CONVEYANCE LOSS MODELING OF RESERVOIR RELEASES IN NATURAL STREAMS OF WYOMING

J. Joel Farber　　Victor Hasfurther

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ABSTRACT

Two Wyoming streams are investigated to predict conveyance losses in natural streams. The computer model J349, originally developed by the United States Geological Survey, is adapted for use in assessing conveyance losses. Primary losses are attributed to bank storage and a reduction of groundwater inflow. Periods of steady streamflow are not required to obtain reliable estimates of conveyance loss.

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CHAPTER I

INTRODUCTION

BACKGROUND

The value of water, as with other resources, is measured in proportion to its scarcity. Where scarcity exists it is necessary to contract some method of resource allocation. The allocation of the natural surface waters of Wyoming is founded on the prior appropriation system of water law. This system of water law severs the water use rights from land ownership, and allocates available water in order of priority, such that the rights of senior appropriators may, in time, be satisfied at the expense of juniors in times of shortage.

The central concern in the allocation of surface water under the prior appropriation system historically was not the apportionment of a finite depleting supply, but rather the geographic and temporal segregation between supply and demand. Accordingly, the water law of Wyoming allows for:

i) the owner of a right to transfer the right to a different place of use and/or to a different kind of use.

ii) the owner to petition to change the point of diversion.

iii) exchanges between any combination of stored, direct flow, or ground water rights between appropriators.

Explicit to each of these entitlements is a stipulation that the action may not, in any manner, injure other existing lawful appropriators, whether their water right be senior or junior to the water right in question. In addition, implicit to each of the above entitlements is the possibility for augmented flows to be conveyed in the natural water course. An accurate assessment of the conveyance losses associated with augmentation water flowing in natural streams is necessary for determining potential injury to prior rights.

Article 8 of the Constitution of the State of Wyoming proclaims all waters within the State are the property of the State. Furthermore, the responsibility for supervision of the waters of the state and of their appropriation, distribution and diversion is placed with a Board of Control, composed of the State Engineer and the Superintendents of the water divisions.

The State Engineer's Office and Board of Control have been encouraging research through the Wyoming Water Resources Center to quantify conveyance losses on selected streams throughout Wyoming since 1983. The investigations conducted to date have focused on three principal aspects of the problem:

i) defining the factors which influence conveyance losses in Wyoming streams.

ii) acquiring a data base of measured stream flows suitable for use in the assessment of conveyance losses.

iii) the development of one or more methodologies to allow the prediction of conveyance losses in Wyoming streams and rivers.

An extensive list of factors influencing conveyance losses is described in a thesis by Pahl (1985). In addition, Pahl collected data on three Wyoming streams to determine conveyance losses. In an effort to expand this data base Hanlin (1988) collected additional data on two streams and compiled historical records for the North Platte River.

The methodologies investigated to this point have been based on a water budget analysis. Such an approach is based on the principle of conservation of mass. This principle states the difference between the inflow of mass to a system and the outflow equals the change in storage of mass within the system with respect to time. The application of this principle to the investigation of conveyance losses in a stream system required the stream system be defined such that the change in storage with respect to time was set equal to zero. This was accomplished by limiting the analysis of the stream system to periods of stable flow. Under such conditions the inflow was considered to equal the outflow to the stream system and any difference was characterized as a conveyance loss. Thus measured inflows were subtracted from measured outflows during stable flow periods to assess conveyance losses.

The primary drawback in the methodologies investigated to date has been the limitation that only steady flow periods may be used to evaluate conveyance losses. Additionally, it has been difficult to quantify the various components of loss (evaporation,bank storage etc.). These limitations can be overcome through the use of hydrologic modeling.

PURPOSE AND OBJECTIVES

The purpose of this paper is to contribute a methodology based on hydrologic modeling to address the prediction of losses of waters being conveyed in natural streams in Wyoming. The objectives of this paper are:

i) develop or acquire a hydrologic model which may be used to predict conveyance losses and test the model against the existing data base.

ii) determine a practical methodology based on a hydrologic model from which the Board of Control may access conveyance losses.

iii) extend the existing data base on the Green River between Fontenelle Reservoir and the Green River Golf Course, with data taken during the summer and fall of 1989.

In the initial phases of this study, inquiries were made to obtain one or more hydrologic models which may be used to predict conveyance losses. Concurrently, a literature search was begun to assess available mathematical expressions relevant to the prediction of the various components of conveyance losses. This information is presented in Chapter II. A hydrologic model available from the United States Geological Survey, known as J349, was chosen to be used in this study. Chapter III presents an overview of the operation of the J349 model and the modifications made to it.

A discussion of the methodology used for selection of a data base, data collection at the Green River and methods of investigation are given in Chapter IV. A discussion of the study results is presented in Chapter V. Conclusions and recommendations derived from the investigation are given in Chapter VI.

CHAPTER II

REVIEW OF LITERATURE

A summary of the available literature relevant to this study is contained in this chapter. Topics discussed in this chapter include: (1) Factors Affecting Conveyance Losses; (2) Review of Past Model Studies; and, (3) Relevant Research.

FACTORS AFFECTING CONVEYANCE LOSSES

There are a wide variety of factors which influence losses associated with the conveyance of augmentation flows in a natural water course. Research conducted on streams in Wyoming (Pahl,1985) identified a long list of potential factors which may influence conveyance losses, including:

- Length of Reach
- Natural flow in river
- Size of increase in flow
- Precipitation
- Elevation and slope of water table
- Stream channel characteristics
- Silt layer characteristics
- Evaporation
- Evapotranspiration
- Hydraulic characteristics of the aquifer
- Irrigation return flows
- Diversions
- Valley cross sections

In order to simplify the quantification of losses, Pahl addressed five factors; bank storage, channel storage, evapotranspiration, inadvertent diversions and groundwater inflow reduction, which are believed to be most significant. Bank storage is that loss which occurs from the infiltration of water into the banks of a stream due to an increase in the streams flow level. Typically, only a portion of water taken into bank storage is considered lost as some stored water returns to the stream as flows recess. Channel storage is a term used to describe the tendency of a stream channel to act as a reservoir. As flow levels in a stream increase, the channel of the stream stores increasingly more water. As in the case of bank storage, channel storage is a time dependent phenomena where stored water is released as stream levels lower.

Evapotranspiration is the consumptive loss of water through plants to the atmosphere (transpiration) and the evaporation of water from soil or water surfaces. Inadvertent diversions describe the excess water delivered by diversion structures as a result of a rise in the stage of the stream which may be caused by augmentation flows.

Many of the stream reaches investigated by Pahl were found to have significant groundwater flow contributions. During a rise in the stage of the stream, the hydraulic gradient near the stream-aquifer boundary is reversed and groundwater flows are temporarily deterred from entering the stream. This phenomena was termed a reduction in groundwater inflow.

The quantification of conveyance losses was also shown to vary by the framework in which these factors are defined. The total loss approach assesses a percentage of the total loss, of the entire streamflow, to the augmentation flow. Alternatively, the incremental loss approach assesses losses in proportion to the incremental increase in streamflow associated with the augmented water. In the case of the incremental approach, conveyance losses may further be defined in relation to the rate, or the total volume of the augmentation flow. The quantity of augmentation water in proportion to the natural streamflow will in main part determine whether the incremental loss or total loss approach affords the greater loss.

In 1985, Hanlin (1985) attempted to quantify conveyance losses using the total loss approach. Pahl's research had been conducted using the incremental approach because it is most easily evaluated utilizing surface flow records. Using surface flow data only, Hanlin determined the total loss approach was not applicable to streams which receive any influent groundwater flow. Streams which receive groundwater inflow are termed "gaining streams" and are most common in Wyoming. Therefore, Hanlin proposed an alternative methodology in which he coined the term "Net Total Loss" value. This value, when incorporated into the water budget analysis for the stream reach in question, affords the losses which should be charged such that the natural stream flow is not altered by the augmented water.

REVIEW OF PAST MODEL STUDIES

A wide variety of studies have been completed to determine conveyance losses using streamflow routing models. A significant share of past work has been conducted to estimate conveyance losses on the Arkansas River in Colorado. Lucky and Livingston (1975) first developed a streamflow routing model which accounted for channel storage, bank storage, inadvertent diversions and travel time. The model was developed for routing reservoir releases from Twin Lakes, near Leadville Colorado, to the Colorado Canal, below Pueblo Colorado, along a 175 mile portion of the Arkansas River. Inadvertent diversions were estimated based on an empirical formulation of conditions which were observed on the river. Each of the time dependent phenomena; stream routing (channel storage and travel time) and bank storage were calculated using theoretical formulas linked in an iterative solution technique. Although the model had difficulty predicting hydrographs during periods when flows in the river were changing rapidly, during relatively stable periods of flow the model produced hydrographs which were in good accord with observed hydrographs. The significance of this investigation is best seen by a review of the history of conveyance loss studies completed for this reach of the Arkansas River.

Shortly after the completion of the Twin Lakes transmountain diversion project, conveyance losses were investigated by Hinderlider (1938). Hinderlider, then State Engineer for Colorado, wrote of the difficulty in accurately determining conveyance losses on a river which gains flow such as the Arkansas:

> "Due to the importance of the problem under consideration the State Engineer has made several attempts to determine the loss of reservoir water turned into a natural stream. With but few exceptions, the results have not been very conclusive, and in fact have been conflicting, due to two principle reasons. First, practically all streams flowing through an irrigated region show and increase or gain in natural flow progressively from their upper to the lower reaches thereof, due to return flow. Hence, it is impossible to determine what losses, if any, occur due to increases in the stages of flow, such as result where the natural flow is augmented by reservoir releases. The only thing disclosed by such measurements from a stream source to its terminus is the net difference between gross gain and gross loss, which as stated, usually shows up in the form of a net gain."

Lacey (1941) studied seven reservoir releases made during 1939-40 to better ascertain conveyance losses on the same reach of the Arkansas. After completing the investigation, in which all reasonable care was taken to monitor the river and its diversions, Lacey concluded:

> "In my opinion, exact determinations as to the loss in transit to the reservoir head in progression are impossible because of the many influencing factors encountered which are beyond control. There are too many variables present in the situation, which tend to obscure the graphic record and make objective conclusions difficult."

As a result of these studies, reservoir releases to the Arkansas from Twin Lakes Reservoir were charged a loss of 0.07

percent per river mile.

Another effort was made to define conveyance losses on the Arkansas River when Wright Water Engineers (1970) conducted a study while under contract with the Colorado State Engineers office. This study proposed the incremental loss approach be used to assess conveyance losses on a 175 mile reach from Leadville to the Colorado Canal below Pueblo. The study considered the effect of 30 reservoir releases performed from 1966-1970. A methodology, based on nomographs, was presented for the estimation of conveyance losses established on three categories of losses. The three categories of losses considered in this investigation were: evaporation, inadvertent diversions, and bank storage. Although the methods used for this investigation were little more sophisticated than earlier work on the river, the study was significant in that it proposed varying the conveyance loss assessed in relation to the magnitude of the reservoir release.

In 1973, Livingston performed another conveyance loss study on the same 175 mile reach of the Arkansas River. This investigation was also based on the incremental approach and apportioned chargeable losses into four categories: evaporation, inadvertent diversions, bank storage, and channel storage. Livingston determined travel time of reservoir releases varied from 29 to 69 hours, depending on the natural flow conditions which existed in the river. Conveyance losses were found to vary between 6 and 28 percent of the rate of the reservoir release, depending on the rate and duration of the reservoir release, and the time of year the release is made. Although this study was similar in approach to the one conducted by Wright Water Engineers, it differed in regard to the approach taken to quantify bank storage. Estimates of bank storage were made based on a theoretical expression describing the physical phenomena which occurs rather than an empirical formulation. This study formed the basis for the development of the computer model Lucky and Livingston first used in 1975 to estimate conveyance losses which was described earlier.

Livingston continued to investigate conveyance losses on the Arkansas River. A 142-mile reach of the Arkansas River from Pueblo Reservoir to John Martin Reservoir in Colorado was investigated by Livingston in 1978 (Livingston, 1978). Construction of Pueblo Reservoir in the early 1970's and a proposed winter-water storage program prompted the study. The upper portion of this reach (approximately 25 miles) encompasses a portion of the study reach of the Arkansas River considered in previous investigations. For this investigation Livingston used a modified version of the hydrologic model developed in 1975 to study the upper reach of the Arkansas River.

In contrast to the upper reach, this reach generally traverses a broad flat flood plain, and the river is characteristically more expansive and slower moving. For this reason, evaporation was expected to account for a greater proportion of conveyance loss and the original source code for the model was revised to include a routine to estimate river evaporation as a function of incremental changes in river width. A stochastic algorithm was used. The algorithm is based on an empirical formulation relating river width to discharge. Evaporation associated with reservoir releases, and coinciding increases in river stage and average width, is subsequently estimated using pan evaporation data.

Other parts of the original model were omitted for this study. The portion of the original 1975 model which dealt with inadvertent diversions was omitted in the revision because observations of the diversion structures along the reach suggested this would not be a significant source of loss. In addition, travel time was estimated from historical data rather than using theoretical relationships as had been done for the 1975 study. This empirical approach was taken because accurate estimates of travel times over a range of variable natural flow conditions was not possible using the theoretical methods included in the original model.

As in the case of earlier studies, conveyance losses were determined to vary significantly with the size and duration of the reservoir release, as well as seasonally. Modeling also showed about 80 percent of all conveyance loss was attributed to bank storage. The remaining 20 percent of loss was equally attributed to channel storage and river evaporation.

The study conducted by Livingston on the lower reach of the Arkansas River demonstrates that not all factors considered in a conveyance loss model need be coded for execution on a computer. The individual characteristics of the hydrologic system under consideration and the specific goals of a study may deem alternative approaches more useful. Wright Water Engineers (1982) estimated conveyance losses for an 80-mile reach of the Fryingpan, Roaring Fork and Colorado Rivers between Ruedi Reservoir and Parachute, Colorado. The study was completed for reservoir releases from Ruedi Reservoir to the Colony Shale Oil Project on the western slope. Theoretical formulations were used to calculate losses associated with bank storage, channel storage and inadvertent diversions. Evapotranspiration was estimated using an empirical formulation. All of the calculations were completed by "hand" because the study was limited to investigating losses associated with three unique sets of flow conditions. An understanding of conveyance losses for the three conditions evaluated was believed sufficient to estimate losses for the range of actual field conditions expected.

To this point, the use of hydrologic models to investigate conveyance losses had been limited to studies completed for rivers in Colorado. In the mid-1980's two studies quantifying the losses associated with reservoir releases during drought conditions were performed by Carswell and Hart (1985) and Jordan and Hart (1985) for two rivers in east-central Kansas. Carswell and Hart investigated travel times and conveyance losses in the Cottonwood River, a tributary to the Nesho River. Jordan and Hart completed a similar study of two reaches of the Nesho River which are located above and below its confluence with the Cottonwood River near John Redmond Reservoir. The two studies were essentially identical in approach. Both studies were completed using a streamflow routing model developed by Land (1977) called J349.

Each study considered two scenarios of antecedent base flow: "severe-drought" and "less-severe-drought". For the severe-drought scenario, it was assumed that only the downstream water use requirement would be released. The less-severe-drought scenario examined minimal base flows. Conveyance losses were estimated using an incremental approach in which bank storage and evaporation were the only two factors considered. All of the reaches studied were considered to be losing flow.

Evaporation from the river was calculated on the basis of pan-evaporation data applied to a statistical relationship found between discharge and mean stream width. Evaporation accounted for a large percentage of all conveyance losses. In the case of the Nesho River study, evaporation accounted for between 32 and 65 percent of all conveyance losses for the severe-drought scenario and from 63 to 79 percent of all loss for the less-severe-drought scenario. The higher evaporation losses determined for the less-severe-drought scenario were justified by the increase in river widths caused by the base flow.

A study conducted by Kuhn (1988), was also completed using the J349 model developed by Land (1977). Kuhn quantified conveyance losses for return flows from a treatment plant which discharges into Fountain Creek near Colorado Springs, Colorado. A portion of Colorado Spring's water supply is obtained from transmountain diversions. Colorado water law provides the owner of water, which is imported from another drainage basin, the right to reuse, sell, lease, exchange or otherwise dispose of such water. The study was prompted by the City's desire to completely use its transmountain diversion water by means of water exchanges with downstream users. Water discharged from the treatment plant minus conveyance losses determined from this study are exchanged for water stored in reservoirs upstream of the city.

Factors considered significant to conveyance losses were bank storage, channel storage, and evaporation. Kuhn used Land's model in conjunction with the method first described by Livingston (1978) to determine evaporation where evaporation was calculated on the basis of pan-evaporation data applied to a statistical relationship found between discharge and mean stream width. Stream evaporation losses were computed based on streamflows estimated from the model simulations.

Kuhn developed an elaborate method for estimating conveyance losses on a daily basis. Real-time data for all measurable surface flows was required. The method involved a two stage computational process. First a "stream-segment computation" was completed for each of four stream segments defined in the study. "Stream-segment computations" were completed to estimate the gain or loss in natural streamflow. In the second stage, computations were completed to estimate conveyance losses for sub-reaches defined within each stream segment. A total of 14 sub-reaches were defined throughout the four stream segments.

Using model simulations, Kuhn developed tabulations of bank storage loss for 10 to 12 natural flow conditions (varying from 0 to 1,000 cubic feet per second) for each of 10 return flow rates (varying between 1 and 100 cubic feet per second). Separate tabulations were developed for each of the 14 sub-reaches defined. Model simulations were also completed to determine an adjustment factor applied to the one day bank storage loss to account for gains or losses in natural stream flow in each sub-reach due to factors such as groundwater withdrawals and tributary inflow. Finally, model simulations for selected return flow and natural streamflow conditions were also used to develop tabulations of the percentage of bank storage returned to the stream for varying recovery periods. Recovery periods ranging from one to 180 days were simulated.

An example application of the methodology developed resulted in an average conveyance loss of 3.1 percent. In comparison, the loss assessed to the city under its interim exchange agreement was 12.3 percent.

The studies discussed above showed that the benefit of using a hydrologic model is that steady flow conditions are not required to determine conveyance losses. Furthermore, once the model has been calibrated it may be used to simulate conveyance losses over a range of combinations of natural and augmentation flows. Two computer models, used in the studies discussed, were considered for use in this investigation. These models are the J349 model (Land, 1977) and the model first developed by Lucky and Livingston (1975) and later revised by Livingston (1978). The algorithms used in these models address the temporal effects of bank storage, channel storage and streamflow routing in different ways. To ascertain which model was most appropriate for this investigation an evaluation of the methods used in each model was also performed as part of the literature review.

THEORETICAL METHODS

The development of hydrologic models used to estimate conveyance losses is predicated on streamflow routing models. These hydrologic models, known as stream-aquifer models, provide a capability to simulate streamflow and the interaction of streamflow with an alluvial aquifer. The distinction between the models used in the model studies discussed in the previous section originates in the numerical methods and solution techniques used to describe this interaction. A review of the more popular theoretical formulations and numerical solution techniques which are contained in the literature follows.

Pinder and Sauer (1971) described a technique for modifying a floodwave due to bank storage by solving one dimensional open channel unsteady flow equations and a two dimensional transient groundwater flow equation simultaneously. To couple the two equations, Darcy's law was used to describe the movement of water between the stream and the aquifer. Finite difference approximations of the flow equations and Darcy's law were solved simultaneously in an iterative procedure. The discretized form of the open channel equations were first solved from initial conditions describing the depth and velocity of flow in the channel. Boundary conditions were based on a prismatic channel. The groundwater equations were then solved based on calculated stream elevations. After the initial conditions were satisfied, the transient solution proceeded by repetitively solving the equations for the stream and aquifer systems until the exchange of flow between the two systems in successive calculations, as described by Darcy's law, was within a predetermined error tolerance. Once the error tolerance criteria was satisfied, the simulation proceeded to the next time step. The open channel equations were solved explicitly whereas the groundwater flow equations were solved implicitly.

Zitta and Wiggert (1971) coupled the open channel flow equations (St. Venant equations) with an equation governing one-dimensional unsteady flow in an unconfined aquifer known as the Boussinesq equation. The Boussinesq equation was solved for the height of the phreatic surface in the aquifer using an explicit finite difference method. Once the height of the phreatic surface was determined, the lateral inflow per unit length was equated to the change in volume associated with the change in storage between successive time steps.

Both of the methods described to this point were limited by the applications which could be made of them. Each method was based on the full dynamic equations describing open channel flow and was used to evaluate the effects of bank storage on a flood wave. To allow for the numerical solution, ideal channels were considered. Less elaborate techniques were also being used to describe the interaction of streamflow with an alluvial aquifer in natural streams.

Jennings and Sauer (1972) outline the basic methods for the solution of the open channel flow. The methods described are categorized as being "complete" or "approximate". Complete methods of flow routing are based on the solution of the St. Venant Equations. The methods vary with regard to the degree the momentum equation is simplified for solution. Most commonly, complete solution methods are based on the diffusion equation. In the diffusion equation an assumption is made that the friction slope is equal to the slope of the water surface. In other words, the principle of conservation of momentum is simplified to a statement of nonuniform flow. A corollary to the diffusion wave equation is the diffusion analogy. The diffusion analogy is an analytical solution to the diffusion wave equation. In addition to flow routing methods based upon the St. Venant equations, the unit response method, included among the approximate methods discussed, is a convolution method where a unit response at a point downstream is convoluted with lagged inflow values of the upstream hydrograph to derive a downstream response.

Sauer (1973) gives a good description of one method based upon the unit response principle. The method is analogous to unit hydrograph theory which predicts runoff from rainfall excess, and the same principles of linearity and superposition are assumed. The unit response method is premised on the theory that an input flow of unit rate and duration will result in a specific flow response downstream. The downstream response is subject to the physical characteristics of the channel. The response function proposed by Sauer is derived by a hydrograph translation technique where a triangular pulse is routed through reservoir-type storage to account for channel storage between the points of inflow and outflow to the system being modeled. The storage routing technique used is linear about a single discharge and results in an approximation of the unit response of given duration for the channel described by a storage coefficient. This storage coefficient is defined by Sauer as the slope of the storage-discharge relation in the routing reach. The unit response for one duration is then transformed to a unit response of another duration by a summation curve technique analogous to the S-curve technique commonly used with unit hydrograph theory. The difficulty with this method is that all inflow to the system is linearized about a single discharge. This results in distortions in the output hydrograph caused by the inability of the model to account for changes in wave celerity and damping with varying discharge. Additionally, this method, as in the case of the unit hydrograph, has little physical significance. The model developed by Lucky and Livingston uses a similar method to that described by Sauer to route streamflows.

The technique used to model streamflow in the J349 model is based upon the work of Keefer and McQuivey (1974). In this work, an equation based on the diffusion analogy is derived which may be convolved with an upstream hydrograph input to determine the response of the stream channel in space and time. This methodology is very similar to the unit response function proposed by Sauer (1973) described above, in that it may be applied based upon the unit response principle. The advantage to this method is that terms of the diffusion analogy equation have some physical basis. Keefer and McQivey also describe a multiple linearization technique whereby the range of discharge is broken up such that the nonlinearities are minimized. Each range of discharge then has its own celerity and dispersion coefficients.

The algorithm used to compute the bank storage discharge used in the J349 model is based on work done by Hall and Moench (1972) in which a solution to a one-dimensional confined flow equation is obtained using the convolution relation or superposition theorem. Darcy's Law is applied to obtain discharge into or out of the stream. Boundary conditions available in the model are: semi-infinite and infinite aquifer with or without a semi-impervious stream bank.

The algorithm used to calculate bank storage in the model developed by Luckey and Livingston is given in their paper (Luckey and Livingston, 1975). The original source for the algorithm used could not be located to evaluate the derivation of the algorithm. However, the solution is based on a formula which approximates the instantaneous bank flow from an unconfined aquifer due to an instantaneous change in head. Lucky and Livingston state the formula assumes the stream fully penetrates an infinite aquifer.

In conclusion, two models were considered for use in this investigation: the model originally developed by Livingston (1975) and then later revised by Lucky and Livingston (1978); and the J349 model presented by Land (1977). Although evaporation is directly accounted for in the model developed by Lucky and Livingston, the stream flow routing component to the model is a weaker algorithm than the routing component used in the J349 model. In addition the bank storage component used in the J349 model has three boundary conditions available whereas the model developed by Lucky and Livingston assumes only an infinite aquifer. Previous investigations of Wyoming streams, completed by Pahl (1985) and Hanlin (1988) established the majority of conveyance loss was due to bank storage and an associated reduction of groundwater inflow. For this reason the J349 model was selected for use in this investigation.

A copy of the J349 model used for this study was obtained from Mr. Gerhard Kuhn with the U.S. Geological Survey. The following chapter presents a description of each of the components of the J349 model and a summary of its operation.

CHAPTER III

J349 STREAMFLOW ROUTING MODEL

A general description of the J349 Model is presented in this chapter. Subjects to be discussed include: (1) Model Components and (2) Model Operation.

INTRODUCTION

The J349 model was published as a United States Geological Survey Computer Contribution (Land, 1977). The source code for the model is written in Fortran IV and the version obtained contains 27 subroutines and approximately 2000 lines of code. The model was updated shortly after its initial release to incorporate the optional multi-linear diffusion analogy technique described by Keefer and McQuivey (1974).

The model consists of three hydrologic components. A streamflow routing component which is based on a one-dimensional diffusion analogy and convolution technique. A bank storage component based on analytical equations for abrupt change in stream stage. Bank storage discharge is computed using a convolution technique. And thirdly, a stream depletion component which allows for diversions, and depletions from well pumping.

Although the theoretical basis for the stream flow routing and bank storage components of this model are generally described in Chapter II, the following section gives a listing of the equations and model parameters incorporated in the model as presented by Land (1977).

MODEL COMPONENTS

CHANNEL HYDRAULICS. The following formulation is derived from the diffusion analogy as presented by Keefer (1974) and Keefer and McQuivey (1974). For an instantaneous unit flow input, the stream system will respond in time and space with a discharge given by:

$$q(x,t) = \frac{1}{\sqrt{4\pi K}} \cdot \frac{x}{t^{3/2}} \cdot \exp\left(-\frac{(C_o t - x)^2}{4Kt}\right)$$

where
q = *unit outflow (fraction of input unit)* (L^3/T),
x = *distance downstream input* (L),
t = *time since input* (T),
π = *constant,*
K = *wave dispersion coefficient* (L^2/T), *and*
C_o = *wave celerity* (L/T).

The wave dispersion coefficient (K) and wave celerity (C_o) can be approximated by:

$$K = \frac{Q_o}{2\, S_o\, W_o} \qquad (L^2/T)$$

where
Q_o = *selected baseline discharge* (L^3/T),
S_o = *channel slope* (L/L), *and*
W_o = *average channel width at baseline discharge* (L).

and

$$C_o = \frac{1}{W_o} \frac{dQ_o}{dY} \qquad (L/T)$$

where $\dfrac{dQ_o}{dY}$ = *slope of stage-discharge rating* (L^2/T).

