POTENTIAL INCREMENTAL SEEPAGE LOSSES IN AN ALLUVIAL CHANNEL IN THE RIO GRANDE BASIN, NEW MEXICO

By Robert L. Gold

U.S. GEOLOGICAL SURVEY
Water-Resources Investigations Report 84-4268

Prepared in cooperation with the
U.S. BUREAU OF RECLAMATION

Albuquerque, New Mexico
1985

UNITED STATES DEPARTMENT OF INTERIOR

WILLIAM P. CLARK, Secretary

GEOLOGICAL SURVEY

Dallas L. Peck, Director

For additional information:
write to:

District Chief
U.S. Geological Survey
Water Resources Division
505 Marquette NW, Room 720
Albuquerque, New Mexico 87102

For sale by:

Open-File Services Section
Branch of Distribution
U.S. Geological Survey, MS 306
Box 25425, Denver Federal Center
Denver, Colorado 80225
(303) 236-7476

CONTENTS

	Page
Abstract	1
Introduction	2
Purpose and scope	2
Description of study area	3
Digital model used to estimate seepage losses	5
General description	5
Model description and results	6
Losing reach	6
Gaining reach	15
Reach affected by ground-water pumpage	17
Model sensitivity	19
Conclusion	21
References	22

ILLUSTRATIONS

Figure 1. Map showing Rio Grande reach between Cochiti Dam and Elephant Butte Reservoir	4
2. Diagrammatic section across river valley	6
3. Diagram of the arrangement of data to be used in model run with applied stress for a losing reach	7
4. Graph showing steady-state hydraulic-head values for a losing reach	8
5. Graph showing incremental flow volumes per specified time step for 1 foot of surcharge, 1 day of application, and 180 days return flow in a losing reach	10
6. Graph showing incremental flow volumes per specified time step for 1 foot of surcharge, 10 days of application, and 180 days return flow in a losing reach	10

ILLUSTRATIONS - Concluded

Page

Figure 7. Graph showing incremental flow volumes per specified time step for 1 foot of surcharge, 100 days of application, and 180 days return flow in a losing reach 11

8. Graph showing seepage rates for 1 day of application for the indicated return-flow period in a losing reach 13

9. Graph showing seepage rates for 10 days of application for the indicated return-flow period in a losing reach 14

10. Graph showing seepage rates for 100 days of application for the indicated return-flow period in a losing reach 14

11. Graph showing steady-state hydraulic-head values for a gaining reach 15

12. Graph showing steady-state hydraulic-head values for a reach affected by pumping of ground water 17

TABLES

Table 1. Discharge rating table for streamflow-gaging station 08330000, Rio Grande at Albuquerque, New Mexico 3

2. Cumulative flow volumes to aquifer from river for 1 foot of surcharge applied for 1 day for a losing reach 11

3. Cumulative flow volumes from aquifer to river for 1 foot of surcharge applied for 1 day with 180 days of return flow for a losing reach 12

4. Net seepage rates for gaining reach 16

5. Net seepage rates for reach affected by pumping of ground water 18

6. Net seepage rates determined during model sensitivity analysis 20

CONVERSION FACTORS

The inch-pound units in this report can be converted to the metric system of units as follows:

Multiply inch-pound unit	By	To obtain metric unit
foot	0.3048	meter
foot per day	0.3048	meter per day
cubic foot per second per mile	0.01759	cubic meter per second per kilometer
mile	1.609	kilometer

POTENTIAL INCREMENTAL SEEPAGE LOSSES IN AN ALLUVIAL CHANNEL IN THE RIO GRANDE BASIN, NEW MEXICO

By Robert L. Gold

ABSTRACT

A two-dimensional, digital, cross-sectional model was used to simulate seepage of water from an alluvial channel, which had the general characteristics of the Rio Grande channel, into the underlying alluvium within the reach from Cochiti Dam to Elephant Butte Reservoir. Incremental seepage rates were determined for losing and gaining reaches and reaches affected by pumping of ground water. The incremental seepage rates were computed for stream surcharges (height of additional water applied on top of base flow) ranging from 0.5 foot to 3 feet and for application periods ranging from 1 to 100 days. The net seepage rates, which were nearly identical for each type of reach, ranged from 0.0 cubic foot per second per mile of channel length for a 0.5-foot surcharge applied for 1 day to 0.37 cubic foot per second per mile of channel length for a 3-foot surcharge applied for 100 days, when each surcharge was followed by a 180-day return flow from the aquifer.

INTRODUCTION

The U.S. Bureau of Reclamation has the responsibility for the accounting of San Juan-Chama Project water as it flows with the natural waters in the Chama River and in the Rio Grande. San Juan-Chama Project water originates in the San Juan River basin from the Rio Blanco, Little Navajo River, and Navajo River and is brought into the Rio Grande basin by means of a transmountain diversion through the Azotea Tunnel.

In the past, San Juan-Chama Project water was stored for water purchasers at Heron, El Vado, and Abiquiu Reservoirs in the Chama River basin. A recreational pool has been established in Elephant Butte Reservoir (Public Law 93-493 title XIV) for storing San Juan-Chama Project water. In addition, Public Law 97-140 authorizes storage of San Juan-Chama Project water in Elephant Butte Reservoir for purchasers of those waters. To store San Juan-Chama Project water in Elephant Butte Reservoir requires conveying the water from the reservoirs in the Chama River basin to the north of Cochiti Dam by way of the Chama River. The water then flows in the Rio Grande to Cochiti Dam and then is released from the dam and conveyed in the Rio Grande to Elephant Butte Reservoir. Stream losses of San Juan-Chama water will result from evaporation, transpiration, and seepage as it flows down the reach of the Rio Grande between Cochiti Dam and Elephant Butte Reservoir. The U.S. Bureau of Reclamation (1955) uses estimates of losses for this reach to determine how much of the San Juan-Chama Project water actually reaches Elephant Butte Reservoir. There is, however, a need for better definition of possible seepage losses caused by San Juan-Chama Project water as it flows between Cochiti Dam and Elephant Butte Reservoir.

Purpose and Scope

The purpose of this study was to provide some indication of the magnitude of potential seepage losses of San Juan-Chama Project water in the Rio Grande between Cochiti Dam and Elephant Butte Reservoir during the non-growing seasons of fall and winter. However, because of the difficulty of actually measuring such losses, a result of discharge-measurement error, a two-dimensional model was used to predict seepage losses in a hypothetical alluvial channel that has the general characteristics of the Rio Grande, excluding the low-flow conveyance channel that begins at San Acacia.

Seepage rates were determined by the model for specific surcharges based on the concept of superposition. Included in the concept is the fact that losses due to incremental increases in river stage can be represented in a model by corresponding increases in the hydraulic head at the river. Surcharge is defined as the height of additional water applied on top of existing flow in the Rio Grande.

Description of Study Area

San Juan-Chama Project water is discharged from Cochiti Lake (fig. 1) located in Sandoval County in the Pueblo de Cochiti Grant. The contributing drainage area upstream from the lake is approximately 12,000 square miles.

The study reach extends from Cochiti Dam to Elephant Butte Reservoir in Sierra County, a distance of 232 river miles. From Cochiti Dam to the San Marcial constriction, the river flows through a series of basins, canyons, and restrictions known as the middle Rio Grande valley. The cities of Bernalillo, Albuquerque, Los Lunas, Belen, and Socorro are adjacent to the Rio Grande within this reach.

Within the middle Rio Grande valley, the Rio Grande is underlain by alluvium of Holocene age. The channel of the Rio Grande, primarily sand, is approximately 800 feet wide. Cottonwood, willow, Russian olive, and saltcedar are the primary natural vegetation within the flood plain. Irrigated farming is practiced along the river throughout the reach.

