FINAL PROJECT REPORT ON MONITORING OF THE UNSATURATED ZONE AND RECHARGE AREAS AT INEL

to the

State of Idaho INEL Oversight Committee

prepared by

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March 1994

INTRODUCTION

This project, begun in March 1991, was originally structured as two separate research efforts: an investigation of the recharge phenomenon and surface water-ground water interactions at the INEL; and a study of water and contaminant movement through the unsaturated zone, including a review of computer models used to describe this process. During the initial months of work, it became obvious to those involved in these studies that the two topic areas were intimately related, and work since that time has proceeded with no firm boundaries between the two efforts. Graduate students and faculty members associated with the project have therefore been cognizant of most of the separate individual efforts being conducted throughout the project's duration.

Much of the Phase I work (March 1991-March 1992) consisted of a detailed review of available literature pertinent to the two research topics and to the INEL site. These literature reviews continued through Phase II (March 1992-March 1993), culminating in a Technical Report, "Abstracts and Parameter Index Database For Reports Addressing the Unsaturated Zone and Surface Water-Ground Water Interactions at the INEL" (State of Idaho INEL Oversight Program Technical Report 93-xx, March, 1993).

At the completion of Phase II in March 1993, an Annual Report was submitted to document the progress on three separate continuing research efforts:

1. An evaluation of computer model algorithms and data requirements for modeling the transport process in the unsaturated zone (Dr. Jim Liou).
2. An effort to predict the growth and decay of the ground water mound beneath the INEL spreading basins, using the computer model UNSAT-2 (Dr. John Finnie).

3. A nearly completed study to examine the recharge rates associated with stream flow in the Big Lost River, and the effects of this recharge on ground water levels at the INEL site (Dr. Dennis Horn).

During this final Phase III of the project, these three separate studies have either been completed or are now nearing completion. Therefore, the purpose of this Phase III Report is to summarize the progress and status of the research, and, in the cases of two of the studies, to present a final completion schedule consistent with the academic progress of the Graduate Assistants who have been involved. The Report is divided into three sections, each documenting the work on each separate study effort.

SECTION I

Prediction of Wetting Front Travel Times

Beneath the Spreading Basins of the Big Lost River

By John Finnie

This report summarizes the activities of Dr. John Finnie and Graduate Assistant Damon McAlister under the research grant for the second and third year of funding. The specific goals addressed by them, along with Dr. Dennis Horn and Graduate Assistant Erik Coats, that pertain to R&D Project 2&3 include the following:

(5) Evaluate existing monitoring systems for capability of detection of ground water impacts. The evaluation should include the location, timing, magnitude, and quality of surface water infiltration including construction, instrumentation, sampling, and analysis methods.

(6) Locate and review data from monitoring and studies of surface water/ground water inter-relationships.

(7) Review modeling investigations on recharge, and by means of limited sensitivity, assess the capability of the models to accurately treat variable inputs of recharge quantity and quality, and the effects of those inputs on ground water flow rates, and water chemistry, including contaminant concentrations and distributions.

(8) Make recommendations to DOE to improve existing monitoring system design and performance and for follow-up research and development projects to fill data gaps and improve our understanding of important system parameters and processes in the unsaturated zone, perched water areas and recharge areas.

Goal #5 was addressed by Horn and Coats - They reviewed existing infiltration data and studies and included them in their report and in the abstracts.

Goal #6 was completed by Horn, Coats, Finnie, and McAlister. The data and studies have been reported in the abstracts as well as in the report of Horn and Coats. The upcoming report of Finnie and McAlister will also review data and studies.

Goal #7 has been partially met by completion of the research of Horn and Coats. They reviewed previous efforts to model surface water/ground water relationships and completed a statistical study that estimated the time lag between surface seepage and elevations of ground water.

The rest of goal #7 will be met upon the completion of the research of Finnie and McAlister. They have reviewed existing data and studies at the spreading basins and the Radioactive Waste Management Complex, and have been applying a computer program to model the flow of water in the unsaturated zone between the surface and the ground water table. Their research is not complete, but they expect to complete it during the spring semester, 1994. A more detailed description of their work is included below.

Goal #8 has been partially met by recommendations included in the report of Horn and Coats. Finnie and McAlister will also include specific recommendations in their forthcoming report. Other recommendations for research have been included in a number of research proposals by Horn, Finnie, and others.

<u>Report on modeling of groundwater travel times:</u>

The following progress has been made by Damon McAlister and John Finnie toward completion of their specific research goals. Damon is planning to complete his thesis during the spring semester. After it is finished, the final report will be sent to the Oversight Committee.

The specific goal of our research project was to predict the growth and decay of the ground water mound beneath the spreading basins with computations using the computer model UNSAT-2. The following paragraphs describe our activity.

During the summer of 1992, we became familiar with the program UNSAT-2 for use in solving unsteady seepage problems. The program was tested on example problems for both time step and grid size independence by varying these parameters in a given soil column. Since the same results were obtained when these parameters were varied, we knew we had achieved time step and grid independence for these example problems.

Since the results of these problems were deemed satisfactory, UNSAT-2 was then tested against the results of other existing computer programs, i.e. UNSAT-H, FLASH, & FEMWATER. Output results were compared to those published in Baca and Magnuson's "Independent Verification and Benchmark Testing of the UNSAT-H Computer Code, Version 2.0." The test results indicated that UNSAT-2 can be used with confidence in attempting to model surface water - ground water interactions in settings similar to the spreading basins.

Many difficulties with UNSAT-2 were experienced. For large array sizes, personal computers could not be used. In the past, the program had been run on the university main-frame computer, an IBM 4300. In order to adopt the program to local computers, changes and modifications had to be made to the FORTRAN code to handle large array sizes, to read data input files, and produce output files that could be more easily used. After finishing these modifications, we found that the CPU-time consumed during computer simulations was too large, with some problems taking days to complete. It became obvious that we needed to either use a Cray supercomputer, like BACA and Magnuson, or find a more efficient computer program. Based on discussions with Jim Liou and his graduate assistant, we decided to switch to the computer program VS2DT. We tested VS2DT against UNSAT-2 on a multiple layer porous media problem and obtained good agreement between them at a great reduction in computation time. Based on this test, we are satisfied that these codes produce similar results.

One important test of a computer program is to compare its results to known solutions. To do this, we applied analytical ("hand") methods to predict conditions under steady state flows for a given flux. Computational results of UNSAT-2 and VS2DT were successfully tested against these analytical results. We did find out that it is more difficult to obtain steady state solutions than unsteady solutions! Apart from their use to verify computer results, these analytical methods could be very valuable for determining the sensitivity of predictions to changes in the soil parameters for steady state flow.

We have completed all of the planned one-dimensional flow simulations for the unsaturated zone under the spreading basins. Since not all of the results have been completely analyzed, the results reported here should be considered preliminary.

We simplified the stratification in the vadose zone to three soil zones: massive basalt, vesicular basalt, and interbed sediment. We are assuming that 90% of the vadose layer is made up of basalt-flow groups. From the normalized distribution of the basalt-flow characteristics, we are assuming the basalt to be 49% massive and 51% vesicular. Van Genutchen parameters for these soils were obtained from reports on the hydraulic characteristics of soils at the RWMC.

Based on our "most-likely" estimates of input data, we have computed travel times for the wetting front from the surface to the ground water of about 10 years. Our calculations do not include the correct number of layers of sediment and basalt, nor are vertical cracks included in the basalt. The computed travel times are longer than indicated by the work of Dennis Horn and Erik Coats.

In order to test the sensitivity of these results to changes in data, we have changed the following parameters and computed new travel times:

Unsaturated "van Genutchen" soil parameters:

These parameters describe the relationship between conductivity, soil tension, and soil moisture content, and have been varied over a range of plus and minus 50%. Travel times for these changes range from 9 to 16 years.

Soil layers:

The number, type, and depth of layers of porous media have been varied to determine their effect on travel time. Our analysis of these results is not complete, but will be reported in Damon McAlister's thesis and in the report to be presented to the INEL Oversight Committee.

Numerical analysis parameters:

During these calculations, we found that parameters relating to the computation method can have as large an effect as physically based parameters. These parameters include the factor that controls damping within each iteration step, and the number that determines when each iteration step is "close enough".

Our original intent was to complete the one-dimensional computer modeling and graduate to a computer model of two-dimensional unsaturated ground water flow. Due to the amount of work required for the one-dimensional simulation, and the computer resources available, we will not be able to complete the two-dimensional simulation.

Final Report

The report on the research of John Finnie and Damon McAlister will be sent to the Oversight Committee after the completion of Damon's thesis, which is expected Spring semester. Their report will examine the effect of variations in soil parameters and strata on infiltration rates. The variables being studied fall into three groups: (1) the van Genutchen parameters which relate unsaturated hydraulic conductivity to moisture content and head (or matric potential), (2) the order of strata within the porous media, and (3) numerical analysis parameters. The calculations being reported are only one-dimensional in nature, and have only 3 different layers of porous media. They will show estimated wetting front travel times from the surface to the groundwater in the order of 10 to 20 years. This estimate is more than that calculated by Coats and Horn, but less than previously published estimates.

SECTION II

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provided by Dr. Jim Liou

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MODELING OF TRANSPORT IN VADOSE ZONE

(Dr. Jim C. P. Liou, Department of Civil Engineering,University of Idaho, May, 1994)

I. Introduction

Water and solute transport in the unsaturated zone at the Idaho National Engineering Laboratory site is one of the focuses of the State of Idaho INEL Oversight Program. This report addresses aspects of water and solute transport modeling.

There are several numerical models that may be used to simulate specific problems at the site. Four are reviewed in detail. Their assumptions, features, and data requirements are compared. Three of the four are being used by INEL for various studies. The fourth one, VS2DT of the U.S. Geological Survey, was made operational on a workstation and on a personal computer. Simulations were made to compare to INEL study results and to explore modeling approaches.

A large amount of information on the geologic and hydraulic properties of the sites is available. The literature revealed highly nonuniform and complex sites at INEL. This report addresses the vadose zone at the Radioactive Waste Management Complex (RWMC). An overview of the site characteristics pertinent to modeling is provided. The advancement of a wet front in the vadose zone following the 1969 flooding of Pit 10 was simulated. The results compared favorably with those in an INEL study, which were obtained from a different model. The results also showed that fractures in the basalt strongly influence water movement.

Model results are uncertain since uncertainties in the models and in the model parameters exist. Model uncertainties can be regarded as bias errors. Bounds for model uncertainties can be estimated by examining modeling assumptions and solution accuracies, and by comparing results from several models. Uncertainties in modeling parameters are attributed to the fact that the parameters are distributed in space and their values are only approximately known at isolated locations. Fractures in the Basalt layers are examples of such uncertainties. Model parameter uncertainties can be regarded as random. For uncertain model results to be meaningful, their central tendencies and the extent of scatter must be quantified. A probabilistic approach to modeling must be adopted.

The Monte Carlo method has been used to estimate water travel time in the vadose zone. Huge numbers of simulations are needed in Monte Carlo simulations. Because the models are complex and CPU intensive, the Monte Carlo method can only handle a few random variables. For vadose zone modeling, a large number of model parameters are random variables, hence it is all but impossible to use the Monte Carlo method. Alternative methods must be sought.

An approximate probabilistic method proposed by Rosenblueth and Harr is outlined and is explored for vadose zone modeling. An example problem is simulated, probabilistically, using both the Monte Carlo method and the approximate method. The latter obtains comparable results with negligible effort relative to that demanded by the former.

2. Review of Computer Simulation Models

There are several computer codes that simulate water flow and solute transport in variably saturated subsurfaces. Case et al. (1989) identified and gave a brief description for FEMWATER/FEMWASTE, PORFLO, SUTRA, and TRACER3D. The codes in use at INEL include FLASH, FLOWMC, PORFLO 2D & 3D, and UNSAT-H. In addition, VS2DT, a code developed by the U.S. Geological Survey also simulates water movement and solute transport in the vadose zone.

