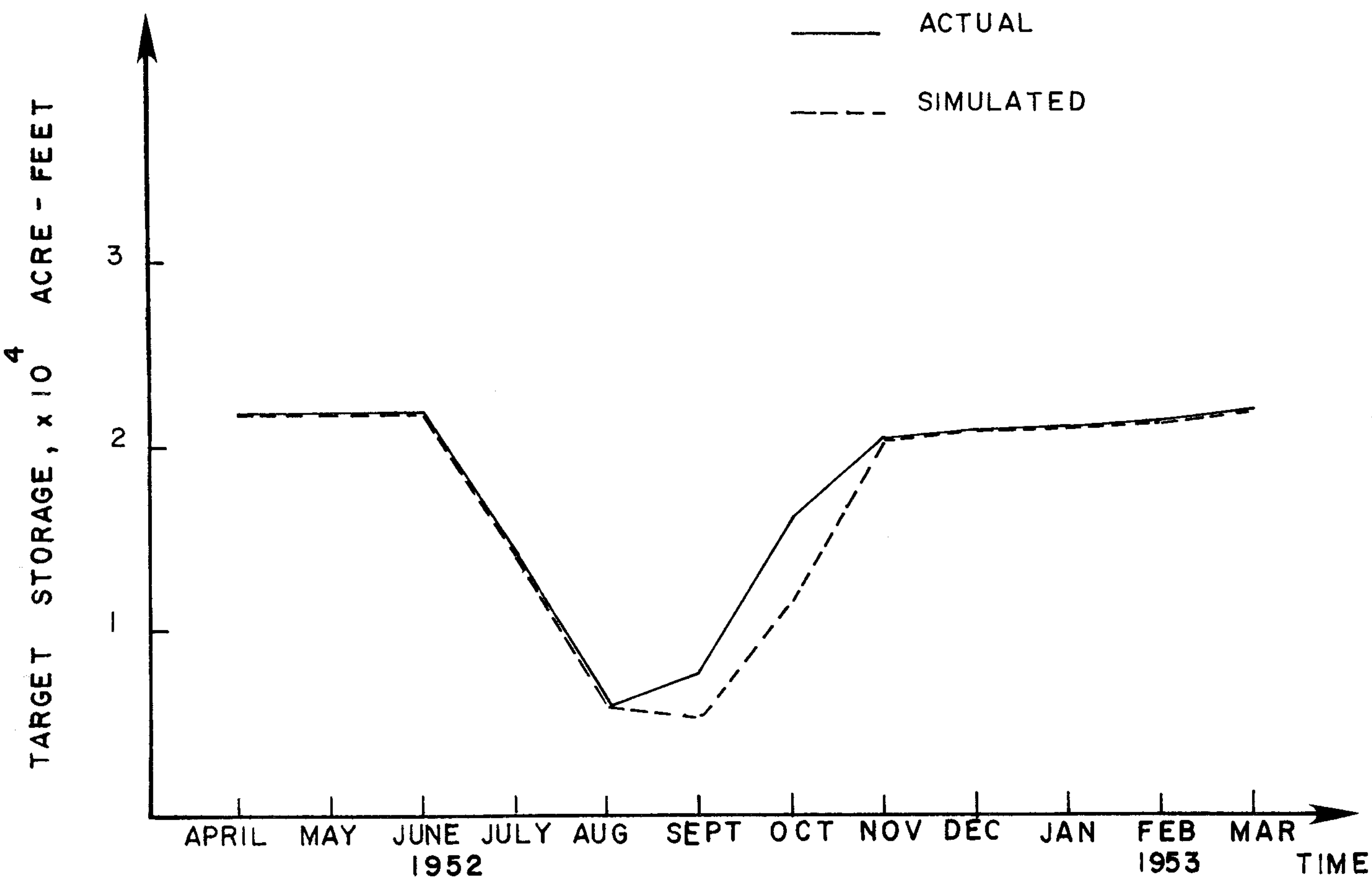

Figure 4.10. Comparison of Actual and Simulated Target Storages for Julesburg Reservoir, 1952-53.

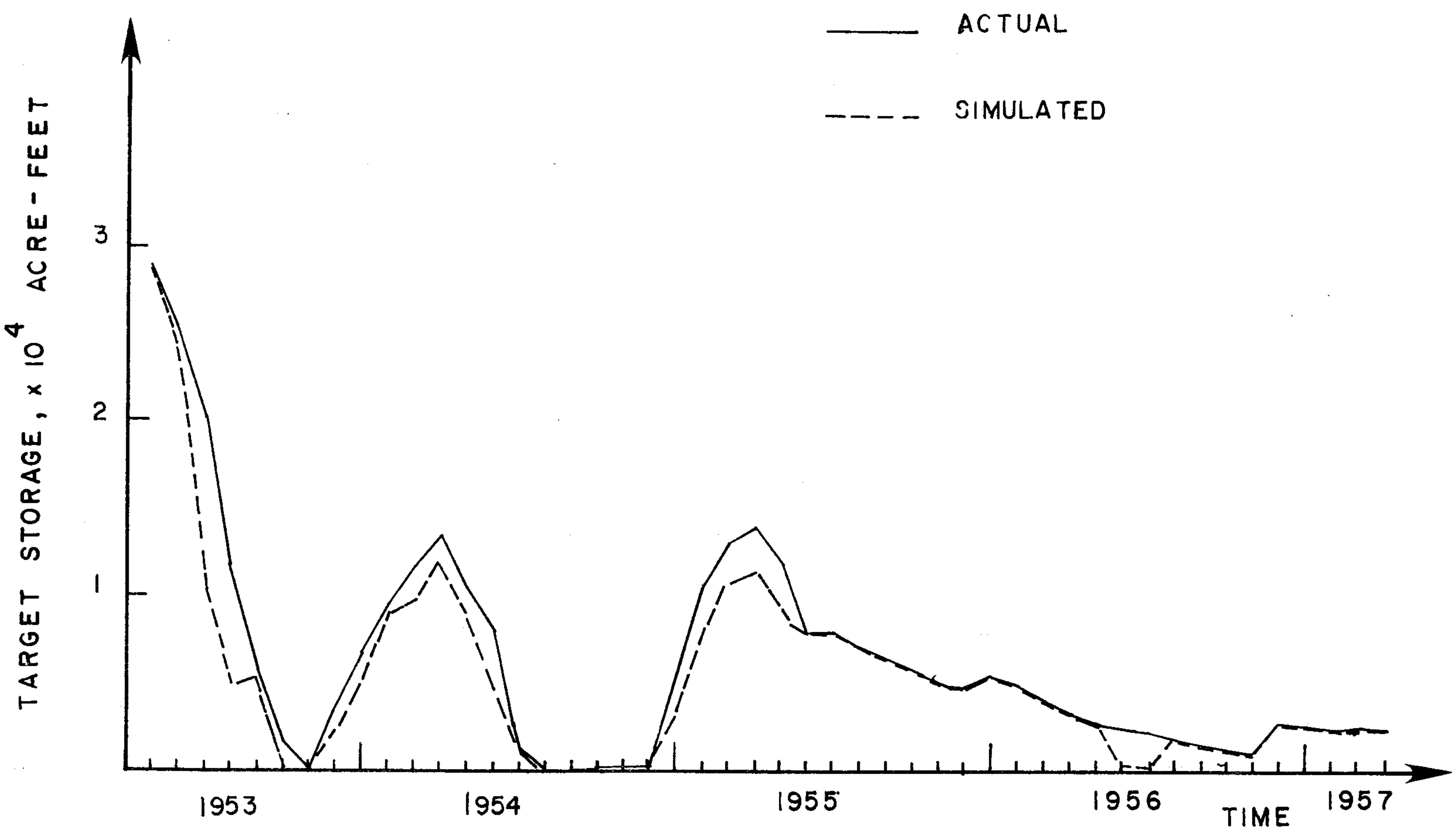

Figure 4.11. Comparison of Actual and Simulated Target Storages for Prewitt Reservoir, 1953-57. (Verification Run)