<u>AQUIFER HYDRAULICS; STREAM-AQUIFER BOUNDARY CONDITIONS</u>. The method used to compute bank storage discharge is given in Hall and Moench (1972). The method also uses a convolution technique to obtain a bank storage discharge hydrograph. Land (1977) terms this hydrograph as a "composite hydrograph" because it represents the average response of the aquifer along the stream reach considered. Analytical equations for three boundary conditions are available to determine the hydraulic gradient at the interface between the stream and aquifer for a unit change in stream stage. The system response in terms of the hydraulic gradient at the stream-aquifer boundary to a unit change in stage for each boundary condition is:

semi-infinite aquifer

$$i(t) = -\frac{1}{\sqrt{\pi t T/S}},$$

finite aquifer

$$i(t) = -\frac{2}{\gamma}\sum_{n=1}^{\infty} \exp\,(-\,c^2 tT/S)$$

and

semi-infinite aquifer with a permeable confining bed

$$i(t) = -\frac{1}{a}\exp\left(\frac{tT/S}{a^2}\right) erfc\left(\frac{(tT/S)^{1/2}}{a}\right)$$

where
$i(t)$ = hydraulic gradient at time (t),
T = transmissivity (L^2/T),
S = storativity (dimensionless),
γ = width of aquifer, one side (L),
$c = (2n-1)\ 2\pi\gamma$,
n = number in summation loop,
$a = m`k/k`$ = retardation coefficient (L),
$m`$ = thickness of confining bed,
$k`$ = hydraulic conductivity of confining bed, and
k = hydraulic conductivity of aquifer.

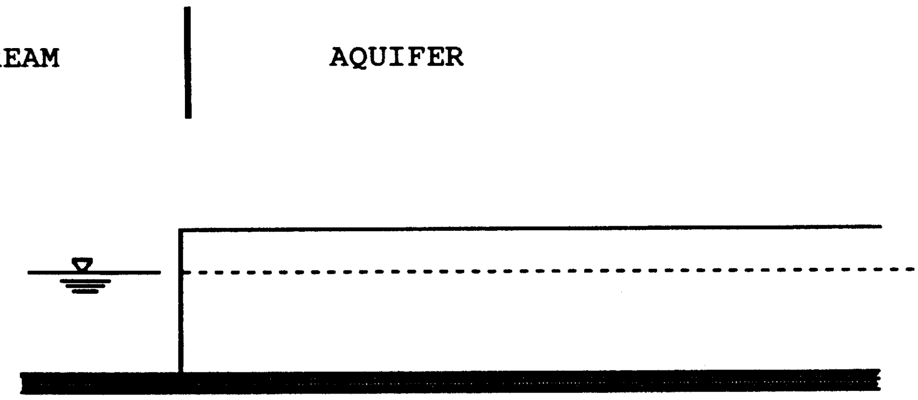
Note the equation for a semi-infinite aquifer with a permeable confining bed presented in Land (1977) is erroneous. The equation above is revised to conform with the original work of Hall and Moench. The stream-aquifer boundary conditions available in the model are illustrated in Figure 1.

DIVERSIONS AND DEPLETIONS FROM WELLS. An analytical expression presented by Glover and Balmer (1954) which computes discharge as a function of time from a stream to an aquifer as a result of pumping wells is used. The expression for a semi-infinite aquifer incorporated into the J349 model is:

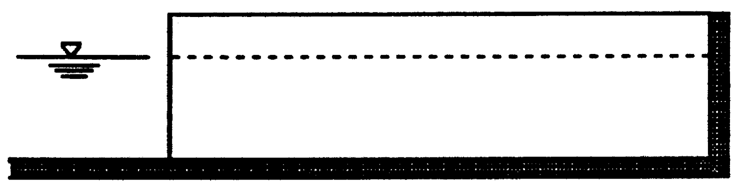
$$q(t) = Q_o\left(1 - erfc\left(\frac{x}{\sqrt{4tT/S}}\right)\right)$$

where
x = distance of well to stream (L),
Q_o = well pumping rate (L^3/T), and
$q(t)$ = stream depletion rate (L^3/T).

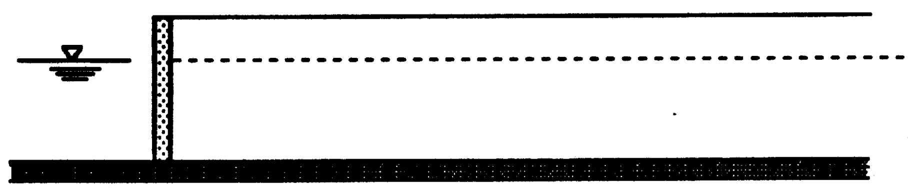
All of the pumped water will eventually reduce the streamflow by an equal amount. Wells within 10 feet of the stream are treated as direct diversions (i.e. q(t) equals Q_o).



Semi-infinite aquifer.



Finite aquifer.



Semi-infinite aquifer with a permeable confining bed separating the stream and the aquifer.

Figure 1. Stream-aquifer boundary conditions

MODEL OPERATION

GENERAL. A complete description of the operation of the model is presented in Land (1977) and is not repeated here. However, aspects of the model and the chronology of its operation which are important to interpreting the results of model simulations in the application for which it is used in this study are discussed.

The model may be operated in one of two ways depending on the objective of the simulation. In one mode of operation, which Land terms the "NON-ROUTE option", the model determines a bank storage discharge hydrograph based upon known hydrographs at both ends of the stream reach being modeled. Alternatively, the model simulates both a bank discharge hydrograph and a downstream discharge hydrograph based on a known upstream hydrograph. Land terms this procedure the "ROUTE option". The "ROUTE option" is was used for this investigation of conveyance loss because this option will allow model calibration of stream and aquifer parameters based on a known downstream hydrograph.

An input file for the model contains approximately 25 "cards" for each reach simulated. Each "card" may consist of one or more lines of data. A description of each card, in sequence, is presented in Appendix A. Input to the model consists of three basic categories: streamflow data, reach parameters and aquifer properties.

Streamflow data input to the model consists of an upstream hydrograph and stage-discharge rating tables for both ends of each reach modeled. Diversions and well depletions are also inputs included in this category. Hydrographs input to the J349 model are limited to 399 time steps with the current dimension statements in the program. This equates to a study period of approximately 33 days for a two hour time step. In addition, the total number of diversions and well depletions for each reach are limited to 25. A constant rate of diversion may last all or any portion of the study period. However, if a varying diversion rate is simulated, each change in rate accounts for one of the total 25 diversions allowed.

Reach parameters include reach length, wave celerity, and wave dispersion. The length of the reach is quantified with two input parameters; the actual stream length along the thalweg of the stream and the distance as measured along the center of the stream valley. In this way, an accounting for channel sinuosity is made. Wave celerity is a measure of the speed of propagation of a water wave through the stream reach. Wave dispersion is a coefficient which describes the amount of attenuation of a water wave within the stream reach. Estimates of wave celerity and dispersion are determined from the equations presented previously.

Aquifer properties input to the model are storativity, transmissivity, and average aquifer width as measured from the stream to a boundary. In the event geologic information on the limits of the alluvium is not available, the boundary may be generally defined by the limits of the stream valley. The storativity and transmissivity are parameters used to describe the volume and rate of flow in an aquifer. The storativity of the aquifer is the volume storage, given or taken by the aquifer, per unit area per unit change in head. Transmissivity is the rate at which water flows through the full saturated thickness of an aquifer having unit width under a hydraulic gradient of one.

The operational sequence used in the model is also significant to interpreting results of model simulations. After the data from the input file is read, the model calculates an instantaneous unit response function and discretizes the response function for the time step used in the simulation. Next the upstream hydrograph is convolved with the response function to compute the downstream hydrograph. The downstream hydrograph is then adjusted for a constant base flow, diversions, and well depletions. After the downstream hydrograph has been corrected for these three components of the flow in the system, an iterative process is begun to estimate losses to bank storage. Bank storage is computed using a mean-stage hydrograph for the reach. The mean-stage hydrograph for the reach is computed by dividing the reach in half. A stage hydrograph is computed for the upper half of the reach using the stage-discharge rating input for the upstream boundary of the reach. Similarly, a stage hydrograph for the lower half of the reach is determined from the downstream stage-discharge rating input. These two stage hydrographs are then averaged. Bank storage is then computed using a convolution technique. For each time step, the hydraulic gradient is multiplied by the transmissivity and the change in mean stage for the reach. This value is then doubled and multiplied by the length of alluvium input for the reach to yield the amount of water taken into or released from bank storage. The downstream hydrograph initially computed is adjusted by the bank storage hydrograph and the process is repeated until closure on bank storage discharge, within a specified tolerance, is achieved.

Once balanced hydrographs of stream discharge and bank storage are computed for a reach, the results of the simulation are output. An example of an output file from the model is presented in Appendix B.

MODEL OPTIONS. The J349 model has a number of optional capabilities which allow the user to tailor the model operation to an individual hydrologic system and database.

The model has an option to route streamflow with a multi-linear diffusion analogy technique presented by Keefer and McQuivey (1974). Tables of wave celerity verses discharge and wave dispersion verses discharge are added inputs. The option is initiated or disabled with a logical variable included in the input file.

An option is available to add a constant base flow to the upstream hydrograph input. The model does not specifically route or compute base flow as a separate component of streamflow.

The model uses the stage-discharge rating table input to compute a stage hydrograph for bank storage calculations. An option is available to apply daily shifts to adjust the actual rating to a rating table. Shifts may be applied to the rating tables input for both the upstream and downstream stations for each reach.

Another option in the model allows a fraction of the bank storage calculated by the model to be retained in the aquifer. This option may be used to simulate losses attributed to a soil moisture deficiency or transpiration. Model input consists of the decimal fraction of the volume bank storage to be reserved from the water budget accounting of the stream-aquifer system.

<u>MODIFICATIONS MADE TO THE MODEL</u>. A number of modifications to the source code of the version of the model received from Mr. Kuhn were made by others. Many minor revisions, which are noted in the source code, were done to allow the compilation on different computer equipment and operation systems. Two substantial changes, made by Mr. Kuhn, affect results of model simulations. Mr. Kuhn incorporated new input and output statements into the source code which allow an observed downstream hydrograph to be included in the input for each reach and then subsequently tabulated in the output file to readily allow a comparison to the computed

downstream hydrograph. The observed downstream hydrograph input is not used in any of the model calculations. Mr. Kuhn also modified the source code which computes the response function for the stream aquifer system. The revision allows the response function to decay 18.5 half-lives as compared to 4.5 half-lives incorporated in the original model. Further discussion of the effect of this revision on the source code follows in later sections of this paper.

In addition to the modifications to the source code noted above, further revisions were made in conjunction with this investigation. The source code was compiled with MicroSoft Fortran so that the model may now be executed on a personal computer. A number of minor changes to the source code were made to overcome compilation errors. For example, an entry statement and the associated source code was assimilated into a separate subroutine to avoid a recursion error. A new subroutine was also written to calculate the complimentary error function to avoid a call to a library function which would not be available on a personal computer.

The program was compiled in two separate blocks, which were subsequently linked to form an executable file approximately 193 kilobytes in size. A math coprocessor is required and run time on an AT type personal computer varies from 15 seconds to a few minutes depending on the complexity of the simulation.

<u>LIMITATIONS OF THE MODEL</u>. Discussion in the previous

sections of this chapter and in Chapter II provide some indication of the basic assumptions and possible sources of error in model simulations. The fundamental assumptions which form the basis for the J349 model are presented in the following paragraphs.

The convolution technique used in the program assumes the hydrologic system is linear. The validity of this assumption is not believed to be a serious limitation in the use of this model to investigate conveyance losses. In great part, the inappropriateness of this assumption may be tempered through the use of the multi-linear streamflow routing option. The hydrologic system being modeled may also be divided into smaller segments which better approximate a linear system.

The assumption is made that the stream fully penetrates the aquifer in the bank-storage algorithm used in the program. This assumption is believed to be valid provided the change in stream stage is not greater than about 1.5 times the original stage (Moench et al, 1974). The limitations of this assumption will be discussed in later sections as it applies to the use of this model for the present study.

The stream is assumed to be in the center of the aquifer. This assumption is not believed to pose any significant limitations to the application of the model to this study because the model allows for the calibration of input data to compensate for variations in the hydrologic system. Land (1977) suggests the alluvial length may be reduced to account for situations where this assumption is not legitimate.

The most alarming assumption implied with the use of the model is that the ground water level in the aquifer is flat when the simulation is initiated. Thus the stream and aquifer are presumed to be initially in equilibrium. In reality, this condition is not likely to exist in a stream-aquifer system, particularly over the short periods of time considered in this investigation.

The methodology employed to estimate conveyance loss to bank storage using the J349 Model is presented in the following chapter. The methodology used to estimate loss by evaporation is also introduced.

CHAPTER IV

METHODOLOGY

The methods used to estimate conveyance losses using the J349 model are described in this chapter. Topics to be discussed include: (1) Selection of Study Areas; (2) Data Collection; (3) Discussion of Study Areas; and (4) Methods of Analysis.

SELECTION OF STUDY AREAS

This is the third investigation in a series of studies funded by the Wyoming Water Resources Center for the State Engineer's Office to quantify conveyance losses on selected streams in Wyoming. In conjunction with the two previous investigations (Pahl, 1985 and Hanlin, 1988), a substantial database of measured streamflows suitable for the assessment of conveyance losses was acquired. A list of the stream reaches included in the existing database is presented in Table I. Flow data for some of the stream reaches listed in Table I was compiled from historical records; surface flow data for other stream segments listed was collected during the two previous investigations. The outcomes of the previous conveyance loss investigations were reviewed to evaluate the suitability of each stream reach to be studied with the J349 model.

TABLE I. STREAM REACHES CONSIDERED FOR STUDY

Water Division 1

North Platte	-	Guernsey Reservoir to the Tri-State Dam.
Laramie River	-	Wheatland Reservoir No.'s 2 and 3 to Confluence with Sybille Creek.

Water Division 2

Piney Creek	-	Lake DeSmet to Clear Creek Confluence.
Clear Creek	-	Confluence with Piney Creek to Carlock Ranch.

Water Division 4

Bear River	-	from Randolph, Utah to Pixley Dam and from Cokeville, Wyoming to the Wyoming-Idaho Border.
Horse Creek	-	from Hunter Ranch on the Little Horse Creek to a point 26 miles downstream where the La Grange Canal diverts, near the Johnson Ranch.
Green River	-	from Fontenelle Reservoir to the City of Green River.

A number of criteria were used to select the stream reaches studied. Stream reaches which were shown to have an incomplete or perverse database were first excluded from consideration. For example, the data for the Laramie River was shown to be corrupted from ungauged runoff from several precipitation events and large shifts in the rating curve used for the inflow record (Pahl, 1985). The results of the previous investigations were also reviewed to determine which stream segments had conveyance losses which were attributed primarily to bank storage and a reduction of groundwater inflow because these factors are best addressed in the J349 model.

Based on the criteria discussed above, two stream reaches with existing databases were selected. The stream reaches selected were Piney Creek and Clear Creek. Both streams are located in Water Division 2 near the City of Buffalo. The segment of Clear Creek considered in this investigation begins at its confluence with Piney Creek. The reach of Piney Creek studied ends at this confluence. For simplicity and because the two stream segments are continuous, the two streams are collectively referred to as the Piney Creek study area in the remainder of this paper. The reach of the Green River between Fontenelle Reservoir and the City of Green River was also modeled for this study. The Green River study reach was included in this paper because the collection of data at that location, as part of the overall objectives of the research being funded, presented the opportunity to tailor the data acquisition phase of this model study to the requirements of the J349 model. The locations of the Piney Creek and Green River study areas are shown in Figure 2.

DATA COLLECTION

<u>PINEY CREEK STUDY AREA</u>. The available database for the Piney Creek study area was obtained from Todd Hanlin. The Piney Creek data consisted of flow records and analyses developed from a monitoring network of stream gauges maintained

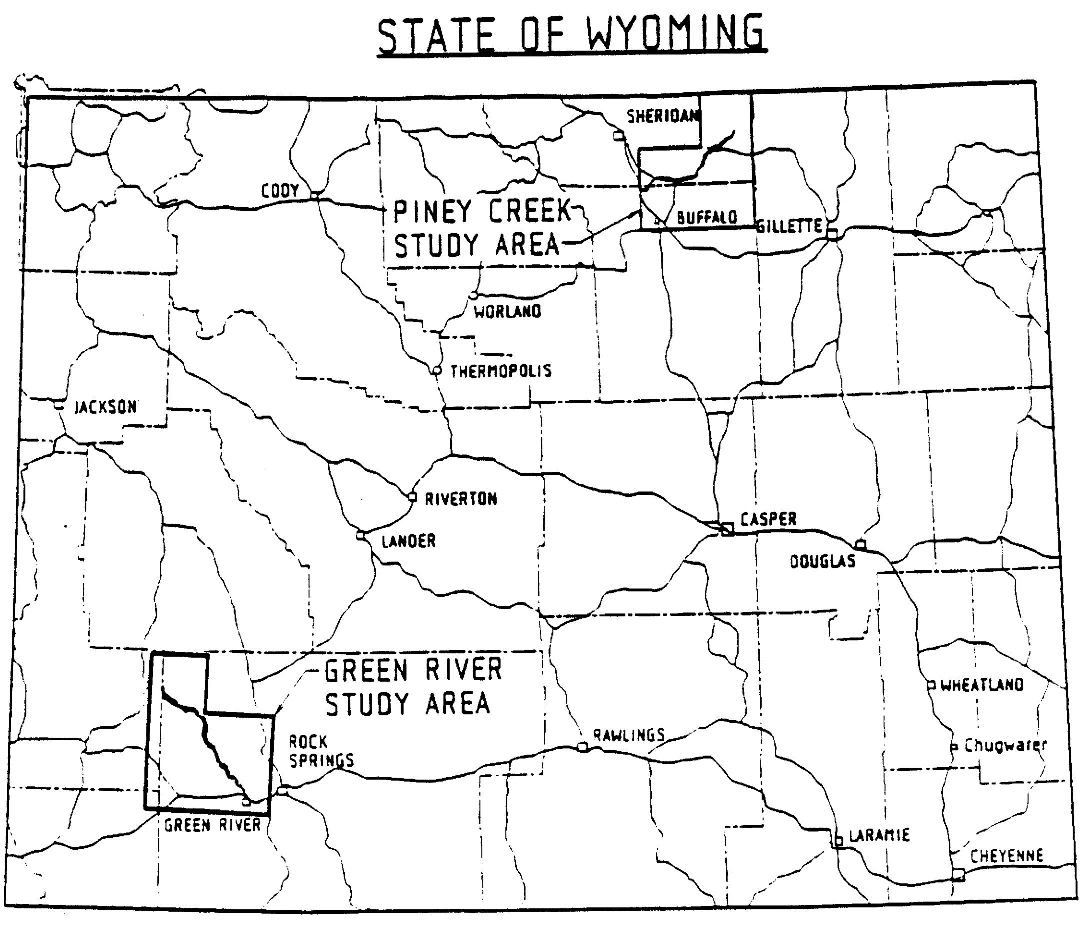


Figure 2. Location Map

during the summers of 1984 and 1985. Continuous stage records, flow measurements and analyses derived from a monitoring network established on Clear Creek during the summers of 1985 and 1986 was also obtained. In addition,during the summer of 1989 a further field investigation was completed to acquire information on channel geometry at selected locations along both streams. During this field investigation a level was used to obtain differential elevations along measured cross sections of each stream.

GREEN RIVER STUDY AREA. At the Green River study site, a stream gauge network was established at operative locations of surface flow into and out of the river. The stream gauge network incorporated an existing USGS gauging station. Records for this station were obtained from the Bureau of Reclamation. Continuous stage recorders were installed at other principal locations in the system. Stage-discharge ratings for these locations were developed from flow measurements taken using Marsh McBirney and Price AA current meters. Flow measurements acquired for stage record stations established on the Green River were taken from the deck of highway bridges near each station. A cable and weight assembly was used with a Price AA current meter because of the high water depths in portions of the river channel. Flow measurements were taken from the downstream side of the bridges. Some small secondary flows to and from the system were not continuously monitored because they remained relatively constant. Flow measurements at these locations were taken using a Marsh McBirney current meter or a 3 inch parshall flume.

DISCUSSION OF STUDY AREAS

PINEY CREEK STUDY AREA. The study area incorporates portions of both Piney and Clear Creeks located near the City of Buffalo, Wyoming (Figure 3). Both streams are considered to be perennial and have their headwaters in the Big Horn Mountains. Lake DeSmet, which receives water diverted from both streams, serves as an off-channel storage reservoir for irrigation water. Storage water from Lake DeSmet is released to Piney Creek during periods of peak irrigation demand.

The reach of Piney Creek considered in this investigation begins at the point where water from Lake DeSmet is discharged to Piney Creek and ends at the confluence with Clear Creek near Ucross, Wyoming (Figure 3). This reach of the stream meanders approximately 22 miles through a narrow alluvial valley. The alluvial valley varies in width from about 2500 to 4000 feet. The reach has Alfalfa hay and native grasses which are grown using flood irrigation practices.

The Piney Creek study reach is bounded by a gauging station installed below where Lake DeSmet water enters Piney Creek and the State Engineer's gauging station near Ucross (06323500). Data for nine diversions and one tributary inflow

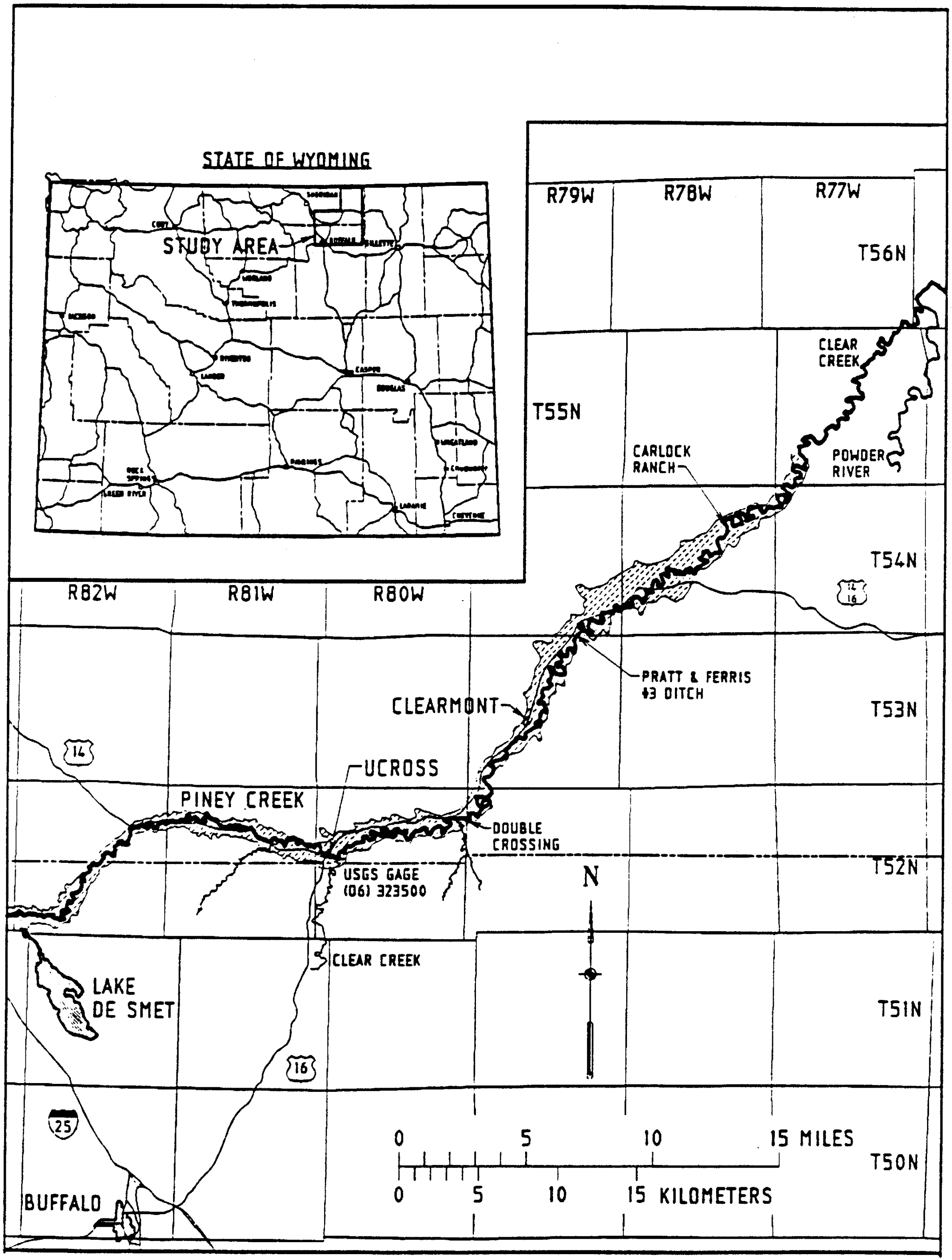


Figure 3. Piney Creek Study Area

located along this reach were also collected. The instrumentation and methods of data collection used for this reach are described in detail by Pahl (1985).

The segment of Clear Creek included in this investigation begins from its confluence with Piney Creek near Ucross, and ends approximately five miles east of the town of Leiter, Wyoming at the Carlock Ranch (Figure 3). The stream travels a total of approximately 50 miles in this segment. The character of this segment of Clear Creek is very similar to Piney Creek. Clear Creek is also a meandering stream, however, the alluvial valley which it traverses is somewhat wider than exists along Piney Creek. The valley ranges from approximately 2500 feet wide to in excess of 10,000 feet along short lengths of the stream. Flood irrigation is predominantly practiced along Clear Creek, although some sprinkler irrigation facilities were observed.

The database for Clear Creek includes information from four gauging stations installed on the stream. A stream gauging station was installed on Clear Creek above its confluence with Piney Creek. Three additional gauging stations installed by Hanlin (1988) allow this segment of Clear Creek to be divided into three reaches. Stream gauge instrumentation was installed near where a small tributary called Double Crossing Creek enters Clear Creek. Another gauging station was constructed below the diversion structure for the Pratt & Ferris #3 Ditch. A third gauging station was installed near the Carlock Ranch. The locations of these gaging stations are shown in Figure 3.

In addition to the data available from the instrumentation installed on the stream, the database for Clear Creek includes records from other locations in a monitoring network designed to account for tributary inflow to, and diversions from, Clear Creek. Data collected for five diversions and one location of tributary inflow was obtained. A detailed description of the instrumentation and methods of data collection completed for the Clear Creek system is presented by Hanlin (1988).

Surface flow data for the Piney Creek study area was collected during the years 1984 through 1986. In total, four stream reaches are defined in order from upstream to downstream; Piney Creek from below Lake DeSmet to the confluence with Clear Creek at Ucross, Clear Creek from the confluence with Piney Creek at Ucross to Double Crossing, Clear Creek from Double Crossing to a point below the Pratt & Ferris Ditch #3 diversion, and Clear Creek from Pratt and Ferris Ditch #3 to the Carlock Ranch.

The Piney Creek reach was first investigated in 1984. The network of gauges was monitored to determine a period of relatively stable flows; i.e., a period during which gains and losses to the stream were constant. Once a stable flow condition was established, augmentation water from Lake DeSmet was released to provide an incremental increase in flow in

Piney Creek. The flow of augmentation water was maintained for a period of several days, after which the flows were reduced to levels that existed prior to the reservoir release.

In the summer of 1985, the monitoring network established on Piney Creek was extended to include the three reaches defined on Clear Creek. Flow data was continuously collected for all of the reaches described (Piney Creek and Clear Creek) during the summers of 1985 and 1986. The streamflow hydrographs generated from these records were examined to assemble data for stable flow periods.