After reviewing the general features, the computer requirements, and the availability of documentation of these codes, TRACER3D, PORFLO-3, UNSAT-H Version 2.0 and VS2DT were chosen for a more in-depth review. Based on the reports documenting these codes (Travis, (1984), Runchal and Sagar (1989), Fayer and Jones (1990), Lappala et al. (1987), and Healy (1990)), the main features of these codes are compared in Table 1. The data requirement and the initial/boundary conditions each code can handle are shown in Tables 2 through 5.

All four codes model time-dependent problems. They all use finite differences in obtaining numerical solutions. Over twenty model parameters and conditions need to be specified. Much greater amounts of data are required for 2 and 3 dimensional problems. All can produce a very large amount of output and all provide user output choices.

According to the VS2DT code developer, about 500 copies have been distributed. A preprocessor for data file preparation is available from a third party. Similar information for the other three codes is unknown.

Table 1 - Model Feature Comparison

FEATURES	TRACR3D	PORFLO-3	USAT-H	VS2DT
Dimensionality	3	3	1	2
Single/Two Phase	T	S	T	S
Deformable Media	Y	N	N	Y
Space Dependence	Y	Y	layered	Y
Anisotropy	Y	Y	N	Y
Heterogeneuty	Y	Y	N	Y
Fractures	H & V	Plane	N	Y
Fracture Network	Y	?	N	?
Randomized Fracture	Y	?	?	?
Variable Saturation	Y	Y	Y	Y
Isothermal Flow	Y	N	Y	Y
Heat Flow Modeled	N	Y	Y	N
Equation of motion	Forvhheim	Darcy	Darcy	Darcy
moisture and relative conductivity function[1]	BC, T	G, BC, T	G,BC, H,P	G,BC, T,H
Porosity - Pressure	Y	N	N	Y
Pore Structures	N	Y	N	N
Chemical Transport	Y	Y	N	Y
Heat Transport	N	Y	Y	N
No. of Species	several	1	0	1
Modes of Transport[2]	ADDDS	ADDDS	AD	ADDS
Computer Platform	mainframe	mainframe	PC	PC

1): BC=Brook and Corey, G=van Genuchten, H=Haverkamp, T=table, P=polynomial
2): A=advection, DDD=diffusion, dispersion, & decay, DD=diffusion & dispersion, S= sorption

Table 2 - Data and Initial/Boundary Conditions for TRACR3D

Hydraulic Properties of the Porous Matrix

1) saturated permeabilities (x, y, z, values)
2) porosity
3) irreducible water saturation
4) pore-size distribution index
5) bubbling pressure
6) constricivity coefficient
7) capillary pressure (matrix potential) vs. air saturation (Brooks & Corey or table)
8) air relative permeability (Brooks & Corey or table)
9) water relative permeability (Brooks & Corey or table)

Mechanical Properties of the Matrix

10) average particle size (used in computing dispersion)
11) compressibility of the matrix
12) bulk matrix density

Transport Properties

13) adsorption coefficient for each tracer
14) desorption coefficient for each tracer
15) adsorption limit for each tracer
16) solubility limit for each tracer
17) molecular diffusivity of tracers in host fluid
18) half lives of tracers
19) molecular weight of tracers

Fluid Properties

20) molecular weight of liquid and gas
21) viscosity of liquid and gas
22) mass density of liquid
23) compressibility of liquid

Initial Conditions

24) initial air-saturation distribution
25) initial pressure distribution
26) initial tracer concentration distribution
27) initial values of adsorbed species
28) stagnant condition assumed?

Table 3 - Data and Initial/Boundary Conditions for PORFLO-3

Hydraulic Properties of the Porous Matrix

1) saturated permeabilities (x, y, z values)
2 porosities (effective or flow, diffusive, and total)
3) effective specific storativity (rate of increase of moisture content r.w.t. soil-moisture potential)
4) relative permeability vs. moisture content (van Genuchten, Brooks & Corey, or table)

Mechanical Properties of the Matrix

5) compressibility of the matrix
6) density of the dry matrix

Transport Properties

7) partition coefficient or retardation factor
8) molecular diffusivity for species in water
9) longitudinal dispersivity of the matrix
10) transverse dispersivity of the matrix (with 9, used to compute dispersion tensor)
11) half lives of tracer or chemical reaction rate

Fluid Properties

12) mass density as a function of temperature and tracer concentration
13) viscosity as a function of temperature

Thermal Properties of the Matrix and the Fluid

14) specific heat of the fluid
15) thermal conductivity of the fluid
16) specific heat of matrix (dry or effective)
17) thermal conductivity (dry or effective)

Initial Conditions

18) initial pressure distribution
19) initial temperature distribution
20) initial tracer concentration distribution
21) initial value of Darcy velocity (x, y, x values)

Table 4 - Data and Initial/Boundary Conditions for UNSAT-H Version 2.0

Hydraulic Properties of the Porous Matrix

1) minimum head for soil to wet up
2) maximum head for soil to dry out when head flow is absent
3) air entry head
4) saturated water content
5) saturated hydraulic conductivity
6) moisture characteristic curve (Brooks & Corey, van Genuchten, Haverkamp, polynomial)
7) relative hydraulic conductivity vs. pressure (Brooks & Corey, van Genuchten, Haverkamp, or polynomial)
8) tortuosity

Mechanical Properties of the Matrix (None explicitly required)

Fluid Properties

9) mass density of liquid
10) diffusion coefficient of water vapor in air

Thermal Properties of the Matrix and the Fluid

11) average soil temperature
12) specific heat of the fluid
13) thermal conductivity of the fluid
14) specific heat of matrix (dry or effective)
15) thermal conductivity (dry or effective)

Initial Conditions

16) initial head (steady state or transient)
17) initial temperature

Boundary Conditions

At the upper boundary:
18) evapotranspiration and heat flow at (may require meteorological data)
19) constant head or specified liquid flux
At the lower boundary:
20) constant head (including static water table)
21) specified flux (including zero)
22) constant temperature, temperature gradient, or heat flux

Table 5 - Data and Initial/Boundary Conditions for VS2DT

Hydraulic Properties of the Porous Matrix

1) horizontal and vertical saturated hydraulic conductivities
2) porosity
3) specific storage
4) moisture characteristic curve (Brooks & Corey, van Genuchten, Haverkamp, or table)
5) relative hydraulic conductivity vs. pressure (Brooks & Corey, van Genuchten, Haverkamp, or table)

Mechanical Properties of the Matrix (none explicitly required)

Fluid Properties

6) mass density of liquid

Transport Properties

7) fluid sources and sinks with specified solute concentrations
8) longitudinal and transverse dispersivities
9) coefficient of molecular diffusion and tortuosity
10) decay constant for linear decay
11) Freundlish, Linear, and Langmuir isotherms for equilibrium adsorption
12) four types of ion exchange

Initial Conditions

13) initial head or moisture content specified (steady state or transient)
14) initial tracer concentration distribution

Boundary Conditions

15) specified head as a function of position and time
16) specified flux as a function of position and time
17) infiltration and ponding
18) evaporation
19) plant-root extraction
20) flow through seepage faces
21) consistent transport boundary conditions with specified solute concentration and specified solute mass flux

3. Overview of VS2DT

On the basis of features, the availability, and the modest computer requirement, VS2DT was chosen for detailed study and implementation. This choice also served the purpose of comparing some INEL study results using different codes. A brief outline of the model is given here.

There are two components of VS2DT: liquid transport and solute transport. For liquid transport, the principle of mass conservation is used to derive the governing equation. The time rate of increase of liquid mass stored in a control volume is equated to the sum of the liquid influx across the control surface and the rate of mass from an internal source (sink). The storage term is quantified by the amount of moisture retainable under a given head and by the specific storage of the medium. The latter is determinable from the porosity and the compressibilities of the medium matrix and the liquid. The liquid flux term is modeled by a generalized Darcy's law using a relative hydraulic conductivity. The amount of retainable liquid and the relative hydraulic conductivity are nonlinear functions of pressure head in the matrix, and are specified by empirical algebraic equations. The source (sink) term refers to liquid addition (removal) within the matrix as opposed to liquid addition (removal) at the boundaries. The principal dependent variable used in VS2DT is the total potential head and is taken as zero at the ground surface.

To solve a particular problem, the initial and the boundary conditions must be specified. The initial condition is specified by providing the total potential head in the solution domain. The initial total potential values can be obtained by hand calculations for static or steady equilibrium conditions, or by a prior simulation. Simple boundary conditions that can be specified a priori include specified liquid flux or specified total hydraulic head. Physical processes such as infiltration, ponding, evaporation, evapotranspiration, and seepage can also be modeled. Unspecified boundary conditions are defaulted to zero flux boundary conditions.

For solute transport, the principle of mass conservation is applied to the solute. The time rate of increase of solute mass in a control volume equals the sum of influx of solute due to liquid advection, influx due to hydrodynamic dispersion, and a source (sink) term. The latter represents processes like decay adsorption and ion exchange. The advection and hydrodynamic dispersion depend on the velocity field that is solved in the liquid transport part of the program. The two components of VS2DT are thus coupled at each time step.

The initial and boundary conditions must be specified according to those in the liquid transport modeling. For example, at a fixed head boundary, if the liquid flow enters the solution domain, then a solute concentration of the inflow must be specified. At a boundary where the liquid exits, no solute concentration can be specified.

Implicit finite differences are used to solve the governing equations for both the liquid and the solute transport. Because of nonlinearities of the equations, numerical stability problems may occur. When this occurs, the time step size needs to be reduced, which considerably slows down the simulation.

4. RWMC Site Characteristics and Pertinent Data for Transport Modeling

At RMWC, a layer of surface sediments lies on top of alternating layers of basalt and sedimentary interbeds. The surface sediments layer is made of alluvial sand, loess, and lacustrine deposits and has a thickness varying from 0 to 25 ft (Barracolough, et al. (1976)). The overall structure of the basalt and interbed layers can be found in Anderson and Lewis (1989). On the basis of borehole data, the structure of the subsurface was shown with several vertical sections. An example section is reproduced in figure 1 to show the general structure and the aerial extent. The shaded layers represent the surface layer and the interbeds at about 110 ft and 240 ft below the surface. The interbeds are made of clay, silt, sand and some gravel. The labels A, B, ... I denote major basalt groups. Significant property variations exist within each interbed as well as within each basalt group. Statistical characterization of the basalt can be found in Knuston, et al. (1990, 1992). Lee (1991) provided statistical data for permeability, porosity, and bulk density for both the surface sediments and the basalt groups.

Sources of data needed for simulating the movement of water beneath RWMC have been compiled by Baca et al. (1992). Two problems were simulated in this interesting report: advancement of wetting front due to Pit 10 flooding, and water travel-time predictions.

For the flooding problem, the vadose zone is represented, from the top down, by horizontal layers of surface sediments, vesicular basalt, massive basalt, 110 interbed, and vesicular basalt. For each layer, the saturated hydraulic conductivity, the effective porosity, the specific storage, the moisture characteristic curves (van Genuchtenthe and Haverkamp), and the relative hydraulic conductivity curves (van Genuchten) were derived from the available data. A base case and a case with a single vertical fracture terminating just above the 110 interbed were simulated. The results demonstrated that fractures can significantly alter the pattern of water movement.

Monte Carlo simulations were used for the travel-time problem. This approach recognizes that uncertain spatial variations allow a wide range of fast and slow flow paths and that a deterministic modeling is not very meaningful. The vadose zone from the ground surface to the water table (about 525 ft in thickness) was considered. The vadose zone was randomized using the uniform distribution of the surface sediments, the statistical information for the thickness of individual basalt layers, and the empirical distribution of the interbed thicknesses and elevations. The random stratigraphy varied from 80 to 110 layers, each always consisting of a surface layer, a 110 interbed, and a 240 interbed. Probabilistic descriptions for the hydraulic conductivity of the surface sediments, the interbeds, and the basalt layers were established. The remaining hydraulic parameters are random variables as well but so far have been treated as deterministic.