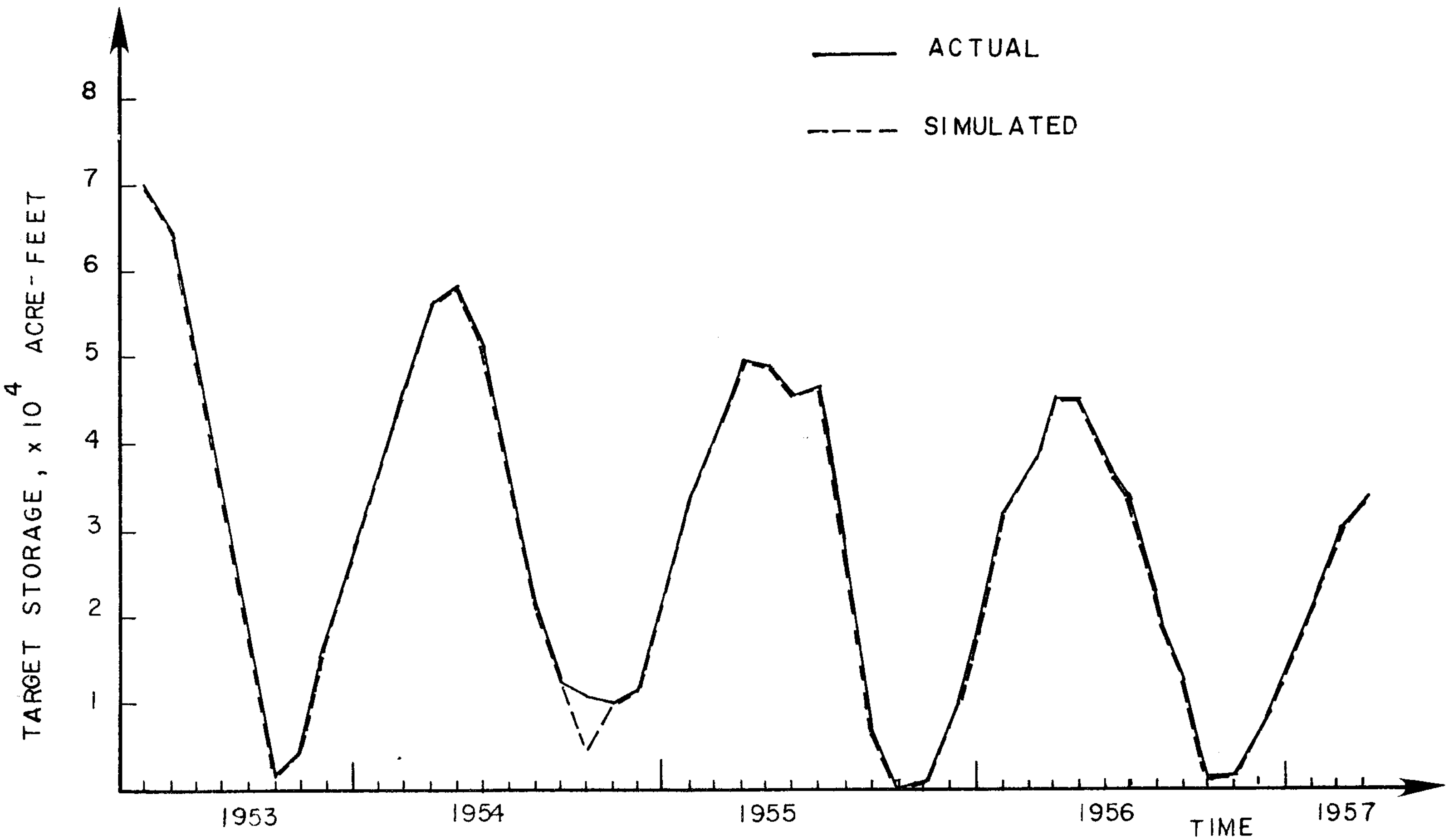

Figure 4.12. Comparison of Actual and Simulated Target Storages for North Sterling Reservoir, 1953-57. (Verification Run)

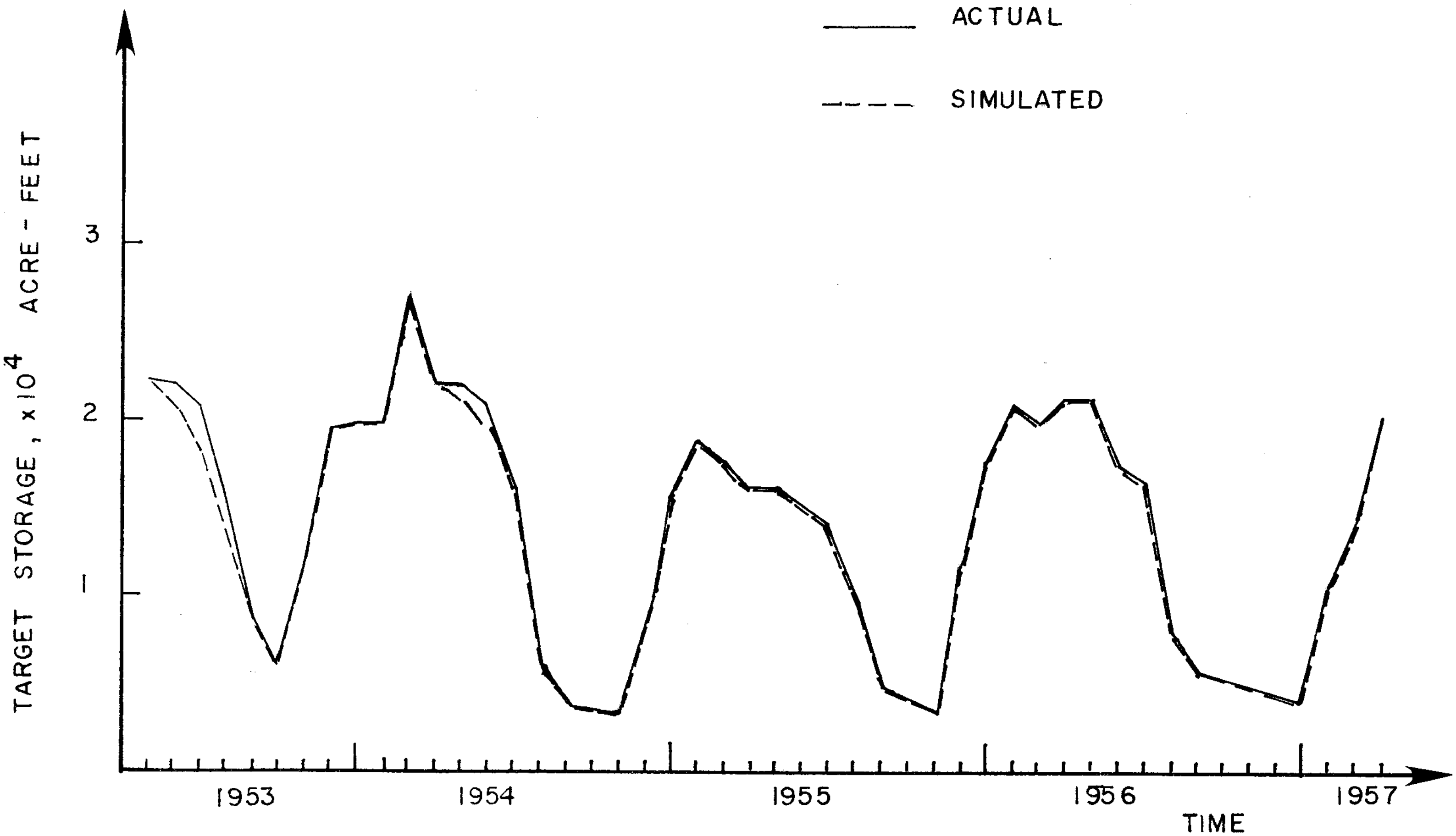

Figure 4.13. Comparison of Actual and Simulated Target Storages for Julesburg Reservoir, 1953-57. (Verification Run)