The slope of the river is about 5 feet per mile from Cochiti to just downstream from Albuquerque, 4 feet per mile thereon to the confluence of the Rio Puerco, and about 3 to 5 feet per mile thereon to Elephant Butte Reservoir (Lagasse, 1980). Elephant Butte Reservoir was formed after completion of Elephant Butte Dam in 1915. Contributing drainage area upstream from the dam is 26,505 square miles. The stage-discharge relationship determined for one streamflow-gaging station, Rio Grande at Albuquerque, is tabulated in table 1. For an increase in stage from 2 feet to 3 feet, the discharge increases 161.6 cubic feet per second.

The reach of the Rio Grande between Cochiti Dam and Elephant Butte Reservoir is part of a complex water-distribution system. Along its course, the river is subjected to many withdrawals and returns for water supply and irrigation. Withdrawals for irrigation flow first into diversion canals and then are channeled into the irrigated fields. Excess irrigation water flows from the fields into drains and wasteways that return water to the Rio Grande. Diversions for irrigation are made at Angostera, Isleta, and San Acacia. The Jemez River, Rio Puerco, and Rio Salado are the main tributaries along this reach.

Table 1. Discharge rating table for streamflow-gaging station 08330000, Rio Grande at Albuquerque, New Mexico

Gage height (feet)	Discharge (cubic feet per second)
2.00	61.0
3.00	222.6
4.00	522.8
5.00	1,060.
6.00	1,993.
7.00	3,689.
8.00	7,632.

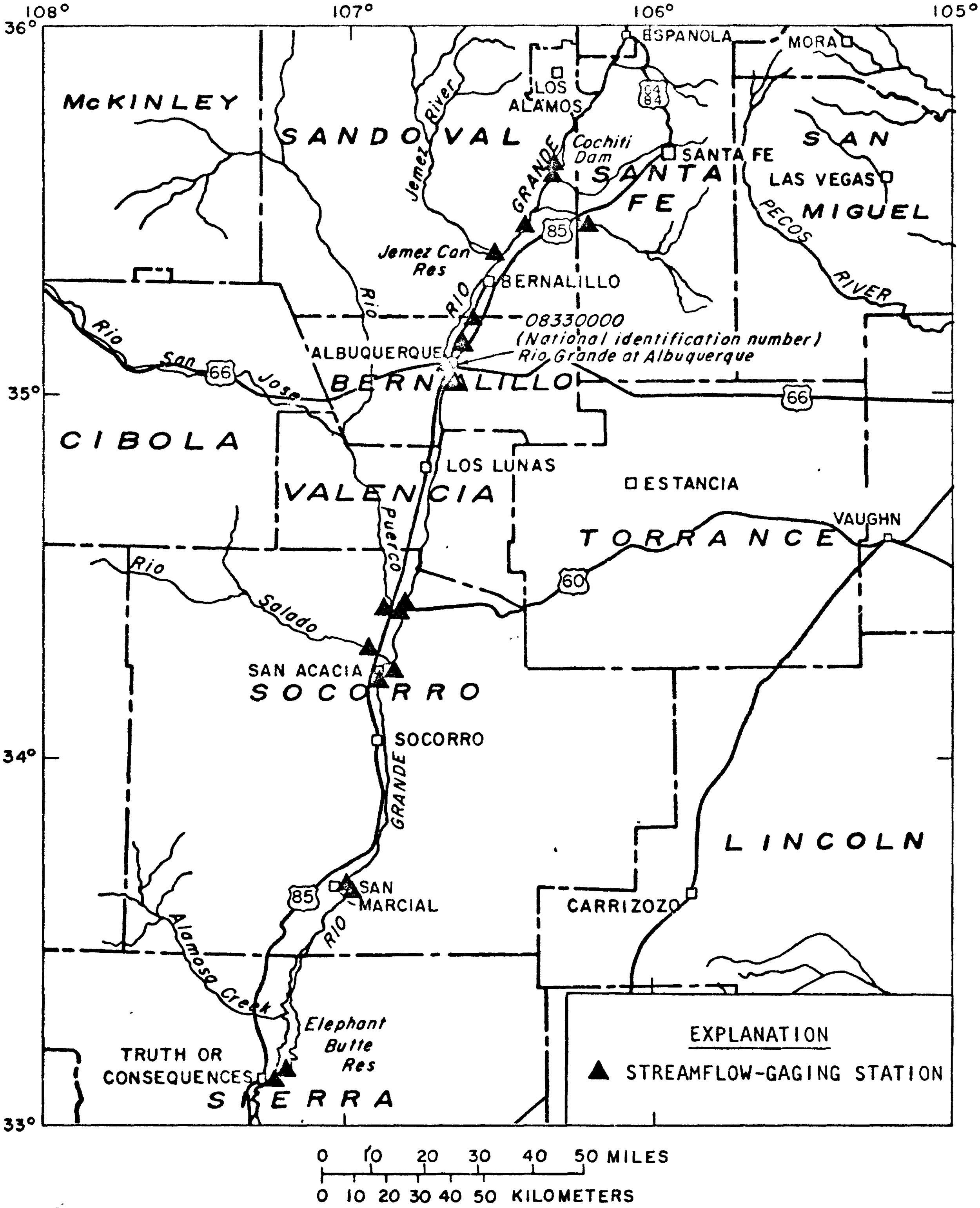

Figure 1.--Rio Grande reach between Cochiti Dam and Elephant Butte Reservoir.

DIGITAL MODEL USED TO ESTIMATE SEEPAGE LOSSES

General Description

A review was made of various digital streamflow and ground-water flow models. The goal was to select a model that best simulated the hydraulic connection between surface streamflow and the underlying alluvial aquifer. The U.S. Geological Survey's two-dimensional model (Trescott and others, 1976) was selected for use in this study. A complete description of the theory, structure, and job-control language for this model is found in Trescott and others (1976) and will not be repeated here.

Elements to be considered in the construction of such a model included width of the flood plain, relative arrangement of drains with respect to the river, and the hydrologic characteristics of underlying water-bearing formations. The diagrammatic section shown in figure 2 displays a typical relationship between the various components that were numerically defined for use in the model. Based on a geologic map of New Mexico (Dane and Bachman, 1965) and various topographic maps as reference, an average flood-plain width of 3 miles was used in the model. The river commonly is flanked by drains within the flood plain. Two such drains were assumed in the model. The flood plain is underlain by alluvium (thickness ranging from 80 to 250 feet) of Holocene age and then by the Santa Fe Group of Pleistocene age (maximum thickness of 9,000 feet) as described by Bjorklund and Maxwell (1961).

Values of hydraulic conductivity and specific storage for the underlying aquifers were also necessary for construction of the model. Hydraulic-conductivity values have been calculated to be 40 feet per day for the alluvial aquifer from work done by Waldron (1956) and to range from 10 to 112 feet per day for the Santa Fe Group from work done by Bjorklund and Maxwell (1961). A hydraulic-conductivity value of 40 feet per day was used in the model for the alluvium and 20 feet per day was used for the Santa Fe Group. Specific storage was 1×10^{-6} per foot of aquifer thickness based on values from Lohman (1972).

Three general hydrologic conditions of streamflow were simulated in the model. These are: losing stream reaches, gaining reaches, and reaches affected by significant ground-water pumpage, such as found in the Albuquerque area. The slope of the water table toward the stream was varied in order to approximate these conditions in the model. Bjorklund and Maxwell (1961) determined the water-table slope to be from 5 to 20 feet per mile. Contours of the water table presented by Bjorklund and Maxwell (1961) indicate a general slope from the east towards the west and from north to the south.