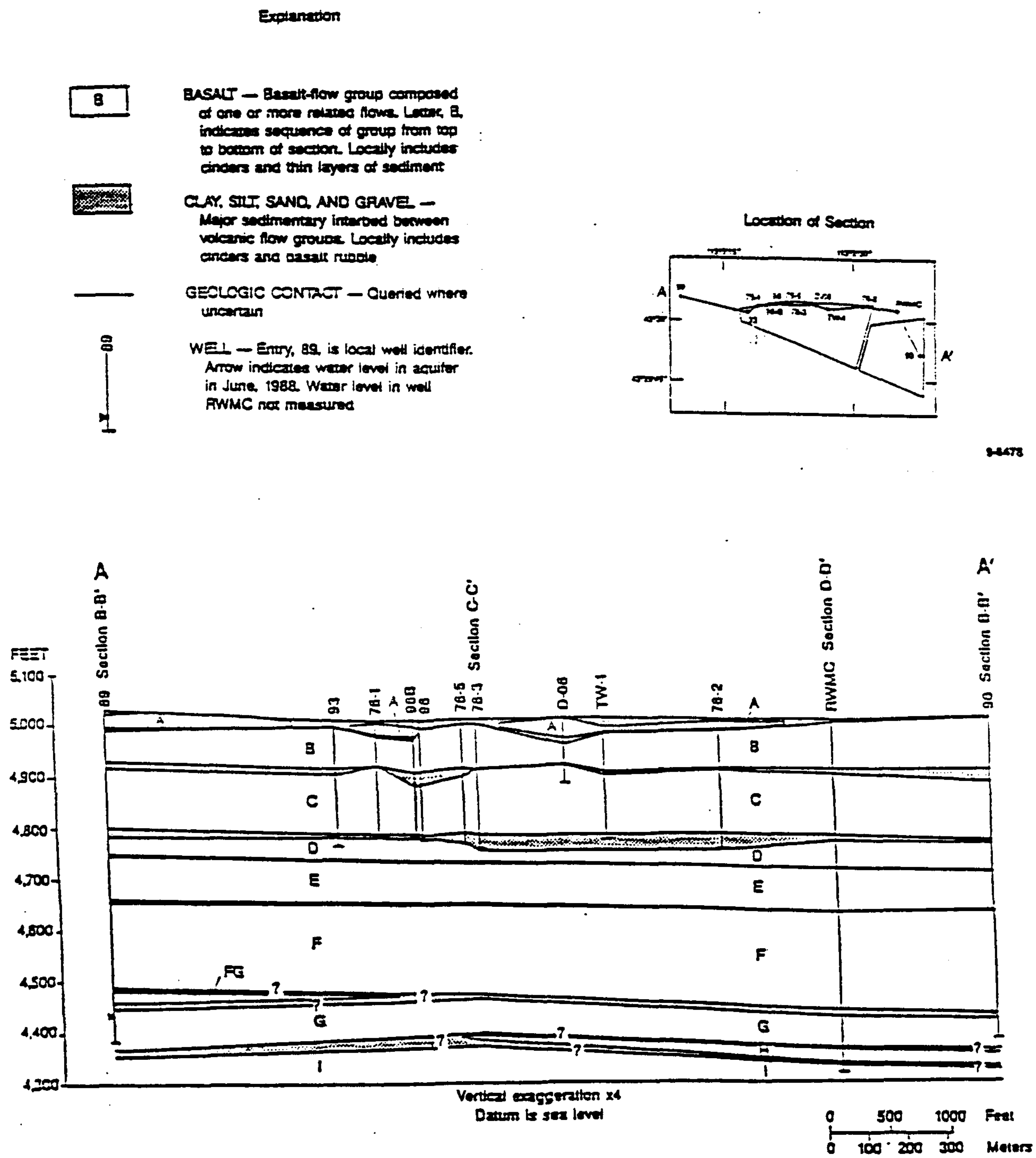

Figure 1 Example ground section at RWMC of INEL (from Anderson and Lewis (1989))

5. Advancement of Wetting Front due to RWMC Pit 10 Flooding as Modeled by VS2DT

Infiltration of water from past flooding events is a likely mechanism for the downward transport of radionuclides at RWMC. Pit 10 is one of many ponds at the site. It is roughly 30 ft wide and about 10 times as long. The movement of flooding water in a cross-section of the ground along the shorter axis of the Pit 10 was simulated by Baca et al. (1992) using PORFLO-3 and/or FLASH. The same problem is simulated here using VS2DT.

The extent of the ground simulated is a 60 m (horizontal) by 37 m (vertical) rectangle with one vertical side aligned with the center of the pit. Taking advantage of symmetry with respect to the longitudinal axis of the pit, only a half of the domain needs to be simulated. The domain of simulation is idealized as five layers: surface sediments, vesicular basalt, massive basalt, 110 interbed, and vesicular basalt. The hydraulic properties for each layer are assumed to be constant. This information, as required by the VS2DT input file, is shown in Table 6.

Table 6 - Hydraulic Properties Used for Pit 10 Flooding Simulation

Layering (m)	K_v/K_h	K_h (m/day)	S_s (m^{-1})	ϕ	$-\alpha'$(m)	θ_r	β'
0 -1 (surface)	1	0.100	0.0001	0.48	0.8224	0.1	1.36
1 - 21 (vesicular)	1/3	29.29	0.0001	0.228	0.2604	0.015	1.474
21 - 24.5 (massive)	1	0.010	0.0001	0.145	0.2604	0.015	1.474
24.5 - 26 (interbed)	1	0.0656	0.0001	0.48	0.8224	0.1	1.36
26 - 40 (vesicular)	1/3	29.29	0.0001	0.228	0.2604	0.015	1.474

In the layering column, 0 is at the bottom of the pit, which is about 8 m below the ground surface. K_v and K_h are saturated hydraulic conductivity in the vertical and horizontal directions respectively, S_s = specific storage, ϕ = effective porosity, α' = the negative of the reciprocal of α in van Genuchten (1980) and in Baca et al. (1992), θ_r = residual moisture content, and β' = n in van Genuchten (1980) and Baca et al. (1992).

The domain is discretized with 60 equi-distant nodes in the horizontal direction and 284 nodes in the vertical direction. The spacing of the vertical nodes varies from 0.05 m to 0.25 m, depending on the location. Shorter spacing at the surface and at the interfaces of layers were used.

At the surface, a flux of 0.2 m/day, or twice the saturated hydraulic conductivity, was imposed over the pit area (0 to 15 m from center). On the surface outside the pit (15 to 60 m from center), a flux of 0.05 m/yr was imposed. A zero-flux was imposed on the left, the right, and the bottom boundaries. The boundary condition so specified is realistic at the left vertical boundary due to symmetry. The zero-flux condition used at the right vertical and the bottom boundaries are realistic only if the water front is far away from these boundaries. A specified head distribution, as established by Baca et al. (1992) was used.

As in Baca et al. (1992), two cases were simulated. The base case is as described above. The simulated head contours at different times are shown in figures 2 to 5. In the second case, a 0.01 m wide vertical fracture begins at the top of the top vesicular basalt layer and ends near the bottom of of massive basalt. The fracture is offset by 8 m from the pit centerline. The results are shown in figures 6 to 9. All the essential features of the wetting front propagation shown by Baca et al. (1992) using POREFLO-3 and/or FLASH are reproduced here with VS2DT. For example, the lateral spreading of the wetting front in the top vesicular layer, the pronounced lateral spreading of the water at the vesicular-massive basalt interface, and the fracture-induced penetration of the wetting front through the 110 interbed can all be seen in these figures. The correct trend of reduced lateral spreading as the water advances deeper into the vadose zone can also been seen by comparing figures 5 and 9.

Although the trends are similar, there are some differences between the results obtained here and those of Baca et al. (1992). In the top vesicular layer, VS2DT produced more moisture retention near the pit, less lateral spread of the wet front, and steeper head gradients at the front. These differences can be seen by comparing figure 5 with figure 10 of Baca et. al, (1992). After the simulations were made, we noticed that a much smaller specific storage value was used here by mistake. This plus differences in the spatial discretization (60 horizontal by 305 vertical in Baca et al. (1992)) may be the causes for the observed discrepancies. Comparing Figure 9 with Figure 12 of Baca et al. 1992 shows a larger extent of water penetration into the 110 interbed and beyond. This may be explained by the fact that the width of the fracture used here is twice as large as that used in Baca et al. (1992).

6. Probabilistic Approach - Water Travel Time Predictions

In addressing nonuniform homogeneous groundwater flows, Freeze (1975) demonstrated the importance of the standard deviation of the model parameters. Changing the standard deviations leads to different means of the output even when the mean values of the model parameters remain fixed. Because of the nonuniformity in model parameters, Freeze contented the validity of deterministic modeling using a single value for each of the model parameters.

At RWMC, significant variations in model parameters exist within each main group of the ground formation. Deterministic modeling using representative or "conservative" model parameters may not be meaningful. Baca et al. (1992) used Monte Carlo simulations to predict the water travel time beneath RWMC. A probabilistic approach, such as the Monte Carlo method, provides information on the uncertainty of the model output.

In Monte Carlo simulations, multiple runs of the model are made. For each run, the model parameters are chosen from pertinent probability density functions. A distribution of the model output emerges as the number of runs increase. The process continues until the means of the outputs converge. When the number of random model parameters is large, a very large number of runs is needed. When the model is complex, such as those used in vadose zone modeling, this method is too CPU intensive to be practical.