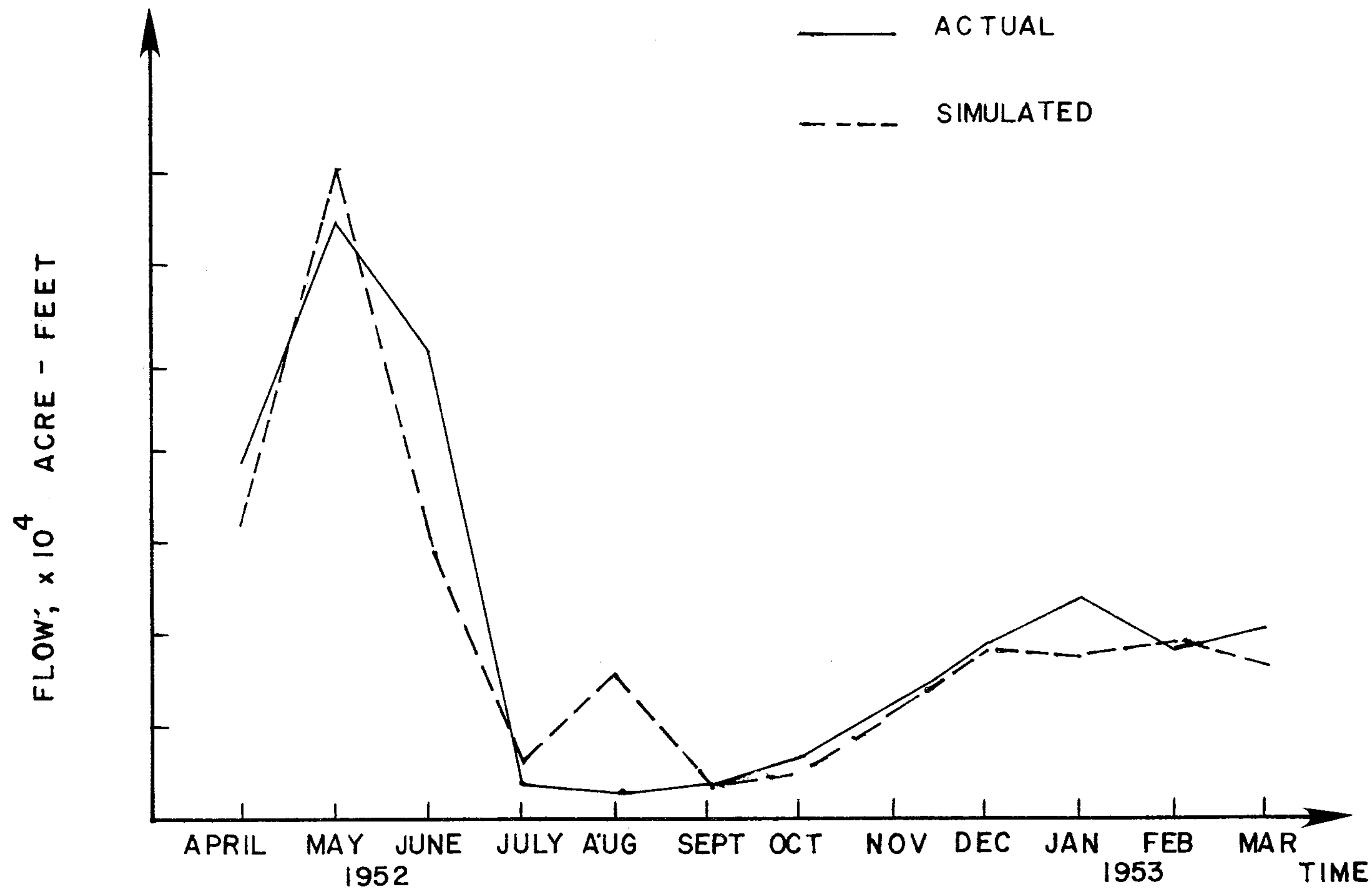

Figure 4.14. Comparison of Actual and Simulated Outflows at Julesburg Gaging Station, 1952-53.

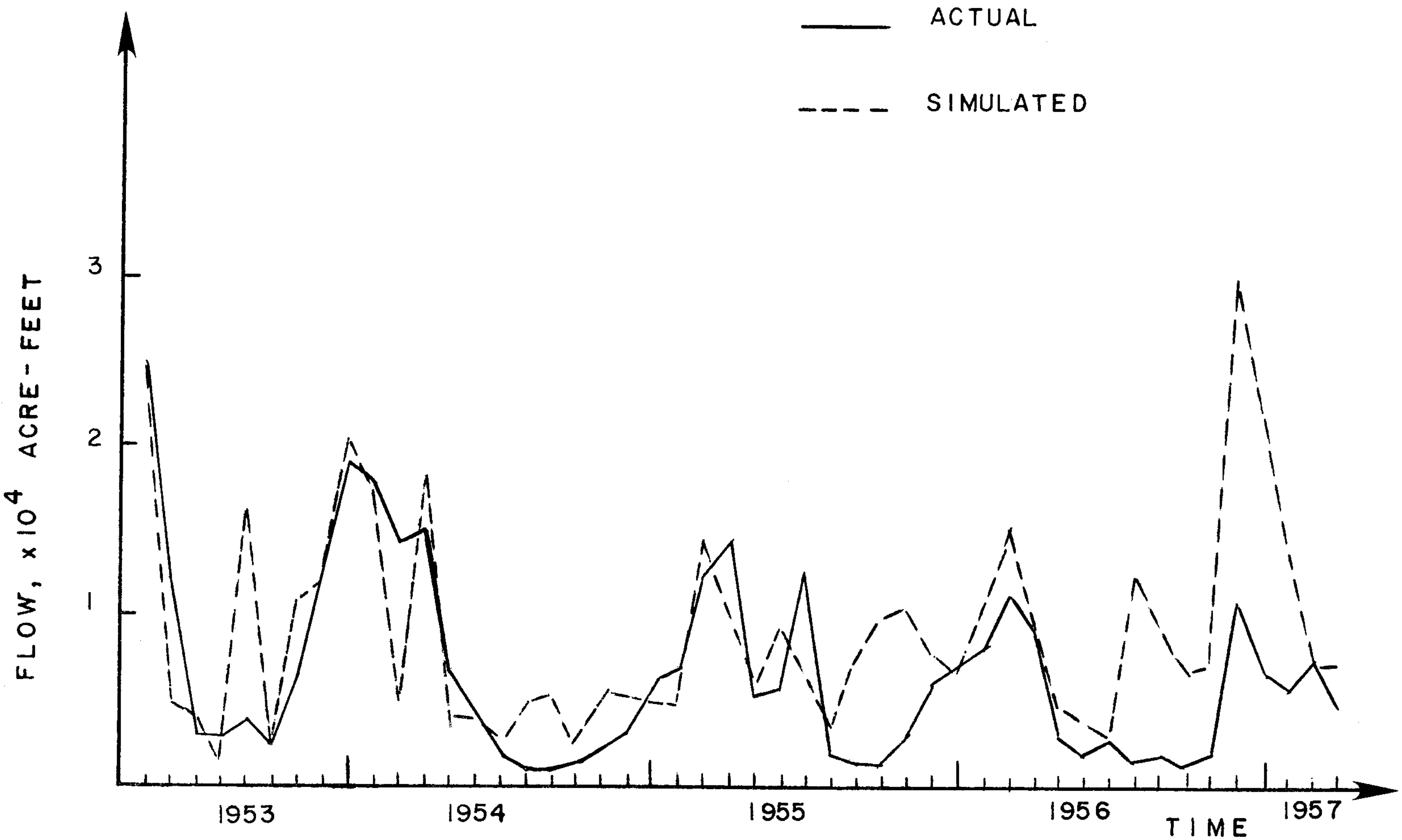

Figure 4.15. Comparison of Actual and Simulated Outflows at Julesburg Gaging Station, 1953-57. (Verification Run)

E. CALIBRATION RESULTS

In the calibration, all of the historical irrigation demands corresponded exactly. However, the model computed reservoir storage levels show good correspondence to observed levels. Deviations from observed gaged flows at Julesburg gaging station are reasonable, except for August 1952 in the calibration. The verification run shows that the model under estimates flows during the late summer and early fall months. It was decided that these results were reasonable enough for screening purposes. Unaccounted for local surface runoff into reservoirs could possibly explain some of the underestimation of flows. The model as calibrated for this case study can be regarded as a safe, conservative predictive tool.

Note that in the calibration phase the monthly pumping capacity of each demand area is set to its actual pumping value with the present priority of a high negative number (in this study -50,000). The allocated flow is therefore always at the upper bound of the pumping link which is equal to the actual pumping.

F. DEMONSTRATION MANAGEMENT STUDIES

The case study area provides a viable demonstration of the ability of the model to examine conjunctive use strategies for a stream-aquifer system. During average or wet years, water demands are usually met. However, excess water leaves the State of Colorado during average and wet years due to lack of surface storage. During drought periods there is severe water shortage because of less precipitation and surface water to meet demands. Farmers who possess junior water rights have to use groundwater as a supplemental source of water, depending on how junior the water rights are. Because of the interaction between the stream and aquifer, groundwater pumping cannot be allowed to deplete the river flow in such a way as to injure senior water right users.

The Office of the State Engineer develops various plans for water augmentation for drought relief. Private organizations such as GASP (Groundwater Appropriators of the South Platte) have also developed their own plans for water augmentation. However, these plans are only for short term relief of water shortages.

With CONSIM as a tool, various management alternatives, whether short or long term, can be studied and evaluated. These alternatives can serve as a basis for integrated water management decision making over an entire river basin through voluntary cooperation of water users. The objective would be to maximize beneficial use of water through coordinated operation and innovated exchange mechanisms as designed through use of the model, while insuring that no user is injured beyond what would have occurred under normal operations. Let us consider some of the management options that could be examined through use of CONSIM.

<u>Artificial recharge</u>: Instead of waiting for a drought year to arrive and trying to solve the problem at hand, it is better to prepare by stocking water for drought relief purposes while water is abundant. To store water in surface reservoirs for this purpose requires large on stream surface reservoirs which are costly, create political controversy, and may be inefficient because of evaporation losses. Subsurface storage is already available with capacities of several orders of magnitude over what could ever be developed in surface storage. Artificial recharge is generally inexpensive, though the main disadvantage of conjunctive use is the need to expend energy to extract water. CONSIM can

be useful for examining the effect of various artificial recharge sites and selection of optimal pumping strategies.

New surface reservoirs: Though large reservoirs are costly, they can be energy producers through hydropower, rather than energy expenders. In some cases, several smaller reservoirs may be a more attractive alternative, including better utilization of many existing small reservoirs. Reservoirs also serve as artificial recharge sources for the aquifer if they are located within the aquifer boundaries. CONSIM can be used to examine optimal siting and sizing of reservoirs, as well as the best ways to operate them.

New demands: Excess water during wet years can be used for supplying demands outside of the existing system. In this particular study case, the Northern High Plains is a prospective new demand. Excess water from the South Platte River can possibly be used for artificial recharge to the Ogallala aquifer in this area. Energy development is also a prospective new demand for this excess water. CONSIM could be used to test the impacts of such demands on the existing system.

Compact change alternative: Should the compact agreement between the States of Colorado and Nebraska be revised? If so, what is its impact on the system? CONSIM is a valuable tool for answering these questions.

Equity of water use alternative: If the existing water right system is modified to a volumetric rather than a flow rate basis, what is its impact on the system?

Of these, and many other possible managment studies, we select only two for demonstration purposes: use of artificial recharge and the compact change analysis.

F.1. Management Alternative #1

Several assumptions are used for this artificial recharge alternative.