Data from suitable periods of flow compiled during these earlier investigations was used for this study. The methods selected to analyze the data compiled are discussed later in this chapter.

<u>GREEN RIVER STUDY AREA</u>. The Green River study area extends from Fontenelle Reservoir to the City of Green River, Wyoming, (Figure 4). The study area encompasses approximately 62 miles of the Green River. This reach of the river meanders through a relatively broad, moderately sloping alluvial valley. The valley is relatively barren of vegetation, with the exception of cottonwoods and other phreatophytes growing along the floodplain of the river. Mean annual potential evapotranspiration is approximately 21 inches in the study area and mean annual precipitation is less than eight inches (Ostrech et al, 1990). Irrigated agriculture is almost non-existent along the river.

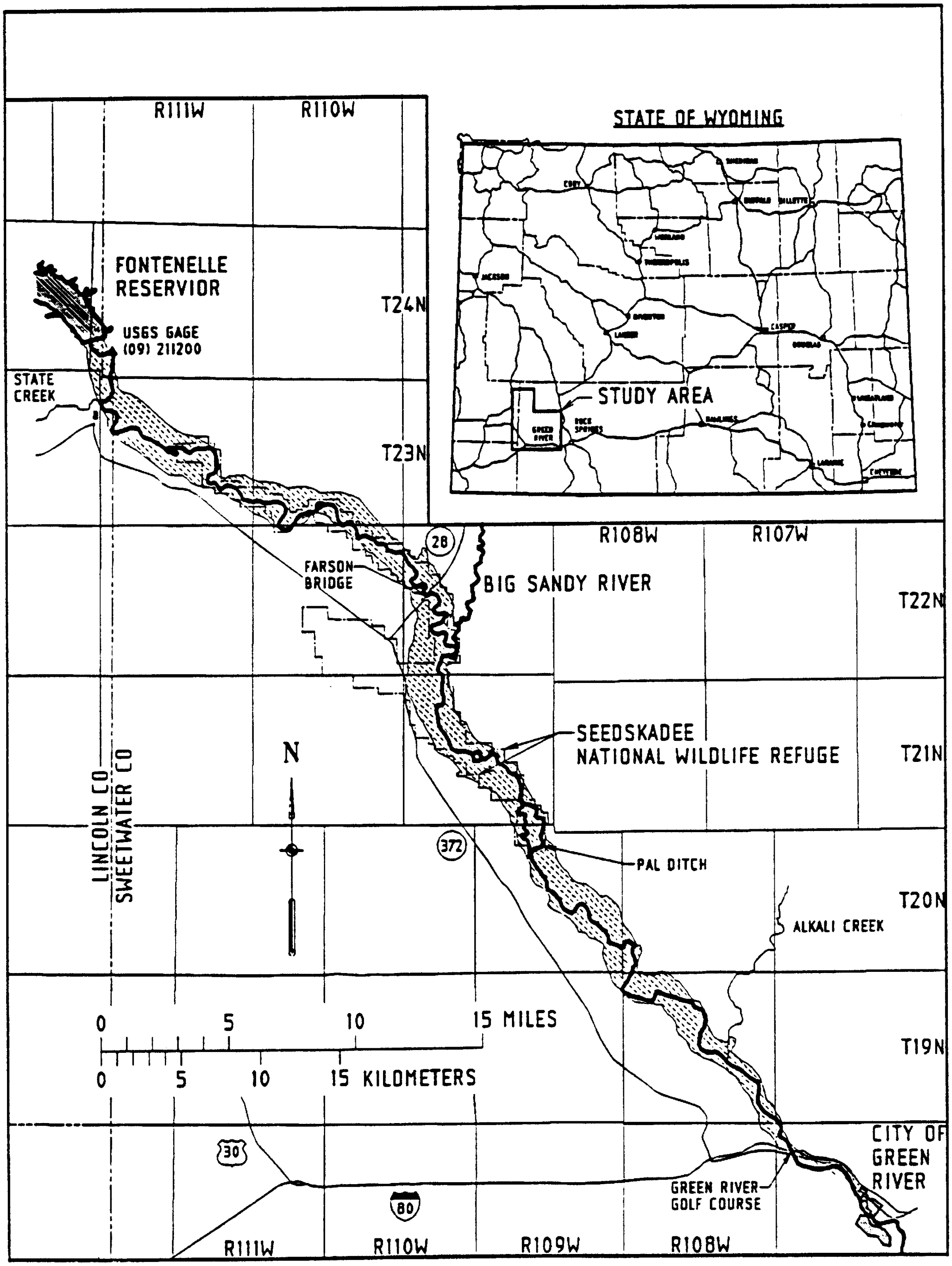


Figure 4. Green River Study Area

Data was collected for this investigation during August and September of 1989. During this period, significant releases of water were being made from Fontenelle Reservoir. Fontenelle Reservoir was constructed as part of the Colorado River Storage Project. It currently provides water based recreational benefits, and generates hydropower from releases of storage water to downstream users.

Seedskadee National Wildlife Refuge is one downstream user. As is shown in Figure 4, a significant portion of the study reach flows through the Seedskadee National Wildlife Refuge. A number of diversions have been constructed at Seedskadee to inundate wetlands established along the river to mitigate the loss of waterfowl habitat caused by the construction of Fontenelle Dam. The primary diversions used by the Wildlife Refuge are the Hamp Ditches No.'s 1 and 2. Only one of these diversions, Hamp Ditch No. 2, was active during the period that data was collected. A continuous stage recorder was installed above a flume located on the ditch about 2000 feet below the diversion point on the river. Water flowing in this ditch was conveyed to a series of ponds along the length of the refuge. Two locations were located where return flows from ponds entered the river. Spot measurements were made at these locations during the course of the study.

Inflow to the upstream end of the study area was monitored using an existing United States Geological Survey

(USGS) gauging station (09-211200) below Fontenelle Reservoir. This station is maintained by the United States Bureau of Reclamation (USBR). Personnel with the Salt Lake City office of the USBR supplied a rating curve and stage records for this location. Outflow from the downstream end of the study area was monitored with a gauging station installed at the Green River golf course. The golf course is located on the upstream side of the Interstate 80 bridge crossing, north of the City of Green River. Stage-discharge measurements were taken from a bridge which serves State Highway 374.

A second continuous stage recorder was installed on the river approximately 300 yards upstream of the bridge crossing for State Highway 28. State Highway 28, which leads to Farson, Wyoming, from State Highway 372, crosses the Green River approximately 24 river miles below the reservoir. This gauging location is referred to as the Farson Bridge location in this paper. This gauge station allowed the study area to be divided into two reaches; an upper reach extending from the USGS gauge station below the reservoir to Farson Bridge, and a lower reach from Farson Bridge to the Green River golf course.

The division of the river, at this location, was considered significant because of the inflow of water from the Big Sandy River nearly two miles downstream. The Big Sandy River is the primary tributary to the Green River in the study area. The flow of the Big Sandy River was gauged, using a continuous stage recorder, approximately 200 yards above its confluence with the Green River. To obtain an accurate model simulation of the Green River system, it was believed important to partition the river so that the inflow of the Big Sandy occurs near the upstream end of a modeling reach. Therefore, the Farson Bridge gauge station was established to allow flows from the Big Sandy to be added only to the affected reach of the river during modeling.

Two other locations of tributary inflow to the Green River were also monitored during the course of the investigation. Slate Creek, which contributes to the river about three miles below Fontenelle Dam, was observed not to be flowing during the course of the field investigation. Flows in Alkali Creek, which contributes water to the lower reach of the river, were measured using a 3 inch Parshall flume. The location of these tributary inflows are shown in Figure 4.

There are numerous diversions made from the segment of the river in the study area. All but one of these diversions are made from pipelines and pumping stations used to convey water from the river to outlying areas. For diversions made from pumping stations, the State Engineer's office contacted owners of water rights and requested data on the water consumed during the study period because this information could not be established in the field. Table II presents a list of the pipeline facilities which divert water from the river, their permit number and the location of the diversion.

TABLE II

PIPELINE DIVERSIONS ALONG STUDY REACH

Facility	Location S	T	R	Permit
Seedskadee Project	30	24N	111W	22365 D
Mt. Fuel Supply Horn Canyon pipeline	34	23N	110W	21923 D
Roberts pipeline	4	20N	109W	25836 D
Texas Gulf sulfur water pipeline	16	20N	109W	22808 D
Wesvaco pipeline	16	20N	109W	20077 D
Stauffer Wyoming pipeline	23	20N	109W	22075 D
Allied pipeline	1	19N	109W	22748 D
Tenneco pipeline	1	19N	109W	26126 D
Shaul sprinkler irrigation system	22	19N	108W	18818 D
Layos Inc. pipeline	36	19N	108W	25565 D
Gaensslen pipeline	36	19N	108W	28148 D
Layos pipeline	6	18N	107W	21137 D
Hodges pipeline #3	6	18N	107W	26271 D
Hodges pipeline #5	6	18N	107W	26272 D

One diversion, which could be monitored in the field, is called Pal Ditch (Figure 4). A continuous stage recorder was installed on a natural section of this ditch and flow measurements were conducted during the course of the field work to establish a discharge rating curve for the location.

Due to the high storage levels which existed in Fontenelle Reservoir during the period of the field investigation, the Bureau of Reclamation was releasing large amounts of water below the dam. It was not operationally feasible to increase the releases from Fontenelle Reservoir during this period without possibly causing harm to downstream facilities. Therefore, to allow changes in the flow of the river to be monitored for this investigation, releases from the reservoir were lowered by increments and then systematically increased again in intervals lasting from two to three days. The methodology used to analyze the data is discussed in the following section.

METHOD OF DATA ANALYSIS

As discussed in previous sections of this paper, a wide variety of factors have been shown to effect conveyance losses on streams in Wyoming. Previous conveyance loss model studies have characterized losses due to bank storage, channel storage, evaporation, and inadvertent diversions to predict the total losses associated with the conveyance of augmentation water in a natural stream. In the previous studies of Wyoming streams, major losses were attributed to bank storage and a reduction of groundwater inflow. In addition, losses were shown to be most effectively quantified using the incremental loss approach because most streams in

Wyoming are gaining streams. A goal of this study was to determine a methodology for estimating conveyance losses using a hydrologic model.

The version of the J349 model discussed in Chapter III was used to analyze the streamflow data compiled for this investigation. The approach taken to estimate conveyance losses for each of the two study areas addressed in this paper consisted of four basic steps: (1) reduction of the raw field data collected to derive stage-discharge rating curves and flow hydrographs for model input; (2) calibration of the J349 model; (3) estimation of evaporation losses; (4) evaluation of conveyance losses from model simulations.

DATA REDUCTION. Standard procedures were used to reduce the available field data. Data from each of the gauging stations was compiled to develop a stage-discharge rating curve using the least squares method as described by Pahl (1985). Rating curves were generated based on an equation of the form:

$$Q = KH^b$$

where Q is the discharge in cubic feet per second

K is a coefficient

H is the stage in feet

and b is an exponent

Equations of this form which were developed previously for the Piney Creek Study Area were also used for this investigation.

Rating curves for each of the continuous gauge locations in the Green River Study Area were generated from equations of the same form. The discharge measurements acquired at the golf course near the City of Green River were adjusted because of the skew of the bridge from which measurements were obtained. The bridge was determined to be skewed 30 degrees from normal to the river. Therefore, the incremental areas used to calculate discharge for each of the flow measurements taken at this location were multiplied by the cosine of 30° to reduce the measured area to an equivalent area for a section normal to the river. The measurements taken from Farson Bridge were not adjusted because the bridge is not skewed to the river channel. Discharge hydrographs were generated from each of the continuous stage records and the corresponding stage-discharge rating equation using a spread sheet program.

<u>MODEL CALIBRATION</u>. Calibration of the model first consisted of selecting an appropriate period from the available hydrographs to be simulated. Ideally, the hydrograph selected should initially have a period of steady flow, followed by an increase of flow, and finally a second period of steady flow similar in magnitude to the flow prior to the increase. In addition, other flows appurtenant to the hydrologic system should remain steady during all three periods.

As was discussed in Chapter III, input to the J349 model consists of three basic categories of data: streamflow data, reach parameters, and aquifer properties. Calibration of the model was achieved by varying the reach parameters and aquifer properties input to the program. The streamflow data acquired in the field and subsequently reduced in the office was not altered to calibrate the model. Streamflow data input to the model included the upstream observed hydrograph, streamflow diversions and tributary inflow. Aggregate values of the diversions and tributary inflow were input to the program by the day and reach in which they occurred. A time step for simulations was selected to maximize the detail of the hydrographs input within the data array storage constraints of the program.

Model parameters such as the stream reach length and aquifer width were estimated from USGS 7.5 minute quadrangle maps. Initial estimates of wave dispersion and wave celerity were made from the equations presented in Chapter III but were successively revised during the calibration process. Adequate information regarding aquifer properties was not available for either of the two study areas. Consequently, estimates of the transmissivity and storativity were determined solely through the calibration process. Land (1977) presented a sensitivity analysis of model parameters in which it was shown transmissivity and storativity are inversely proportional in the effect they have on the model results. A percentage increase of one parameter has the identical effect as the same percentage decrease in the other parameter. This analysis was verified during the calibration process.

Similarly, the choice of the stream-aquifer boundary condition used for model simulations was made based upon the calibration process. The finite aquifer and semi-aquifer boundary case alternatives were evaluated during the calibration process. The semi-infinite aquifer with a permeable confining bed case was not considered for use because of the necessity for additional, unavailable data. The boundary conditions evaluated are believed to have provided an adequate characterization of the conditions which exist at both of the study areas considered.

Base flow input during model calibrations was varied to simulate streamflow gains evident in each reach. This aspect of the calibration process was generally completed last. The approach used was based on the assumption that gains in a stream reach were relatively constant over the period considered in the simulation. The character of the observed hydrograph was first approximated by varying reach and aquifer parameters to the greatest extent possible. Gross disparities between the observed hydrograph and the hydrograph determined in the initial calibration process were offset by varying the base flow.

Calibrations were not strictly constrained to obtain a mass balance in the system during the simulation. Rather, the success of a simulation during the calibration process was also judged on the basis of the precision with which the recession leg of the observed hydrograph was simulated. It was believed some imbalance in the total volume of flow was justified to obtain a reliable simulation of the response of the system during recessions in flow. It is believed a small mass imbalance is likely a result of two principal factors: (1) random and systematic errors associated with the data input to the model; and (2) the presumption, inherent in the model, that the stream and aquifer are initially in equilibrium. Model calibration performed solely on the basis of a mass balance between the observed downstream hydrograph and the hydrograph derived from the model simulation does not overcome either factor. To the contrary, the model studies performed indicated requiring a strict mass balance during calibration of the model served only to skew reach and aquifer parameters, distort the simulated hydrograph, and overall degrade the quality of the simulation. Each of the hydrologic systems modeled are, of course, constrained by the law of conservation of mass. However, because of the limitations of the data input and the assumptions inherent in the J349 model, mass balance was not used as an exclusive basis for determining model calibration.

EVAPORATION. A cursory assessment of the loss due to evaporation is presented in this paper. Evaporation loss is not directly accounted for in the J349 model and was determined independently from model simulations. In each of the study areas, direct evaporation loss from the stream surface was calculated to determine an order of magnitude estimate of this loss for comparison with the loss to bank storage determined from the model. Evaporation loss was estimated using pan evaporation data from the nearby meteorological stations. The measured pan evaporation, in units of inches per hour, was totaled for the period of study under consideration and converted to units of square feet per second per mile (ft^2/sec-mile). This value was then multiplied by the number of stream miles under consideration and a pan coefficient of 0.70 to estimate the evaporation loss per unit change in stream width. This estimate is termed the unit evaporation loss.

The unit evaporation loss was used in conjunction with calculated changes in stream width associated with the average increase in stage over the period of study to estimate total evaporation loss. Changes in stream width associated with a differing stream stage were calculated using available data at gauge locations.

Although the methodology employed to calculate evaporation loss is not rigorous, it was believed appropriate based on the information available and the overall objectives of this investigation.

<u>EVALUATION OF CONVEYANCE LOSSES</u>. The goal of the model studies discussed in this paper was to determine a practical methodology based on a hydrologic model which the Board of Control may use to administer conveyance loss. Therefore, an emphasis was placed on understanding the J349 model operation

and interpreting the results of model simulations rather than determining any firm conveyance loss estimates for either study area. Ideally, once the model is calibrated, an assessment of conveyance loss to bank storage can be made over the range of discharges considered in the calibration process. Evaporation losses, which were determined independently, are subsequently added to the bank storage loss determined from the model simulation of a stream, to arrive at an estimate of the total conveyance loss.

The ideal methodology was employed successfully to varying degrees in the two study areas. The model was successfully used to simulate a variety of flow conditions which might be present in one of the selected study areas. Systematic variations of hydrologic conditions were simulated to derive graphical comparisons of bank storage. The method for the determination of bank storage loss presented is equally applicable to either the total or incremental loss approaches. Differences in both natural, or base flow, and augmentation flow can be established in model simulations to evaluate bank storage loss from the standpoint of either approach. Combinations of natural and augmentation flows evaluated in this paper were restricted to the range of flows used to calibrate the model. Although the J349 model might be used to provide reasonable results for discharges outside the range of those used in the calibration process, model simulations involving extrapolative data was not considered.

CHAPTER V

RESULTS AND DISCUSSION

A discussion of the results of the analyses performed for each of the study areas is presented in this chapter. Topics to be covered included: (1) Piney Creek Study Area; (2) Green River Study Area; and, (3) Comparison of Results.

PINEY CREEK STUDY AREA

In 1984 a stream gauge network was established on Piney Creek. The stream gauge network was expanded in 1985 and maintained through 1986. During the three years when the field data was collected in the study area, four periods of streamflow records were obtaining while reservoir releases were made from Lake DeSmet. The data and results from the analysis of two of these periods are discussed below.

1984 RESERVOIR RELEASE. The existing database of streamflows and diversions along Piney Creek, from below Lake DeSmet to the gauging station near Ucross, Wyoming, was compiled for the period from August 9 to September 2, 1984 for use with the J349 model. The streamflow data input to the model is depicted in Figure 5. The inflow hydrograph in Figure 5 depicts discharges measured below Lake DeSmet and the outflow hydrograph describes the discharges measured at the

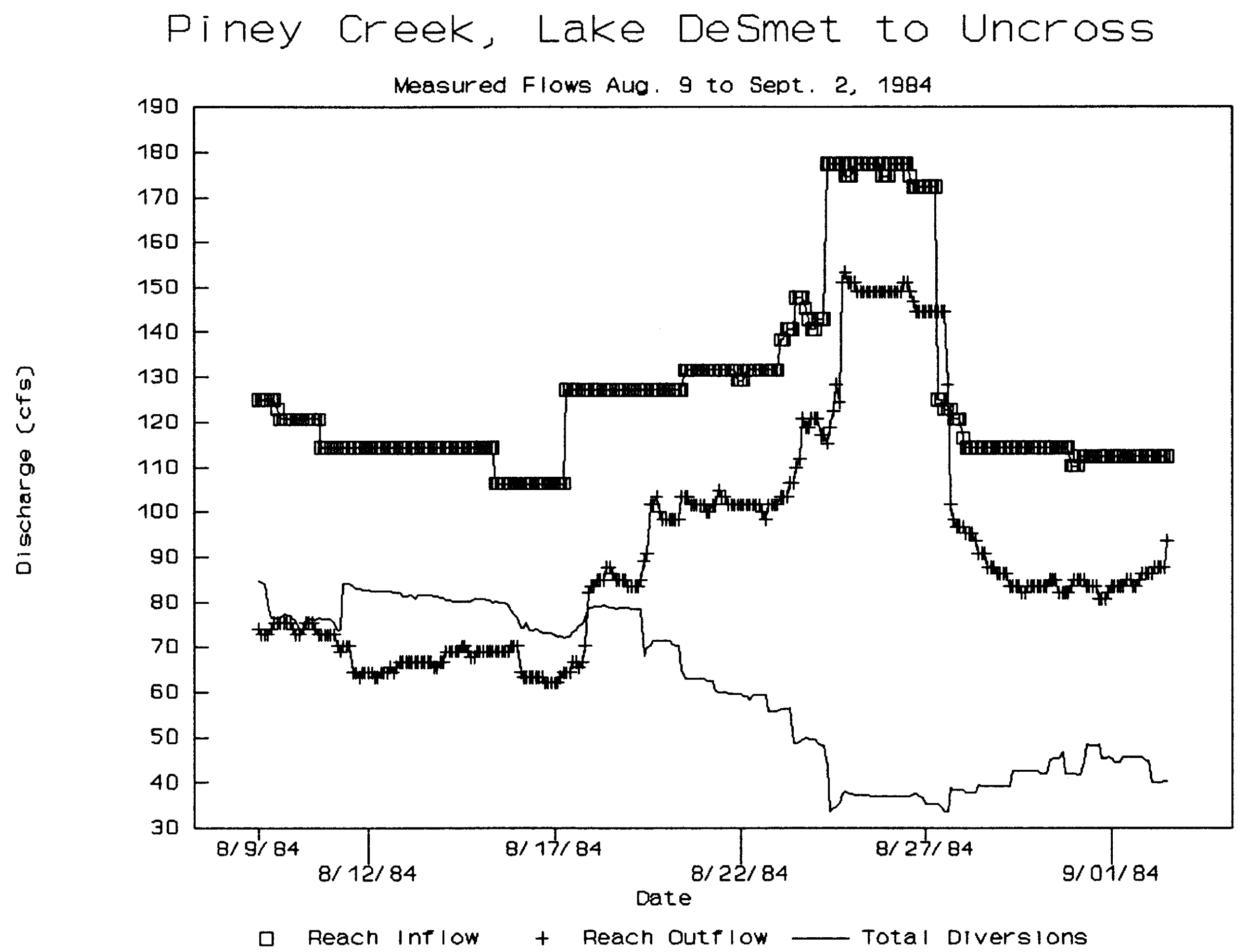


Figure 5. Piney Creek 1984 Reservoir Release,
Measured Streamflow Hydrographs.

Piney Creek at Ucross, Wyoming gauging station. The total of the measured diversions for each of the ditches along the reach is also depicted in Figure 5 in the form of a composite hydrograph. Average daily diversion values were derived from this hydrograph for input to the model.

Reach parameters and aquifer properties input to the model were successively revised during the calibration process. This was the first database compiled for use with the J349 model. Accordingly, over thirty calibrations were completed to evaluate the response of the model to changes in input, as well as to adjust the model input to this specific database. Land (1977) presents the results of a sensitivity analysis completed for the J349 model. These results were confirmed during the calibration process for this database. Table III presents the model parameters determined from the calibration and the range of values evaluated during the process.

Figure 6 graphically presents the results from the model calibration for this stream reach. In general, the model simulation yields poor results for the period of record. The initial three to four days of the model simulation as shown in Figure 6 have considerable fluctuation in response. The irregularity seen in this portion of the simulation results from the inability of the model to obtain closure between the streamflow, diversions, and bank storage. The J349 model, as with other hydrologic models, requires some "warm up time" at

TABLE III

PINEY CREEK 1984 RELEASE MODEL PARAMETERS

Input Parameter	Employed	Range Evaluated
Stream-aquifer boundary	Case 2	Case 1, Case 2
Transmissivity (ft^2/d)	10,000	100 - 10,000
Storativity	0.30	0.01 - 0.30
Aquifer Width (ft)	1500	--
Soil Retention	0.0	0.0 - 1.0
Wave Dispersion (ft^2/s)	400-470	100 - 600
Wave Celerity (ft/s)	1.8-2.6	1.0 - 2.6
Base Flow (cfs)	40	20 - 50
Length of Channel (mi)	22	--
Length of Alluvium (mi)	13.5	--

the beginning of the simulation. Generally the variations observed are not this dynamic, however, the simulation of this stream reach is complicated by high diversion rates relative to the streamflow in Piney Creek.