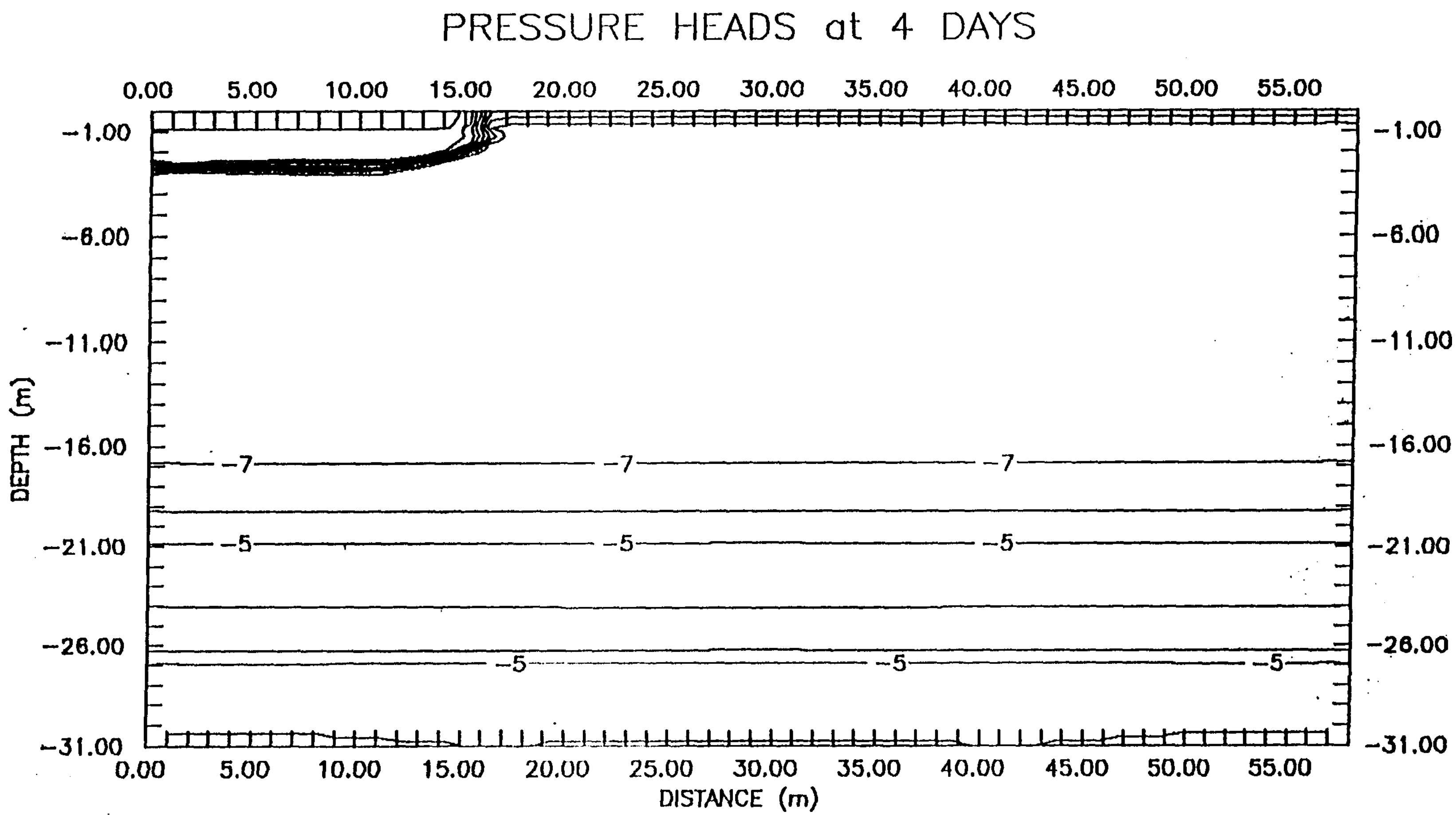

Figure 2 Base case - pressure heads at 4 days

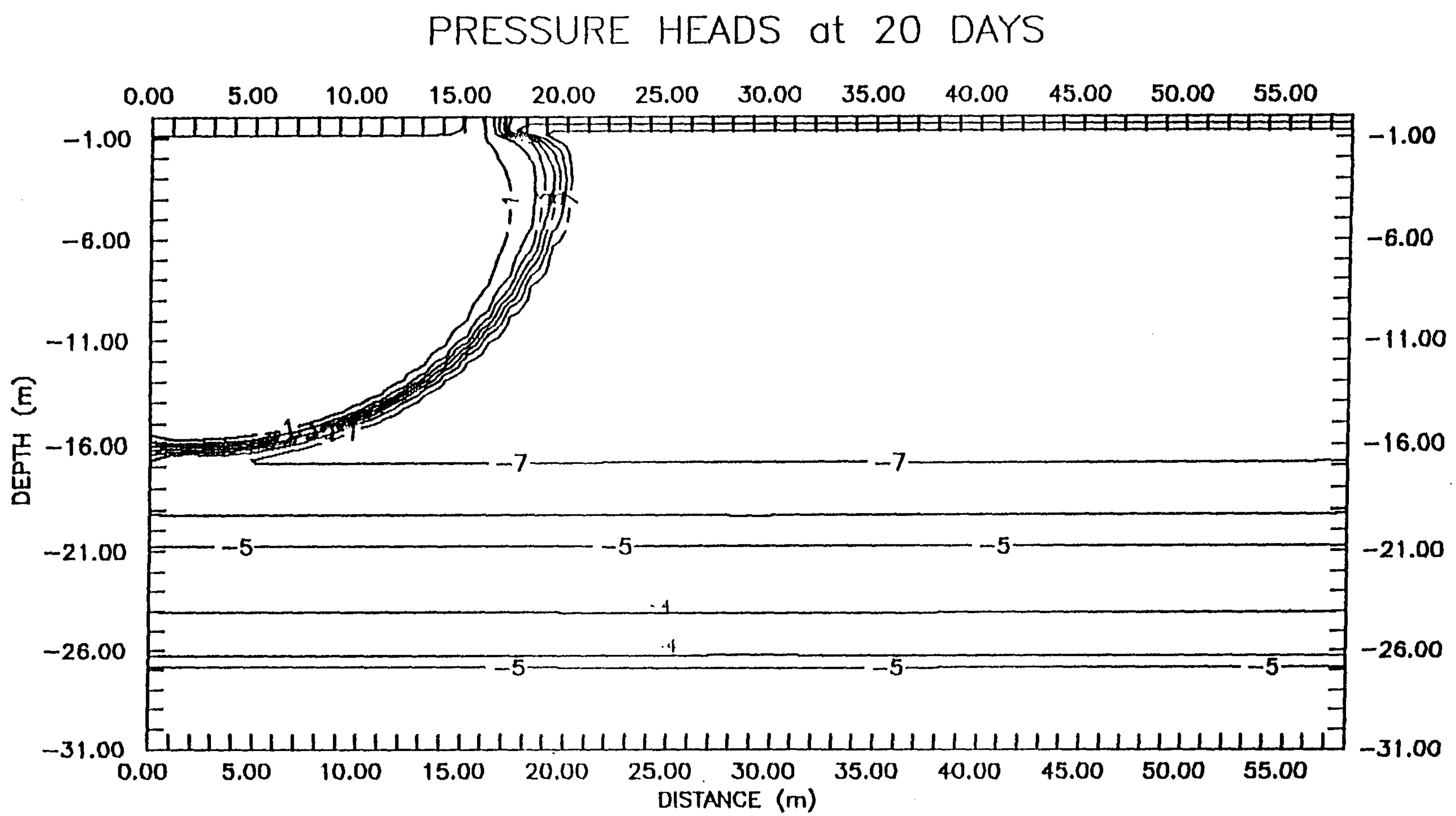

Figure 3 Base case - pressure heads at 20 days

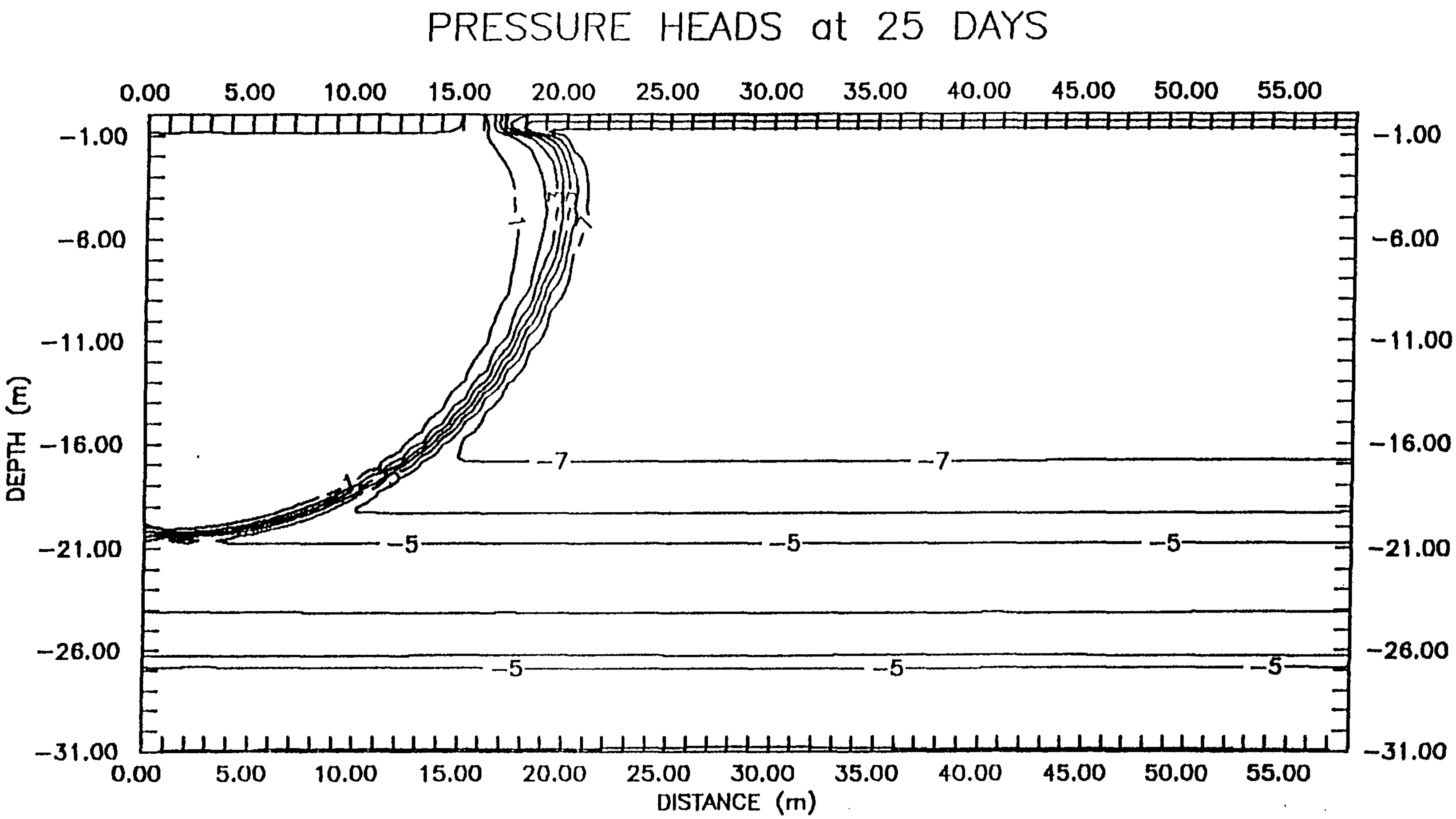

Figure 4 Base case - pressure heads at 25 days

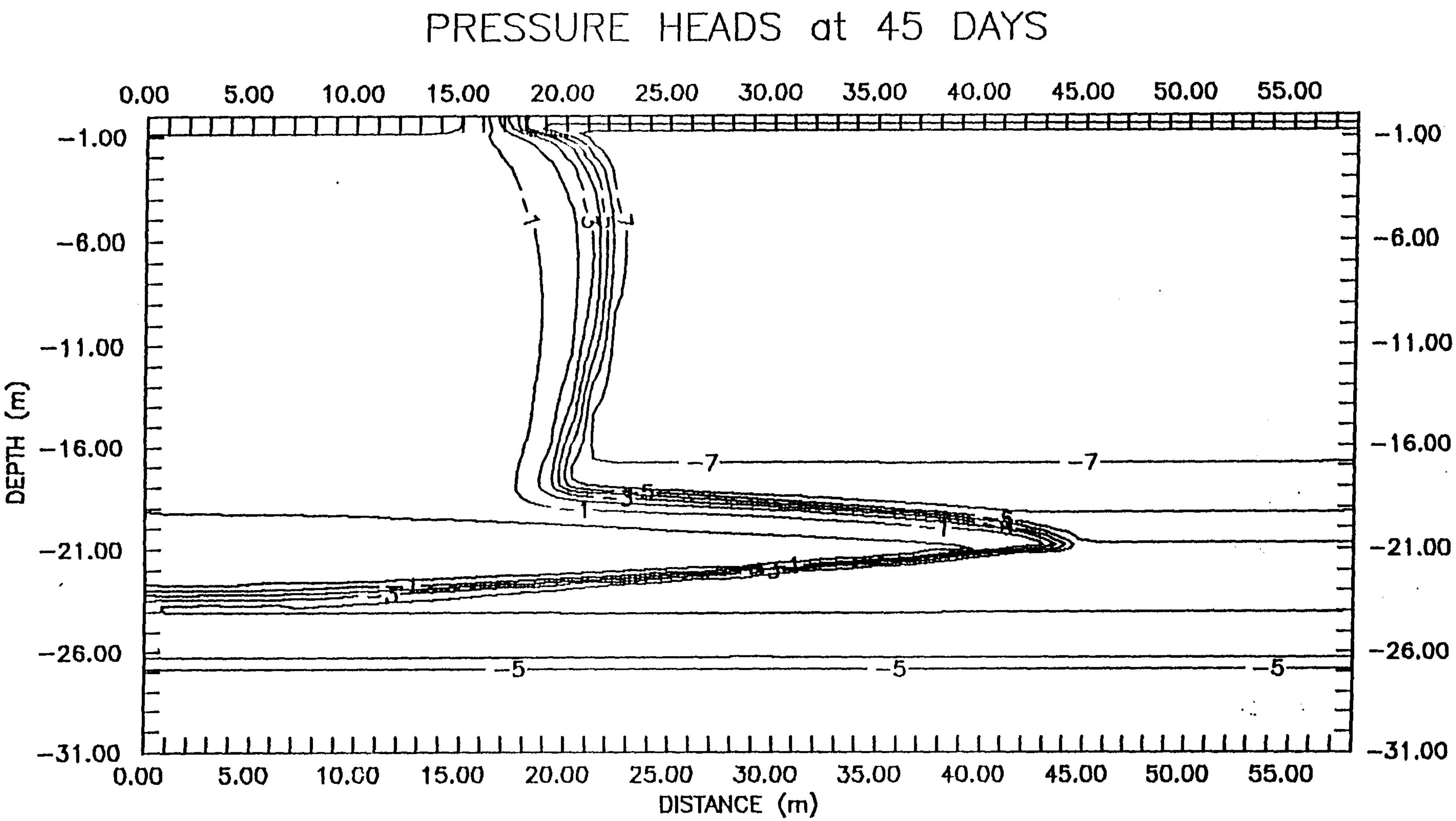

Figure 5 Base case - pressure heads at 45 days

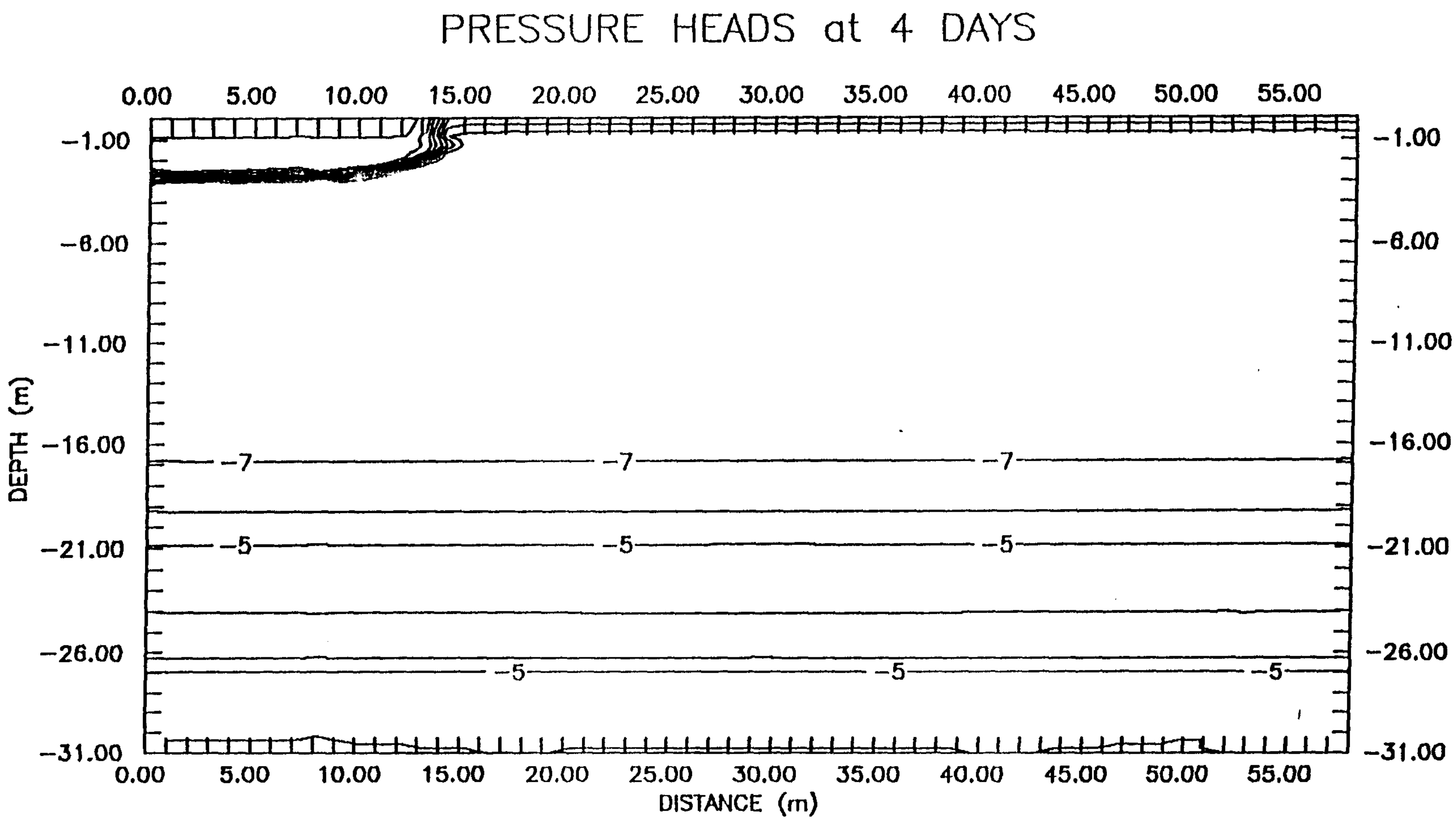

Figure 6 Fractured case - pressure heads at 4 days

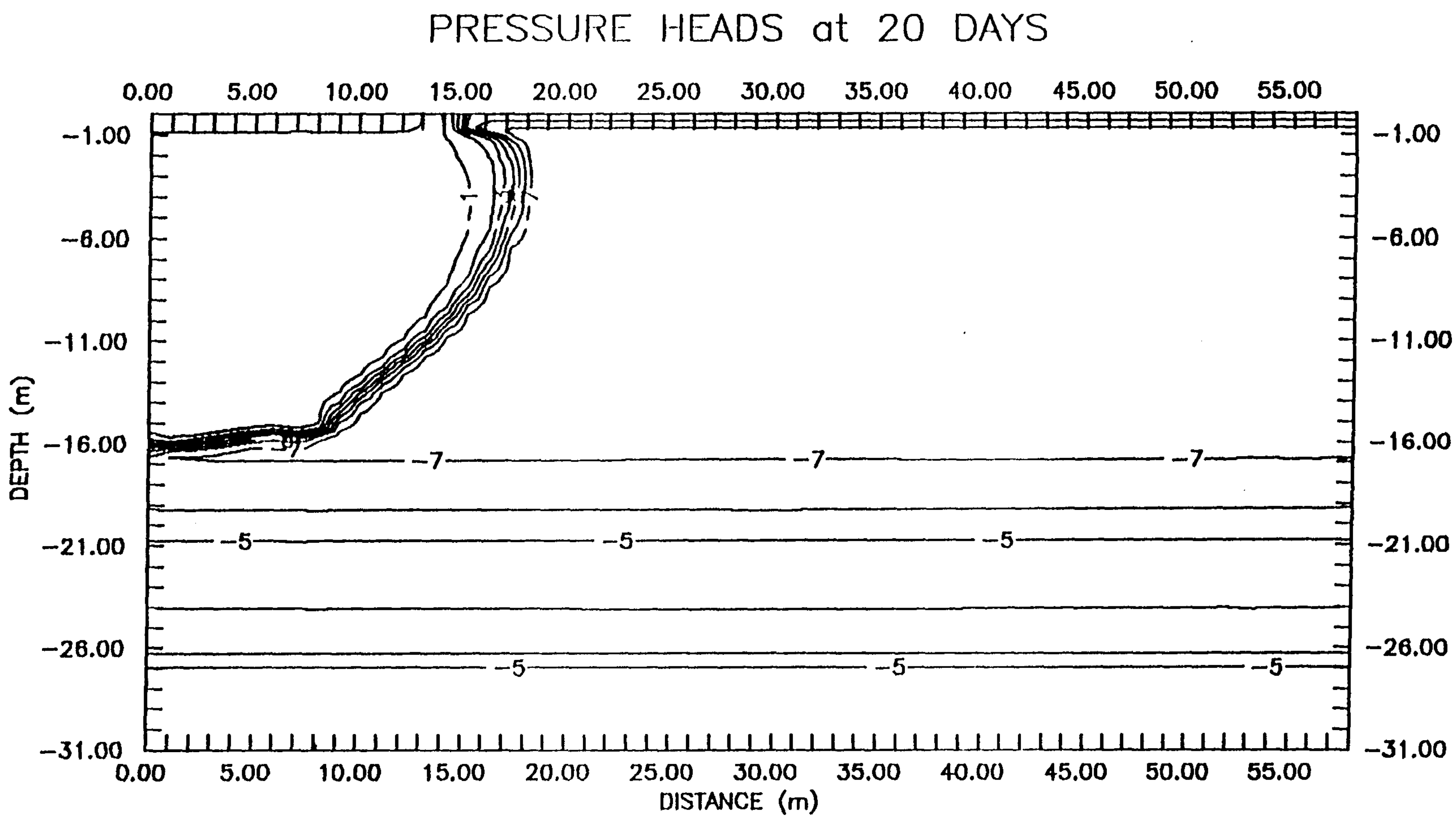

Figure 7 Fractured case - pressure heads at 20 days

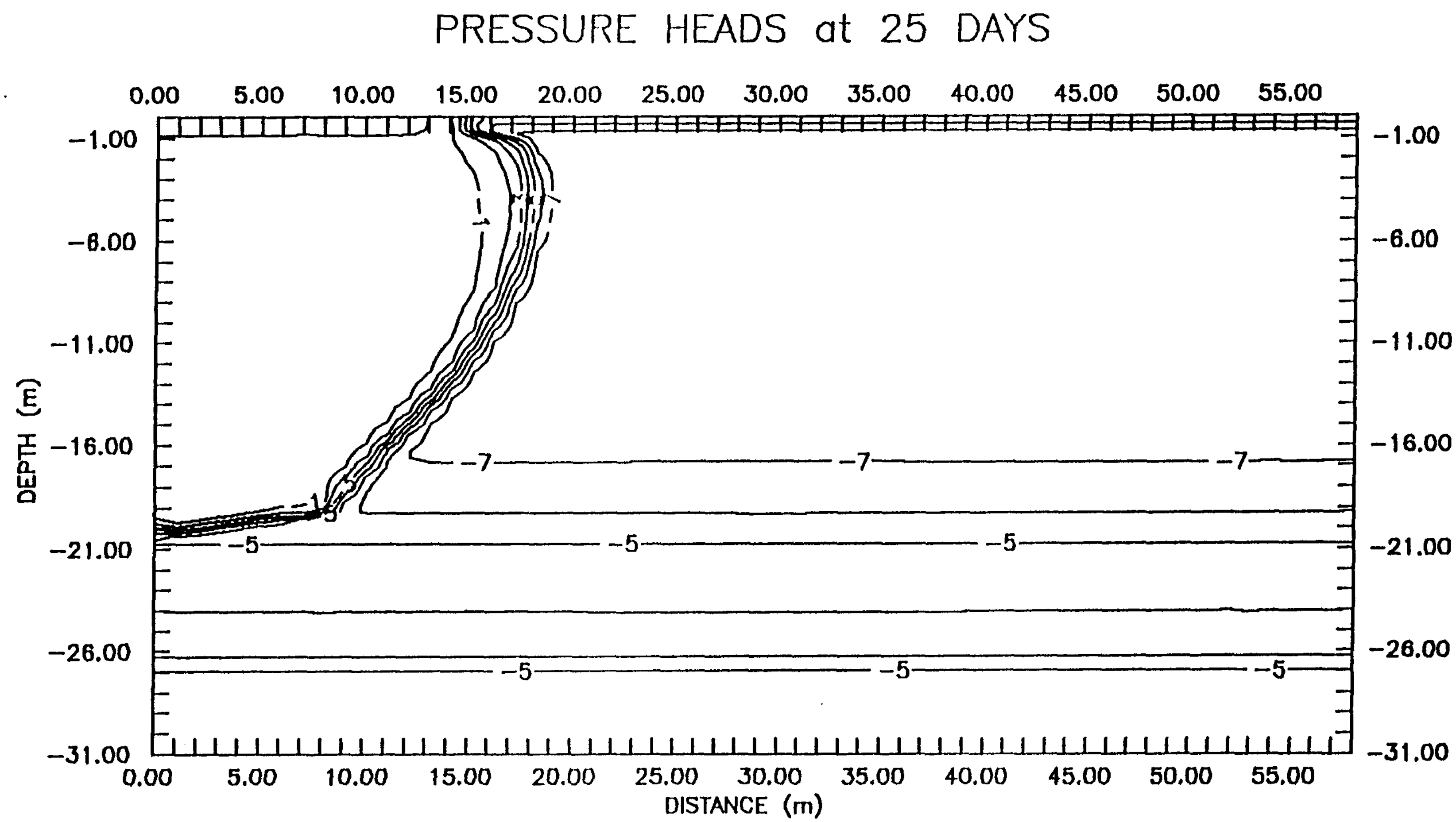

Figure 8 Fractured case - pressure heads at 25 days

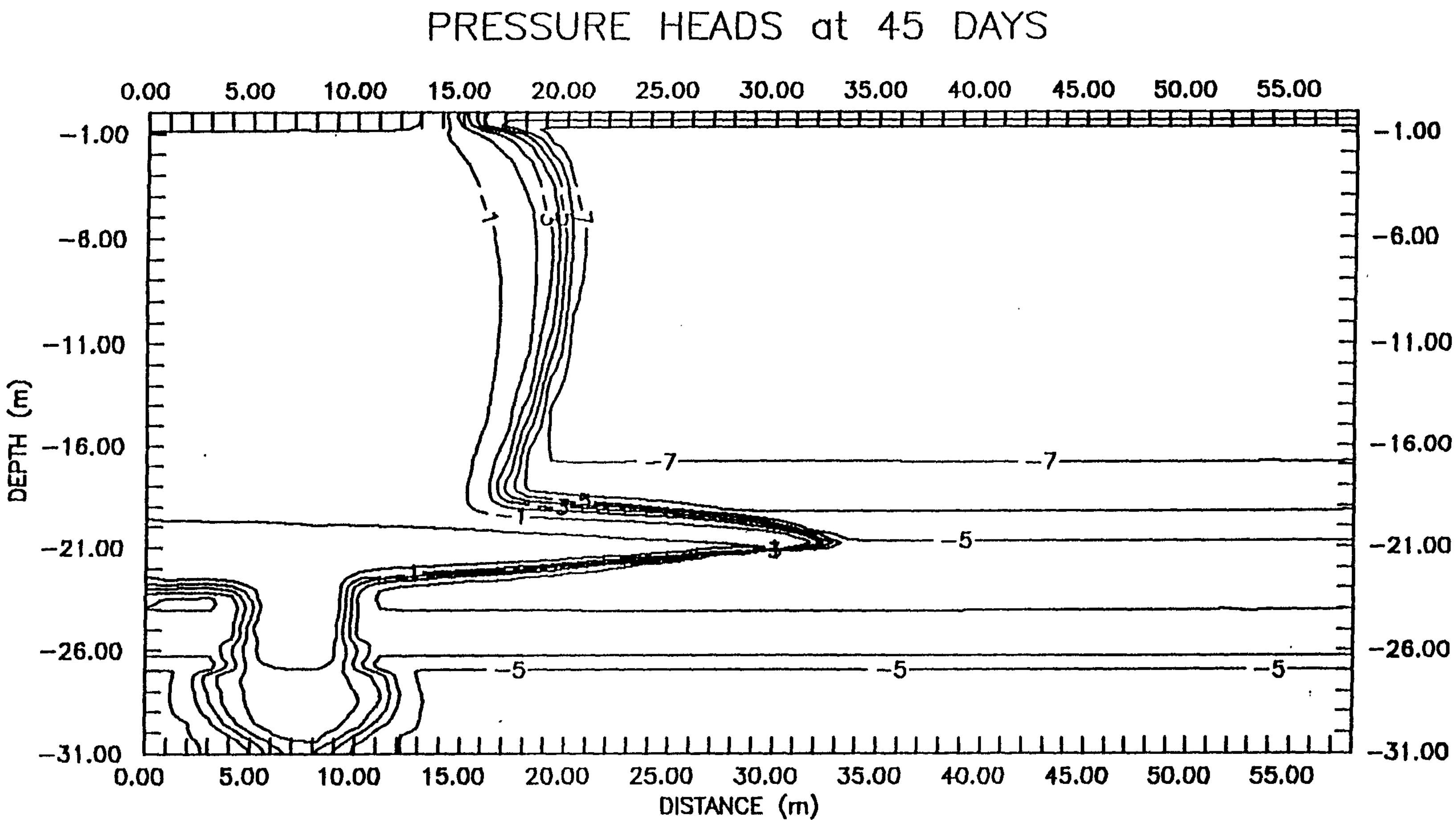

Figure 9 Fractured case - pressure heads at 45 days

The randomized field used by Baca et al. (1992) was described in section 4. The variables randomized are the net infiltration rate, the saturated hydraulic conductivity, and the thicknesses of the surface sediments, the basalt layers, and the inter beds. It took 1881 runs of the model for the arithmetic means of the computed water travel times to converge!

For better efficiency, Schanz and Salhotra (1992) evaluated the Rackwitz-Fiessler uncertainty analysis for transport modeling in saturated media using three models. The random model parameters considered are Darcy velocity, dispersivity, and porosity. They showed that this method can be efficient, requiring 10 to 30 model runs to provide estimates for tracer concentration within 10% of that produced by the Monte Carlo method with 10,000 runs. However, this method is not always accurate, and the number of runs increases, presumably rapidly, as the number of uncertain model parameters increase.

Another approximate probabilistic method was proposed by Rosenblueth (1981), and later extended by Harr (1989). This method as originally proposed by Rosenblueth is very simple. Harr's extension makes this method applicable to problems with large numbers of uncertain model parameters. No literature on the application of this method to the transport process in the vadose zone has been found.

7. Approximate Probabilistic Methods by Rosenblueth and Harr

The Rosenblueth method is outlined first in terms of a function of a single random variable first. Consider the analogy between a probability distribution and a distributed vertical load on a horizontal rigid beam with two supports. The zero-th and the first three moments of the two reactions can be equated to unity, and to the mean, the standard deviation, and the skewness of the distribution respectively. Knowing the mean, the standard deviation and the skewness, the two reactions and their points of application can be computed. The function is then evaluated at the points of application of the two reactions. The resulting values can be viewed as the point of application of the two reactions in a second beam analogy. In this analogy, the vertical load is the same as before but the length of the beam is now the value of the function instead of the value of the random variable itself. The mean, the standard derivation, and the skewness of the function are then computed from the first three moments of the second beam analogy.