1. All irrigation demand areas (nodes) have access to water that could be used for artificial recharge purposes.
2. Artificial recharge is accomplished by simply applying excess water uniformly over the demand areas.
3. Artificial recharge water is assumed to eventually reach the groundwater table during a particular month regardless of weather conditions. For example, in winter months the top soil may be subject to thawing during certain intermittent warm periods during which recharge water can percolate to the groundwater table. Most artificial recharge is assumed to occur during high flow spring months.

The objective is to capture excess water currently exported from the system and store it in subsurface aquifers so that it can be used at a later time. The time from November to March is chosen as the artificial recharge period.

For this analysis, irrigation demand is set equal to potential evapotranspiration, multiplied by a factor greater than one to compensate for irrigation efficiencies and various other water losses. The factor chosen in this study is 1.5. The 50 percent increase is considered to be adequate to compensate for an average of 20 percent channel loss, 22 percent of tailwater loss and 8 percent of other minor losses.

For the nonirrigation season, the water demands for areas chosen for artificial recharge are set equal to a fraction of the total amount of water to be recharged. This fraction is obtained by the following equation

$$AR_{ik} = EX_k \cdot \frac{RA_i}{\sum_{i=1}^{I} RA_i} \tag{4.3}$$

where AR_{ik} = amount of water to be recharged over area i during period k

EX_k = total amount of water available to be recharged during time period k

RA_i = user input priority for artificial recharge of area (node) i

I = total number of areas (nodes) receiving artificial recharge

Note that in this case, higher numbers of RA_i mean higher priority.

The criteria to choose the priorities for areas to receive artificial recharge water are as follows.

1. Areas which are heavily pumped during the irrigation season, regardless of their locations. Upstream artificial recharge is considered to be more attractive than downstream artificial recharge. However, downstream area artificial recharge is also essential to bring groundwater levels back to normal conditions or higher for the next irrigation season pumping.

2. Areas which rely entirely on groundwater use. These areas are usually junior in surface water rights. They usually have canals for surface water diversion but rarely are able to use them.

From these criteria, area numbers (node numbers) 25, 27, 28, 29, 30, 31, 33, 34, 35, 40, 41 and 48 were chosen. All of the surface reservoir operating target storages were set to 100 percent of capacity. The model computes best target storage levels for these reservoirs according to their priorities in relation to all other demands and available water. Excess water available for artificial recharge was set equal to 150 percent of total unregulated inflow. This number is not unreasonable since from historical operation of this system, total surface water diversions greatly exceed total stream inflows because of successive downstream use of return flows. The values of RA were chosen arbitrarily by giving higher priorities to those areas where subsurface storage depletions were more severe.

The time period from April 1953 to March 1957 was chosen for illustration of this management alternative. This period was chosen since it was considered to be the driest period, with three dry years in a row (1953-1955) and an average year (1956). The computer runs were made for this period with no artificial recharge, and then with 150 percent of total unregulated inflow assumed available for artificial recharge.

In this management alternative, the pumping policy is determined in an iterative fashion. In the first iteration, all pumping is set equal to zero, and shortages are computed. Then, pumping is set equal to shortages incurred in the first iteration for the second iteration. For the third iteration, pumping is set equal to pumping from the second iteration plus new shortages, and so on. The process repeats until shortages vanish or are close to zero. A computer run was made to determine the optimal conjunctive use for this management alternative using the above procedure. The results are intuitively reasonable and are summarized as follows:

1. For irrigation areas with high priorities, demands are met by surface water alone.
2. For irrigation demand areas with intermediate priorities, demands are met using both surface water and groundwater conjunctively.
3. For irrigation demand areas with low priorities, demands are met almost totally by groundwater.

Examples of some irrigation demand area pumping patterns are shown in Figures 4.16 and 4.17 for intermediate and low priority areas, respectively.

The optimal pumping policy obtained by the above procedure does not resemble the actual pumping activity in the system. Generally, most irrigation domand areas, whether with high, intermediate or low priorities, use both surface and groundwater. Therefore, the optimal pumping policy obtained from the above procedure would require cooperative, voluntary exchange agreements for implementation.

Since it is known that all users pump groundwater to a certain degree, then instead of setting initial pumping to zero in the first iteration, they can be set to fractions of demand. Each fraction may be a long-term average pumping percentage of demand for each area. Another computer run was made using assumed initial pumping equal to 20 percent of demands for all areas and all irrigation months. The 20 percent

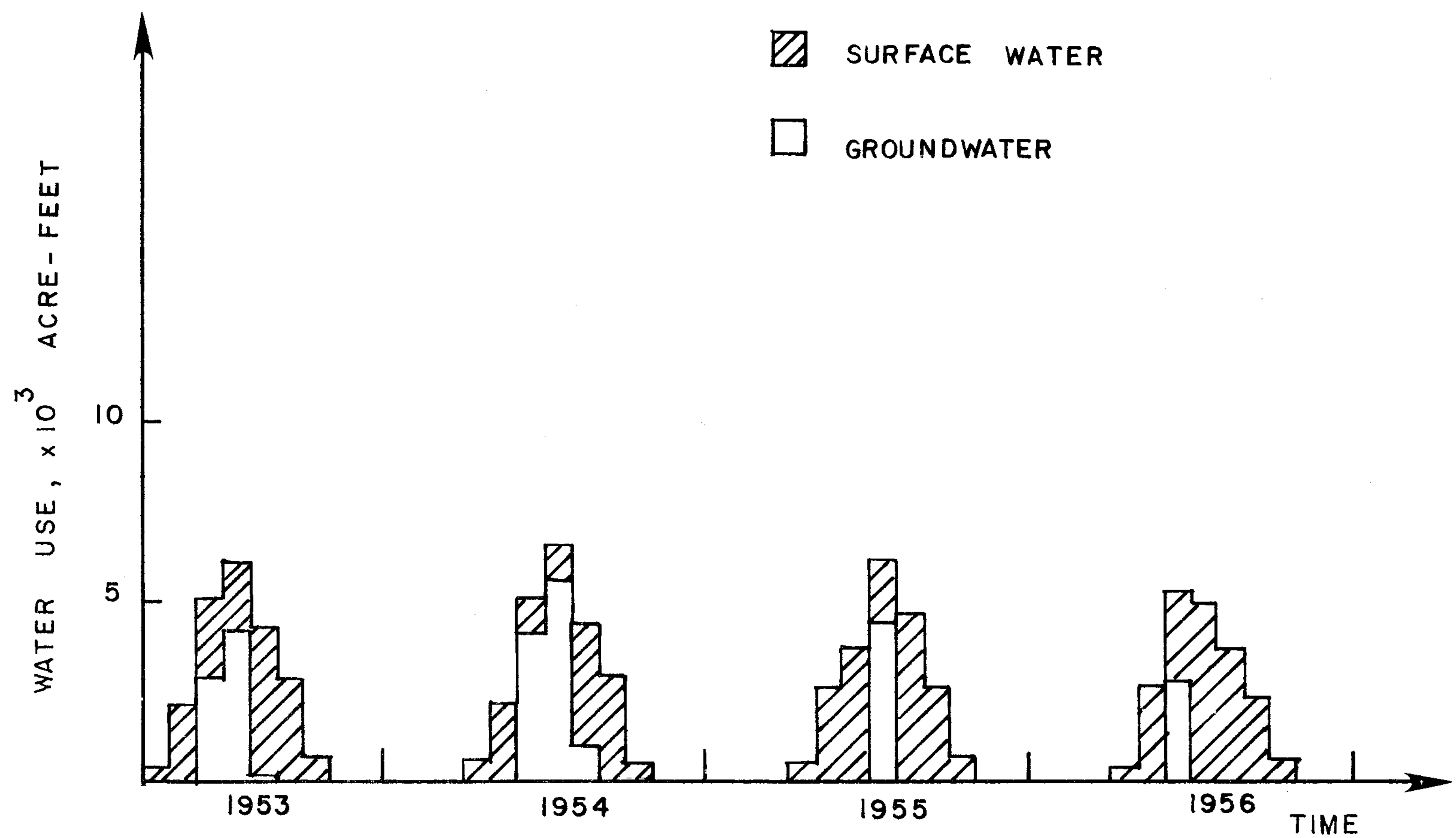

Figure 4.16. Example of Demand Satisfaction of Intermediate Priority Area with Zero Initial Pumping for Node 32, Iliff and Platte Valley.