Model Description and Results

Losing Reach

The basic structure used in defining the properties and limits of a typical cross section is shown in figure 2. The subsurface structure is divided into two parts. The upper part is shallow alluvium with a thickness of 100 feet into which the Rio Grande and the two flanking drains are partly penetrating. The lower part is the Santa Fe Group. The Santa Fe Group was assumed to be 1,000 feet thick for the model. A no-flow boundary was assumed to exist below the Santa Fe Group. This allowed the lower part of the flow system to be active. The short time scale of the problem did not require a deeper boundary.

The data used in the model were arranged in a typical model-grid pattern (fig. 3). The various data values are identified for model use by row and column. The variables that were to be assigned values for use in the model were hydraulic head, specific storage (the volume of water released from or taken into storage per unit volume of the aquifer per unit change in head) and hydraulic conductivity. A cross-sectional thickness of 50 feet also was assumed.

The procedure followed was to first simulate a steady-state condition in which storage of water in the aquifer does not vary with time. The values for the variables used in the model while the surcharges were applied are shown in figure 3.

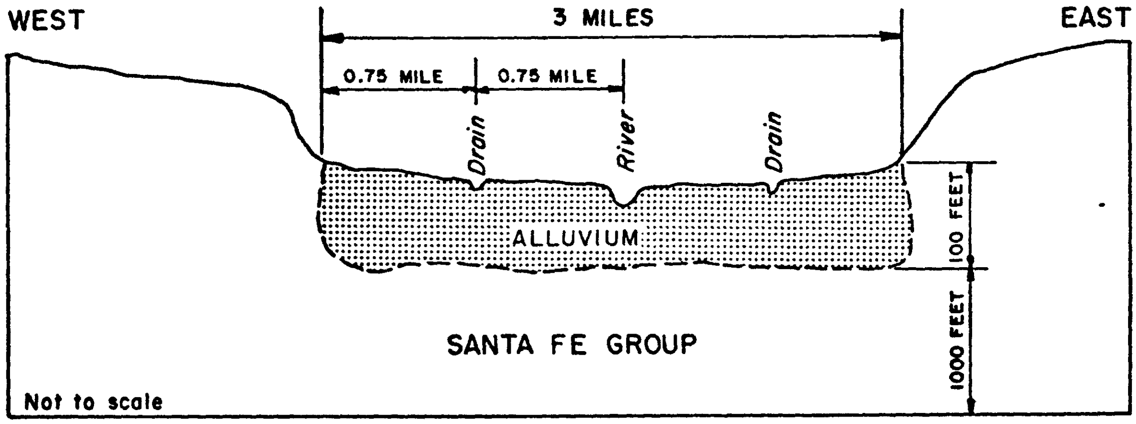

Figure 2.--Diagrammatic section across river valley.

To simulate a losing reach, the water-table surface was defined to slope at a rate of 20 feet per mile from right to left (east to west) across the flood plain (Bjorkland and Maxwell, 1961). The hydraulic-head values used to define that water-table slope were assumed to be a constant 130 feet at the east boundary and a constant 70 feet at the western boundary. Starting hydraulic-head values within the cross section were defined to be 100 feet. The storage coefficient was equal to 0.1 (unconfined) for the top row at land surface. Constant hydraulic heads were assigned to the river, drains and the end boundaries, and no-flow boundaries were assigned for all other points. Hydraulic-conductivity values were 40 feet per day for the shallow alluvium and 20 feet per day for the Santa Fe Group. The ratio of vertical to horizontal hydraulic conductivity was assumed to be 1:250 from work done by Wilson and White (1984).

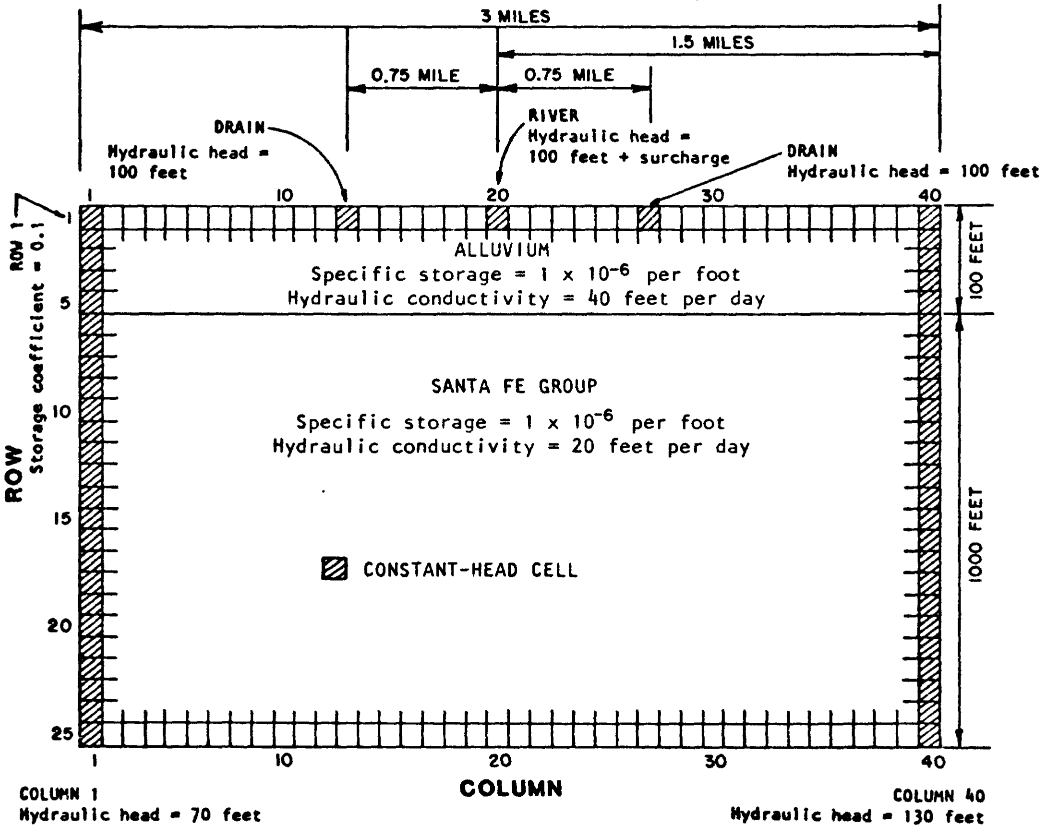

Figure 3.--Arrangement of data to be used in model run with applied stress for a losing reach.

The model was then used to determine steady-state values of hydraulic head (fig. 4) and rates of recharge for the cross section. These values were saved for the next step in the simulation where transient stress in terms of river surcharge was applied to the system.

The steady-state hydraulic-head values were used in subsequent model runs to determine subsurface flow and storage values for the cross section at different river surcharges and durations of application. For these model runs the specific storage for the subsurface was 1 x 10^{-6} per foot of aquifer thickness based on values from Lohman (1972). The storage coefficient of 0.1 (unconfined) at the surface was kept constant in both the steady-state and transient-state model runs.