We note that the actual distribution of the random variable does not have to be known. In addition, Rosenblueth showed, at least for simple functions, that the mean and the standard deviation of the function are not sensitive to the skewness of the independent random variable. Thus, if there is no reason to suspect the variable to be skewed, the skewness coefficient can be assumed to be zero. Therefore, knowing the mean and the standard deviation of the random variable alone enables us to obtain reasonable estimates for the central tendency and the scatter of the function. This is an important advantage over the Monte Carlo method which requires the distribution of the random variable to be known. In vadose zone modeling, the means and the standard deviations of model parameters are often known but their distributions are not. Also, note that only two deterministic function evaluations or model runs are needed.

For a function with n random variables, the mean and the standard deviation for each variable, and a set of $n(n - 1)/2$ correlation coefficients among the random variables can be established. Based on the means and the standard deviations of the random variables, 2^n values of the function can be evaluated. Meanwhile, 2^n reactions are computed from the correlation coefficients (assuming zero skew of the independent random variables). The mean and the standard deviation of the function can then be calculated. In short, the information about the n independent variables (means, standard deviations, and correlation coefficients) produces 2^n estimates for the dependent random variable (i.e., the function) and 2^n reactions. The reactions scale these estimates according to the correlation amongst the independent random variables. For large n, the Rosenblueth method can still be very CPU intensive as 2^n increases rapidly as n increases.

When there are n independent random variables, an n by n symmetrical correlation matrix can be established from repetitive sampling of the independent random variables. The eigenvalues and eigenvectors of the correlation matrix can be readily computed. The correlation matrix can be rotated in the direction of the eigenvectors so that the standardized independent variables are not correlated. Harr (1989) demonstrated that the correlation matrix can be represented by a hypersphere (for $n > 3$) with a radius $n^{0.5}$. The hypersphere is centered at the mean point of the Cartesian coordinate system of the random variables. Each eigenvector emanates from the center and intersects the sphere at two points. Thus there are $2n$ intersections on the sphere. The coordinates of these $2n$ points replace those of the 2^n points in the Rosenblueth method. Only $2n$ evaluations of the function are needed. Due to the fact that the standardized independent random variables are not correlated, the magnitude of the reactions in the Harr's method is simply $1/n$. The mean and the standard deviation of the dependent random variable, i.e., the values of the function, can then be established. The advantage of Harr's extension to the Rosenblueth method is substantial for large n. For example, for $n = 15$, which is likely in vadose zone simulations, 2x15 = 30 while $2^{15} = 32{,}768$.