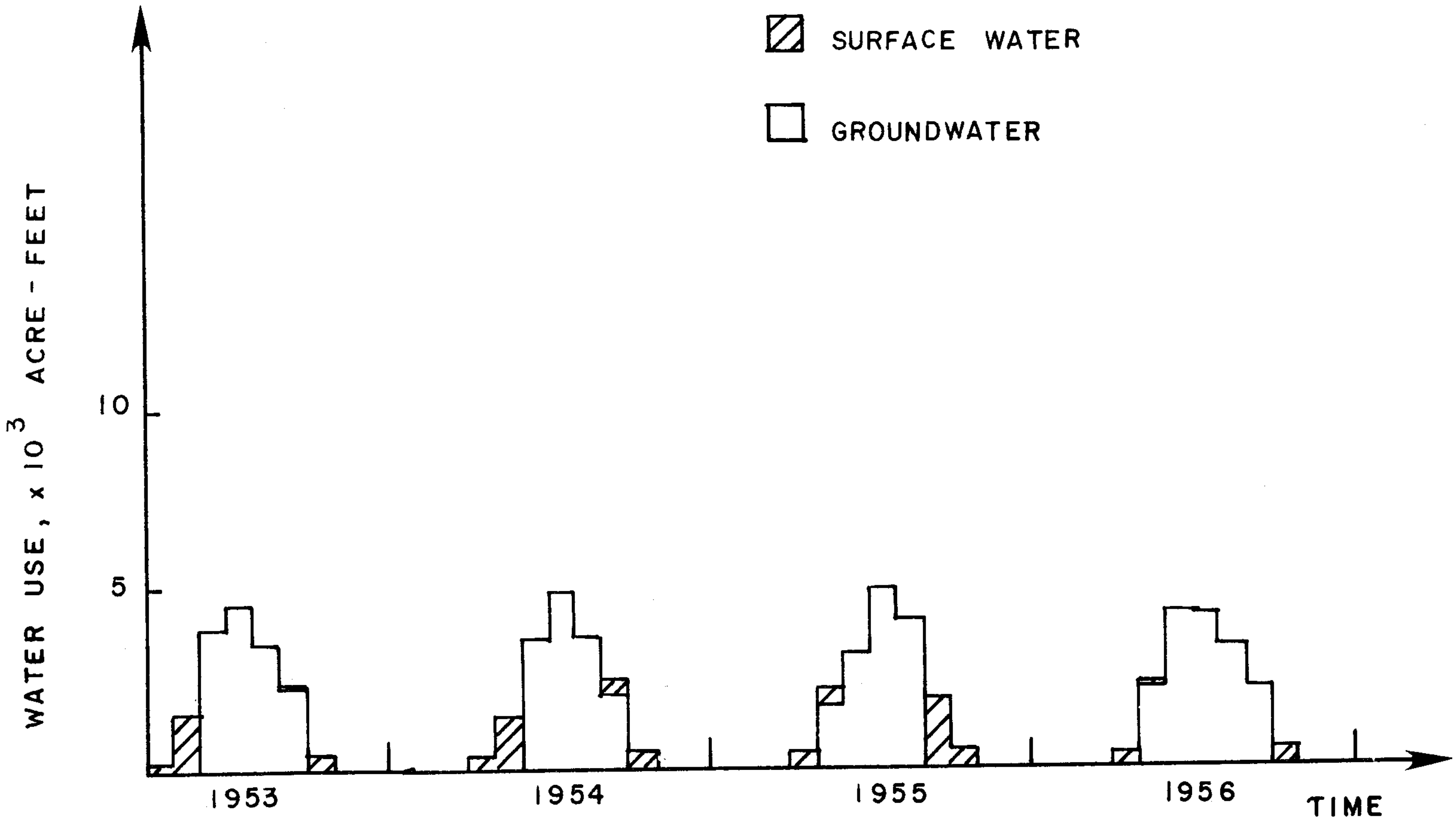

Figure 4.17. Example of Demand Satisfaction of Low Priority Area with Zero Initial Pumping for Node 50, Tamarack.

figure used is for illustration purposes only. Pumping patterns obtained are shown in Figures 4.18 and 4.19 for the same intermediate and low priority areas, respectively. For high priority areas, pumping was always kept at the preset initial pumping levels. Model users may wish to run CONSIM for several initial pumping fraction sets to evaluate various pumping policies and select a suitable one.

A comparison of the artificial recharge alternative and no artificial recharge alternative can be made using the total accumulated subsurface storage changes with time from the two alternatives. This is shown in Figure 4.20.

It can be seen that the artificial recharge alternative is a much better alternative than the no artificial recharge alternative. The overall accumulated subsurface storage change improves from -84,139 acre-ft with no artificial recharge alternative to -10,888 acre-ft for artificial recharge alternative, respectively. Note that the comparison is made using 20 percent initial pumping for artificial recharge alternative.

F.2. <u>Management Alternative #2</u>

In this management alternative, it is assumed that the compact between the States of Colorado and Nebraska may be subject to revision. The current compact agreement specifies that during April 1 to October 15, the outflow from the State of Colorado to the State of Nebraska must not be less than 120 cubic feet per second.

To study the impact of a compact change, the network system configuration is modified so that the last node of the network represents a demand node whose monthly demand is set equal to the compact agreement amount. Three schemes were studied for this management alternative.

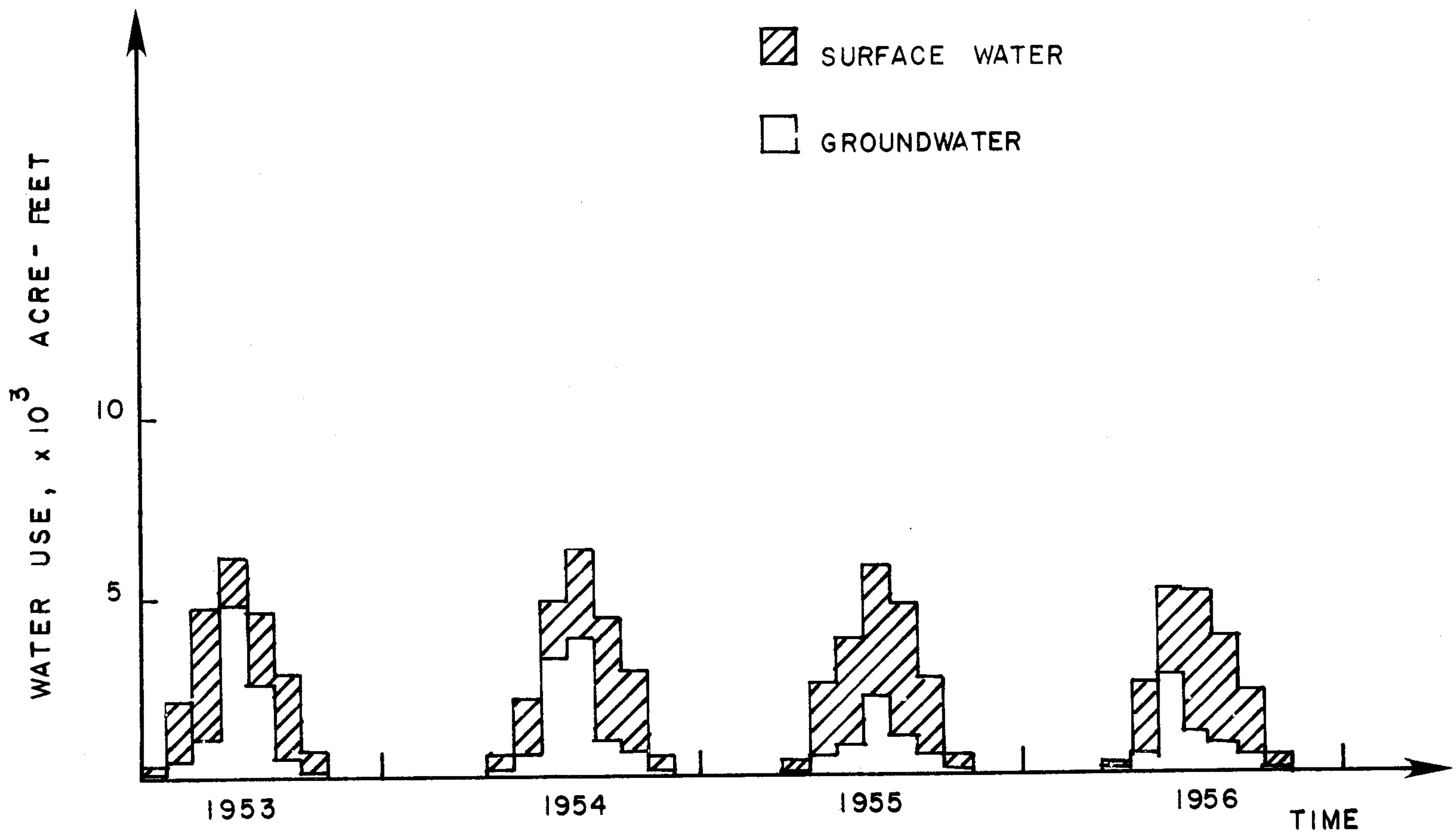

Figure 4.18. Example of Demand Satisfaction of Intermediate Priority Area with 20% of Demand Initial Pumping for Node 32, Iliff and Platte Valley.