The poor results observed for the remaining portion of the simulation of this study period are believed to result from the magnitude of gains to streamflow occurring in this reach. Figure 7 presents the measured net reach inflow as compared to the measured outflow. The net reach inflow hydrograph is the difference between the measured inflow hydrograph and the total diversions hydrograph shown in Figure 5. Figure 7 shows the variation in gains to the study reach

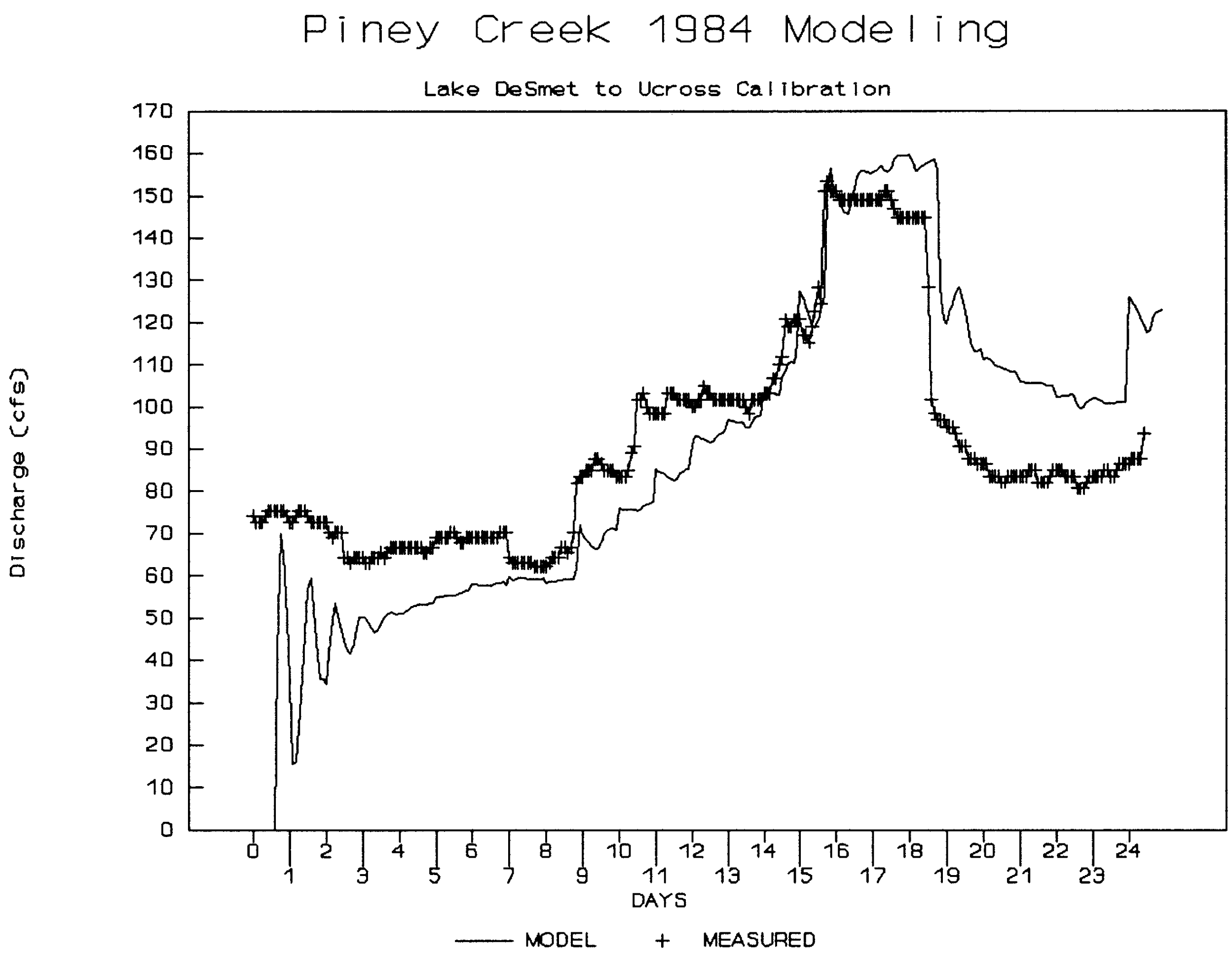


Figure 6. Piney Creek 1984 Reservoir Release, Lake DeSmet to Ucross Model Calibration

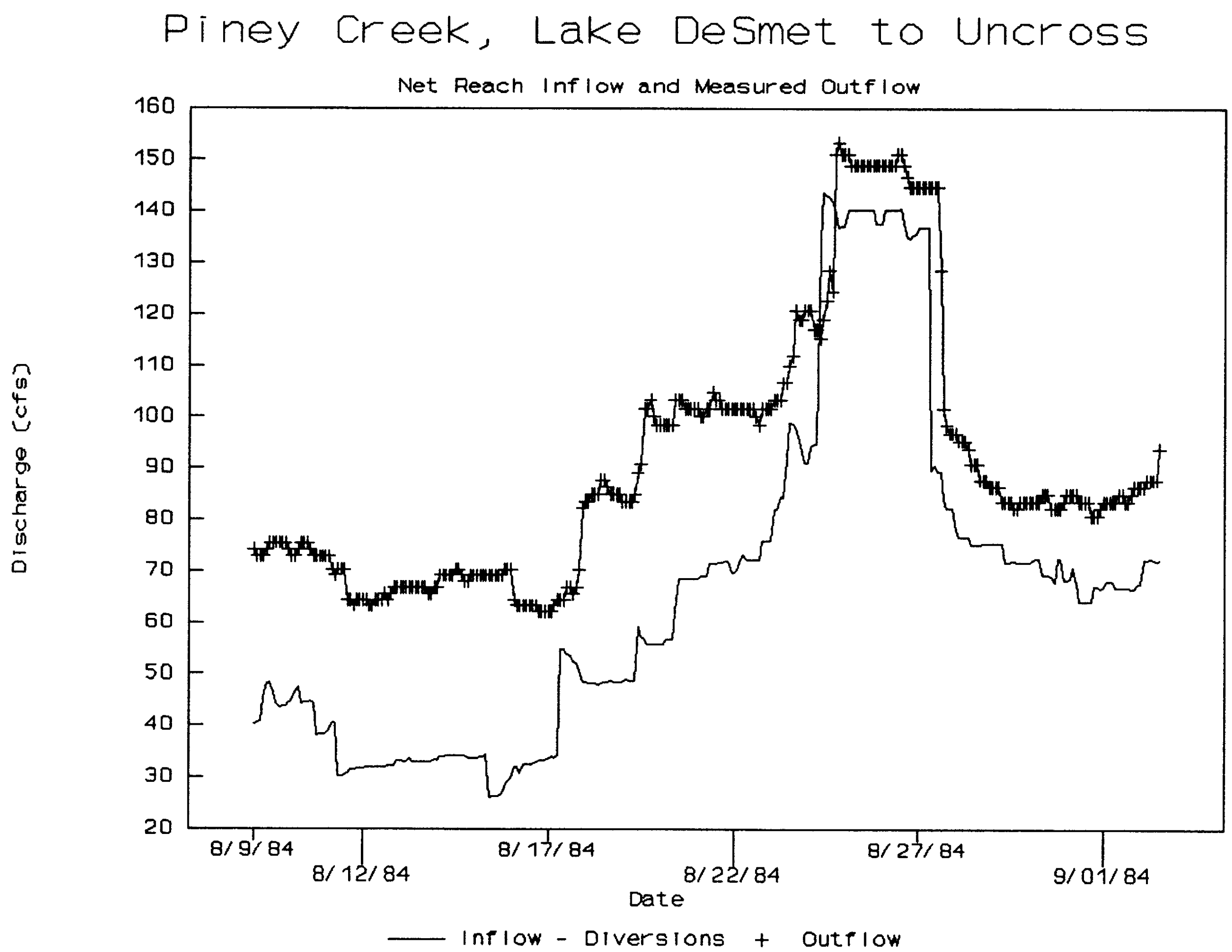


Figure 7. Piney Creek 1984 Reservoir Release, Net Reach Inflow and Measured Outflow.

during the period of the reservoir release. The difference between the two hydrographs shown in Figure 7 illustrates the unmeasured gains to this reach of Piney Creek. These gains are believed to primarily result from groundwater inflow. Of particular importance with regard to the poor results obtained with the J349 model is the variability in gains which is observed in Figure 7. Gains observed during the peak period of the reservoir release are significantly less than those seen prior to and after the release. Prior to the reservoir release, gains from groundwater along the reach contribute approximately 50 percent of the measured outflow. During the peak of the reservoir release, gains contribute less than 10 percent of the measured outflow. As discussed in Chapter III, the assumption that the groundwater surface is flat and in equilibrium with the stream is intrinsic to the J349 model. Figure 7 shows graphically this assumption is not valid for this reach of Piney Creek.

It is important to draw the distinction that the failing of the J349 model to simulate streamflows in this reach primarily results from the variability of the gains to Piney Creek. Although the assumption that the groundwater surface is flat would not be any more valid if the gains were constant, the model would likely be able to adequately simulate streamflows in a situation where gains do not vary significantly. In addition, the magnitude of the gains in relation to the magnitude of observed streamflows also contributes to the inability of the J349 model to accurately simulate measured streamflows. The significant contribution to streamflows from gains, attributed to groundwater inflow along this reach of Piney Creek, contributes to the poor results obtained from the model.

In spite of the poor quality of the streamflow simulation which was obtained with the J349 model, quantitatively the results agreed well with previous estimates. Conveyance loss to bank-storage over the period of the simulation was estimated to be 14.3 percent of the volume of flow. This estimate equates to an average loss of 0.65 percent per stream mile.

Evaporation was estimated to additionally contribute losses between 0.05 and 0.23 cubic feet per second (c.f.s.) per day. Evaporation data used for these estimates were taken from records for the period from the Sheridan field station. The 0.05 c.f.s. per day value was calculated considering an average increase in stream width of 1 foot over the period of the release. The 0.23 c.f.s. per day loss estimate was calculated based on an average increase in stream width of 5 feet. The range of increase in stream width considered was determined based upon an average increase in stage of Piney Creek during the release of 0.18 feet. This range of evaporation loss is tantamount to less than 0.2 percent of the total volume of flow during the period of study. Therefore evaporation was believed to be insignificant to the total conveyance loss attributed to this reach of Piney Creek.

1985 RESERVOIR RELEASE. Streamflows and diversions along Piney and Clear Creeks from below Lake DeSmet to the Carlock Ranch were compiled from the existing database for the period from July 6 to July 31, 1985 for use with the J349 model. As discussed in Chapter IV, the study area encompassed four contiguous stream reaches. The first stream reach, Piney Creek from Lake DeSmet to Ucross, is identical to the reach modeled from the 1984 database. The remaining three consecutive reaches are located on Clear Creek. The streamflow data input to the model for each of the four reaches are shown in Figures 8, 9, 10, and 11. The hydrographs in Figure 8 depict discharges measured for the same stream reach considered in the 1984 study period discussed previously. Figures 9, 10, and 11 present measured streamflow data for the three reaches on Clear Creek in order from upstream to downstream. The inflow hydrograph shown in each of these figures traces observed flows at the upstream limit of the reach and the outflow hydrograph describes the discharges measured at the downstream limit of each reach. The total of the measured daily diversions for each of the ditches along a reach, as input to the J349 model, is also depicted in the figures as discrete values. The most downstream reach, Pratt and Ferris Ditch No. 3 to Carlock Ranch (Figure 11), did not have any diversions.

Each of the four reaches were evaluated separately with

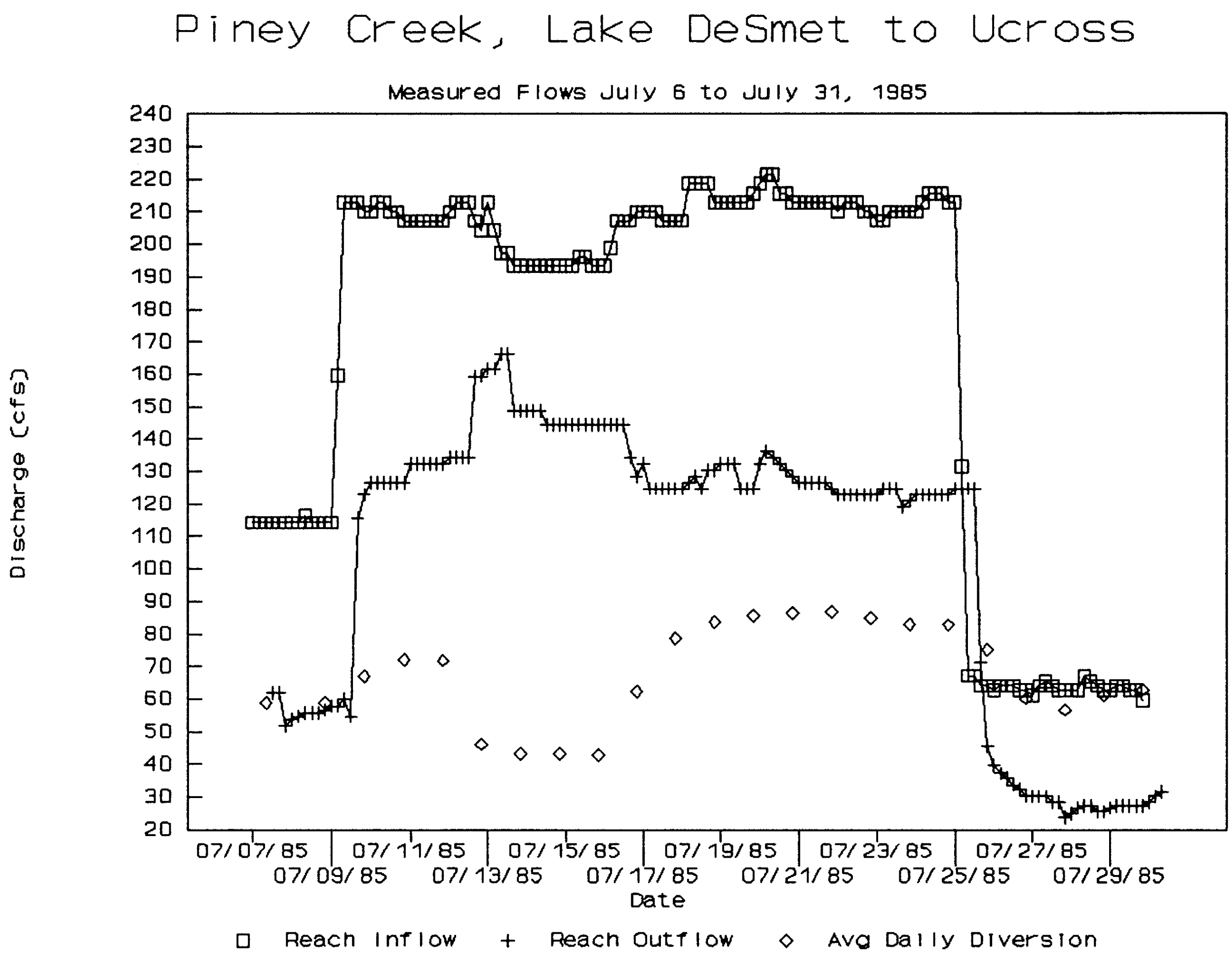


Figure 8. Piney Creek 1985 Reservoir Release, Lake DeSmet to Ucross Measured Streamflows.

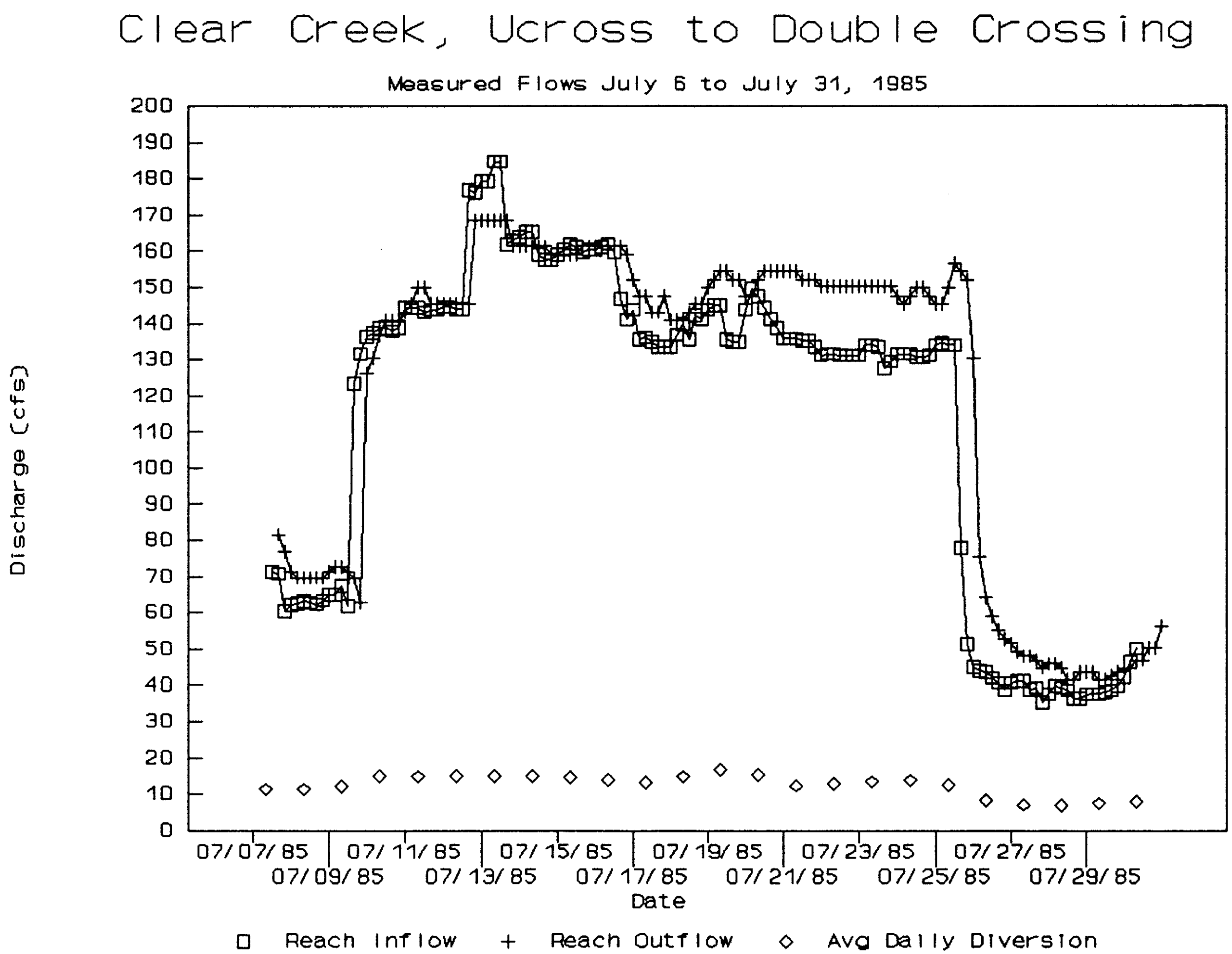


Figure 9. Clear Creek 1985 Reservoir Release, Ucross to Double Crossing Measured Streamflows.

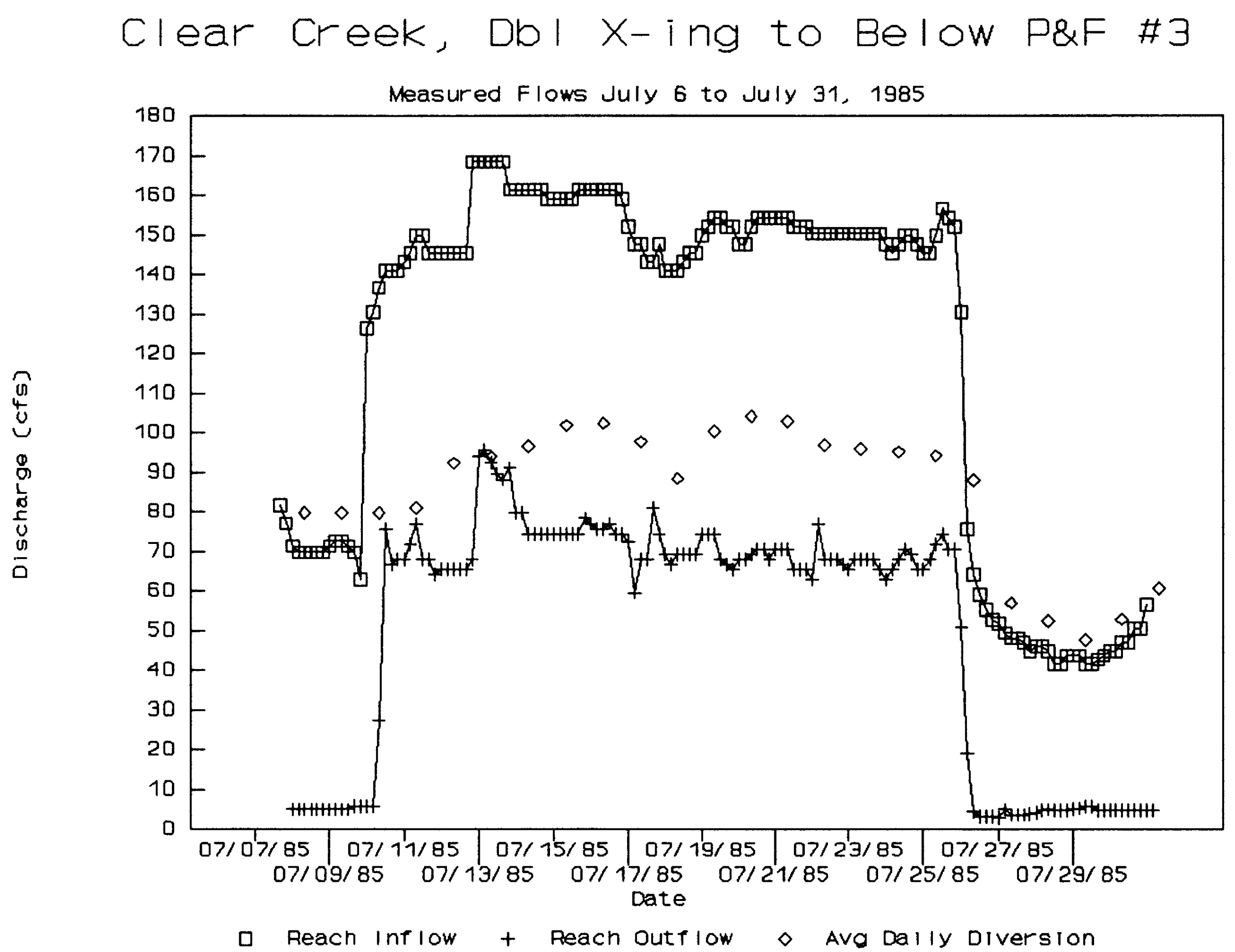


Figure 10. Clear Creek 1985 Reservoir Release, Double Crossing to Below Pratt & Ferris Ditch No. 3 Measured Streamflows.

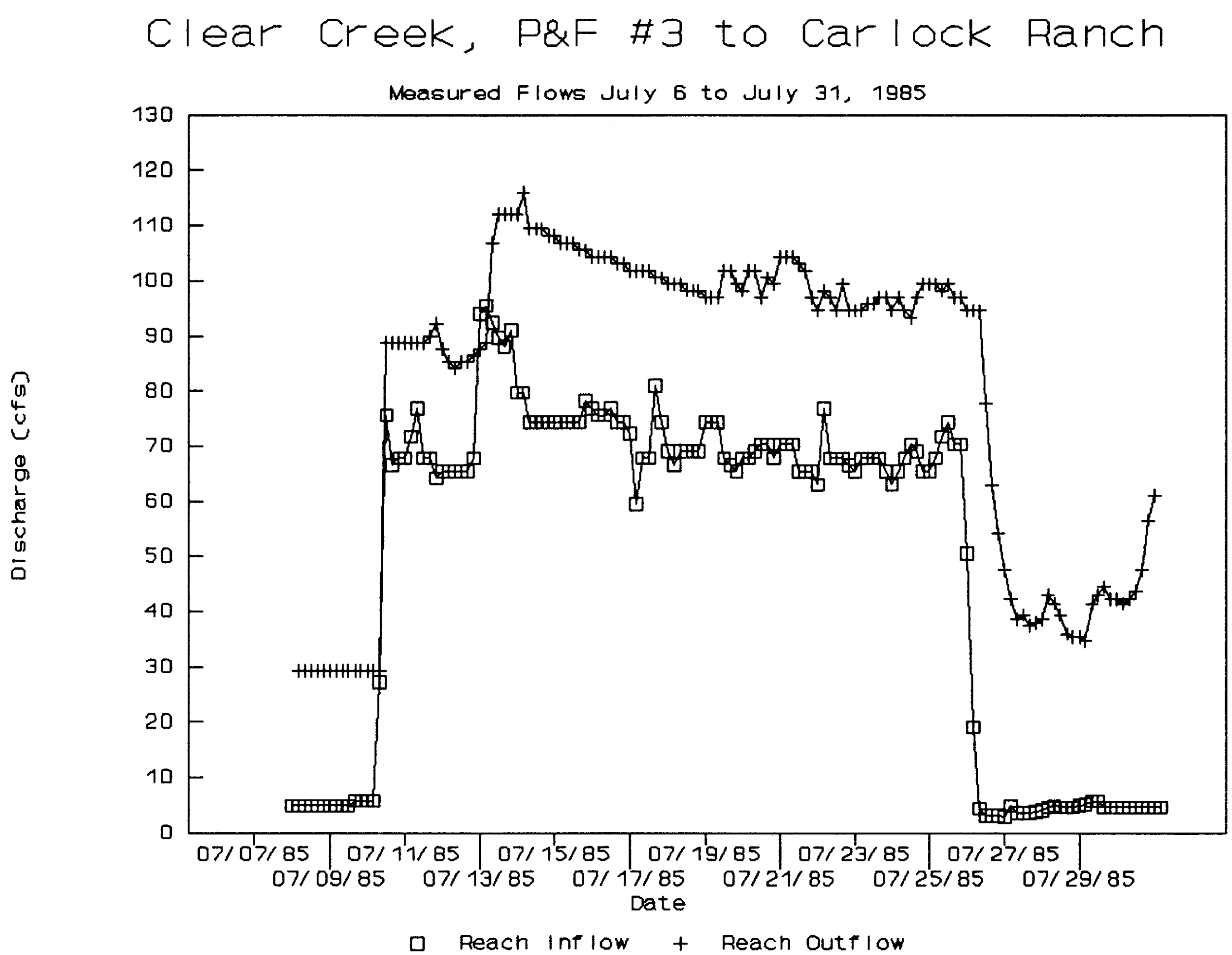


Figure 11. Clear Creek 1985 Reservoir Release, Below Pratt & Ferris Ditch No. 3 to Carlock Ranch Measured Streamflows.

the model. The J349 model is capable of continuously routing streamflows in consecutive reaches, however, the model results from the upstream reach are used for the next downstream reach. Based upon the results of the simulation of the 1984 release, it was believed a more accurate (with respect to measured streamflows) calibration would be obtained by evaluating each reach separately. Table IV presents the model parameters determined from the calibration for each reach. The multi-linear routing option was used for all of the reaches. The range of values for wave dispersion and wave celerity which were determined for each reach are listed. The base flow was used to approximate the gain estimated along each stream reach. All four of the reaches simulated were demonstrated to be gaining flow, presumably from groundwater inflow.

Except for base flow, the input parameters determined in the calibration of the 1984 data were also employed for the 1985 data acquired for the Piney Creek reach (first reach). Figure 12 shows the results of the calibration of the first reach. As in the case of the model simulation of the 1984 data, the results of the calibration are generally poor. Alternative simulations to that presented in Figure 12 were evaluated, however, the model results could not be calibrated to address the dynamic range of streamflows observed at the downstream limit of the reach. During the calibration process it appeared the alluvium along the stream behaved like a

TABLE IV

PINEY CREEK 1985 RELEASE MODEL PARAMETERS

Input Parameter	Reach 1, Piney Creek Lake DeSmet to Ucross	Reach 2, Clear Creek Ucross to Double X-ing	Reach 3, Clear Creek Double X-ing to P&F #3	Reach 4, Clear Creek P&F #3 to Carlock
Stream-aquifer boundary	Case 2	Case 2	Case 2	Case 2
Transmissivity (ft^2/d)	10,000	10,000	100	2,000
Storativity	0.30	0.30	0.05	0.20
Aquifer Width (ft)	1500	2,000	2,000	2,500
Soil Retention	0.0	0.0	0.0	0.0
Wave Dispersion (ft^2/s)	400-470	400-470	100-107	800-870
Wave Celerity (ft/s)	1.8-2.5	1.8-2.6	1.8-2.6	1.8-2.6
Base Flow (cfs)	30	25	15	28
Length of Channel (mi)	22.0	9.2	12.2	19.7
Length of Alluvium (mi)	13.5	4.6	7.9	8.9

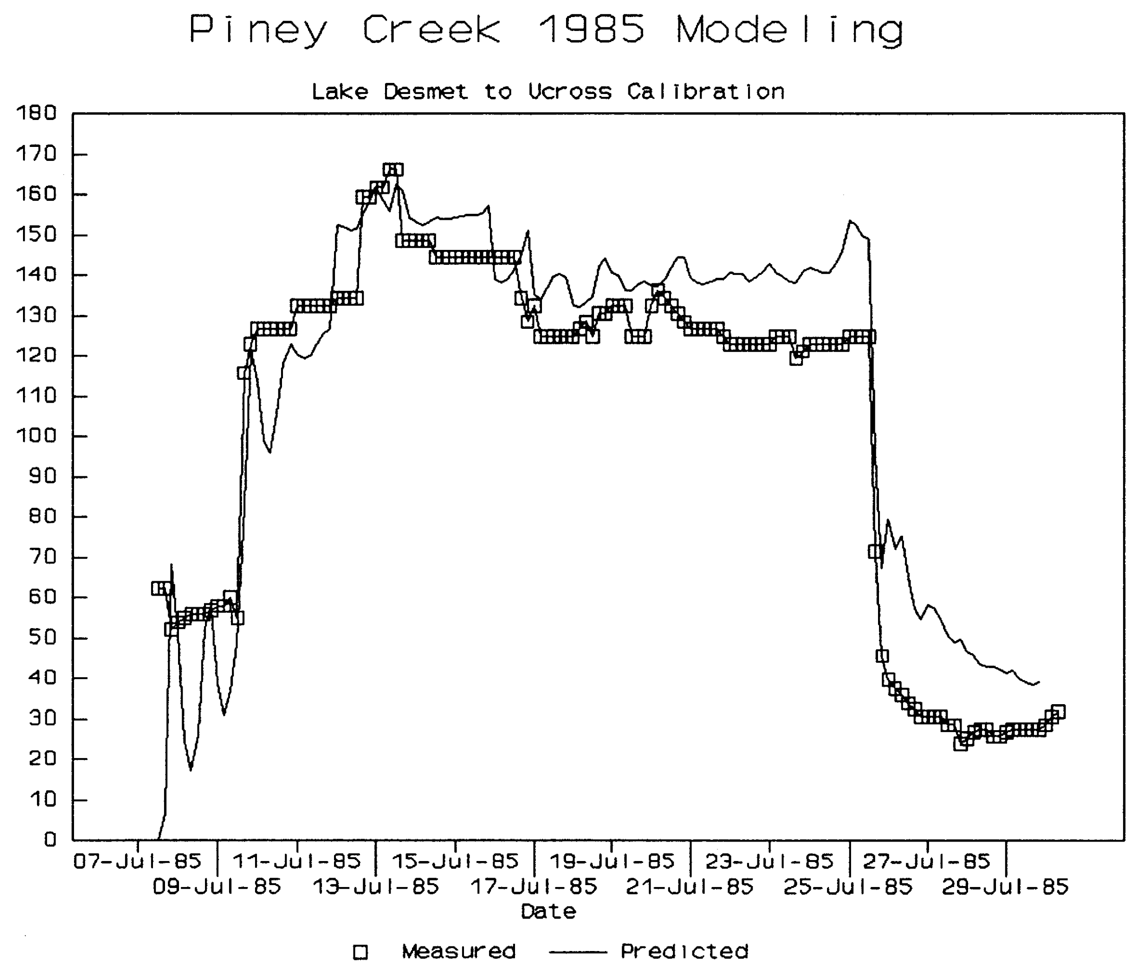


Figure 12. Piney Creek 1985 Reservoir Release, Lake DeSmet to Ucross Model Calibration.

sponge in that it seemed to have an almost infinite capability to take in water from the stream as the reservoir release occurred. Once the higher flows associated with the reservoir release began to recede, the alluvium responded in an analogous manner by refraining from discharging stored water. These contrary results could, in small part, be offset by using the soil retention option in the model. However, even when all of the water accrued as bank storage was restricted from returning to the stream, the simulation showed little improvement. The result presented in Figure 12 provides an acceptable calibration and allows for a comparison with the simulation completed for the 1984 database. Conveyance loss to bank-storage over the period of the simulation was determined to be 11.3 percent of the volume of flow. This estimate equates to an average loss of 0.52 percent per stream mile. These results are reasonable in comparison to those obtained from the simulation completed with the 1984 reservoir release.