Despite the apparent advantages of the Harr's method, doubts exist. The method is approximate. The examples in Rosenblueth (1981) and in Harr (1989) used very simple functions. How well will it work for complex nonlinear problems such as transport in the vadose zone? How good are the approximations for specific applications?

8. Application of the Harr's Method

To obtain an indication on how well Harr's method applies to vadose zone transport problems, the downward water travel time in a hypothetical two-layer system was estimated by Harr's method and by Monte Carlo simulations. A 3 m thick layer of surface sediments lies over a 3.3 m massive basalt. A uniform head of -5 m is imposed initially. Water is infiltrated into the ground from the top. Zero flux is imposed at the bottom. The problem conceived is one-dimensional and thus a zero flux is also imposed on the sides in the VS2DT simulations. The arrival of the wetting front at the bottom signifies the water travel time. The depth of the problem is made small (relative to that used by Baca et al. (1992)) so that a large number of VS2DT simulations can be made on a microcomputer in the Monte Carlo approach.

The saturated hydraulic conductivity of the layers and the infiltration rate were chosen as the three random model parameters. Their means and standard deviations are shown in Table 7. The remaining hydraulic properties are shown in Table 8. The values in these tables need to be verified and may be revised at a later date.

Table 7 - Mean and Standard Deviations of Randomized Model Parameters

Parameter	Mean	Standard Deviation
infiltration rate (m/day)	0.00015	0.0000072
K_h of top layer (m/day)	0.22683	0.13268
K_h of bottom layer (m/day)	0.06378	0.02866

Table 8 - Hydraulic Properties Used in Water Travel Time Problem

Layering (m)	S_s (m^{-1})	ϕ	$-\alpha'$(m)	θ_r	β'
0 -1	0.001	0.48	0.8224	0.1	1.36
3 - 6.3	0.001	0.21	0.2604	0.1	1.474

Normal distributions for the randomized model parameters were assumed in generating the infiltration rate and the saturated hydraulic conductivities for the Monte Carlo simulations. The mean and the standard deviation of the travel time as functions of the number of simulations made are shown in figure 10. It appears that several hundred simulations are needed to obtain convergence. After completing the 400-th simulation, the mean travel time is 6471 days with a mean standard deviation of 3693 days.

The mean and the standard deviation obtained by Harr's method is 6550 days and 3314 days. Only six VS2DT simulations were needed to obtain these results!

It is noteworthy that Freeze (1975) contended that there is no simple way to define an equivalent uniform porous medium for transient flow in nonuniform media. Harr's method, in spirit, is consistent with Freeze's contention. At the same time, the method provides a way to define a collection of uniform media that, collectively, give the mean and the scatter of the process in the nonuniform medium.