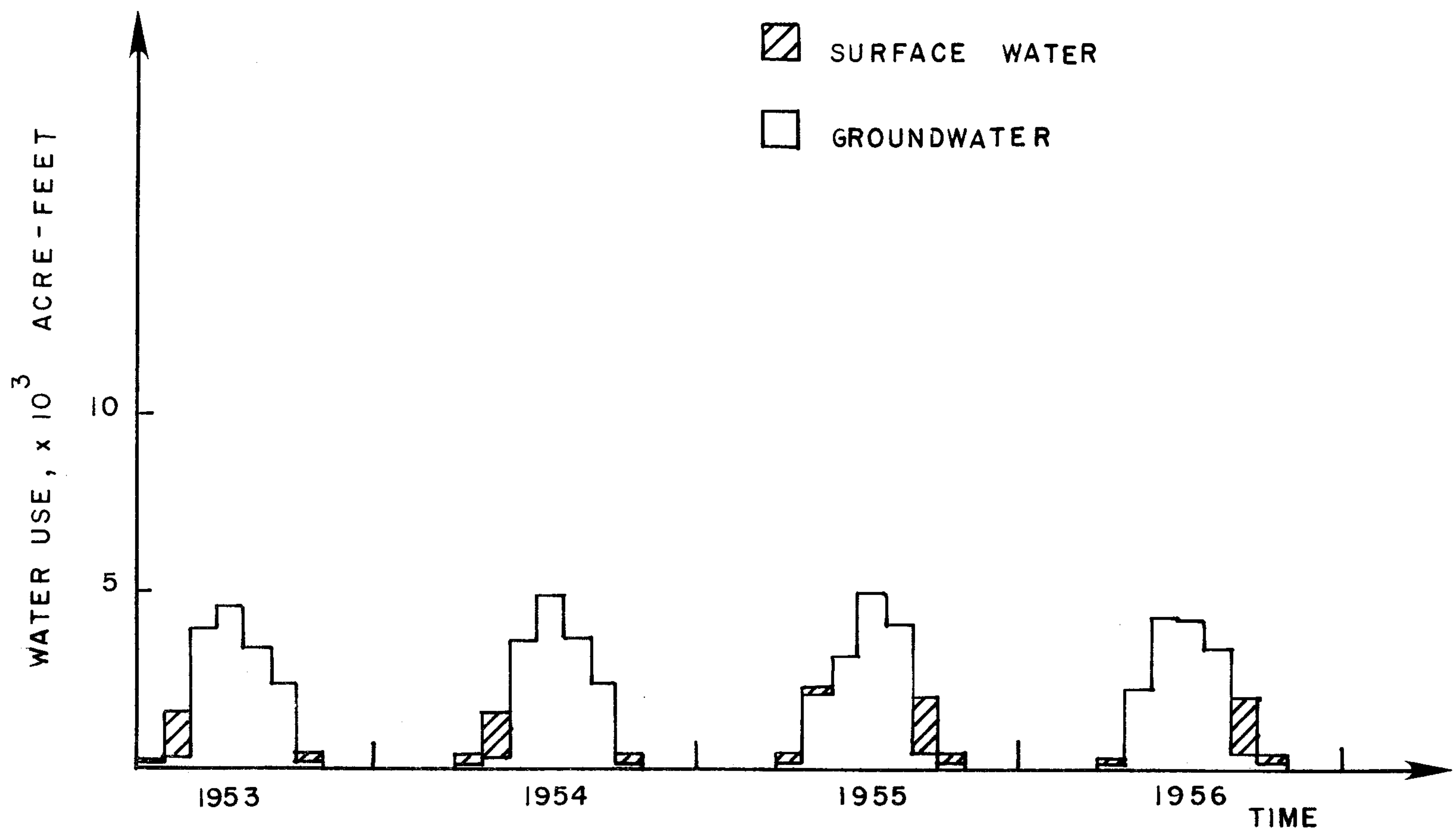

Figure 4.19. Example of Demand Satisfaction of Low Priority Area with 20% of Demand Initial Pumping for Node 50, Tamarack.

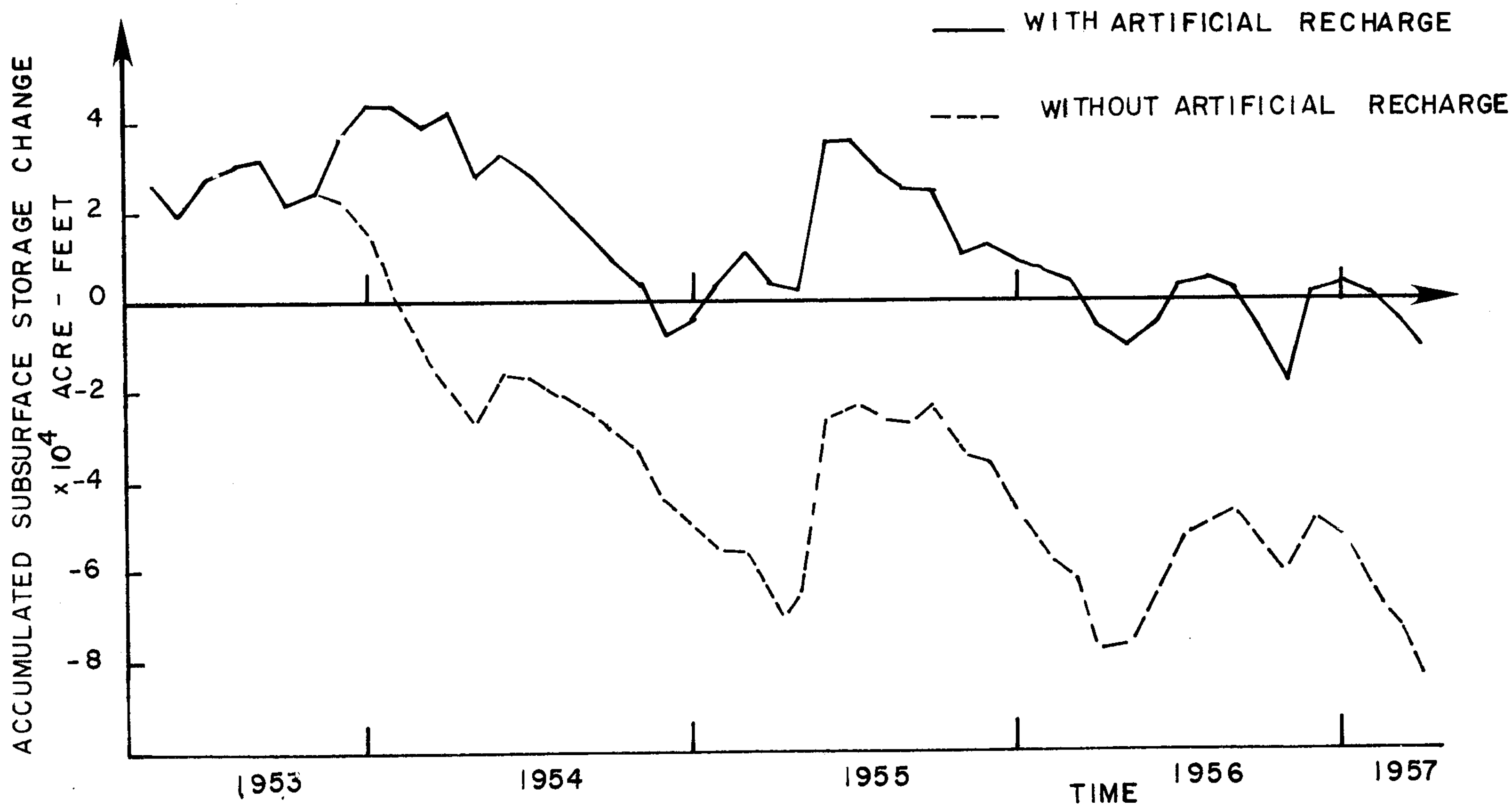

Figure 4.20. Comparison of Subsurface Storage Changes between Alternatives with Artificial Recharge and Without Artificial Recharge.

The first scheme represents the original compact agreement. The second scheme represents an assumed new compact agreement that would double the original compact amount. The third scheme represents the assumed new compact agreement to decrease the original compact amount to a minimum flow requirement. This minimum flow requirement is necessary to keep the solution feasible. Zero flow causes an infeasible solution for this type of network setup. This is because the last river reach always has some return flows from the aquifer, and these return flows cannot contribute to any node if the last node has zero demand.

Actual data for year 1952 were used for this management alternative since these flows represent an average flow year for this case study area. Results of the computer run are shown in Figures 4.21 to 4.23. Figure 4.21 shows monthly storage levels for one of the reservoirs in this system. Figure 4.22 shows monthly excess water available after meeting all demands. In this management alternative, all reservoir operating rules were set equal to the reservoir maximum capacities. It can be seen that reservoir storage levels and excess water are decreasing with increasing downstream flow requirements.

From these results, there are shortages associated with the first and second schemes. Shortages from the first scheme are quite small and can be neglected. However, shortages from the second scheme are considerable. This indicates that any revision of a compact agreement to substantially increase the amount of flow is undesirable for average and dry years. For wet years shortages are obviously low or even zero and the flows out of the State of Colorado are high.