The results from the model simulation completed for the second reach; Clear Creek from Ucross to Double Crossing, are shown in Figure 13. The difficulties associated with the simulation of the first reach (Piney Creek) also persisted during the calibration of the model with this database. Although the predicted discharge hydrograph shown in Figure 13 simulates observed flows quite well over the majority of the study period, the results are less favorable during the

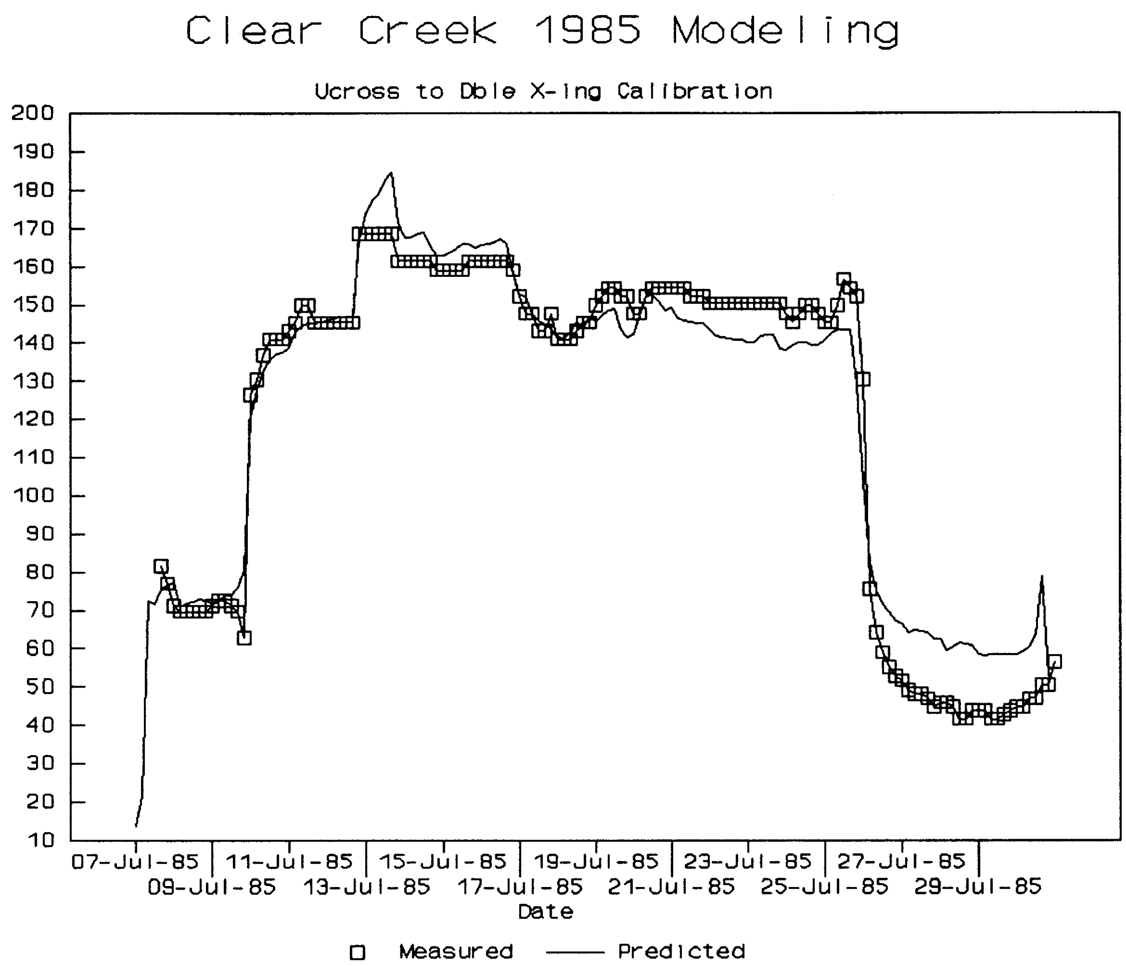


Figure 13. Clear Creek 1985 Reservoir Release, Ucross to Double Crossing Model Calibration.

recession of the reservoir release. The improved simulation results are most likely attributable to the stability in diversions observed for this reach (Figure 9). The reason for this incongruity during the flow recession is not clear. However, these results are similar to those obtained for the first reach, and therefore it is likely this difficulty is due to some physical phenomena rather than a shortcoming in the database of measured streamflows. The simulation results in a total loss to bank-storage over the period of the simulation of 2.26 percent of the volume of flow. This estimate equates to an average loss of 0.25 percent per stream mile.

Results of the simulation of the third reach; Clear Creek from Double Crossing to Below Pratt and Ferris Ditch No. 3, are presented in Figure 14. The model calibration resulted in significantly changed values for aquifer parameters. Both the values input for transmissivity and storativity were changed to reflect the relative incapacity of the alluvium to interact with the stream (Table IV). Also significant was the modification to the wave dispersion parameter. The lower values input for this reach, in comparison to the previous two upstream reaches, evidence the limited attenuation of the flood wave associated with the reservoir release observed. The results of the model simulation of this reach of Clear Creek were generally good. Total loss to bank-storage over the period of the simulation amounted to only 0.13 percent of the volume of flow. This estimate equates to an average loss

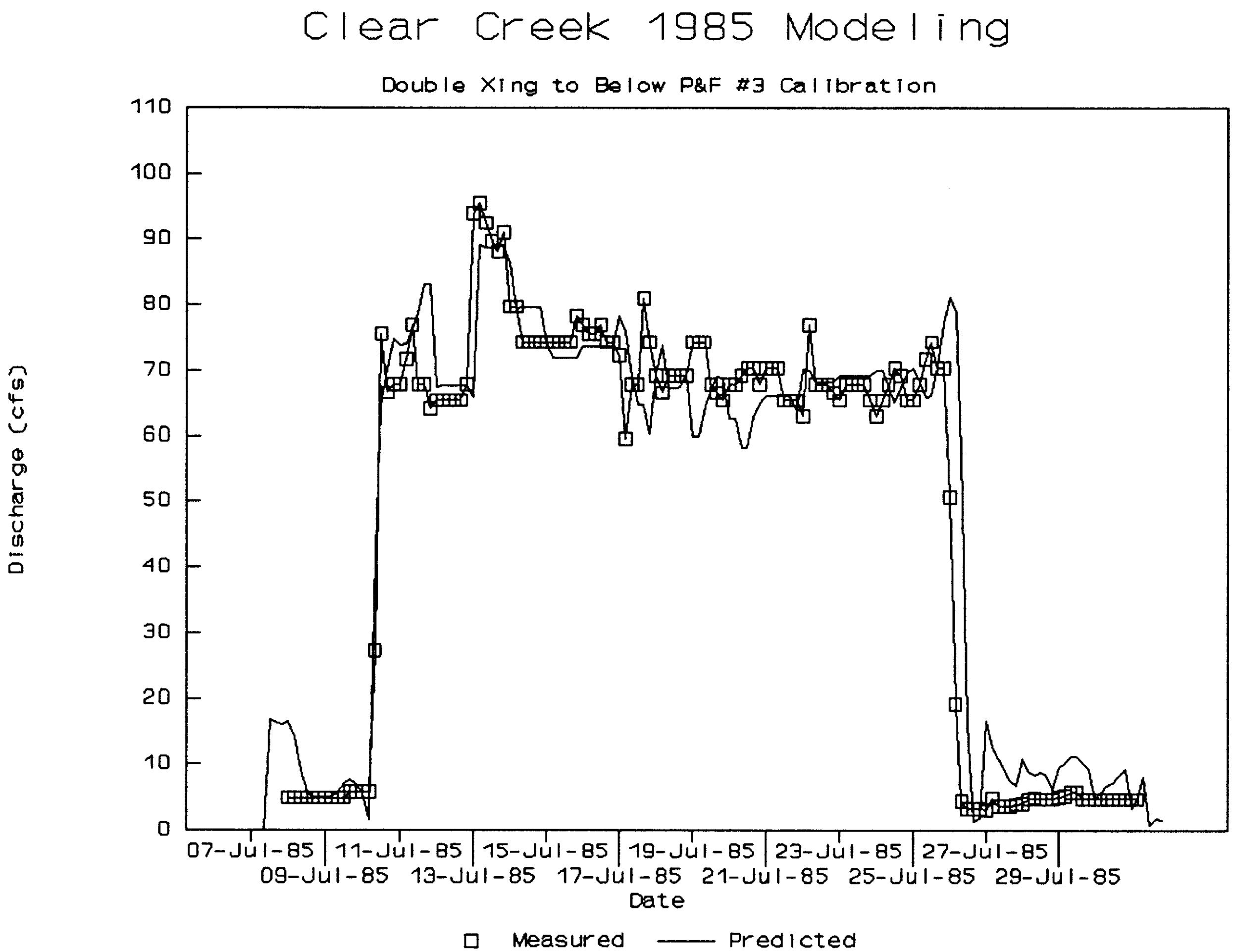


Figure 14. Clear Creek 1985 Reservoir Release, Double Crossing to Below Pratt & Ferris Ditch No. 3 Model Calibration.

of only 0.011 percent per stream mile.

The database for the fourth and most downstream reach of the study area investigated; Pratt and Ferris Ditch No. 3 to Carlock Ranch, also resulted in a reasonably good model simulation. Figure 15 presents the outcome of the model calibration for this reach. The aquifer parameters determined for this reach are shown in Table IV to be intermediate between those determined for the first two reaches and the third reach. However, the value used for wave dispersion along this reach was calibrated to be much greater than any of the three upstream reaches. Also of significance to the results obtained for this stream reach is the disparity between the predicted and measured hydrograph at the beginning of the reservoir release. This disparity may be a result of a perversion in the data for the measured streamflows. The data used for this investigation were obtained from streamflow tabulations which had been shifted in time to account for travel time. The shifts to the streamflow data were removed to allow input of the data to the model. A review of these corrections made to the database for the streamflows at the Pratt and Ferris Ditch station did not reveal any errors, however, the database and results from this reach are suspect from the standpoint of the time base of the reservoir release. The model simulation for this reach concluded with an estimate for total loss to bank-storage of 3.25 percent of the volume of flow. This estimate equates to an average loss of 0.17

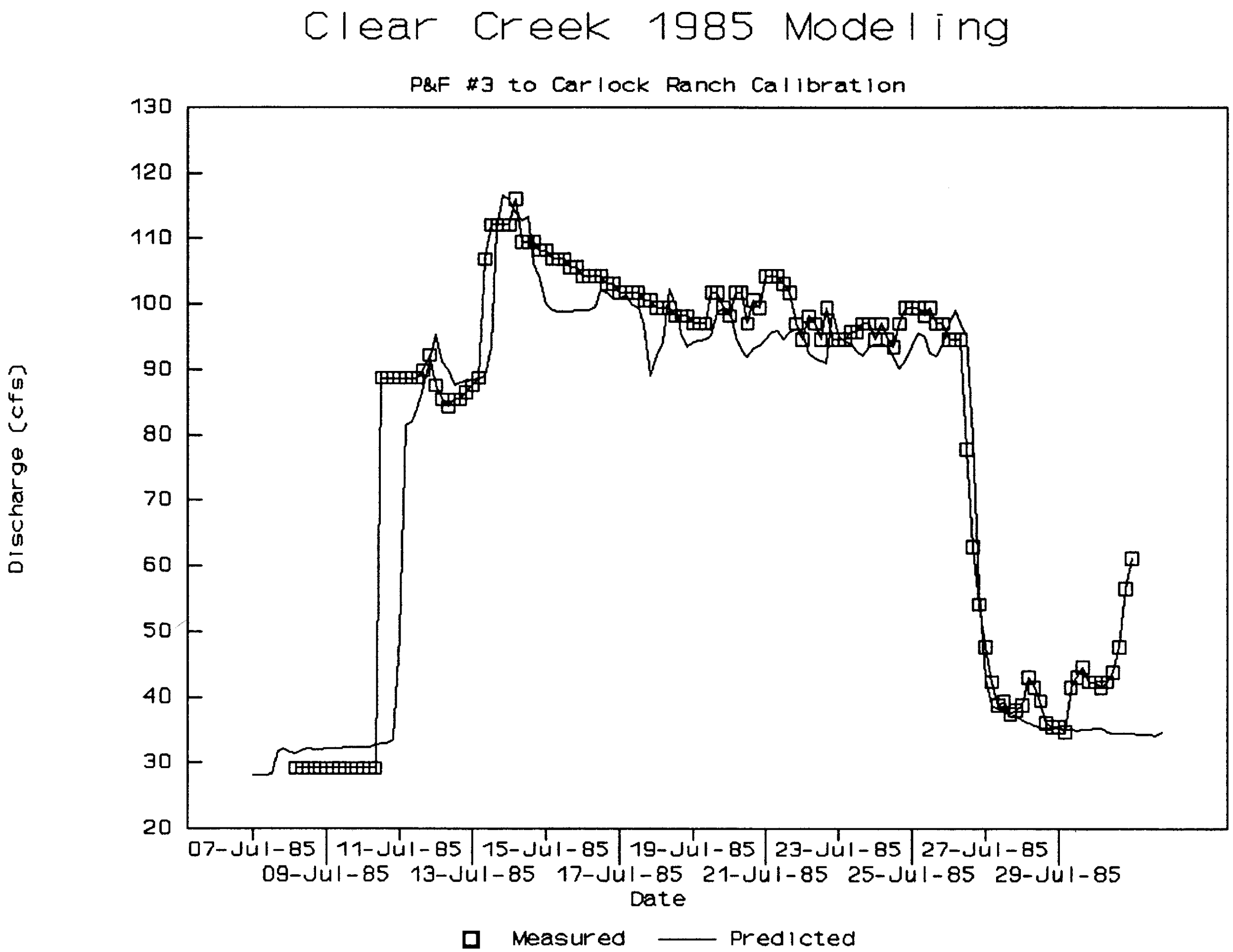


Figure 15. Clear Creek 1985 Reservoir Release, Below Pratt & Ferris Ditch No. 3 to Carlock Ranch Model Calibration.

percent per stream mile.

Estimates of evaporation loss were not calculated for the 1985 reservoir release. The results of the 1984 release established evaporation losses are not significant to the objectives of this investigation of the Piney Creek study area. Although evaporation losses likely represent a meaningful percentage of the total conveyance loss associated with the reach of Clear Creek between Double Crossing and the Pratt and Ferris Ditch No. 3 the volume of loss in this reach which is attributable to evaporation was still trivial.

Furthermore, model simulations, other than those completed for the calibrations discussed, were not completed for this study area. Initially, additional modeling was postponed because of the generally poor results obtained from the two most upstream reaches and a desire to obtain some preliminary results from the Green River study area. It was believed more prudent to complete an evaluation of the J349 model using the Green River database prior to continuing an assessment of its value based on the data for this study area. The modeling results obtained using the data on the Green River were notably more positive and ultimately further work using this database was abandoned.

GREEN RIVER STUDY AREA

During August and September of 1989 a stream gauge network was established on the Green River between Fontenelle

Reservoir and the City of Green River. At that time the pool level at Fontenelle Reservoir was near the maximum permissible elevation. Significant releases of water were being made from the reservoir to maintain the pool below the lip of the primary spillway. Due to these unusual conditions, it was not feasible to further increase the release of water from Fontenelle Reservoir because of potential harm to downstream structures along the banks of the Green River. In lieu of increasing discharges from the reservoir, the USBR agreed to systematically lower and then increase the rate of reservoir releases over a period of a few days to assist in this investigation of conveyance losses. The data and results from the analysis of the period during which the discharges from Fontenelle Reservoir were varied are discussed below.

MODEL CALIBRATION. Discharges were monitored at three locations on the Green River during the period of study: below Fontenelle Reservoir, at the Farson Bridge and near the Green River Golf Course. Figure 16 presents the measured flows at each of the three locations during the period from August 5 to September 19, 1989. The period during which releases from Fontenelle Reservoir were varied to facilitate conveyance loss modeling occurred from August 11 to August 23, 1989. The study area was subdivided into two reaches, from below Fontenelle Reservoir to Farson Bridge and from Farson Bridge to the Green River Golf Course. It is apparent from Figure 16 that the river is gaining flow in both reaches.

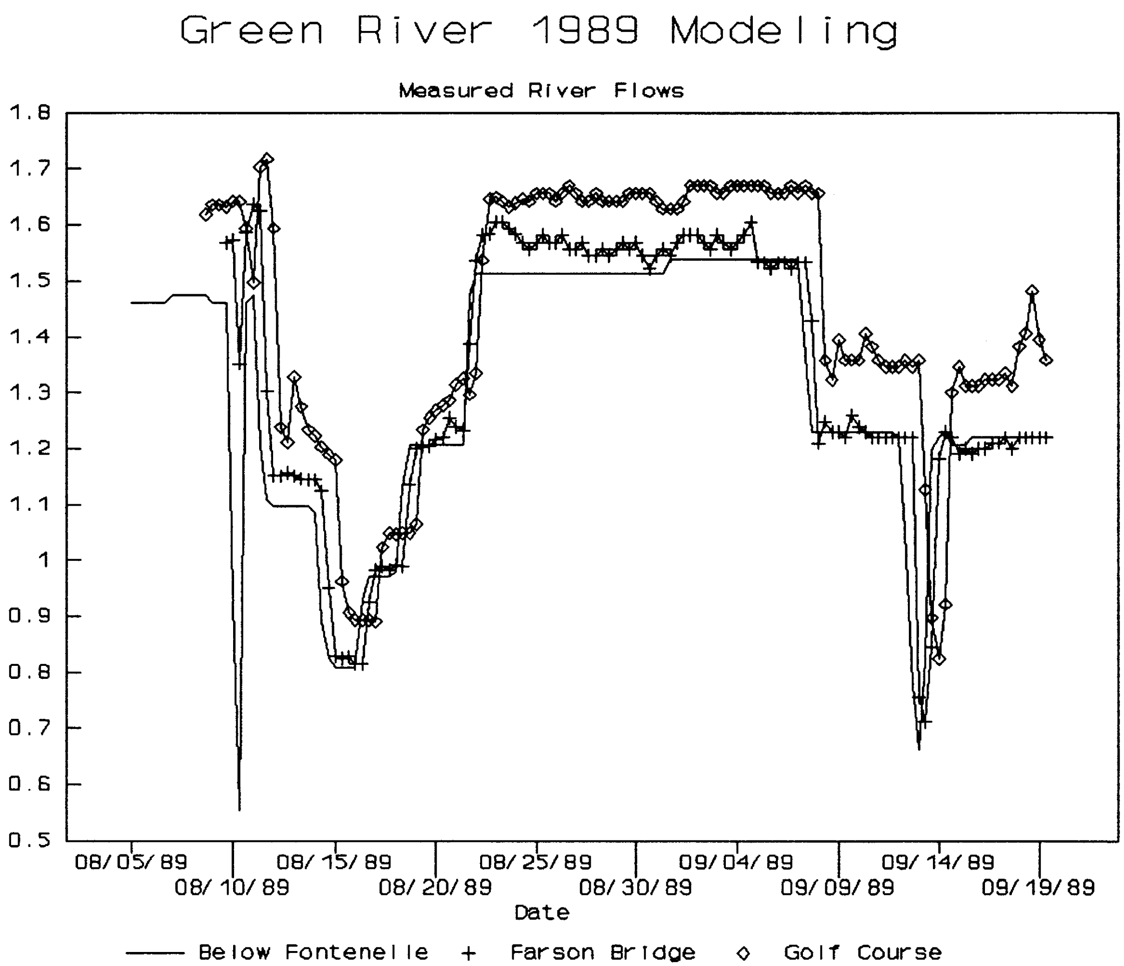


Figure 16. Green River 1989 Reservoir Release, Measured River Flow Hydrographs.

The J349 model was used to estimate conveyance loss which resulted from bank storage in each of the two reaches considered. Tabulations of the parameters determined from the calibration to be used for modeling each reach are given in Table V. This table shows aquifer properties input to the model were identical for the two reaches. Other than measured parameters, such as reach length and aquifer width, only the wave celerity and base flows were determined to vary between the two reaches. The reach below Farson Bridge was found to have generally higher wave celerities over the range of discharges considered in the calibration. Also note the disparity between the range of wave celerity determined for the reaches above and below Farson Bridge. The reason for this disparity is not obvious from the character of the river. Both reaches of the river were resolved to be modeled with the Case 2 - Finite Aquifer boundary condition.

Figure 17 presents the results of the model calibration for the reach above Farson Bridge. The calibration was limited to the period from August 9 to September 2, 1989 because only limited diversion data was available for the remainder of the period of study. The observed discharge hydrograph is shown to vary between approximately 1650 and 800 cubic feet per second. The model simulation results shown in Figure 17 are good. Despite the lack of stability in discharge during the initial portion of the database, prior to the first planned decrease in discharge from Fontenelle on

TABLE V

GREEN RIVER MODEL PARAMETERS

Input Parameter	Above Farson Bridge	Below Farson Bridge
Stream-aquifer boundary	Case 2	Case 2
Transmissivity (ft^2/d)	1,000	1,000
Storativity	0.10	0.10
Aquifer Width (ft)	3500	5000
Soil Retention	0.0	0.0
Wave Dispersion (ft^2/s)	235 - 5600	235 - 5600
Wave Celerity (ft/s)	2.0 - 3.75	3.55 - 3.62
Base Flow (cfs)	60	100
Length of Channel (mi)	24.2	37.6
Length of Alluvium (mi)	16.5	28.0

August 11, the model simulation predicts observed flows quite well during the remainder of the study period. The model calibration is shown to have a limited response to the range of flows observed. The model could not be calibrated to produce a simulated hydrograph which accurately predicts both the trough in the observed hydrograph (August 15 through 16) and the peak discharge observed subsequent to August 23. The base flow input to the model was adjusted to calibrate the predicted streamflow for the first incremental decrease in flow which is shown in Figure 17 to have occurred from August 12 to August 14.

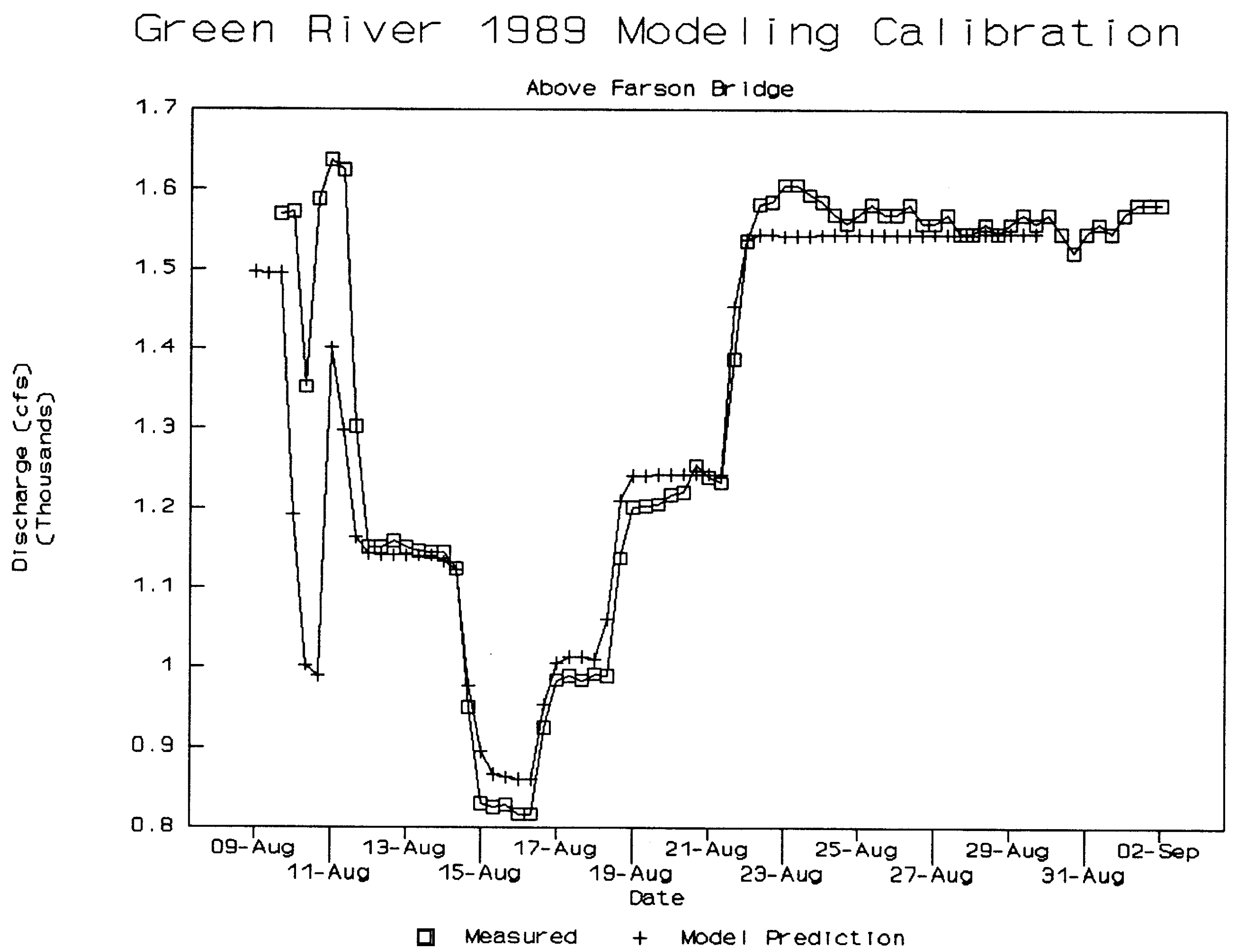


Figure 17. Green River Model Calibration of the Reach Above Farson Bridge.