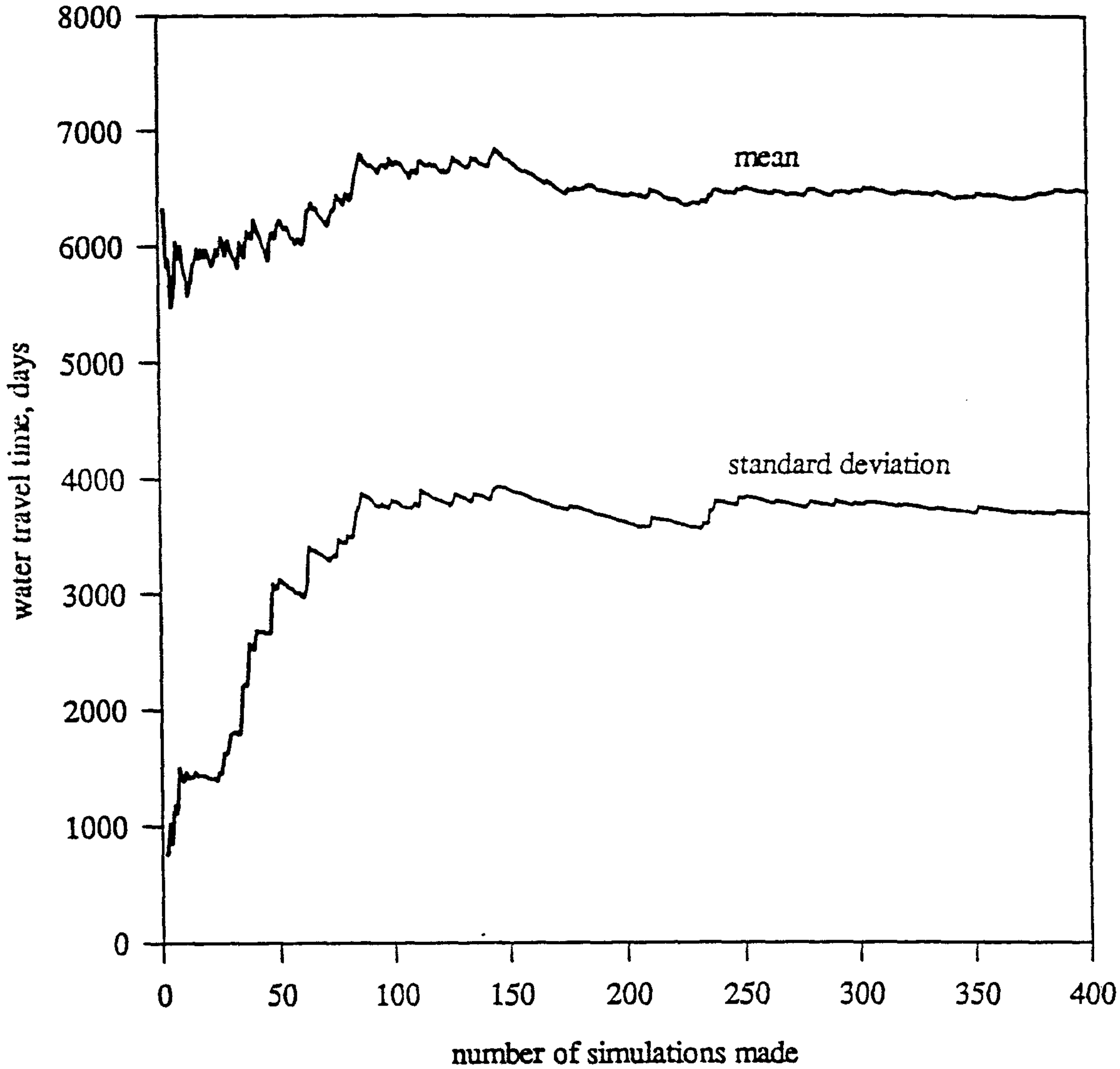

Figure 10 Convergence of travel time and standard deviation by Monte Carlo simulations

9. Sensitivity of Water Travel Time to Model Parameters

Two, four, and six hundred samples were extracted from the model parameters and the sample mean and standard deviations computed. They were then used to estimate the water travel time using Harr's method. The results are shown in Table 9.

Table 9 - Sensitivity of Harr's Method to the Example Water Travel Time

Sample Size	Sample Mean, m/day	Travel Time, days	Standard Deviation, days
200	.000150/.21598/.06309	5937	2200
400	.000150/.22445/.06532	6684	3335
600	.000155/.22632/.06544	6550	3314

Under the second column of Table 9, the mean values of the infiltration rate and the hydraulic conductivities of the top and the bottom layers are shown. Around 10% change in the travel time is noted. Is this sensitivity caused by the approximate nature of Harr's method? Will the Monte Carlo approach also exhibit some sensitivity? More importantly, is the sensitivity a trait of the problem itself?

The great utility of Harr's method is apparent in trying to answer the last question posed. The number of VS2DT simulations required is 3x6 = 18 for Harr's method and 3x400 = 1200 for the Monte Carlo Method. Harr's method makes probabilistic studies feasible on personal computers and workstations when many random model parameters are involved. This is especially true for 2 and 3 dimensional problems.

10. Summary and Conclusion

This study reviewed four computer codes applicable for simulating water and solute transport in the unsaturated zone at the INEL sites. They are: TRACR3D, POREFLO-3, UNSAT-H, and VS2DT. A comparison of the assumptions, capabilities, and data requirements was made. VS2DT is developed and maintained by the U. S. Geological Survey. It was made functional on a HP workstation and on a personal computer. An overview of the modeling basis and feature of VS2DT was given.

A large amount of information exists for the geologic and hydraulic properties pertaining to transport modeling in the vadose zone underneath the RWMC of INEL. The data indicates that the site characteristics are highly nonuniform. Using VS2DT, the advancement of a wet front due to the 1969 Pit 10 flooding at RWMC was simulated deterministically. The results compare favorably with those obtained by INEL researchers using PORFLO-3 or FLASH codes. The results also show a strong influence of water movement by fractures in the basalt that can not be identified without uncertainty. Nonuniform and uncertain model parameters suggest that a probabilistic modeling approach should be used.

INEL researchers have estimated the travel time of surface infiltration to reach the water table using the Monte Carlo method, a direct probabilistic approach. Since the numerical models are complex and CPU intensive, and since a large number of model parameters can be considered as random variables, the Monte Carlo method is not workable for 2 and 3 dimensional problems.

An approximate probabilistic method of Rosenblueth and Harr is described. A hypothetical two-layer problem was devised to explore this method. The infiltration rate and the saturated hydraulic conductivities of the two layers were randomized. Sufficiently accurate mean travel time and its standard deviation were obtained by Harr's method with only six VS2DT simulations while the Monte Carlo method required about three hundred simulations. The efficiency would be even greater if more model parameters were considered random.

Harr's method will make probabilistic modeling of vadose zone problems at INEL feasible. The method provides the needed tool to meaningfully ranking model parameters for specific problems. Further investigation of Harr's method is recommended.

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SECTION III

Supplemental Study: Correlation Between

Big Lost River Seepage and Groundwater Elevations

Introduction:

A draft version of a Master's Thesis by Erik Coats, "Seepage Rates from the Big Lost River and Effects on the Groundwater at INEL," was submitted with the Phase II Annual Report in March, 1993. Since that time, the thesis has undergone significant revisions in response to various review comments, and was finally completed and accepted by the Graduate College in January, 1994. A copy of this final version is attached to this report as an Appendix.

As part of the review process, Dr. Warren Barrash, Environmental Hydrogeologist with the INEL Oversight Program, suggested that several supplemental analyses might be performed. The work by Coats had determined that there was a strong, time-lagged correlation between seepage in the reach of the Big Lost River above the spreading area diversion, and water levels in one of the USGS observation wells (USGS 9). Similar strong correlation was observed between spreading area seepage and another well, USGS 8. The correlation was very poor, however, between the other two river reaches and the well data that had been selected from wells 9 and 18. Dr. Barrash believed that these correlations may be improved by using data from two other USGS wells, 12 and 84, and requested that the supporting analyses be performed as a supplement to the original study.

This brief report summarizes the results of these additional correlation studies, and describes the data bases and procedures used in the analyses.

Well Data:

Water level data for USGS wells 12 and 84 were obtained from USGS Open File Reports 84-239 and 92-643. These reports covered the two time intervals used in the study, 1969 through 1976 and 1983 through 1987.

The observations at USGS 12 were taken very frequently, often resulting in 4 or 5 values for each month. These values were used to obtain a monthly averages of the levels, which were entered into a QUATTRO PRO spreadsheet. A similar procedure was followed with the USGS 84 data, although it was not sampled nearly as frequently. Interpolation was often necessary to estimate levels during months in which no data were available and this was a significant problem in 1986 and 1987 when a total of only 8 observations were made.

Analysis Procedures:

Using procedures described in detail in the thesis by Coats, 12-month moving averages were calculated for the monthly well data, and those were compared to the corresponding seepage data from Reaches 2 and 3. Dr. Barrash had suggested a possible correlation between Reach 2 seepage and USGS 84, and Reach 3 seepage with USGS 12, due to the proximity of the wells to the reach locations.

Before performing any quantitative correlation analyses, the data sets were plotted, using both the moving averaged and raw monthly values for both time periods. These 8 graphs are included as Figures 1 through 8. By observation, it is apparent that there is almost no correlation between the seepage and water level time-series in either reach or either time period. Eventual calculations of r and r^2 confirmed the visual conclusions, with maximum r^2 values during the 1969-1976 period of 0.07 (log of n = 7 months) for Reach 3,