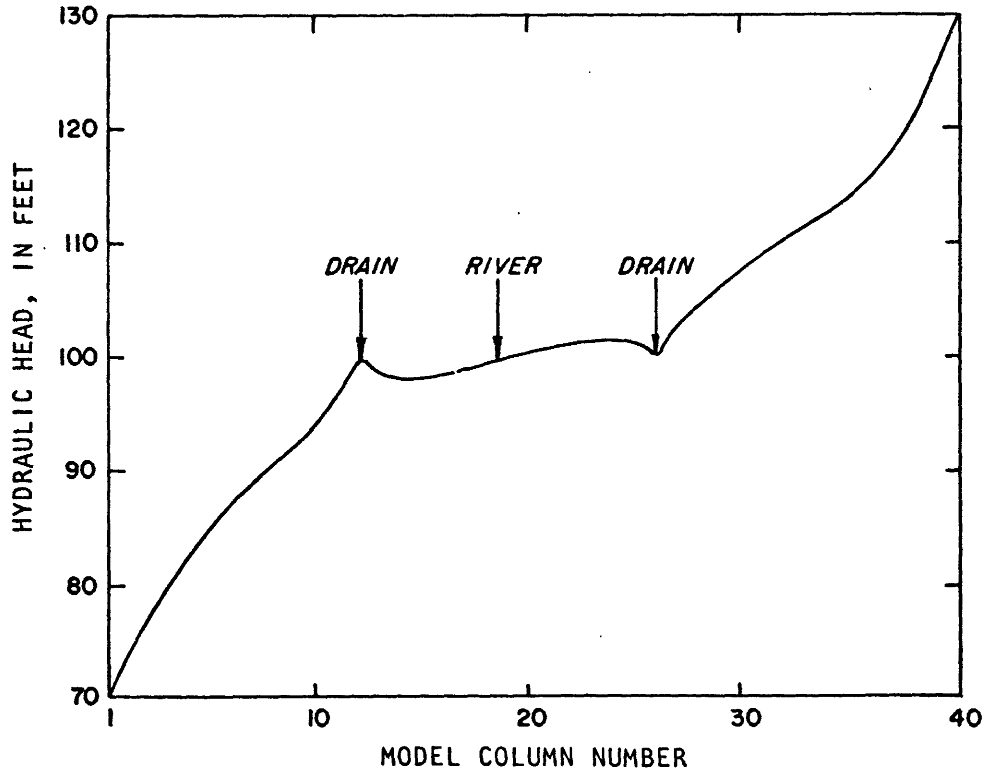

Figure 4.--Steady-state hydraulic-head values for a losing reach.

Surcharges, applied only at the river, were varied in separate model runs using the aforementioned values of hydaulic head, hydraulic conductivity, and specific storage to determine the rates of loss from the river and drains. Steady-state values of flow were subtracted from the final values of flow to arrive at seepage losses due only to the increased water level in the river. The resulting rates of loss, integrated over time, yielded a volume of seepage for a river length of 50 feet. The surcharges ranged from 0.5 to 3 feet for durations ranging from 1 to 100 days. For example, for one model run, the water surface of the river was raised 1 foot (1 foot of surcharge) and applied for 1 day (1 day of duration).

Once the volume of water lost by the surface system during surcharge was determined, it was then necessary to determine the quantity of water that would return to that system once the surcharge on the river was removed. The difference between the original losses and the return flow represents the net seepage loss for a particular surcharge that was applied for a specified period of time.

To accomplish this, the constant hydraulic-head value of the river was returned to 100 (the initial value) and the model was used to calculate flow rates back into the surface flow system for 180 days. It was determined that 180 days were sufficient to account for most of the return flow without experiencing significant model error. The rates of gain or loss in the surface system were again adjusted by subtracting steady-state rates.

The incremental volumes of seepage per specified time step determined by the model for 1 foot of surcharge applied for 1, 10, and 100 days with 180-day return flow are plotted in figures 5-7. The positive values of volume represent water flowing into the aquifer (stream loss). The negative volumes represent water flowing back into the river from the aquifer (stream gain). The increased flow volumes calculated for 10 days and 100 days of application tend to minimize the rounding errors. A tabulation of the volumes of both losses and gains for 1 foot of surcharge applied for 1 day and 180 days of return flow is shown in tables 2 and 3 for the river and drains for the specified time steps. The shape of the curves for 0.5, 2, and 3 feet of surcharge is similar to that shown for the 1-foot curves. The magnitudes of the volumes for the 0.5, 2, and 3 feet surcharge obviously are different.

Flux (discharge) rates computed by the model at both the eastern and western boundaries ranged from 0.002 to 0.0006 cubic foot per second (unadjusted for steady state) at the end of the surcharge application period for 50 feet of channel length. The flux rates at both boundaries also ranged from 0.002 to 0.0006 cubic foot per second (unadjusted for steady state) at the end of the 180-day return period.

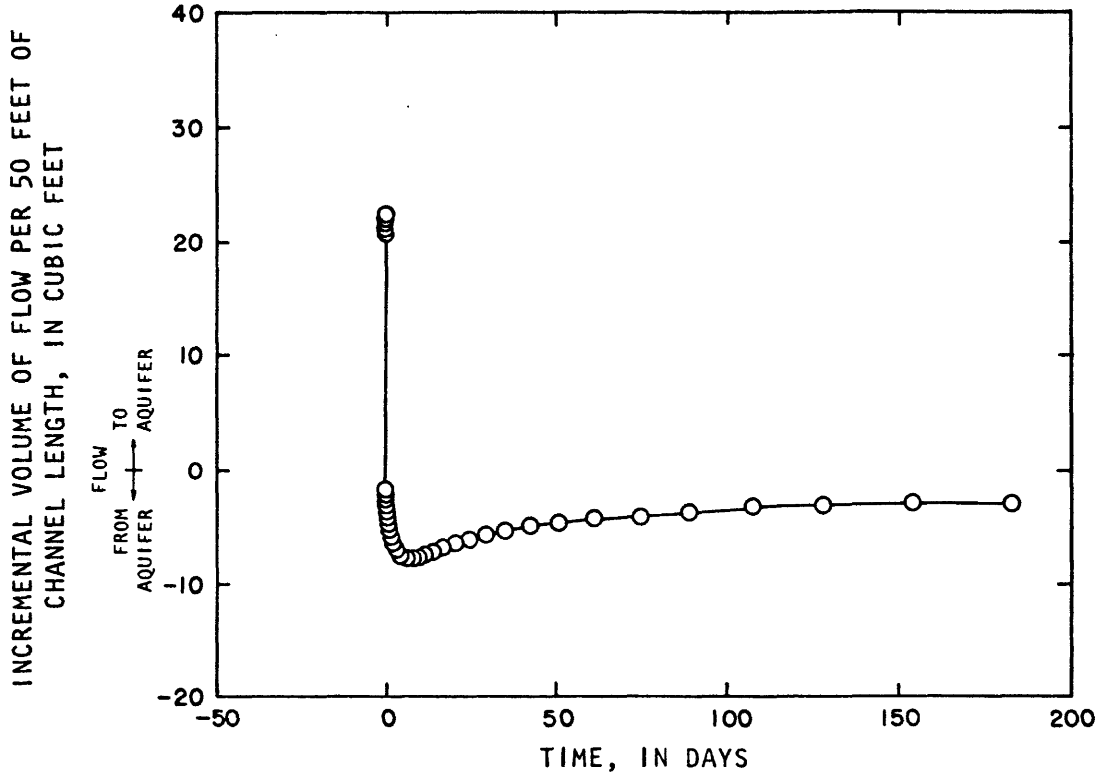

Figure 5.--Incremental flow volumes per specified time step for 1 foot of surcharge, 1 day of application, and 180 days return flow in a losing reach.