The results of the model calibration for the downstream reach, between Farson Bridge and the Golf Course are presented in Figure 18. The character and period of the measured discharge hydrograph are similar to that of the upstream reach. However, the river has additional gains in this reach so the discharges have increased by roughly 100 cubic feet per second. As was the case for the upstream reach, the model simulation results are generally good. The lack of stability in discharge during the initial portion of the database, prior to the first planned decrease in discharge, became more profound in the lower reach. This portion of the measured hydrograph could not be replicated during the calibration. With the exception of the highest flows observed during the end of the study period, the model simulation predicts observed flows quite well.

The base flow input to the model was adjusted to calibrate the predicted streamflow for the incremental increases in flow shown in Figure 18 to have occurred from August 17 to August 23. The model calibration is shown to have a limited response to the range of flows observed. As was discussed for the upstream reach, the model could not be calibrated to produce a simulated hydrograph which accurately predicted both the observed trough and peak in the hydrograph. This limitation is seen to be more extreme for the lower reach. This was believed to be, in part, due to the degradation of the accuracy with which discharges were

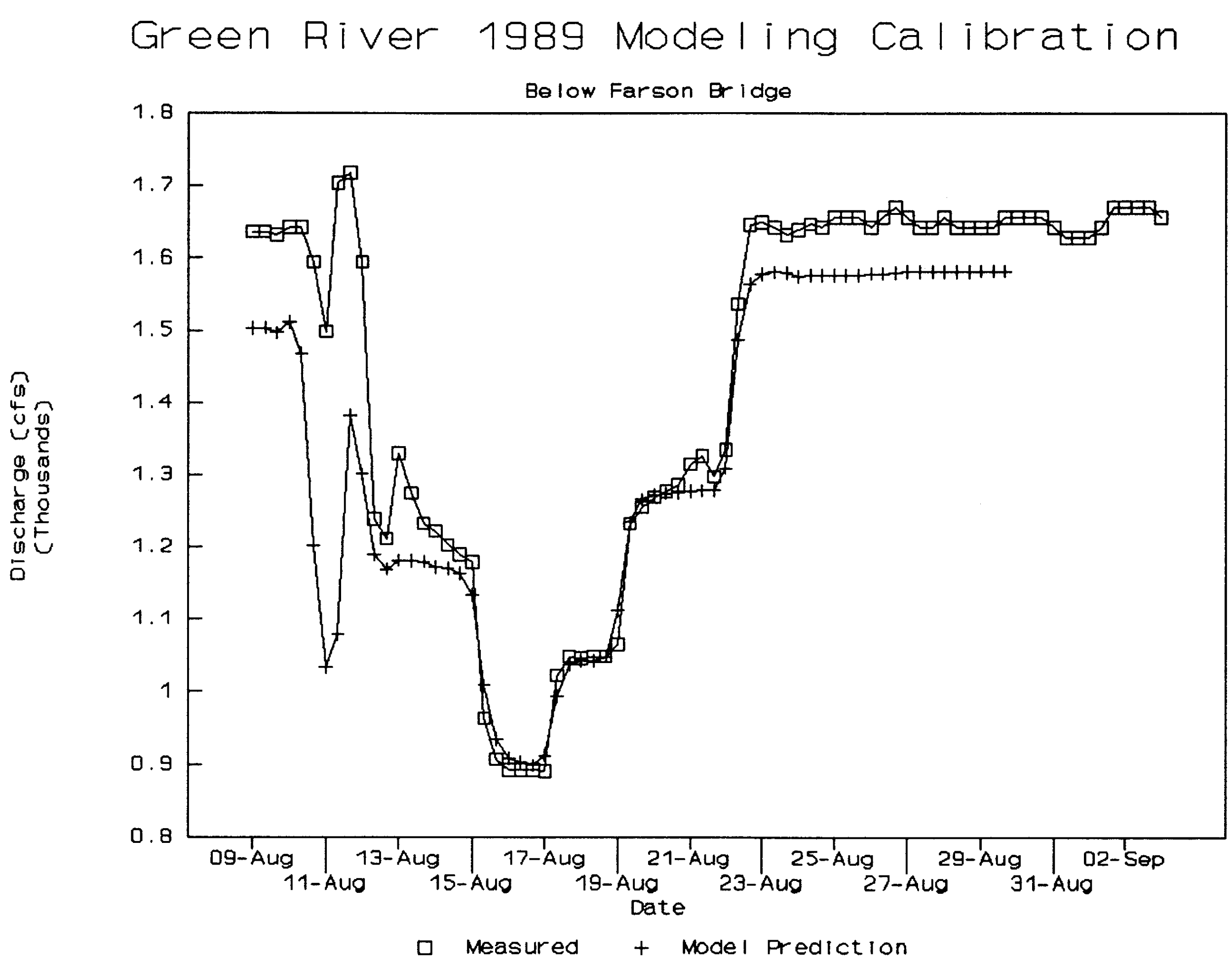


Figure 18. Green River Model Calibration of the Reach Below Farson Bridge.

measured from the deck of bridges as flows increased in the river. The effect of turbulence and eddies in the flow of the river due to the bridge piers was believed to have an increasing effect on the rating curves at higher flows. The quality of the discharge measurements was believed to be lower at the Golf Course gauge station, which may account for some of the discrepancy between the observed and simulated hydrographs during intervals of higher flow.

As a result of releases from Fontenelle Reservoir being decreased, rather than increased, for this investigation it was not possible to determine any specific loss to bank storage associated with the period of study as was done for the Piney Creek Study Area. As would be expected, gains to the river from the return of bank storage water were predicted in the model calibration. Because gains from the return of bank storage are contrary to the conclusions to be drawn from this investigation they are not reported here. Rather, the model calibration which resulted from the period of study was subsequently used to investigate bank storage loss from a range of simulated reservoir release rates and periods.

MODEL SIMULATIONS. The J349 Model was used to study bank storage loss for varying conditions of discharge on the Green River. By use of the model, 25 simulations were completed to determine bank storage loss on the Green River from Fontenelle Reservoir to the City of Green River. Bank storage loss was determined for augmentation flows of 100, 500, and 1000 cubic feet per second for each of six reservoir release periods: 1, 2, 3, 4, 5, and 6 days. The results of these model simulations were used to develop graphs which characterize bank storage loss along the river. Figure 19 shows the variation in the percent of bank storage returned to the river with respect to the length of the period of the reservoir release. Bank storage loss for reservoir releases of 100 cubic feet per second which varied from one to six days in duration are compared. As would be expected, the longer the duration of a release, the more prolonged is the period of recovery. Figure 19 also shows the decreasing effect of extending the period of reservoir release has on the recovery period with increasing duration of reservoir release. For example, note the difference between the affect of increasing the period of release from one to two days verses increasing the period of recovery from five to six days. This result is due to the inability of longer periods of increased river stage to proportionally increase loss to bank storage. An incremental increase in the length of the reservoir release has a diminishing effect on the loss to bank storage as the period of the reservoir release becomes longer.

The independent affect of varying the magnitude of the reservoir release on bank storage loss was also considered in the model simulations completed for the Green River. Figure 20 shows the result of varying the magnitude of the reservoir release on the percentage of the bank storage water returned

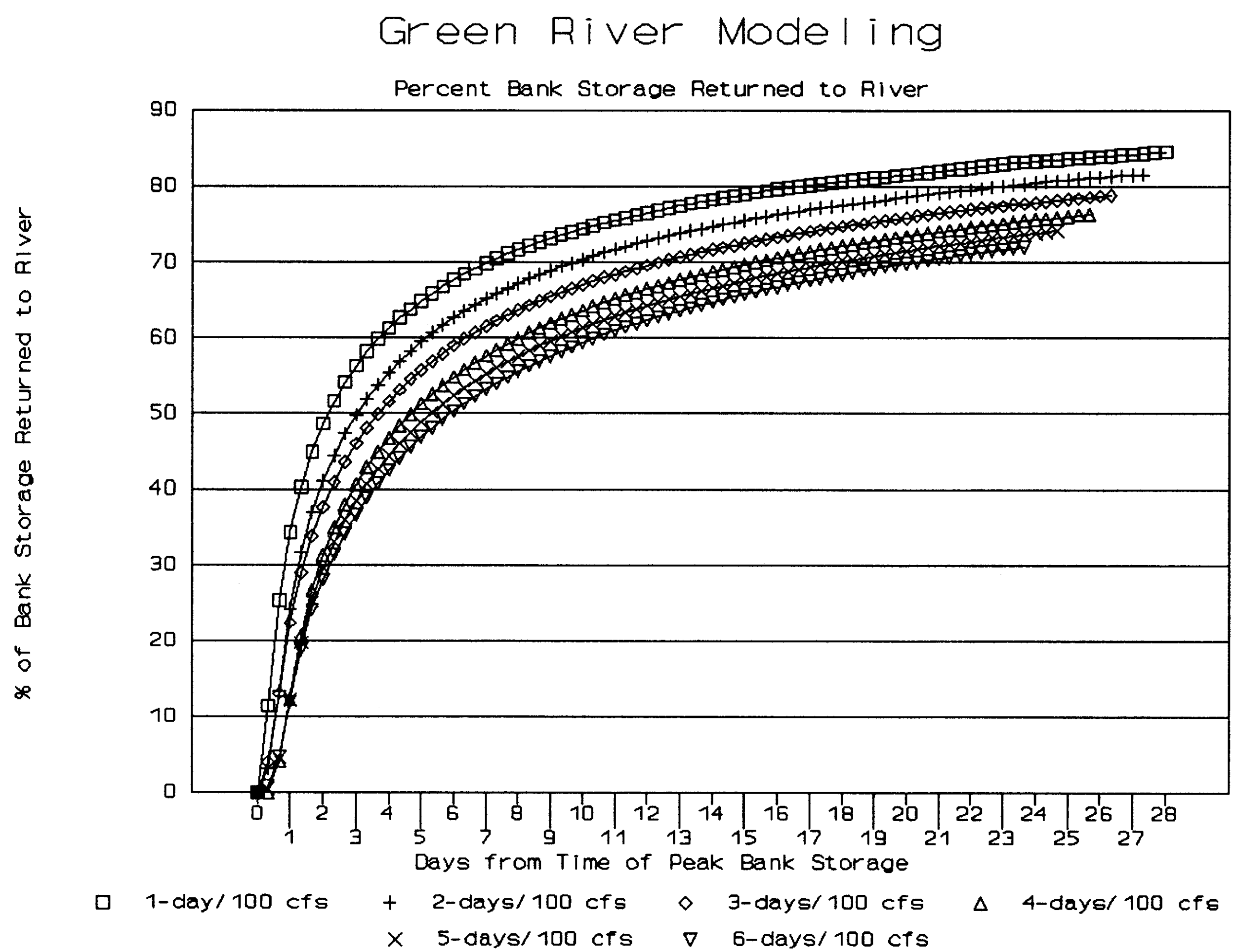


Figure 19. Green River Modeling Results of the Percent of Bank Storage Returned to the River with Varying Duration of Reservoir Release.

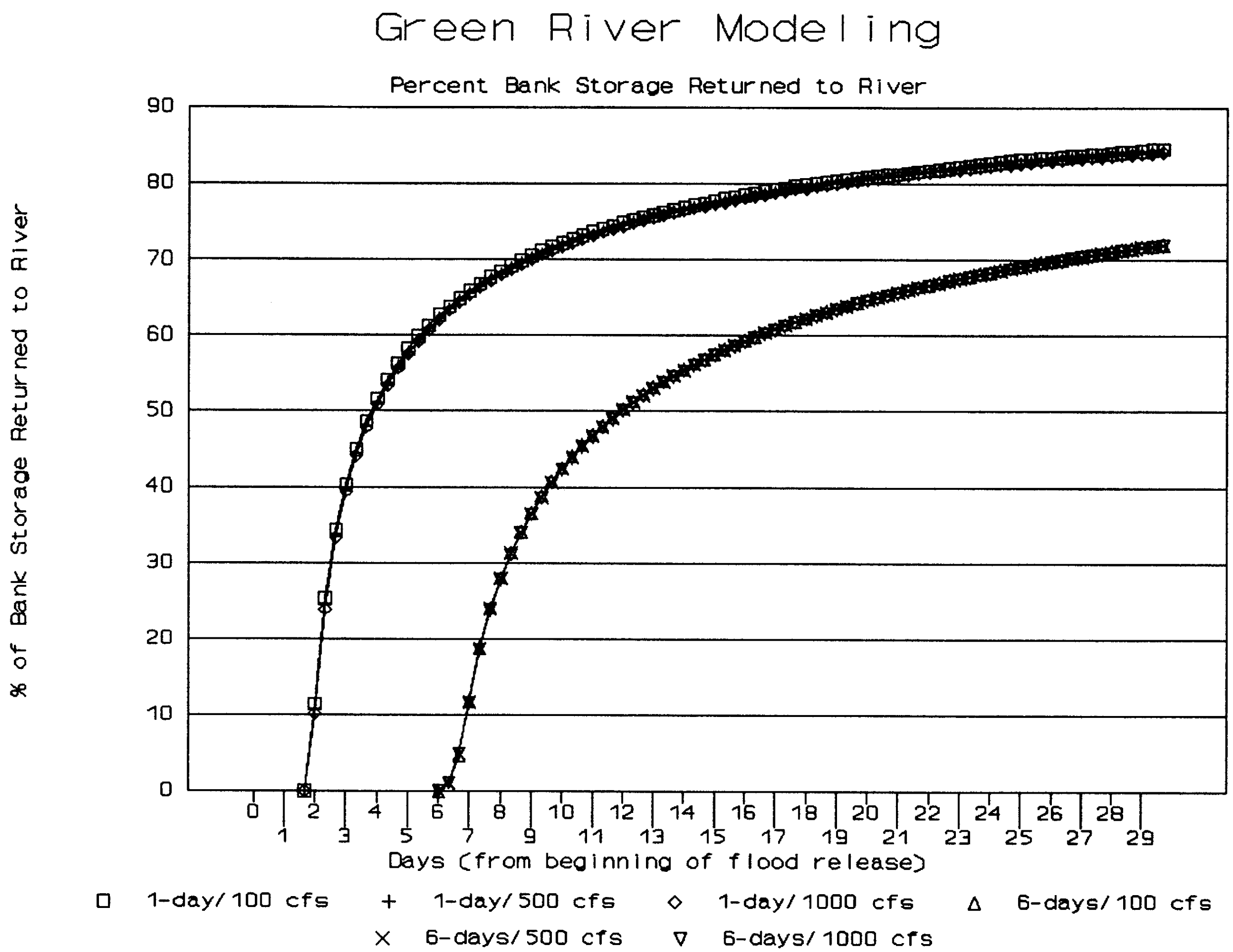


Figure 20. Green River Modeling Results of the Percent of Bank Storage Returned to the River with Varying Magnitude of Reservoir Release.

to the river with time of recovery. The model simulations predict no change in the rate of return with varying magnitude of reservoir release. These results are not surprising considering the assumptions of linearity and superposition inherent in the model. Bank discharge hydrographs associated with reservoir releases of constant duration and increasing magnitude are shown in Figure 21. Note that positive discharge is defined as flow from the aquifer to the river.

The bank storage discharge hydrographs shown in Figure 21 are presented to clarify the phenomena of bank storage loss. It can be seen that bank storage loss is a time dependent phenomena. The magnitude of the loss associated with this phenomena is conditional on the length of the recovery period allowed under the administrative scheme used to protect other water rights on the water course. The longer the recovery period allowed, the lower loss will be associated with any given magnitude of augmentation flow.

To this point the discussion of the model simulations of bank storage loss have focused on the percentage of the loss returned to the river with time. Alternatively, the model results may be used to view bank storage loss from the perspective of the percent of the flood volume (or volume of augmentation flow) which remains in bank storage with time. Figure 22 presents this perspective for the three magnitudes of reservoir release studied. It can be seen that the percent of the total flood volume (or volume of augmentation flow)

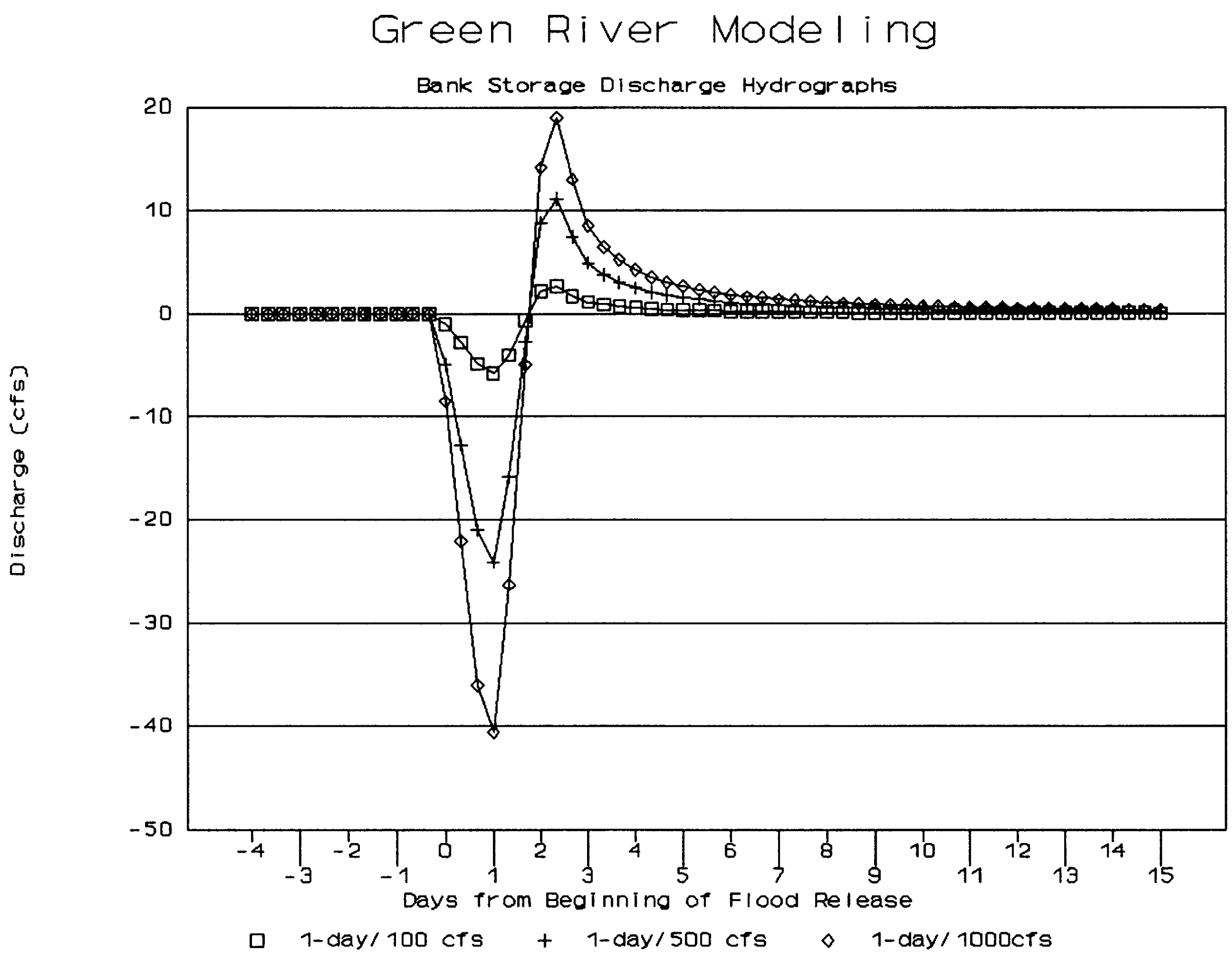


Figure 21. Green River Modeling Results, Bank Storage Discharge Hydrographs.

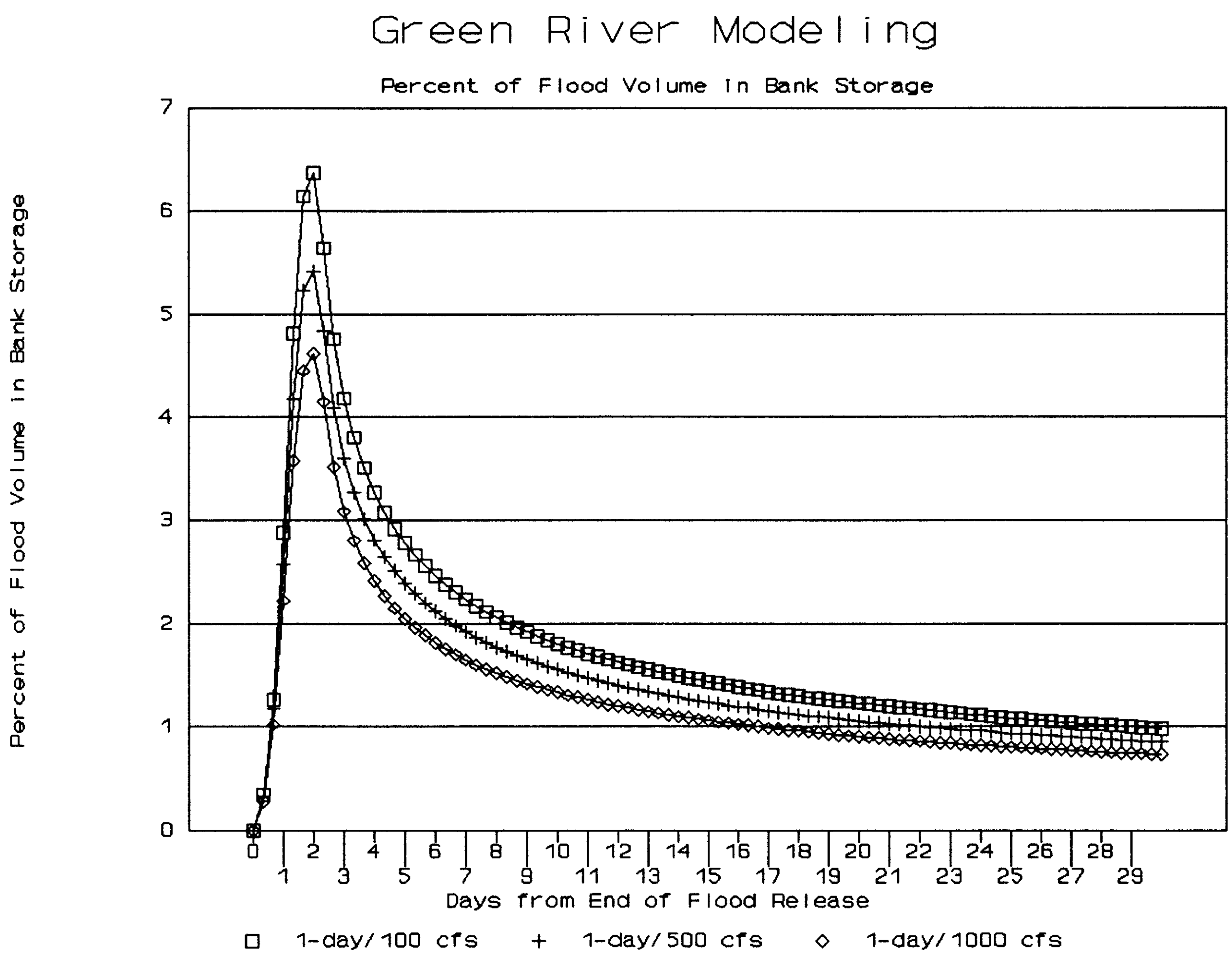


Figure 22. Green River Modeling Results, Percent of Flood Volume in Bank Storage with Varying Magnitude of Reservoir Release.

which remains in bank storage increases with decreasing discharge. Comparing Figure 22 with Figure 20 reveals an important distinction between the two perspectives. Figure 20 shows the percentage of the total bank storage volume taken which returns to the river is constant with respect to the magnitude of the reservoir release. Figure 22 shows the percent of the flood volume which remains in bank storage is dependent on the magnitude of the reservoir release. Although at first this distinction may seem frivolous, it shows conveyance loss to bank storage is dependent on both duration of the reservoir release and the magnitude of the flood release.

The percent of a flood volume which remains in bank storage with time is shown to vary with the duration of the reservoir release in Figure 23. This figure shows that the percentage of the flood volume which remains in bank storage only varies significantly with the magnitude and duration of the reservoir release for a period of up to 15 to 20 days from the beginning of the flood release. This happens because the rise in bank storage loss with increasing duration of reservoir release is offset by the associated rise in flood volume.

Three factors will contribute to the estimate of bank storage loss for a given reservoir release: (1) the magnitude of the reservoir release; (2) the duration of the reservoir release; and, (3) the recovery period allowed. Although the

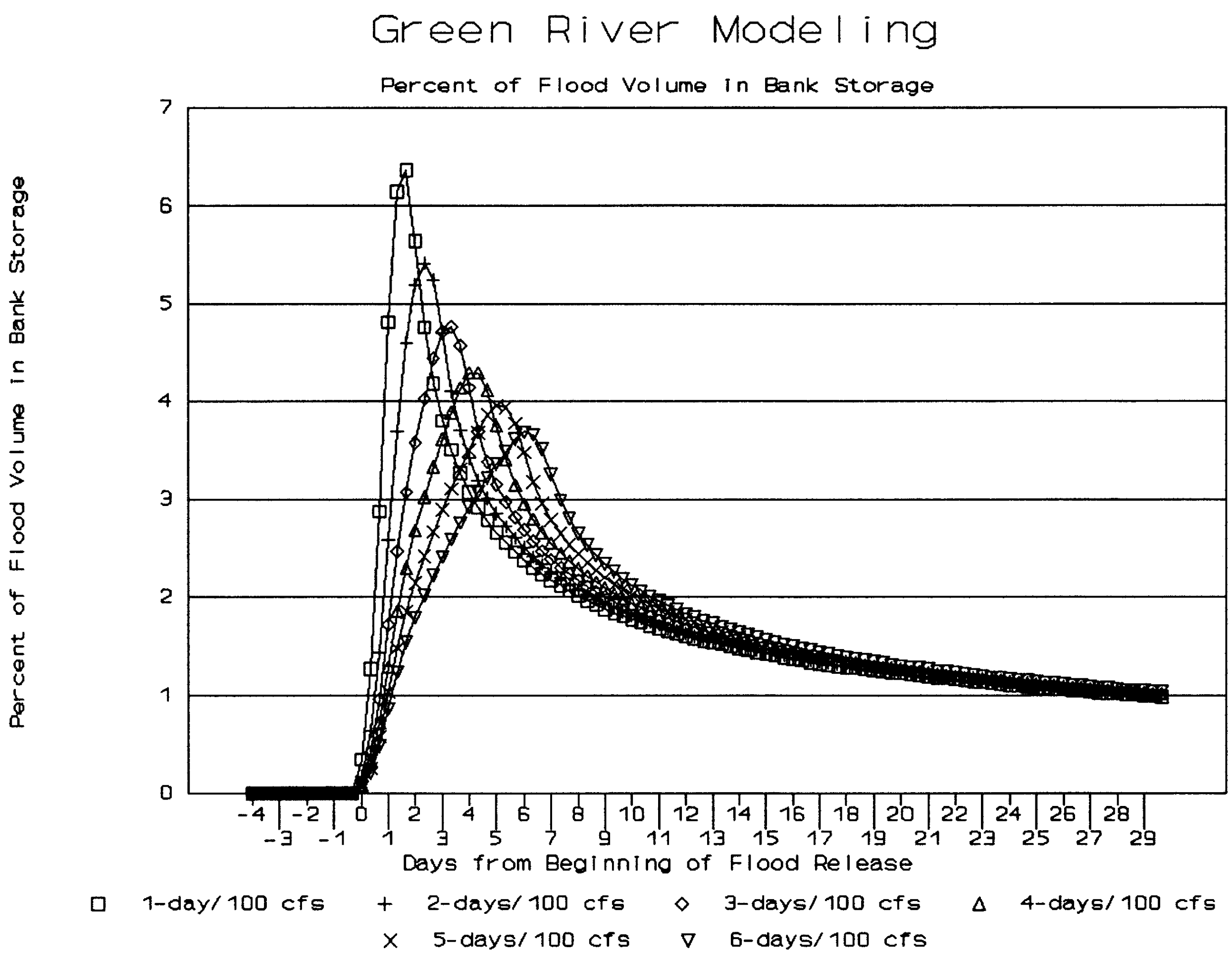


Figure 23. Green River Modeling Results, Percent of Flood Volume in Bank Storage with Varying Duration of Reservoir Release.

results of the model simulations presented in Figures 19 through 23 allow some general inferences to be drawn about bank storage loss, the results provided in these figures are specific to the Green River Study Area. A prediction of the conveyance loss to bank storage for a reservoir release in this study area dictates a judgment be made of an allowable recovery period for the return of this water. The results demonstrate conveyance loss to bank storage on the Green River will vary from over six percent to less than one percent of the volume of a reservoir release for the range of flows considered.