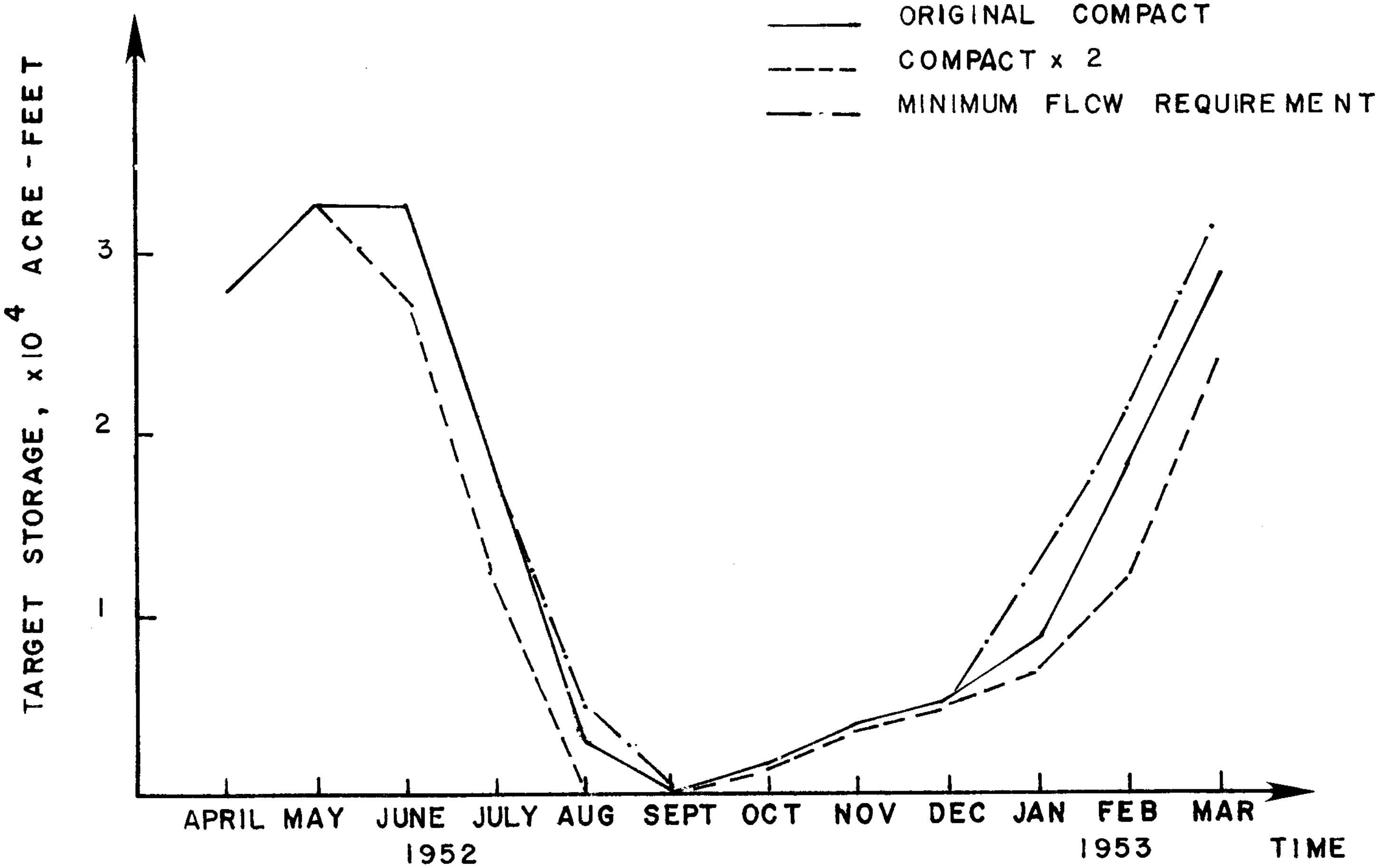

Figure 4.21. Comparison of Prewitt Reservoir Storages with Changes in Compact Agreement.

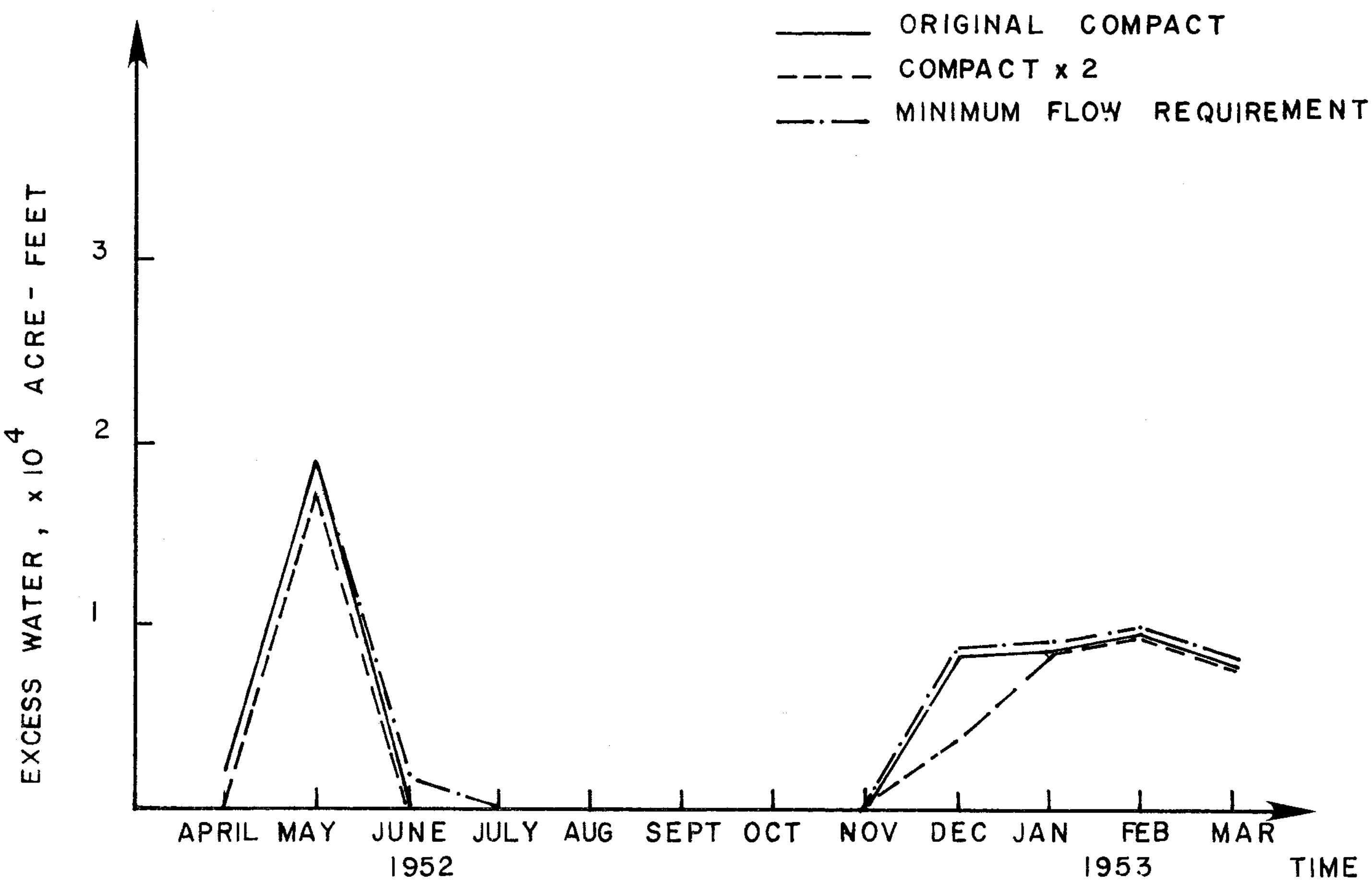

Figure 4.22. Comparison of Excess Water from Various Compact Alternatives.