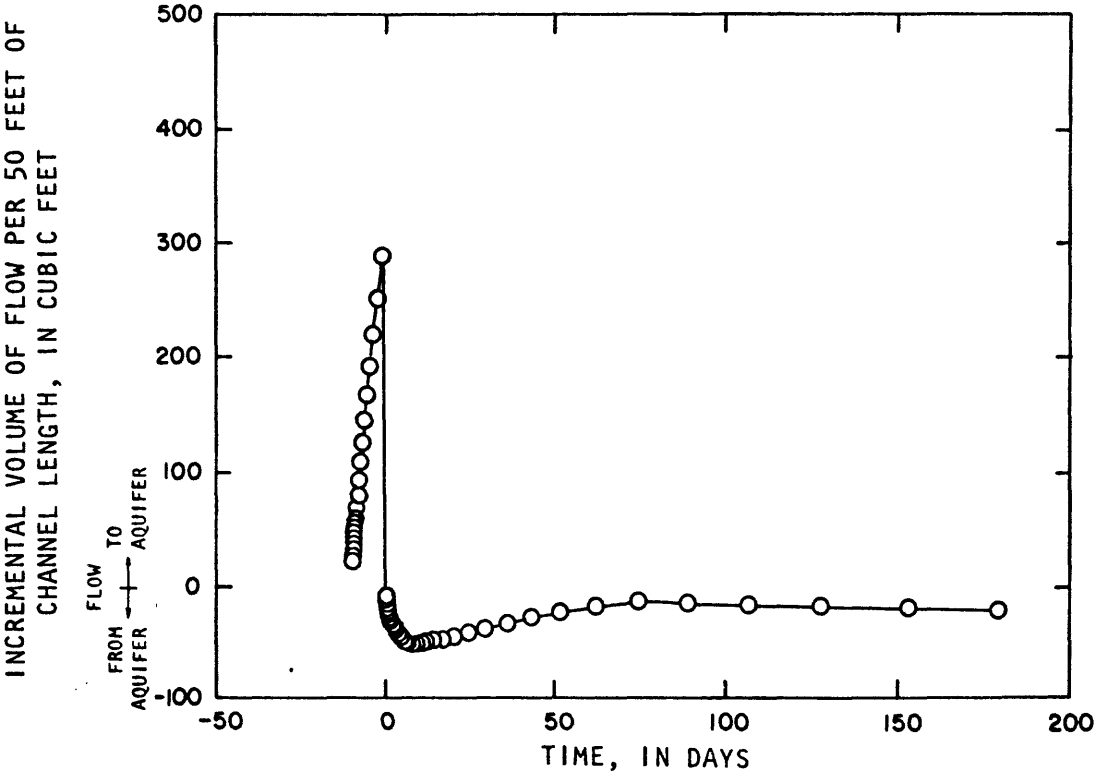

Figure 6.--Incremental flow volumes per specified time step for 1 foot of surcharge, 10 days of application, and 180 days return flow in a losing reach.

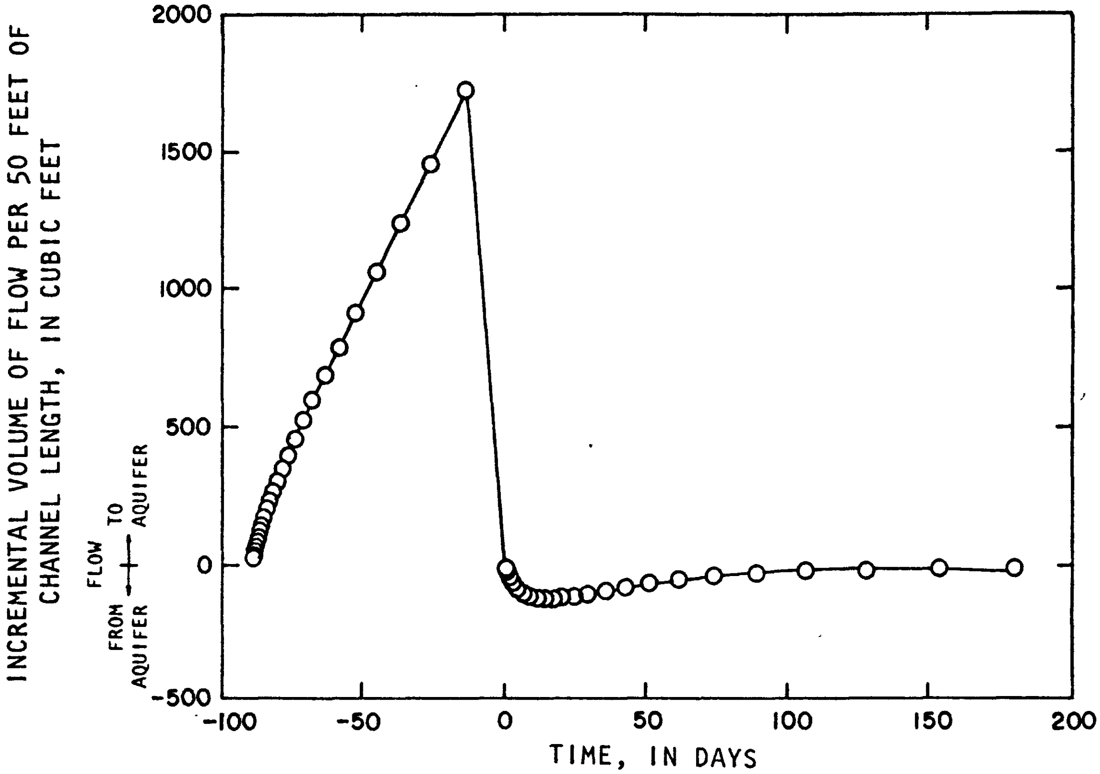

Figure 7.--Incremental flow volumes per specified time step for 1 foot of surcharge, 100 days of application, and 180 days return flow in a losing reach.

Table 2. Cumulative flow volumes to aquifer from river for 1 foot of surcharge applied for 1 day for a losing reach

Cumulative storage for drain (cubic feet)	Cumulative storage for river (cubic feet)	Cumulative storage for drain (cubic feet)	Time (hours)
0.02	22.70	0.02	2.0
.06	44.99	.06	4.0
.11	67.08	.11	6.0
.18	89.01	.17	8.0
.24	110.78	.24	10.0
.32	132.39	.31	12.0
.39	153.86	.39	14.0
.47	175.18	.47	16.0
.56	196.35	.55	18.0
.65	217.38	.64	20.0
.73	238.27	.73	22.0
.83	259.03	.82	24.0

Table 3. Cumulative flow volumes from aquifer to river for 1 foot of surcharge applied for 1 day with 180 days of return flow for a losing reach

Cumulative storage for drain (cubic feet)	Cumulative storage for river (cubic feet)	Cumulative storage for drain (cubic feet)	Time (hours)
0.07	1.82	0.07	1.5
.13	3.68	.13	1.8
.20	5.80	.19	4.0
.27	8.27	.26	6.6
.35	11.13	.33	9.7
.44	14.43	.42	13.4
.54	18.21	.52	17.8
.67	22.51	.65	23.2
.83	27.37	.80	29.7
1.01	32.79	.98	37.4
1.24	38.76	1.20	46.7
1.53	45.28	1.47	57.9
1.87	52.26	1.79	71.2
2.30	59.64	2.20	87.3
2.82	67.32	2.69	106.6
3.47	75.19	3.30	129.7
4.27	83.13	4.04	157.4
5.2	91.01	4.96	190.7
6.46	98.80	6.10	230.6
7.92	106.37	7.48	278.5
9.68	113.67	9.14	336.0
11.75	120.64	11.08	405.0
14.14	127.23	13.30	487.9
16.81	133.42	15.76	587.2
19.71	139.17	18.41	706.5
23.01	145.31	21.48	849.6
26.83	151.83	24.95	1,021.3
31.17	158.79	28.86	1,227.3
36.06	166.24	33.24	1,474.6
41.69	174.31	38.26	1,771.3
48.00	183.12	43.84	2,127.4
55.22	192.63	50.18	2,554.6
63.47	203.00	57.16	3,067.4
72.85	214.41	65.03	3,682.6
83.51	226.90	73.86	4,320.0