EVAPORATION LOSS. In contrast to the temporal nature of bank storage loss, evaporation loss is a permanent loss to the stream system. Evaporation loss for this study was estimated using pan evaporation data from the Green River meteorological station. The average daily evaporation loss for the month of August in 1989 was calculated to be 0.283 inches per day. This value equates to 0.00101 feet squared per second per mile assuming a pan coefficient of 0.70. Based on the data acquired for the discharge rating curves, the average increase in river width for an increase in discharge of 100 cubic feet per second is estimated to be approximately two feet. The length of the river from Fontenelle Reservoir was determined to be 61.8 miles. Therefore, the resulting loss to evaporation for a one day reservoir release of 100 cubic feet per second is calculated to be 0.12 cubic feet per second per day or 0.12 percent of the volume of the reservoir release. This loss is an order of magnitude less than calculated bank storage loss for an equivalent reservoir release after a thirty day recovery period.

COMPARISON OF RESULTS

The database acquired for the two study areas chosen for this investigation were altogether different in character. The database for the Piney Creek study area included significantly larger diversion rates relative to the total streamflow. The hydrologic system in the Piney Creek study area was influenced to a greater degree by groundwater than was the Green River study area. The streamflows considered in the Green River hydrologic system were considerably greater than those encountered in Piney and Clear Creeks. All of these factors contributed to the differences in the results obtained from the model studies completed for each area.

Despite the differences in the character of the two study areas, the modeling results obtained from both areas were generally indicative of the basic assumptions and limitations of the J349 Model. The difficulties experienced in obtaining accurate model simulations of the stream reaches (particularly the two most upstream reaches) in the Piney Creek study area were believed to be a consequence of the model assumptions that the groundwater water table is initially flat and in equilibrium with the stream. The magnitude of the gains relative to the streamflows influent to the stream reaches suggested these assumptions made the J349 model incapable of simulating the existing hydrologic system with high accuracy.

The modeling results obtained from the database for the Green River study area were generally much better than the results obtained from the Piney Creek study area. This improvement in the model simulations for the Green River can be attributed to the fact that the assumptions intrinsic to the model were not as contradictory to the hydrologic system under consideration. The model calibration of the Green River data merited the completion of model simulations to evaluate a variety of streamflow conditions. These model simulations were also shown to evidence the basic assumptions of linearity and superposition which are intrinsic to the J349 model.

The model simulations completed for the Green River study area can not be compared directly with the results obtained for the Piney Creek study area without making some assumption as to an appropriate recovery period. The recovery period for the Piney Creek data simulations was limited to the period of the database. If the recovery period for the Green River simulations are similarly truncated the percent of the flood volume lost to bank storage ranges from roughly three to four percent depending on the duration of the flood release and its magnitude. This equates to a loss to bank storage of roughly 0.05 percent per mile. Although this loss is significantly less than the loss estimated for the Piney Creek study area, these results are consistent with the relative differences in stream and aquifer properties between the two study areas.

The measured evaporation rates at the two areas were very comparable. This is consistent with the similarity in pan evaporation data available from the meteorological stations near the two study areas. In both study areas calculated evaporation loss from the surface of the water course was found to be insignificant relative to the estimates of bank storage loss calculated.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

The measurement and prediction of conveyance losses in natural water courses is an important goal for the effective management of Wyoming's water resources. Conveyance losses may be influenced by a wide variety of factors and have been quantified by various methods. This investigation was intended to develop a methodology, based on a hydrologic model, whereby conveyance losses in Wyoming streams could be assessed.

The measurement of conveyance losses to Wyoming streams is applicable to a broad range of management practices. The use of conveyance loss modeling to better schedule the duration and magnitude of reservoir releases to downstream appropriators is an obvious water management application of the methods evaluated in this investigation. The assessment of conveyance losses is also pertinent to an evaluation of the feasibility of a change in the point of diversion for a water right whereby a deviation in natural streamflows resulting from the transfer potentially harms other appropriators. The methods described in this thesis are generally applicable to such customary water management issues.

Less traditional applications for the J349 model and the methods described in this paper may also be envisioned. As one example, an understanding of conveyance losses to bank storage could be important to an evaluation of the feasibility of providing instream flows. Instream flow rights in Wyoming are defined at the downstream end of a stream segment. The methods reported in this investigation could be used to provide an evaluation of the quantity and timing of water availability for an instream flow water right filing by generating simulated streamflow records at the downstream end of the instream flow segment based upon a known upstream hydrograph.

The applicability of the J349 model for use in water management objectives in Wyoming is considerable. The conclusions attained from this investigation provide a framework from which the feasibility of using the J349 model for diverse applications may be judged.

CONCLUSIONS

The results of this investigation and of those previous investigations considered in the literature review completed in conjunction with this paper yield the following conclusions:

1. The primary advantage to using a hydrologic model to assess conveyance loss to bank storage is that steady flows are not required to obtain reliable estimates of this loss.

2. The J349 model used for this investigation is capable of providing reasonable estimates of conveyance losses to bank storage using measured streamflow data and commonly available information such as U.S.G.S. 7.5 minute quadrangle maps.

3. The merit of bank storage loss estimates derived using the J349 model are constrained by the assumptions inherent in the model. Most significant is the assumption that the water table in the aquifer is initially level and in equilibrium with the stream. This assumption is most problematic for the model simulation of hydrologic systems in which the aquifer contributes a significant share of the streamflow through groundwater inflow.

4. Loss to bank storage is a transient phenomena. The conveyance loss due to bank storage is defined by the recovery period allowed for the return of water from the aquifer to the water course.

5. Generally the following relationships regarding bank storage loss were determined to result from reservoir releases to the Green River:

a. The percentage of the total volume of a reservoir release lost to bank storage decreases as the magnitude of the reservoir release increases.

b. The percentage of the total volume of a reservoir release lost to bank storage decreases as the duration of the reservoir release increases.

c. The percentage of the flood volume which remains in bank storage only varies significantly with the magnitude and duration of the reservoir release for some finite period of time from the beginning of the flood release. This occurs because the rise in bank storage loss with increasing magnitude or duration of reservoir release is eventually offset by the associated rise in flood volume.

6. Hydrologic models, such as the J349 model, provide a framework for which conveyance loss along a stream-aquifer system may be evaluated. Even for those cases where the model does not accurately simulate observed streamflows, a better understanding of the physical phenomena which affect conveyance losses is obtained.

7. Evaporation loss was determined to be insignificant in comparison to bank storage loss for the water courses considered in the two study areas addressed in this investigation.

RECOMMENDATIONS

Several recommendations can be advanced regarding future work with conveyance losses employing the J349 Model or other models discussed in this paper. These recommendations are:

1. Perform additional research concerning the initial

boundary conditions present in the aquifer on streams which are determined to be gaining would provide valuable insight to future model studies.

2. Develop a standard definition of conveyance loss based on the incremental approach such that incremental losses are assessed with respect to a base flow scenario defined such that other appropriators may not be harmed.

3. Perform additional research to determine the physical phenomena which govern the loss of bank storage water. The central goal of this research would be the development of a practical methodology to assess recovery periods for ideal stream-aquifer systems described in the literature.

4. Evaluate the methodology presented in this paper on additional Wyoming stream-aquifer systems which may provide additional insight to the definition of conveyance loss to bank storage.

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APPENDIX A

J349 MODEL INPUT FILE STRUCTURE

Card No.	Description	Variable	Columns (Justify)
1	Information card. Generally with basin name, period of record, date of run, etc.	INFO (20)	1-80 (F)
2	Upstream station number (USGS 8 digit).	STANO1 (2)	1-8 (L)
	Station name (48 characters or less).	STANM1 (12)	11-58 (L)
3	Identify input source of hydrograph data = 1 for cards = 2 for disk	ISOURC	10
	Identify objective of model run = 1 for bank storage discharge hydrograph = 2 for bank storage discharge and downstream hydrographs	IDATA	20
4	Number of reaches in this model run.	NRCHS	1-10 (R)
	Number of upstream reaches previously studied or numbered.	NPREVR	11-20 (R)
	Duration of study period, in days.	ITMAX	21-30 (R)
	Duration of time step, in hours (must be 24.0 for DISK option.	DT	31-40 (R)

Card No.	Description	Variable	Columns (Justify)
5	Starting date of study period		
	month	INITMO	1-10 (R)
	day	INITDY	11-20 (R)
	year	INITYR	21-30 (R)
	Ending date of study period		
	month	LASTMO	31-40 (R)
	day	LASTDY	41-50 (R)
	year	LASTYR	51-60 (R)
	Number of record spaces to be reserved for longest station record, required for DISK option.	NRECDS	61-70 (R)
6	Number of stage-discharge rating table points at upstream station. (max of 20)	NUSRP	1-10 (R)
	Shifts are used to correct rating table. Permitted for CARDS option.	ZUSHIFT	11-20 (F)
	Minimum flow during study period at upstream station, in ft^3/s. (DON'T READ)	USQB	21-30 (R)
7	Stage-discharge rating table; stage, in ft., discharge in ft^3/s. Repeat in pairs, using all 80 columns of card until table is completed. Number of pairs equals NUSRP.	SRAT(1,) QRAT(1,)	1-10, 11-20,.. 61-70, 71-80 (R)

Card No.	Description	Variable	Columns (Justify)
8	Daily shift, in ft., that is needed to adjust actual rating to rating table. Permitted for CARDS option. Required when ZUSHIFT is TRUE.	SHIFT(1,)	1-10,.., 71-80 (R)
9	Upstream discharge hydrograph, in ft^3/s. The first value corresponds to DT hours after study period starts. Required for CARDS option.	USQ()	1-10,.., 51-60 (R)
10	Information card. Generally used to identify reach.	INFO(20)	1-80 (F)
11	Downstream station of reach		
	Number (USGS 8 digit)	STANO2 (2)	1-8 (L)
	Name	STANM2 (12)	10-58 (L)
12	Identify stream-aquifer boundary conditions of reach = 1 for semi-infinite aquifer = 2 for finite aquifer = 3 for stream lined with permeable confining bed and semi-infinite aquifer	ICASE	10
	An account is made for streamflow losses to diversions or well pumpage. Permitted for ROUTE option.	ZLOSS	11-20 (F)
	Discharge hydrographs are to be plotted on line printer. (USE FALSE)	ZPLOT	21-30 (F)

Card No.	Description	Variable	Columns (Justify)
	Hydrographs are to be tabulated. (USE TRUE)	ZPRINT	31-40 (F)
	Downstream hydrograph values are to be punched. (USE FALSE)	ZPUNCH	41-50 (F)
	Multi-linear routing option is to be used.	ZMULT	51-60 (F)
	Observed downstream hydrograph is input for comparison.	ZDSQO	61-70 (F)
	Observed downstream hydrograph is output for comparison.	ZOUTPUT	71-80 (F)
13	Estimated travel time, in hours, of flood wave for reach. Required when IDDATA = 1. Otherwise, information only.	TT	1-10 (R)
	Channel length of reach, in miles.	CHLGTH	11-20 (R)
	Alluvial length of reach, in miles.	ALLGTH	21-30 (R)
14	Transmissivity of aquifer for reach, in ft^2 per day.	T	1-10 (R)
	Storage coefficient of aquifer, in dimesionless terms.	SS	11-20 (R)

Card No.	Description	Variable	Columns (Justify)
	Fraction of bank storage retained in aquifer. This water may go to satisfy a soil moisture deficiency above the original water table or to plants.	SOILRT	21-30 (R)
15	Wave dispersion coefficient. Generally describes the spreading of a hydrograph pulse from the upstream to downstream points of a reach. Realistic value needed only for ROUTE option.	XK	1-10 (R)
	Wave celerity. Generally describes the travel time between ends of a reach for a hydrograph pulse. Realistic value needed only for ROUTE option.	CZERO	11-20 (R)
	Error criteria, in ft^3/s, for closure in iteration process used in ROUTE option. A value of 1.0 is commonly used.	TOLRNC	21-30 (R)
	Retardation coefficient, in ft. Generally describes the impedance to flow between a stream and an aquifer due to a permeable confining bed covering the streambank. Use only in Case 3 option.	XKA	31-40 (R)
	Width of aquifer from stream to boundary, in ft. Used only in Case 2 option.	XL	41-50 (R)

Card No.	Description	Variable	Columns (Justify)
16	Similar to card no. 6, except for downstream station.		
	Number of stage-discharge rating table points at downstream station. (Maximum of 20).	NDSRP	1-10 (R)
	Shifts are used to correct rating table. Permitted for CARDS option.	ZDSHFT	11-20 (F)
	Minimum flow during study period at downstream station, in ft^3/s.	DSQB	21-30 (R)
17	Stage-discharge rating table; stage, in ft, discharge, in ft^3/s. Repeat in pairs, using all 80 columns of card, until number of pairs equals NDSRP. Similar to card set no. 7 for downstream station.	SRAT(2,), QRAT(2,)	1-10, 11-20,.. 61-70, 71-80 (R)
18	Daily shift, in ft, that is needed to adjust actual rating to rating table. Similar to card set no. 8, except for downstream station. Permitted for CARDS option. Required when ZDSHIFT is TRUE.	SHIFT(2,)	1-10,.., 71-80 (R)
18a	Expected lower limit of flow to be routed in this reach.	QMIN	1-10 (R)
	Expected upper limit of flow to be routed in this reach. Required when ZMULT is TRUE.	QMAX	11-20 (R)

Card No.	Description	Variable	Columns (Justify)
18b	Wave celerity - discharge rating table; wave celerity, in ft/s, discharge in ft^3/s. Repeat in pairs, using all 80 columns. Two cards, up to 8 pairs of data, are required. Required when ZMULT is TRUE.	CORAT( ) COZRAT( )	1-10, 11-20,.. 61-70, 71-80 (R)
18c	Wave dispersion - discharge rating table; wave dispersion in ft^2/s, discharge, in ft^3/s. Repeat in pairs, using all 80 columns. Two cards, up to 8 pairs of data are required. Required when ZMULT is TRUE.	XKRAT( ) XKQRAT( )	1-10, 11-20,.. 61-70, 71-80 (R)
19	Downstream discharge hydrograph, in ft^3/s. The first value corresponds to DT hours after study period starts. Similar to card set no. 9 except for downstream station. Required for NON-ROUTE option with card input.	DSQ( )	1-10,.., 51-60 (R)
20	Required for DISK option, omitted for CARDS option. (OMMIT)		
21	Number of diversions and wells for reach (CARDS option). Number of diversions and wells for a given reach in a given water year (DISK option). NLOSS is limited to 25. Permitted for ROUTE option. Required when ZLOSS defined as TRUE.	NLOSS	1-10 (R)

Card No.	Description	Variable	Columns (Justify)
22	Required when NLOSS greater than 0. Number of cards equals NLOSS value.		
	Distance from stream, in ft. A direct diversion is assumed for 10 ft. or less.	X( )	1-10 (R)
	Rate of diversion or well pumpage, in ft^3/s. A negative value assumes withdrawl. A positve value assumes recharge.	QLOSS( )	11-20 (R)
	Starting date of diversion or well pumpage		
	month	JIMO	21-30 (R)
	day	JIDY	31-40 (R)
	year	JIYR	41-50 (R)
	Ending date of diversion or well pumpage		
	month	JLMO	51-60 (R)
	day	JLDY	61-70 (R)
	year	JLYR	71-80 (R)

If NRCHS, specified on card no. 4, is greater than 1, the data set on cards 10 through 22 is repeated until the number of data sets equals NRCHS. Each data set represents a reach and its downstream station.

APPENDIX B

J349 MODEL OUTPUT FILE EXAMPLE

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GREEN RIVER 1989 - FIELD WORK - RUN9

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PROPERTIES AND CHARACTERISTICS OF MODEL RUN

BEGINNING DATE	8/ 5/1989
ENDING DATE	8/29/1989
OBJECTIVES ARE TO COMPUTE - FOR EACH REACH	1) DOWNSTREAM HYDROGRAPH
	2) BANK STORAGE DISCHARGE HYDROGRAPH
LENGTH OF TIME STEP (HOURS)	8.0
NUMBER OF REACHES IN THIS RUN	2
NUMBER OF UPSTREAM REACHES	0
BASE FLOW AT UPSTREAM STATION (CFS)	.0

UPSTREAM STATION DATA

RATING TABLE

STAGE	DISCHARGE
10.60	404.00
10.70	472.00
10.80	547.00
10.90	628.00
11.00	715.00
11.10	808.00
11.20	902.00
11.30	1002.00
11.40	1108.00
11.50	1220.00
11.60	1332.00
11.70	1450.00
11.80	1577.00
11.90	1711.00
12.00	1850.00

GREEN RIVER 1989 - FROM FONTENELLE RESERVOIR TO FARSON BRIDGE (REACH NO. 1)

PROPERTIES AND CHARACTERISTICS OF REACH

LENGTH OF CHANNEL (MILES)	24.2
LENGTH OF ALLUVIUM (MILES)	16.5
TRAVEL TIME (ESTIMATED HOURS)	10.0
TRAVEL TIME TO BEGINNING OF RESPONSE (HOURS)	10.6 Cumulative from start of first reach = .44 DAYS
TRAVEL TIME TO CENTER OF RESPONSE (HOURS)	13.7
TRAVEL TIME BETWEEN BREAKS IN HYDROGRAPHS (HOURS)	12.2
NUMBER OF SUBREACHES USED IN COMPUTATIONS	2
TRANSMISSIVITY OF AQUIFER (SQ.FT./DAY)	1000.0
STORAGE COEFFICIENT OF AQUIFER (CU.FT./CU.FT)	.10
AQUIFER IS ASSUMED TO BE 3500. (FT) WIDE (STREAM TO BOUNDARY)	CASE 2
SOIL RETENTION FACTOR	.00
BASE FLOW AT DOWNSTREAM STATION	60.0
MINIMUN EXPECTED DISCHARGE TO BE ROUTED	50.0
MAXIMUN EXPECTED DISCHARGE TO BE ROUTED	2100.0

CELERITY AND DISPERSION RATING TABLE	W. CELERITY	DISCHARGE	DISP. COEF.	DISCHARGE
	2.00	50.0	235.0	50.0
	3.45	300.0	890.0	300.0
	3.50	600.0	1675.0	600.0
	3.55	900.0	2460.0	900.0
	3.60	1200.0	3245.0	1200.0
	3.65	1500.0	4030.0	1500.0
	3.70	1800.0	4815.0	1800.0
	3.75	2100.0	5600.0	2100.0

FAMILY OF FLOW ROUTING UNIT-RESPONSE FUNCTIONS

NO.	W. CELERITY FT/SEC	DISP.COEF SQ FT/SEC	TRAVEL TIME TIME STEPS	DISCHARGE CU FT/SEC	ORDINATES
1	2.00	235.0	2	1075.0	1) .7814 2) .2186
2	3.75	5600.0	1	2100.0	1) .7898 2) .2102

STREAM-AQUIFER UNIT-RESPONSE FUNCTION
NOTE: CLOSURE WAS NOT OBTAINED FOR THE FIRST 4 NUMBERS.
COMPUTATIONS WERE MADE USING CASE 1 CONDITIONS FOR THESE NUMBERS.
NOTE: THIS RESPONSE FUNCTION (EXPONENTIAL DECAY TYPE IS EVALUATED FOR 18.5 HALF-LIVES.
IT HAS 75 ORDINATES.

1) -.013820	2) -.007979	3) -.006180	4) -.005223	5) -.004606	6) -.004166
7) -.003833	8) -.003568	9) -.003351	10) -.003170	11) -.003016	12) -.002881
13) -.002764	14) -.002659	15) -.002566	16) -.002482	17) -.002405	18) -.002336

19) -.002272	20) -.002213	21) -.002158	22) -.002107	23) -.002060	24) -.002016
25) -.001974	26) -.001935	27) -.001898	28) -.001863	29) -.001830	30) -.001799
31) -.001769	32) -.001741	33) -.001714	34) -.001688	35) -.001664	36) -.001640
37) -.001617	38) -.001596	39) -.001575	40) -.001555	41) -.001535	42) -.001517
43) -.001499	44) -.001482	45) -.001465	46) -.001449	47) -.001433	48) -.001418
49) -.001403	50) -.001389	51) -.001375	52) -.001362	53) -.001349	54) -.001336
55) -.001324	56) -.001312	57) -.001300	58) -.001289	59) -.001278	60) -.001267
61) -.001256	62) -.001246	63) -.001236	64) -.001226	65) -.001217	66) -.001207
67) -.001198	68) -.001189	69) -.001181	70) -.001172	71) -.001164	72) -.001156
73) -.001148	74) -.001140	75) -.001132			

DOWNSTREAM STATION DATA

RATING TABLE

STAGE	DISCHARGE
9.30	549.00
9.40	600.00
9.50	654.00
9.60	713.00
9.70	776.00
9.80	844.00
9.90	917.00
10.00	996.00
10.10	1081.00
10.20	1172.00
10.30	1269.00
10.40	1374.00
10.50	1486.00
10.60	1606.00
10.70	1735.00
10.80	1872.00
10.90	2019.00

SUMMARY OF ITERATION DATA FOR ROUTING OPTION

	CHANGES BETWEEN ITERATIONS		VOLUMES AT END OF ITERATION	
ITERATION NO.	MAXIMUM CHANGE IN BANK STORAGE DISCHARGE (CFS)	ABSOLUTE CHANGE IN BANK STORAGE VOLUME (CFS - DAYS)	NET VOLUME OF BANK STORAGE (CFS - DAYS)	VOLUME OF FLOW AT DOWNSTREAM STATION (CFS - DAYS)
1	18.2	110.	-92.	32262.
2	.1	1.	-91.	32262.

CLOSURE WAS OBTAINED AFTER 2 ITERATIONS

CRITERIA FOR CLOSURE	1.0 CFS
GREATEST CHANGE IN LAST ITERATION	.1 CFS

BANK STORAGE DISCHARGE AFFECTED DOWNSTREAM ROUTED DISCHARGE 2 TIME STEPS LATER.