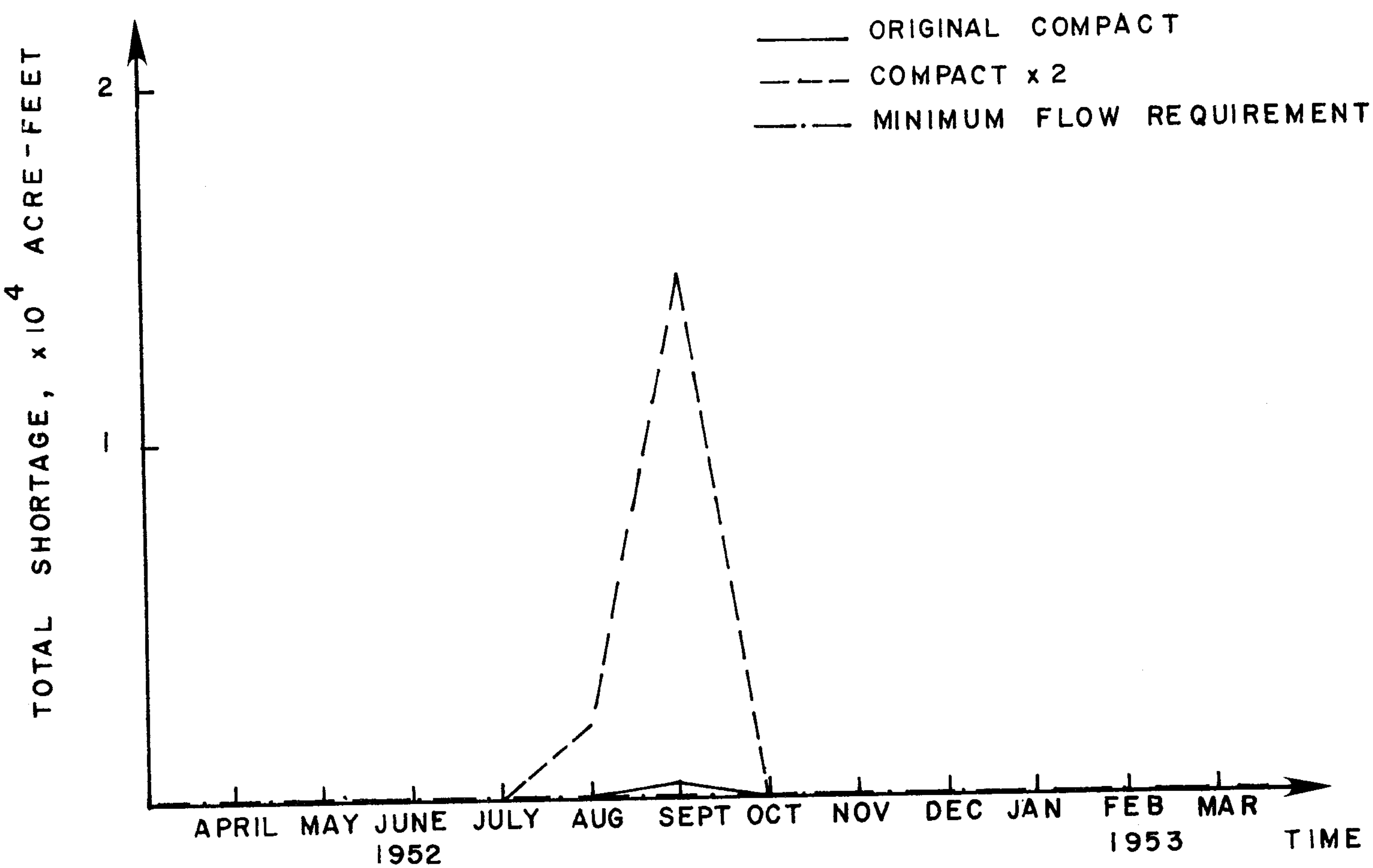

Figure 4.23. Comparison of Total Shortages from Various Compact Alternatives.

REFERENCES

Ault, D. V., "Calibration of a River Basin Simulation Model," Department of Civil Engineering, Colorado State University, Fort Collins, CO, 1981.

Bercha, F. G. and Assoc., Ltd., "Alberta Environment WBSM: Model Guide," Report #WBSM-99-99, Alberta, Canada, March 1981.

Bittinger, M., "Groundwater-Surface Water Conflicts," Journal of the Water Resources Planning and Management Division, ASCE, Vol. 16, No. WR2, July 1980, pp. 467-475.

Burns, Alan W. and Theodore Hurr, U.S. Geological Survey, Denver Office, personal communication.

Culver, C. F. and C. B. Millham, "Hydropower Losses from Navigation in the Snake-Columbia Rivers: 1978-1979," Water Resources Bulletin, Vol. 17, No. 3, June 1981, pp. 501-503.

Evans, N., Colorado Drought Workshop, Colorado Water Resources Research Institute, November 29, 1977, Denver, Colorado.

Farr, W. D., "Challenge to Innovation," Colorado Drought Workshop, Colorado Water Resources Research Institute, November 29, 1977, Denver, Colorado, pp. 19-22.

Fleming, D. E., "The CORSIM Project," presented at the Annual Meeting of the American Society of Civil Engineers, Denver, Colorado, November 4, 1975.

Fulkerson, D. R., "An Out-of-Kilter Method for Minimum Cost Flow Problems," Journal of the Society of Industrial and Applied Mathematics, Vol. 9, No. 1, March 1961, pp. 18-27.

Gerlek, Steve, "Water Supply Analysis of the South Platte River Basin," Technical Appendix Volume 2, Report for U.S. Army Engineer District, Omaha Corps of Engineers, Omaha, Nebraska, April 1977.

Glover, R. E., "Transient Groundwater Hydraulics," Department of Civil Engineering, Colorado State University, Fort Collins, CO, 1974.

Hayes, E. T., "Assessing Energy Available to the U.S. in 2000," in Hydropower: A National Energy Resource, Engineering Foundation, New York, NY, March 1979.

Hurr, R. Theodore, et al., "Hydrology of the South Platte River Valley, Northeastern Colorado," USGS, Colorado Water Conservation Board and Colorado Division of Water Resources, Water Resources Circular No. 28, 1975.

Hurr, R. Theodore, et al., "Hydrogeologic Characteristics of the Valleyfill Aquifer in the Brush, Sterling and Julesburg Reaches of the South Platte River Valley, Colorado," U.S. Geological Survey Open File Report prepared in cooperation with the Colorado Water Conservation Board, 1972.

Illangasekare, T. H., "Influence Coefficients Generator Suitable for Stream-Aquifer Management," Ph.D. Dissertation, Colorado State University, Fort Collins, CO, 1978.

Jensen, M. E. (ed), Design and Operation of Farm Irrigation Systems, Monograph No. 3, American Society of Agricultural Engineers, St. Joseph, MI, 1980.

Konikow, L. F. and J. D. Bredehoeft, "Modeling Flow and Chemical Quality Changes in an Irrigated Stream-Aquifer System," Water Resources Research, Vol. 10, No. 3, pp. 546-562, 1974.

Labadie, J. W., "A Surrogate-Parameter Approach to Modeling Groundwater Basins," Water Resources Bulletin, Vol. 11, No. 1, February 1975, pp. 97-114.

Labadie, J. W. and I. Khan, "River Basin Salinity Management via the ASTRAN Method: (2), Case Study," Journal of Hydrology, July 1979.

Labadie, J. W. and J. M. Shafer, "Water Management Model for Front Range River Basins," Technical Report No. 16, Colorado Water Resources Research Institute, Colorado State University, Fort Collins, CO, April 1979.

Labadie, J. W., J. M. Shafer and R. Aukerman, "Recreational Enhancement of High Country Reservoirs," Water Resources Bulletin, Vol. 16, No. 3, June 1980.

Lazaro, R., J. W. Labadie and J. Salas, "State-Space Streamflow Forecasting Model for Optimal River Basin Management," presented at the International Symposium on Real-Time Operation of Hydrosystems, University of Waterloo, Waterloo, Ontario, Canada, June 24-26, 1981.

Longenbaugh, R. A. and I. F. Wymore, "Analyses of Methods for the Determination of Water Availability for Energy Development," FEA/G-77/059, Federal Energy Administration, May 1977.

Maasland, M., "Water Table Fluctuations Induced by Intermittent Recharge," Journal of Geophysical Research, Vol. 64, No. 5, 1949, pp. 549-559.

Maddock, T., III, "Algebraic Technological Function for a Simulation Model," Water Resources Research, Vol. 8, No. 1, February 1972, pp. 129-134.

McWhorter, D. B., "Lecture Notes for CE 631: Solutions to Groundwater Problems," Department of Civil Engineering, Colorado State University, Fort Collins, CO, Spring 1978.

McWhorter, D. B., et al., "Analytical Model of Stream-Aquifer Systems with Pumping and Recharge," a completion report, submitted to Colorado State Engineer's Office, Denver, CO, October 1, 1972.

Morel-Seytoux, H. J. and C. J. Daly, "A Discrete Kernel Generator for Stream-Aquifer Studies," Water Resources Research, Vol. 11, No. 2, April 1975, pp. 253-260.

Phamwon, S., "Network Model for Optimal Management of Stream-Aquifer Systems," Ph.D. Dissertation, Colorado State University, Ft. Collins, CO, Spring 1982.

Powell, M. J. D., "An Efficient Method for Finding the Minimum of a Function of Several Variables without Calculating Derivatives," Computer Journal, Vol. 7, 1964, pp. 155-162.

Schreiber, R. P., "A Digital Simulation Model for Groundwater-Surface Water Systems," M.S. Thesis, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1976.

Shafer, J., "An Interactive River Basin Water Management Model: Synthesis and Application," Technical Report No. 18, Colorado Water Resources Research Institute, Colorado State University, Fort Collins, CO, August 1979.

Shafer, J. M. and J. W. Labadie, "Synthesis and Calibration of a River Basin Water Management Model," Completion Report No. 89, Colorado Water Resources Research Institute, Colorado State University, Fort Collins, CO, October 1978.

Texas Water Development Board, "Economic Optimization and Simulation Techniques for Management of Regional Water Resource Systems: River Basin Simulation Model SIMYLO II -- Program Description," Systems Engineering Division, Austin, Texas, July 1972.

Thaemert, R. L., "Mathematical Model of Water Allocation Methods," Ph.D. Dissertation, Colorado State University, Ft. Collins, CO, 1976.

Toups Corporation, "Water Resources Investigation in the South Platte River Valley Reach, No. 2, Balzac Gaging Station to the Kersey Gaging Station," a report submitted to Colorado Water Conservation Board, Colorado, 1978.

U.S. Army Corps of Engineers, "HEC-3 Reservoir System Analysis for Conservation - User's Manual," Hydrologic Engineering Center, Davis, CA, July 1974.

U.S. Department of Agriculture, "Irrigation Water Requirement," U.S. Soil Conservation Service, Engineering Division, Technical Release No. 21, Revised September 1970, p. 88.

Westen, L. K. and R. E. Swain, "Artificial Recharge in the South Platte River Basin," Journal of the Irrigation and Drainage Division, ASCE, Vol. 105, No. IR2, June 1979, pp. 117-127.

Wurbs, R. A., "Flood Damage Reduction System Optimization," Ph.D. Dissertation, Department of Civil Engineering, Colorado State University, Fort Collins, CO, 1978.

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