Estimated net seepage from the river is calculated by tabulating the total adjusted volume of flow into the aquifer and subtracting the cumulative volume of return flow at a particular time. In order to calculate the incremental net seepage rate per mile of stream length, the net volume of seepage computed by the model for 50 feet of channel length was multiplied by a factor to determine the volume for 1 mile. This volume of water is the seepage loss determined for the application period in days. A factor was used to compute the volume of loss per second. For example, for 1 foot of surcharge, 1 day of application, and 180 days of return flow from the aquifer (tables 2 and 3), the net seepage rate was determined as follows:

$$\frac{(259 - 227) \text{ cubic feet}}{(50 \text{ feet of stream length}) (1 \text{ day of application})} \times \frac{5{,}280 \text{ feet}}{1 \text{ mile}} \times \frac{1 \text{ day}}{86{,}400 \text{ seconds}}$$

$= 0.04$ cubic foot per second per mile of stream length

The net seepage values reported as cubic feet per second per mile of channel length have been plotted in figures 8, 9, and 10 for 7, 30, and 180 days of return flow and for 1, 10, and 100 days of applied surcharge. The graphs indicate that only very small changes in the rate of net seepage occur as the surcharge is applied for longer periods of time. Additionally, the rate of net seepage decreases as the return period is increased. The rate of net seepage increases significantly as the height of surcharge is increased.

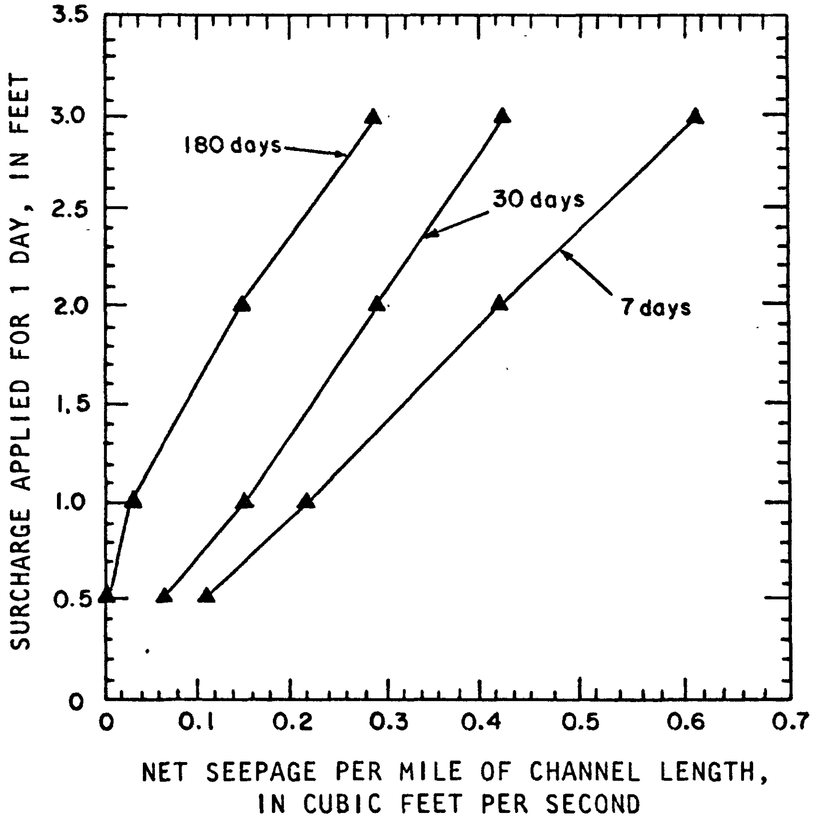

Figure 8.--Seepage rates for 1 day of application for the indicated return-flow period in a losing reach.

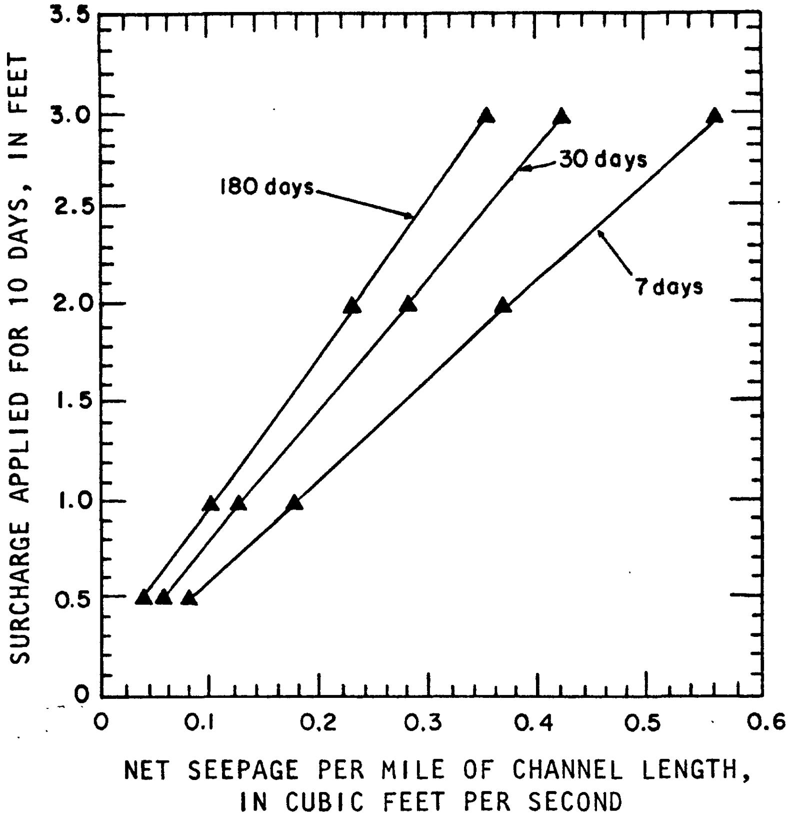

Figure 9.--Seepage rates for 10 days of application for the indicated return-flow period in a losing reach.

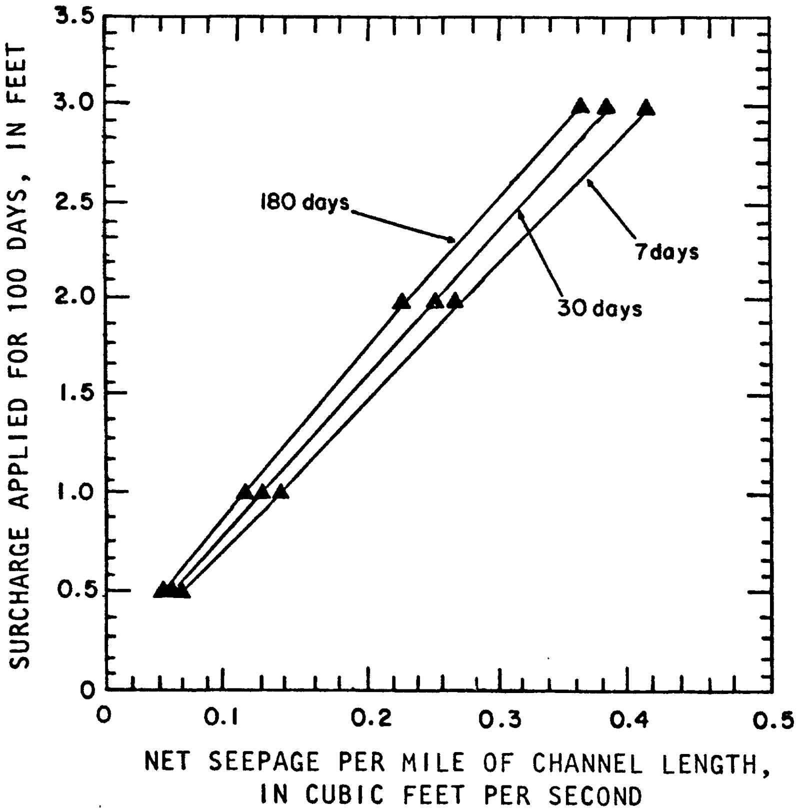

Figure 10.--Seepage rates for 100 days of application for the indicated return-flow period in a losing reach.