REACH NO. 1:	BEGINS AT GAGING STATION 0000001	GREEN RIVER BELOW FONTENELLE RESERVOIR
	ENDS AT GAGING STATION 000002	FARSON BRIDGE

TOTAL STUDY PERIOD: BEGINS 8/ 5/1989
ENDS 8/29/1989

THIS SIMULATION PERIOD BEGINS 8/ 5/1989 AND ENDS 8/29/1989

SUMMARY OF STREAMFLOW DIVERSIONS AND DEPLETIONS

DISTANCE FROM STREAM	DISCHARGE	STARTING DAY	ENDING DAY
FEET	CFS	NUMBER OF DAY	FROM BEGINNING OF MODEL RUN
.00	-21.40	1	1
.00	-21.40	2	2
.00	-21.40	3	3
.00	-21.40	4	4
.00	-21.40	5	5
.00	-19.70	6	6
.00	-19.20	7	7
.00	-15.60	8	8
.00	-15.20	9	9
.00	-14.20	10	10
.00	-8.50	11	11
.00	-10.20	12	12
.00	-14.90	13	13
.00	-19.50	14	14
.00	-21.70	15	15
.00	-22.00	16	16
.00	-22.50	17	17
.00	-21.50	18	18
.00	-25.40	19	19
.00	-25.10	20	20
.00	-25.10	21	21
.00	-25.10	22	22
.00	-24.90	23	23

.00	-24.90	24	24
.00	-24.90	25	25

SUMMARY OF DATA AND RESULTS

DATE	TIME	OBSERVED UPSTREAM DISCHARGE	PREDICTED DOWNST. DISCHARGE	DOWNST. DISCHARGE	DOWNST. Q W/O BANK STORAGE AND LOSSES	BANK STORAGE DISCHARGE	DIVERSIONS AND DEPLETIONS	UPSTREAM STAGE	DOWNSTREAM STAGE	CHANGE IN STAGE
8/ 5/1989	800	1462.00	.00	38.60 *	60.00	-3.83	-21.40	11.71	8.30	.14
8/ 5/1989	1600	1462.00	.00	344.25	365.65	-15.42	-21.40	11.71	8.90	.47
8/ 5/1989	2400	1462.00	.00	1261.79	1287.03	-18.06	-21.40	11.71	10.29	.31
8/ 6/1989	800	1462.00	.00	1485.18	1522.00	-13.46	-21.40	11.71	10.50	.04
8/ 6/1989	1600	1462.00	.00	1482.54	1522.00	-10.82	-21.40	11.71	10.50	.00
8/ 6/1989	2400	1475.00	.00	1487.14	1522.00	-9.51	-21.40	11.72	10.50	.01
8/ 7/1989	800	1475.00	.00	1500.05	1532.27	-8.61	-21.40	11.72	10.51	.01
8/ 7/1989	1600	1475.00	.00	1504.09	1535.00	-7.85	-21.40	11.72	10.52	.00
8/ 7/1989	2400	1475.00	.00	1504.99	1535.00	-7.25	-21.40	11.72	10.52	.00
8/ 8/1989	800	1475.00	.00	1505.75	1535.00	-6.78	-21.40	11.72	10.52	.00
8/ 8/1989	1600	1475.00	.00	1506.35	1535.00	-6.38	-21.40	11.72	10.52	.00
8/ 8/1989	2400	1462.00	.00	1506.82	1535.00	-5.94	-21.40	11.71	10.52	-.00
8/ 9/1989	800	1462.00	.00	1496.95	1524.73	-5.58	-21.40	11.71	10.51	-.00
8/ 9/1989	1600	1462.00	.00	1494.66	1522.00	-4.88	-21.40	11.71	10.51	-.02
8/ 9/1989	2400	931.00	.00	1495.02	1522.00	-.03	-21.40	11.23	10.51	-.18
8/10/1989	800	554.00	.00	1191.77	1216.35	6.57	-19.70	10.81	10.22	-.31
8/10/1989	1600	1462.00	.00	1002.75	1022.48	1.98	-19.70	11.71	10.01	.02
8/10/1989	2400	1475.00	.00	988.93	1002.06	-7.51	-19.70	11.72	9.99	.30
8/11/1989	800	1208.00	.00	1401.17	1418.39	-6.39	-19.20	11.49	10.42	.06
8/11/1989	1600	1108.00	.00	1297.42	1324.12	-1.38	-19.20	11.40	10.33	-.13
8/11/1989	2400	1097.00	.00	1163.43	1189.02	-.38	-19.20	11.39	10.19	-.07
8/12/1989	800	1097.00	.00	1142.33	1159.31	-1.00	-15.60	11.39	10.17	-.01
8/12/1989	1600	1097.00	.00	1141.02	1157.00	-1.41	-15.60	11.39	10.17	-.00
8/12/1989	2400	1097.00	.00	1140.40	1157.00	-1.60	-15.60	11.39	10.17	.00
8/13/1989	800	1097.00	.00	1140.39	1157.00	-1.71	-15.20	11.39	10.17	.00
8/13/1989	1600	1097.00	.00	1140.20	1157.00	-1.75	-15.20	11.39	10.17	.00
8/13/1989	2400	1086.00	.00	1140.09	1157.00	-1.66	-15.20	11.38	10.16	-.00
8/14/1989	800	892.00	.00	1132.36	1148.31	-.26	-14.20	11.19	10.16	-.05
8/14/1989	1600	826.00	.00	1121.45	1137.31	2.21	-14.20	11.12	10.14	-.11
8/14/1989	2400	808.00	.00	977.54	992.00	3.30	-14.20	11.10	9.98	-.09
8/15/1989	800	808.00	.00	894.14	900.43	2.77	-8.50	11.10	9.87	-.04
8/15/1989	1600	808.00	.00	866.73	871.93	1.96	-8.50	11.10	9.83	-.01
8/15/1989	2400	808.00	.00	862.27	868.00	1.39	-8.50	11.10	9.83	-.00
8/16/1989	800	931.00	.00	859.76	868.00	.14	-10.20	11.23	9.82	.03

8/16/1989	1600	971.00	.00	859.19	868.00	-1.84	-10.20	11.27	9.82	.08
8/16/1989	2400	971.00	.00	954.05	964.11	-2.66	-10.20	11.27	9.95	.06
8/17/1989	800	971.00	.00	1005.51	1022.26	-2.26	-14.90	11.27	10.01	.02
8/17/1989	1600	971.00	.00	1013.44	1031.00	-1.83	-14.90	11.27	10.02	.00
8/17/1989	2400	981.00	.00	1013.84	1031.00	-1.73	-14.90	11.28	10.02	.00
8/18/1989	800	1130.00	.00	1009.67	1031.00	-3.04	-19.50	11.42	10.02	.05
8/18/1989	1600	1208.00	.00	1061.02	1082.25	-5.19	-19.50	11.49	10.08	.10
8/18/1989	2400	1208.00	.00	1208.51	1231.06	-5.53	-19.50	11.49	10.24	.06
8/19/1989	800	1208.00	.00	1241.11	1268.00	-4.54	-21.70	11.49	10.27	.01
8/19/1989	1600	1208.00	.00	1240.77	1268.00	-3.89	-21.70	11.49	10.27	.00
8/19/1989	2400	1208.00	.00	1241.76	1268.00	-3.54	-21.70	11.49	10.27	.00
8/20/1989	800	1208.00	.00	1242.11	1268.00	-3.29	-22.00	11.49	10.27	.00
8/20/1989	1600	1208.00	.00	1242.46	1268.00	-3.10	-22.00	11.49	10.27	.00
8/20/1989	2400	1208.00	.00	1242.71	1268.00	-2.95	-22.00	11.49	10.27	.00
8/21/1989	800	1208.00	.00	1242.40	1268.00	-3.17	-22.50	11.49	10.27	.01
8/21/1989	1600	1475.00	.00	1242.55	1268.00	-5.64	-22.50	11.72	10.27	.00
8/21/1989	2400	1513.00	.00	1453.20	1478.88	-7.67	-22.50	11.75	10.47	.12
8/22/1989	800	1513.00	.00	1537.87	1565.01	-6.84	-21.50	11.75	10.54	.04
8/22/1989	1600	1513.00	.00	1543.83	1573.00	-5.79	-21.50	11.75	10.55	.00
8/22/1989	2400	1513.00	.00	1544.66	1573.00	-5.19	-21.50	11.75	10.55	.00
8/23/1989	800	1513.00	.00	1541.81	1573.00	-4.81	-25.40	11.75	10.55	.00
8/23/1989	1600	1513.00	.00	1542.41	1573.00	-4.54	-25.40	11.75	10.55	.00
8/23/1989	2400	1513.00	.00	1542.79	1573.00	-4.33	-25.40	11.75	10.55	.00
8/24/1989	800	1513.00	.00	1543.36	1573.00	-4.15	-25.10	11.75	10.55	.00
8/24/1989	1600	1513.00	.00	1543.57	1573.00	-4.00	-25.10	11.75	10.55	.00
8/24/1989	2400	1513.00	.00	1543.75	1573.00	-3.87	-25.10	11.75	10.55	.00
8/25/1989	800	1513.00	.00	1543.90	1573.00	-3.76	-25.10	11.75	10.55	.00
8/25/1989	1600	1513.00	.00	1544.03	1573.00	-3.66	-25.10	11.75	10.55	.00
8/25/1989	2400	1513.00	.00	1544.14	1573.00	-3.57	-25.10	11.75	10.55	.00
8/26/1989	800	1513.00	.00	1544.24	1573.00	-3.49	-25.10	11.75	10.55	.00
8/26/1989	1600	1513.00	.00	1544.33	1573.00	-3.41	-25.10	11.75	10.55	.00
8/26/1989	2400	1513.00	.00	1544.41	1573.00	-3.34	-25.10	11.75	10.55	.00
8/27/1989	800	1513.00	.00	1544.69	1573.00	-3.28	-24.90	11.75	10.55	.00
8/27/1989	1600	1513.00	.00	1544.76	1573.00	-3.22	-24.90	11.75	10.55	.00
8/27/1989	2400	1513.00	.00	1544.82	1573.00	-3.16	-24.90	11.75	10.55	.00
8/28/1989	800	1513.00	.00	1544.88	1573.00	-3.11	-24.90	11.75	10.55	.00
8/28/1989	1600	1513.00	.00	1544.94	1573.00	-3.06	-24.90	11.75	10.55	.00
8/28/1989	2400	1513.00	.00	1544.99	1573.00	-3.02	-24.90	11.75	10.55	.00
8/29/1989	800	1513.00	.00	1545.04	1573.00	-2.97	-24.90	11.75	10.55	.00
8/29/1989	1600	1513.00	.00	1545.08	1573.00	-2.93	-24.90	11.75	10.55	.00
8/29/1989	2400	.00	.00	1545.13	1573.00	6.22	-24.90	10.01	10.55	-.33
COLUMN TOTALS:		95487.00	98584.95	96786.03	98584.95	-274.34	-1521.30			

FOOTNOTE: * DOWNSTREAM DISCHARGE IS LESS THAN SPECIFIED MINIMUM FLOW.
THIS MAY BE CAUSED BY THE MODEL WHEN A SHARP RISE IN STAGE OCCURS.
OR THIS MAY ALSO BE CAUSED BY A HIGH DIVERSION OR DEPLETION.
** DIVERSIONS AND DEPLETIONS WERE REDUCED TO PREVENT NEGATIVE FLOW AT ONSET.
DOWNSTREAM DISCHARGES SHOWN RESULT FROM BANK STORAGE.

VOLUME OF FLOW (CFS-DAYS)

UPSTREAM STATION		REACH		DOWNSTREAM STATION	
TOTAL	31828.98			TOTAL (W/O BANK STORAGE + LOSSES)	32861.63
				TOTAL (W/ BANK STORAGE + LOSSES)	32262.01
BASE FLOW	.00			BASE FLOW	1500.00
RELEASE OR FLOOD	31828.98	STREAMFLOW LOSS OR GAIN	-598.55	RELEASE OR FLOOD	30762.01

BANK STORAGE:	
FLOW FROM STREAM	100.29
STORED IN AQUIFER	91.45
LOST TO SOIL	.00
RETURNED TO STREAM	8.84
NET BANK STORAGE DISCHARGE	-91.45
DIVERSIONS AND WELL LOSSES	-507.10

FIRST REACH RELEASE OR FLOOD VOLUME = 31829.0 CFS-DAYS
WELL LOSS, CUMULATIVE FROM FIRST REACH = .00 CFS-DAYS
CUMULATIVE TOTAL LOSS = -598.55 CFS-DAYS
CUMULATIVE LOSS EXCLUDING WELL LOSS = -598.55 CFS-DAYS = -1.88 PERCENT OF FIRST-REACH RELEASE OR FLOOD VOLUME

NOTE: UNLESS STATED OTHERWISE
(-) INDICATES FLOW FROM STREAM
(+) INDICATES FLOW INTO STREAM

GREEN RIVER 1989 - FROM FARSON BRIDGE TO GREEN RIVER GOLF COURSE (REACH NO. 2)

PROPERTIES AND CHARACTERISTICS OF REACH

LENGTH OF CHANNEL (MILES)	37.6
LENGTH OF ALLUVIUM (MILES)	28.0
TRAVEL TIME (ESTIMATED HOURS)	14.0

TRAVEL TIME TO BEGINNING OF RESPONSE (HOURS)	10.1	Cumulative from start of first reach =	.86 DAYS	
TRAVEL TIME TO CENTER OF RESPONSE (HOURS)	15.5			
TRAVEL TIME BETWEEN BREAKS IN HYDROGRAPHS (HOURS)	12.8			
NUMBER OF SUBREACHES USED IN COMPUTATIONS	2			
TRANSMISSIVITY OF AQUIFER (SQ.FT./DAY)	1000.0			
STORAGE COEFFICIENT OF AQUIFER (CU.FT./CU.FT.)	.10			
AQUIFER IS ASSUMED TO BE 5000. (FT) WIDE				
(STREAM TO BOUNDARY)	CASE 2			
SOIL RETENTION FACTOR	.00			
BASE FLOW AT DOWNSTREAM STATION	100.0			
MINIMUN EXPECTED DISCHARGE TO BE ROUTED	50.0			
MAXIMUN EXPECTED DISCHARGE TO BE ROUTED	2100.0			

CELERITY AND DISPERSION RATING TABLE

W. CELERITY	DISCHARGE	DISP. COEF.	DISCHARGE
3.55	50.0	235.0	50.0
3.56	300.0	890.0	300.0
3.57	600.0	1675.0	600.0
3.58	900.0	2460.0	900.0
3.59	1200.0	3245.0	1200.0
3.60	1500.0	4030.0	1500.0
3.61	1800.0	4815.0	1800.0
3.62	2100.0	5600.0	2100.0

FAMILY OF FLOW ROUTING UNIT-RESPONSE FUNCTIONS

NO.	W. CELERITY FT/SEC	DISP.COEF SQ FT/SEC	TRAVEL TIME TIME STEPS	DISCHARGE CU FT/SEC	ORDINATES
1	3.62	5600.0	1	2100.0	1) .1509 2) .7939 3) .0552

STREAM-AQUIFER UNIT-RESPONSE FUNCTION

NOTE: CLOSURE WAS NOT OBTAINED FOR THE FIRST 9 NUMBERS.
COMPUTATIONS WERE MADE USING CASE 1 CONDITIONS FOR THESE NUMBERS.

NOTE: THIS RESPONSE FUNCTION (EXPONENTIAL DECAY TYPE IS EVALUATED FOR 18.5 HALF-LIVES.
IT HAS 75 ORDINATES.

1) -.013820	2) -.007979	3) -.006180	4) -.005223	5) -.004607	6) -.004167
7) -.003833	8) -.003568	9) -.003352	10) -.003170	11) -.003015	12) -.002881
13) -.002764	14) -.002659	15) -.002566	16) -.002482	17) -.002406	18) -.002336
19) -.002272	20) -.002213	21) -.002158	22) -.002107	23) -.002060	24) -.002016
25) -.001974	26) -.001935	27) -.001898	28) -.001863	29) -.001830	30) -.001799
31) -.001769	32) -.001741	33) -.001714	34) -.001688	35) -.001664	36) -.001640
37) -.001617	38) -.001596	39) -.001575	40) -.001555	41) -.001535	42) -.001517
43) -.001499	44) -.001482	45) -.001465	46) -.001449	47) -.001433	48) -.001418
49) -.001403	50) -.001389	51) -.001375	52) -.001362	53) -.001349	54) -.001336
55) -.001324	56) -.001312	57) -.001300	58) -.001289	59) -.001278	60) -.001267
61) -.001256	62) -.001246	63) -.001236	64) -.001226	65) -.001217	66) -.001207
67) -.001198	68) -.001189	69) -.001181	70) -.001172	71) -.001164	72) -.001156
73) -.001148	74) -.001140	75) -.001132			

DOWNSTREAM STATION DATA

RATING TABLE

STAGE	DISCHARGE
6.50	412.00
6.60	457.00
6.70	506.00
6.80	559.00
6.90	617.00
7.00	681.00
7.10	750.00
7.20	824.00
7.30	905.00
7.40	993.00
7.50	1087.00
7.60	1190.00
7.70	1300.00
7.80	1419.00
7.90	1547.00
8.00	1685.00
8.10	1833.00
8.20	1993.00

SUMMARY OF ITERATION DATA FOR ROUTING OPTION

	CHANGES BETWEEN ITERATIONS		VOLUMES AT END OF ITERATION	
ITERATION NO.	MAXIMUM CHANGE IN BANK STORAGE DISCHARGE (CFS)	ABSOLUTE CHANGE IN BANK STORAGE VOLUME (CFS - DAYS)	NET VOLUME OF BANK STORAGE (CFS - DAYS)	VOLUME OF FLOW AT DOWNSTREAM STATION (CFS - DAYS)
1	67.9	430.	-430.	31944.
2	.6	2.	-428.	31945.

CLOSURE WAS OBTAINED AFTER 2 ITERATIONS

CRITERIA FOR CLOSURE 1.0 CFS

GREATEST CHANGE IN LAST ITERATION .6 CFS

BANK STORAGE DISCHARGE AFFECTED DOWNSTREAM ROUTED DISCHARGE 2 TIME STEPS LATER.

REACH NO. 2: BEGINS AT GAGING STATION 000002 FARSON BRIDGE
ENDS AT GAGING STATION 000003 GOLF COURSE

TOTAL STUDY PERIOD: BEGINS 8/ 5/1989
ENDS 8/29/1989

THIS SIMULATION PERIOD BEGINS 8/ 5/1989 AND ENDS 8/29/1989

SUMMARY OF STREAMFLOW DIVERSIONS AND DEPLETIONS

DISTANCE FROM STREAM FEET	DISCHARGE CFS	STARTING DAY	ENDING DAY
		NUMBER OF DAY FROM BEGINNING OF MODEL RUN	
.00	-13.60	1	1
.00	-12.50	2	2
.00	-13.30	3	3
.00	-14.40	4	4
.00	-14.60	5	5
.00	1.50	6	6
.00	-6.50	7	7
.00	-.50	8	8
.00	12.90	9	9
.00	4.90	10	10
.00	5.80	11	11
.00	4.90	12	12
.00	5.40	13	13
.00	2.10	14	14
.00	3.10	15	15
.00	7.40	16	16
.00	8.20	17	17
.00	7.50	18	18
.00	13.50	19	19
.00	7.30	20	20
.00	5.80	21	21
.00	7.30	22	22
.00	8.60	23	23
.00	7.70	24	24
.00	8.30	25	25

SUMMARY OF DATA AND RESULTS

DATE	TIME	OBSERVED UPSTREAM DISCHARGE	PREDICTED DOWNST. DISCHARGE	DOWNST. DISCHARGE	DOWNST. Q W/O BANK STORAGE AND LOSSES	BANK STORAGE DISCHARGE	DIVERSIONS AND DEPLETIONS	UPSTREAM STAGE	DOWNSTREAM STAGE	CHANGE IN STAGE
8/ 5/1989	800	38.60	.00	86.40	*100.00	-13.41	-13.60	8.30	5.78	.28
8/ 5/1989	1600	344.25	.00	86.40	*100.00	-15.79	-13.60	8.90	5.78	.17
8/ 5/1989	2400	1261.79	.00	95.60	*122.60	-43.19	-13.60	10.29	5.80	.69
8/ 6/1989	800	1485.18	.00	477.59	505.88	-67.31	-12.50	10.50	6.64	.84
8/ 6/1989	1600	1482.54	.00	1229.21	1284.90	-62.54	-12.50	10.50	7.64	.37
8/ 6/1989	2400	1487.14	.00	1432.66	1512.46	-49.28	-12.50	10.50	7.81	.05
8/ 7/1989	800	1500.05	.00	1447.55	1523.38	-41.68	-13.30	10.51	7.82	.01
8/ 7/1989	1600	1504.09	.00	1466.25	1528.84	-37.20	-13.30	10.52	7.84	.01
8/ 7/1989	2400	1504.99	.00	1484.97	1539.95	-33.86	-13.30	10.52	7.85	.01
8/ 8/1989	800	1505.75	.00	1492.41	1544.01	-31.22	-14.40	10.52	7.86	.00
8/ 8/1989	1600	1506.35	.00	1496.79	1545.05	-29.13	-14.40	10.52	7.86	.00
8/ 8/1989	2400	1506.82	.00	1500.18	1545.80	-27.42	-14.40	10.52	7.86	.00
8/ 9/1989	800	1496.95	.00	1502.66	1546.39	-25.88	-14.60	10.51	7.87	-.00
8/ 9/1989	1600	1494.66	.00	1503.29	1545.30	-24.52	-14.60	10.51	7.87	-.00
8/ 9/1989	2400	1495.02	.00	1496.68	1537.15	-23.45	-14.60	10.51	7.86	-.00
8/10/1989	800	1191.77	.00	1511.82	1534.84	-19.03	1.50	10.22	7.87	-.07
8/10/1989	1600	1002.75	.00	1467.27	1489.23	-10.94	1.50	10.01	7.84	-.19
8/10/1989	2400	988.93	.00	1202.44	1219.97	-7.00	1.50	9.99	7.61	-.16
8/11/1989	800	1401.17	.00	1033.64	1051.09	-13.74	-6.50	10.42	7.44	.06
8/11/1989	1600	1297.42	.00	1078.41	1091.91	-21.34	-6.50	10.33	7.49	.16
8/11/1989	2400	1163.43	.00	1382.53	1402.77	-18.43	-6.50	10.19	7.77	.01
8/12/1989	800	1142.33	.00	1301.07	1322.92	-13.31	-.50	10.17	7.70	-.08
8/12/1989	1600	1141.02	.00	1188.70	1207.64	-12.55	-.50	10.17	7.60	-.04
8/12/1989	2400	1140.40	.00	1169.48	1183.30	-13.20	-.50	10.17	7.58	-.00
8/13/1989	800	1140.39	.00	1181.35	1181.00	-13.43	12.90	10.17	7.59	.00
8/13/1989	1600	1140.20	.00	1180.13	1180.43	-13.31	12.90	10.17	7.59	.00
8/13/1989	2400	1140.09	.00	1179.84	1180.36	-13.10	12.90	10.16	7.59	-.00
8/14/1989	800	1132.36	.00	1171.79	1180.19	-12.82	4.90	10.16	7.58	-.00
8/14/1989	1600	1121.45	.00	1170.73	1178.93	-12.36	4.90	10.14	7.58	-.01
8/14/1989	2400	977.54	.00	1163.22	1171.14	-9.95	4.90	9.98	7.57	-.05
8/15/1989	800	894.14	.00	1133.77	1140.33	-5.88	5.80	9.87	7.55	-.11
8/15/1989	1600	866.73	.00	1008.74	1012.89	-3.87	5.80	9.83	7.42	-.09
8/15/1989	2400	862.27	.00	934.52	934.60	-4.45	5.80	9.83	7.33	-.04
8/16/1989	800	859.76	.00	908.60	907.57	-5.57	4.90	9.82	7.30	-.01
8/16/1989	1600	859.19	.00	902.58	902.13	-6.39	4.90	9.82	7.30	-.00
8/16/1989	2400	954.05	.00	899.14	899.81	-8.44	4.90	9.95	7.29	.03
8/17/1989	800	1005.51	.00	912.55	913.54	-11.53	5.40	10.01	7.31	.07

8/17/1989	1600	1013.44	.00	993.55	996.59	-12.69	5.40	10.02	7.40	.06
8/17/1989	2400	1013.84	.00	1037.74	1043.87	-11.95	5.40	10.02	7.45	.02
8/18/1989	800	1009.67	.00	1042.48	1053.06	-11.12	2.10	10.02	7.45	.00
8/18/1989	1600	1061.02	.00	1043.34	1053.19	-11.42	2.10	10.08	7.45	.02
8/18/1989	2400	1208.51	.00	1048.63	1057.65	-14.10	2.10	10.24	7.46	.07
8/19/1989	800	1241.11	.00	1112.13	1120.45	-16.81	3.10	10.27	7.52	.09
8/19/1989	1600	1240.77	.00	1234.30	1245.30	-16.54	3.10	10.27	7.64	.05
8/19/1989	2400	1241.76	.00	1265.56	1279.26	-15.02	3.10	10.27	7.67	.01
8/20/1989	800	1242.11	.00	1271.80	1280.94	-14.03	7.40	10.27	7.67	.00
8/20/1989	1600	1242.46	.00	1274.13	1281.75	-13.39	7.40	10.27	7.68	.00
8/20/1989	2400	1242.71	.00	1275.51	1282.14	-12.92	7.40	10.27	7.68	.00
8/21/1989	800	1242.40	.00	1277.29	1282.48	-12.53	8.20	10.27	7.68	.00
8/21/1989	1600	1242.55	.00	1277.93	1282.65	-12.29	8.20	10.27	7.68	.00
8/21/1989	2400	1453.20	.00	1278.10	1282.44	-14.49	8.20	10.47	7.68	.05
8/22/1989	800	1537.87	.00	1309.55	1314.34	-18.29	7.50	10.54	7.71	.11
8/22/1989	1600	1543.83	.00	1487.38	1494.36	-19.07	7.50	10.55	7.85	.07
8/22/1989	2400	1544.66	.00	1563.31	1574.10	-17.48	7.50	10.55	7.91	.02
8/23/1989	800	1541.81	.00	1578.05	1583.63	-16.10	13.50	10.55	7.92	.00
8/23/1989	1600	1542.41	.00	1580.20	1584.18	-15.19	13.50	10.55	7.92	.00
8/23/1989	2400	1542.79	.00	1579.46	1582.06	-14.54	13.50	10.55	7.92	.00
8/24/1989	800	1543.36	.00	1574.54	1582.43	-14.08	7.30	10.55	7.92	.00
8/24/1989	1600	1543.57	.00	1575.61	1582.85	-13.72	7.30	10.55	7.92	.00
8/24/1989	2400	1543.75	.00	1576.58	1583.36	-13.39	7.30	10.55	7.92	.00
8/25/1989	800	1543.90	.00	1575.66	1583.58	-13.09	5.80	10.55	7.92	.00
8/25/1989	1600	1544.03	.00	1576.17	1583.76	-12.85	5.80	10.55	7.92	.00
8/25/1989	2400	1544.14	.00	1576.61	1583.91	-12.63	5.80	10.55	7.92	.00
8/26/1989	800	1544.24	.00	1578.49	1584.04	-12.42	7.30	10.55	7.92	.00
8/26/1989	1600	1544.33	.00	1578.82	1584.15	-12.23	7.30	10.55	7.92	.00
8/26/1989	2400	1544.41	.00	1579.13	1584.25	-12.05	7.30	10.55	7.92	.00
8/27/1989	800	1544.69	.00	1580.71	1584.34	-11.89	8.60	10.55	7.92	.00
8/27/1989	1600	1544.76	.00	1581.00	1584.45	-11.73	8.60	10.55	7.92	.00
8/27/1989	2400	1544.82	.00	1581.39	1584.68	-11.57	8.60	10.55	7.92	.00
8/28/1989	800	1544.88	.00	1580.74	1584.76	-11.42	7.70	10.55	7.92	.00
8/28/1989	1600	1544.94	.00	1580.96	1584.83	-11.29	7.70	10.55	7.92	.00
8/28/1989	2400	1544.99	.00	1581.16	1584.89	-11.17	7.70	10.55	7.92	.00
8/29/1989	800	1545.04	.00	1581.95	1584.94	-11.05	8.30	10.55	7.93	.00
8/29/1989	1600	1545.08	.00	1582.13	1584.99	-10.93	8.30	10.55	7.93	.00
8/29/1989	2400	1545.13	.00	1582.29	1585.04	-10.81	8.30	10.55	7.93	.00
COLUMN TOTALS:		96786.03	96957.98	95835.34	96957.98	-1284.78	140.40			

FOOTNOTE: * DOWNSTREAM DISCHARGE IS LESS THAN SPECIFIED MINIMUM FLOW.
THIS MAY BE CAUSED BY THE MODEL WHEN A SHARP RISE IN STAGE OCCURS.
OR THIS MAY ALSO BE CAUSED BY A HIGH DIVERSION OR DEPLETION.

** DIVERSIONS AND DEPLETIONS WERE REDUCED TO PREVENT NEGATIVE FLOW AT ONSET.
DOWNSTREAM DISCHARGES SHOWN RESULT FROM BANK STORAGE.

VOLUME OF FLOW (CFS-DAYS)

UPSTREAM STATION		REACH		DOWNSTREAM STATION	
TOTAL	32262.01			TOTAL (W/O BANK STORAGE + LOSSES)	32319.34
				TOTAL (W/ BANK STORAGE + LOSSES)	31945.12
BASE FLOW	1500.00			BASE FLOW	2500.00
RELEASE OR FLOOD	30762.01	STREAMFLOW LOSS OR GAIN	-381.46	RELEASE OR FLOOD	29445.12

BANK STORAGE:	
FLOW FROM STREAM	428.26
STORED IN AQUIFER	428.26
LOST TO SOIL	.00
RETURNED TO STREAM	.00
NET BANK STORAGE DISCHARGE	-428.26

DIVERSIONS AND WELL LOSSES 46.80
FIRST REACH RELEASE OR FLOOD VOLUME = 31829.0 CFS-DAYS
WELL LOSS, CUMULATIVE FROM FIRST REACH = .00 CFS-DAYS
CUMULATIVE TOTAL LOSS = -980.01 CFS-DAYS
CUMULATIVE LOSS EXCLUDING WELL LOSS = -980.01 CFS-DAYS = -3.08 PERCENT OF FIRST-REACH RELEASE OR FLOOD VOLUME

NOTE: UNLESS STATED OTHERWISE
(-) INDICATES FLOW FROM STREAM
(+) INDICATES FLOW INTO STREAM