Gaining Reach

The physical relationship between the elements in the cross section for a gaining reach is the same as that for the losing reach (fig. 2). However, for a gaining reach the water table slopes toward the river from both directions.

The values of hydraulic conductivity, specific storage, and hydraulic head assigned for use in the model are the same as those used for the losing reach except that the constant hydraulic-head values at the west boundary are 130 feet. Hydraulic-head values at steady state are plotted in figure 11. The procedure followed in determining rates of gains and losses and the subsequent calculation of net seepage volumes are the same as used for the losing reach.

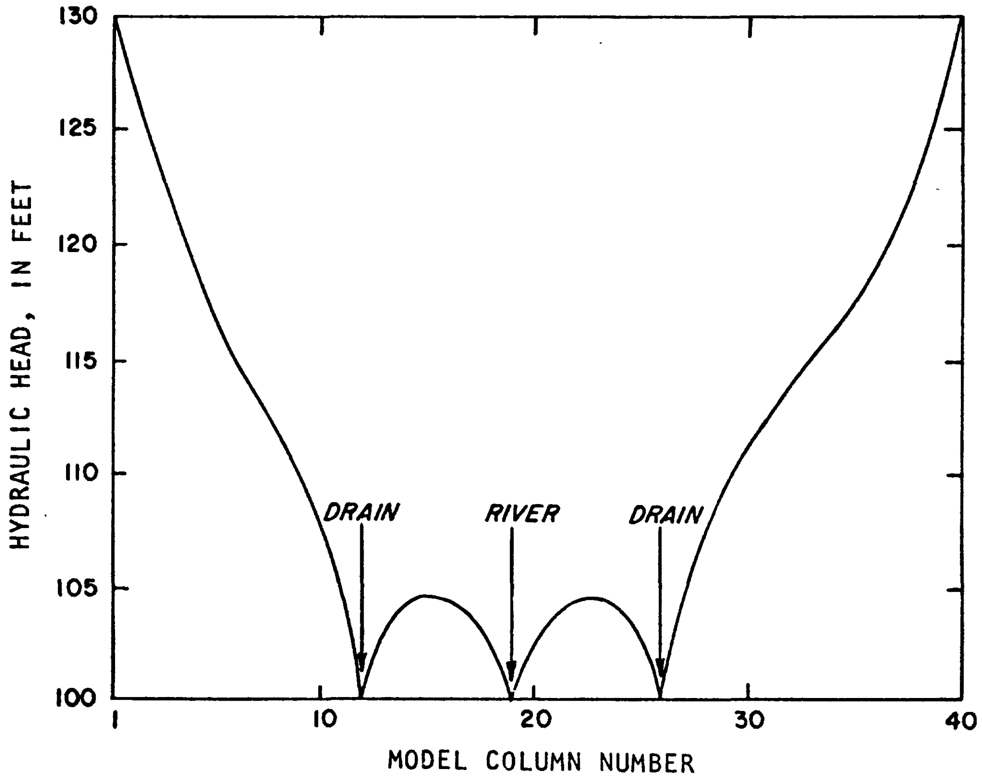

Figure 11.--Steady-state hydraulic-head values for a gaining reach.

The rates of net seepage computed from model results for a gaining reach were nearly identical to rates calculated for the losing reach for the same surcharge and duration of application. The rates of net seepage for the gaining reach have been listed in table 4.

Table 4. Net seepage rates for gaining reach

[cubic feet per second per mile of channel length]

Surcharge (feet)	Duration of application (days)		
	1	10	100
	180-DAY RETURN FLOW		
0.5	0.0	0.06	0.06
1.0	.04	.12	.12
2.0	.11	.24	.24
3.0	.31	.36	.37
	30-DAY RETURN FLOW		
0.5	0.07	0.07	0.06
1.0	.15	.14	.13
2.0	.24	.29	.26
3.0	.45	.43	.39
	7-DAY RETURN FLOW		
0.5	0.11	0.10	0.07
1.0	.22	.19	.14
2.0	.30	.38	.28
3.0	.64	.57	.42

Reach Affected by Ground-Water Pumpage

The effects of ground-water pumpage on the Rio Grande are assumed significant only in the Albuquerque area, where pumpage rates have been reported to be 67,951 acre-feet per year (Kelley, 1982), and are great enough to affect the water table in the flood plain. The physical dimensions for the pumping-affected cross section defined for the model are the same as those for the losing reach (fig. 2). In order to represent the effects of pumpage, the water table to the east of the river was assumed to have no slope. The constant hydraulic head at the eastern boundary was 100 feet.

The procedures followed in the computation of the rates of net seepage for various surcharges are the same as those followed for the losing reach. Hydraulic-head values computed at steady state are plotted in figure 12. The net seepage rates determined for the reach effected by pumpage are nearly identical to those computed for the losing reach (figs. 8-10). The net seepage rates for the pumping-affected reach are listed in table 5.

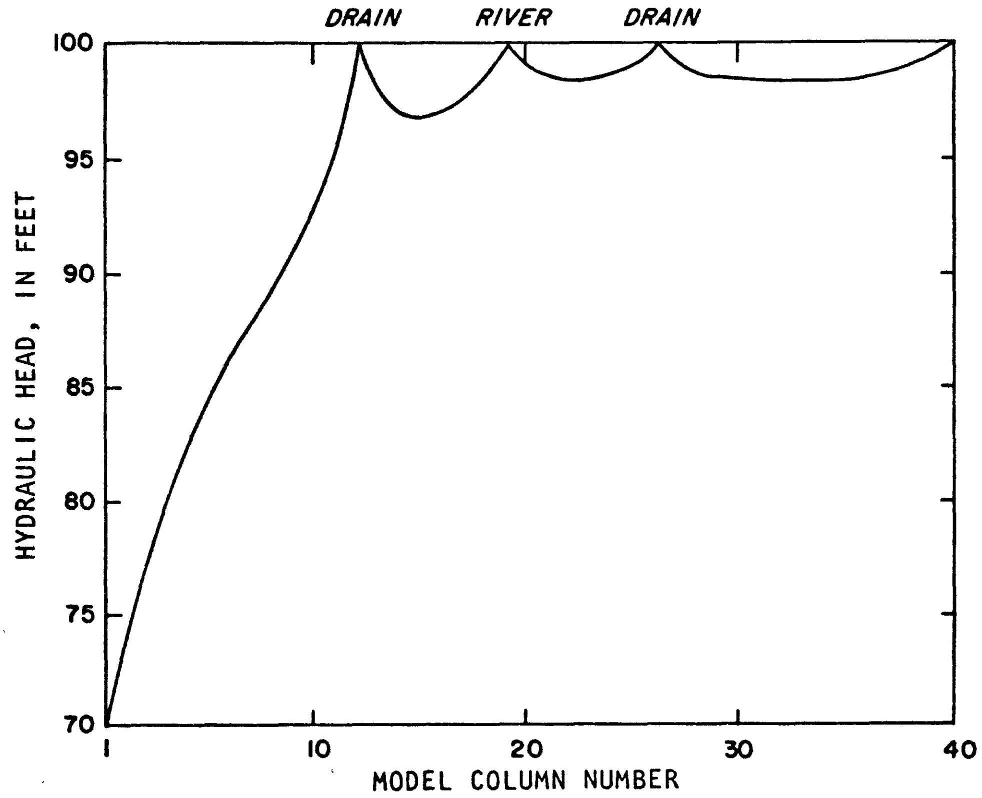

Figure 12.--Steady-state hydraulic-head values for a reach affected by pumping of ground water.

Table 5. Net seepage rates for reach affected by pumping of ground water (cubic feet per second per mile of channel length)

Surcharge (feet)	Duration of Application (days) 1	10	100
	180-DAY RETURN FLOW		
0.5	0.0	0.05	0.06
1.0	.03	.12	.12
2.0	.15	.24	.24
3.0	.30	.36	.37
	30-DAY RETURN FLOW		
0.5	0.07	0.07	0.06
1.0	.15	.14	.13
2.0	.30	.29	.26
3.0	.44	.43	.39
	7-DAY RETURN FLOW		
0.50	0.15	0.09	0.07
1.0	.22	.19	.14
2.0	.43	.38	.28
3.0	.65	.57	.42

Model Sensitivity

The values assigned to the variables (hydraulic head, specific storage, hydraulic conductivity) were chosen to represent typical properties of the stream-aquifer system. The values were then investigated to see how the resulting rates of net seepage would be affected by changes in those values.

The values assigned to the various hydraulic heads have already been investigated in the model runs previously described. The hydraulic-head values that were used to represent the differing streamflow conditions (losing reaches, gaining reaches, and reaches affected by ground-water pumpage) also served as an analysis of the effects of hydraulic-head values on the net seepage rates. Model runs using the different hydraulic-head values resulted in net seepage rates nearly identical for each surcharge simulated. Therefore, varying the hydraulic-head values used in the analysis can be seen to have little effect on the results.

In order to determine the effects that the other variables have on the seepage rates, the losing reach model was rerun with 1 foot of surcharge for 1 day using different values of the variables. Hydraulic conductivity was first varied from 30 to 50 feet per day for the alluvium and 10 to 30 feet per day for the Santa Fe Group. The resulting rates of net seepage ranged from 0.03 to 0.04 cubic foot per second per mile of channel length (table 6). The rate computed using the original hydraulic conductivities of 40 feet per day for the alluvium and 20 feet per day for the Santa Fe Group is 0.03 cubic foot per second per mile of channel length (table 6).

Specific storage was varied from 2×10^{-7} to 2×10^{-6} per foot of aquifer thickness for confined conditions. The net seepage rates resulting from these model runs are listed in table 6. These rates of 0.03 and 0.01 cubic foot per second per mile of channel length did not differ significantly from the value of 0.03 cubic foot per second per mile of channel length for the model simulation with specific storage set at 1×10^{-6}.

The ratio of vertical to horizontal hydraulic conductivity was varied from 1:10 to 1:1,000. The results in table 6 show a net seepage rate of 0 cubic foot per second per mile of channel length for the ratio 1:10 and 0.05 cubic foot per second per mile of channel length for the ratio 1:1,000.

The effect of changes in the physical boundaries of the cross section upon the rates of net seepage was investigated. First, the eastern and western boundaries of the cross section were extended an additional 0.5 mile while holding all other variables constant. Second, the flood-plain width was held at 3 miles while the thickness of the alluvium was increased from 100 to 200 feet and the thickness of the Santa Fe Group was increased from 1,000 feet to 1,500 feet. The net seepage rates for 1 foot of surcharge are 0.02 cubic foot per second per mile of channel length for the simulation with the increased flood-plain width and 0.09 cubic foot per second per mile of channel length for the simulation with the increased thickness of the underlying formations (table 6).

The sensitivity analysis indicated that only changes in the thickness of underlying formations caused significant changes in the net seepage rates. Further refinement for these generalized estimates was not considered necessary.

Table 6. Net seepage rates determined during model sensitivity analysis

Hydraulic conductivity (feet per day)	Specific storage (per foot of aquifer thickness)	Flood-plain width (miles)	Ratio of vertical to horizontal hydraulic conductivity	Alluvium thickness (feet)	Santa Fe Group thickness (feet)	Net seepage (1 foot of surcharge, 1 day of application, 180 days of return flow, in cubic feet per second per mile of channel length)
Alluvium = 40 Santa Fe = 20	1×10^{-6}	3.0	1:250	100	1,000	0.03
Alluvium = 30 Santa Fe = 10	1×10^{-6}	3.0	1:250	100	1,000	.03
Alluvium = 50 Santa Fe = 30	1×10^{-6}	3.0	1:250	100	1,000	.04
Alluvium = 40 Santa Fe = 20	2×10^{-7}	3.0	1:250	100	1,000	.03
Alluvium = 40 Santa Fe = 20	2×10^{-6}	3.0	1:250	100	1,000	.01
Alluvium = 40 Santa Fe = 20	1×10^{-6}	3.0	1:10	100	1,000	0.0
Alluvium = 40 Santa Fe = 20	1×10^{-6}	3.0	1:1000	100	1,000	.05
Alluvium = 40 Santa Fe = 20	1×10^{-6}	4.0	1:250	100	1,000	.02
Alluvium = 40 Santa Fe = 20	1×10^{-6}	3.0	1:250	200	1,500	.09

CONCLUSION

The magnitude of potential seepage losses has been found to be nearly identical for the three cases simulated in the model (losing reach, gaining reach, and pumpage-affected reach). The seepage values ranged from 0.0 cubic foot per second per mile of channel length for a 0.5-foot surcharge applied for 1 day to 0.37 cubic foot per second per mile of channel length for a 3-foot surcharge applied for 100 days, when each surcharge was followed by a 180-day return flow from the aquifer.

The values of aquifer properties used in the model simulations generally are representative of actual values found within the study area. The physical boundaries used in the model are representative of three types of flow conditions (losing reaches, gaining reaches, and pumpage-affected reaches). However, the model simulations were not representative of conditions found along the low-flow conveyance channel beginning at San Acacia.

The results are only estimates of potential seepage losses for the study area. Their main value is to indicate the potential for surcharge seepage losses. In order to further refine the estimates of seepage losses, more detailed investigations would need to be performed.

REFERENCES

Bjorklund, L. J., and Maxwell, B. W., 1961, Availability of ground water in Albuquerque, Bernalillo and Sandoval Counties, New Mexico: New Mexico State Engineer Technical Report 21, 117 p.

Dane, C. H., and Bachman, G. O., 1965, Geologic map of New Mexico: U.S. Geological Survey map, scale 1:500,000.

Kelley, V. C., 1982, Albuquerque, its mountains, valley, water and volcanoes: New Mexico Bureau of Mines and Mineral Resources Scenic Trips to the Geologic Past No. 9, 106 p.

Lagasse, P. F., 1980, An assessment of the response of the Rio Grande to dam construction--Cochiti to Isleta Dam: U.S. Army Corps of Engineers Technical Report, var. p.

Lohman, S. W., 1972, Ground-water hydraulics: U.S. Geological Survey Professional Paper 708, 70 p.

Trescott, P. C., Pinder, G. F., and Larson, S. P., 1976, Finite-difference model for aquifer simulation in two dimensions with results of numerical experiments: U.S. Geological Survey Techniques of Water-Resources Investigations, Book 7, Chapter C1, 116p.

U.S. Bureau of Reclamation, 1955, San Juan-Chama project-- Definite plan report, Appendix D-- Hydrology: 475 p.

Waldron, J. F., 1956, Reconnaissance geology and ground water study of a part of Socorro County, New Mexico: Stanford University, School of Mineral Sciences, Palo Alto, California, unpublished Ph.D. dissertation, 255 p.

Wilson, C. A. and White, R. R., 1984, Ground-water conditions in the central Mesilla Valley, Dona Ana County, New Mexico: U.S. Geological Survey Water-Resources Investigations Report 82-555, 144 p.