SEEPAGE (REACH 2) VS. LEVELS IN USGS 84
Monthly Data

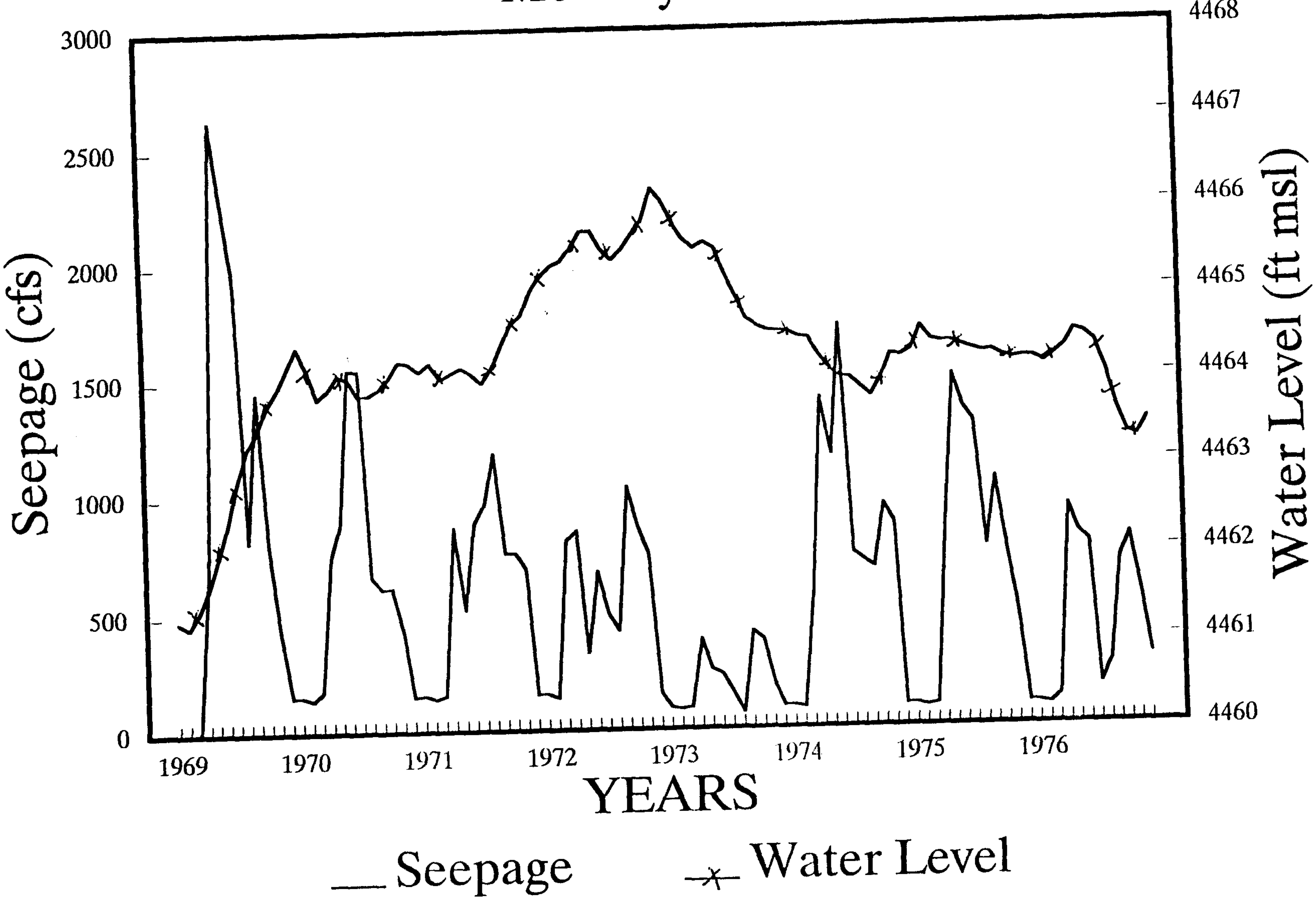

FIGURE 1

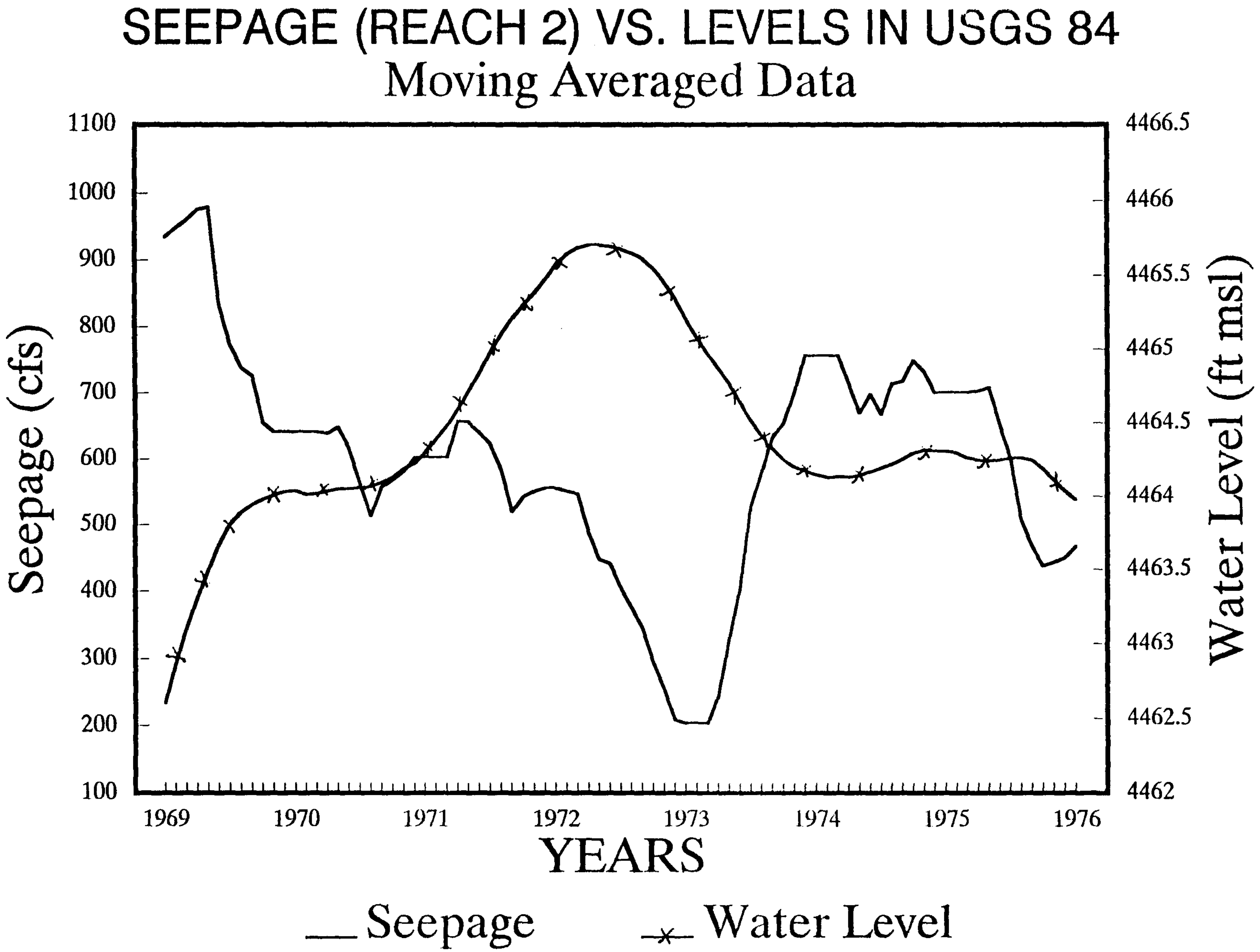
SEEPAGE (REACH 2) VS. LEVELS IN USGS 84
Moving Averaged Data
Seepage (cfs)
1100
1000
900
800
700
600
500
400
300
200
100
Water Level (ft msl)
4466.5
4466
4465.5
4465
4464.5
4464
4463.5
4463
4462.5
4462
1969
1970
1971
1972
1973
1974
1975
1976
YEARS
Seepage
Water Level

FIGURE 2

SEEPAGE (REACH 2) VS. LEVELS IN USGS 84

Monthly Data

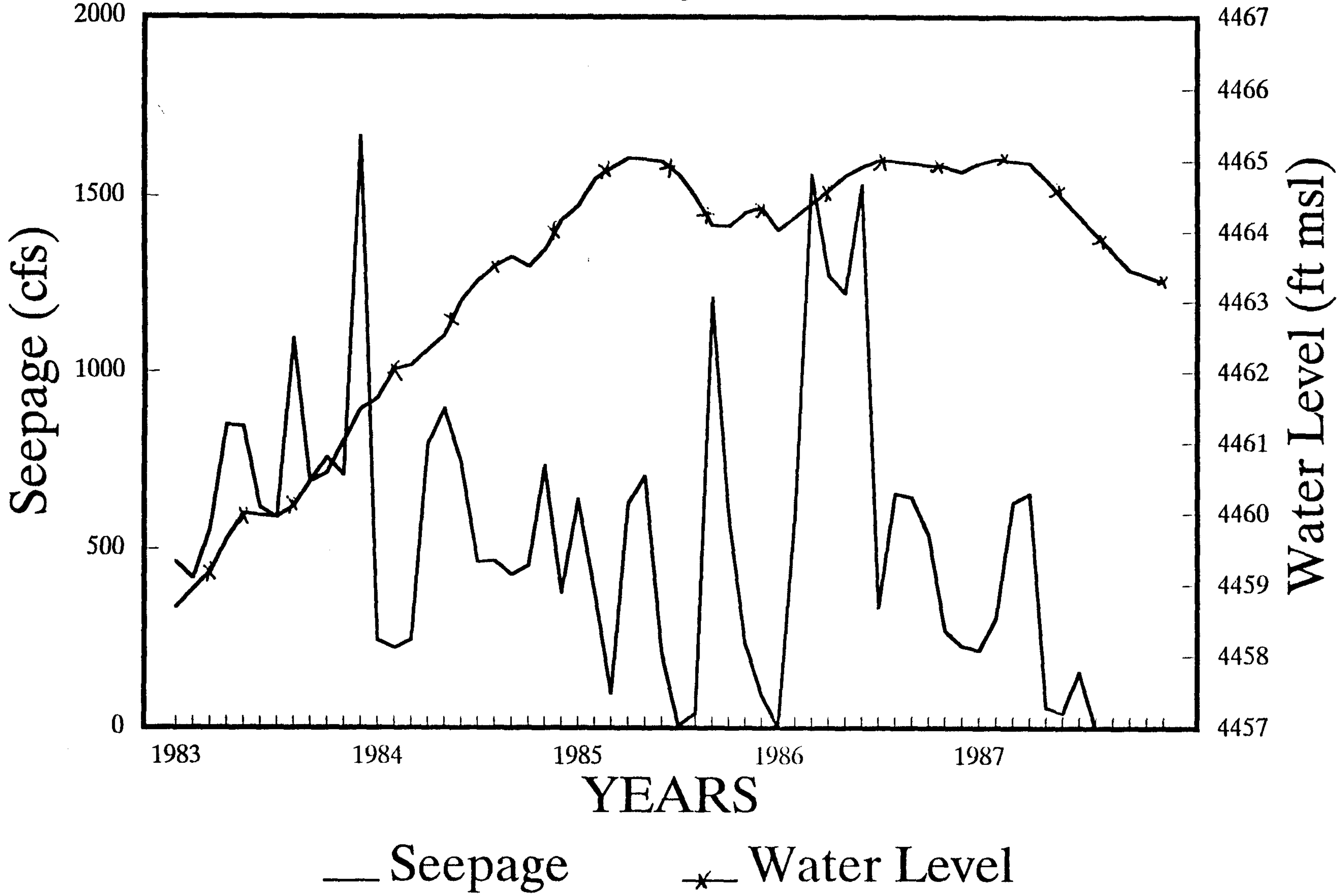

FIGURE 3

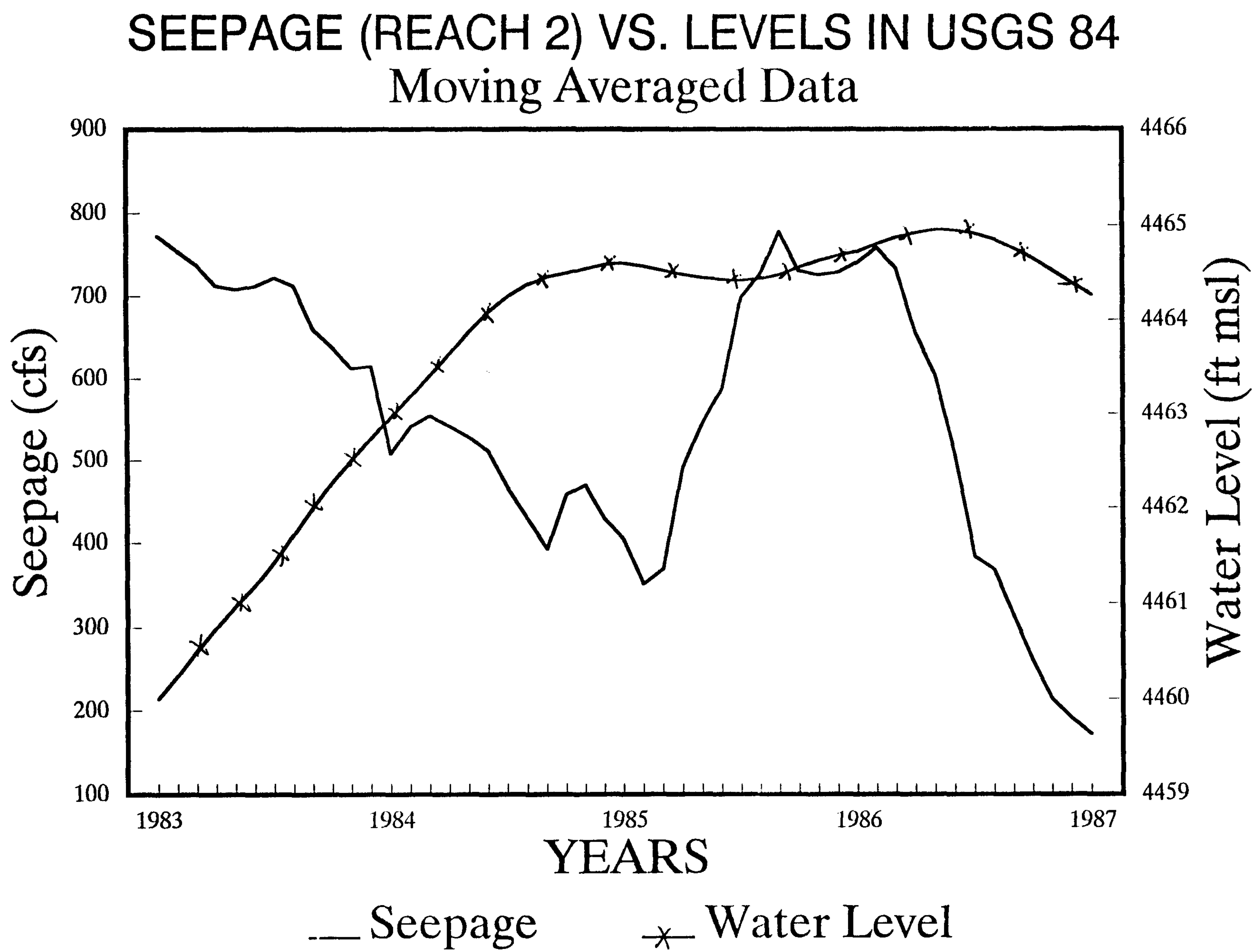
SEEPAGE (REACH 2) VS. LEVELS IN USGS 84
Moving Averaged Data
900
800
700
600
500
400
300
200
100
Seepage (cfs)
4466
4465
4464
4463
4462
4461
4460
4459
Water Level (ft msl)
1983
1984
1985
1986
1987
YEARS
Seepage
Water Level

FIGURE 4

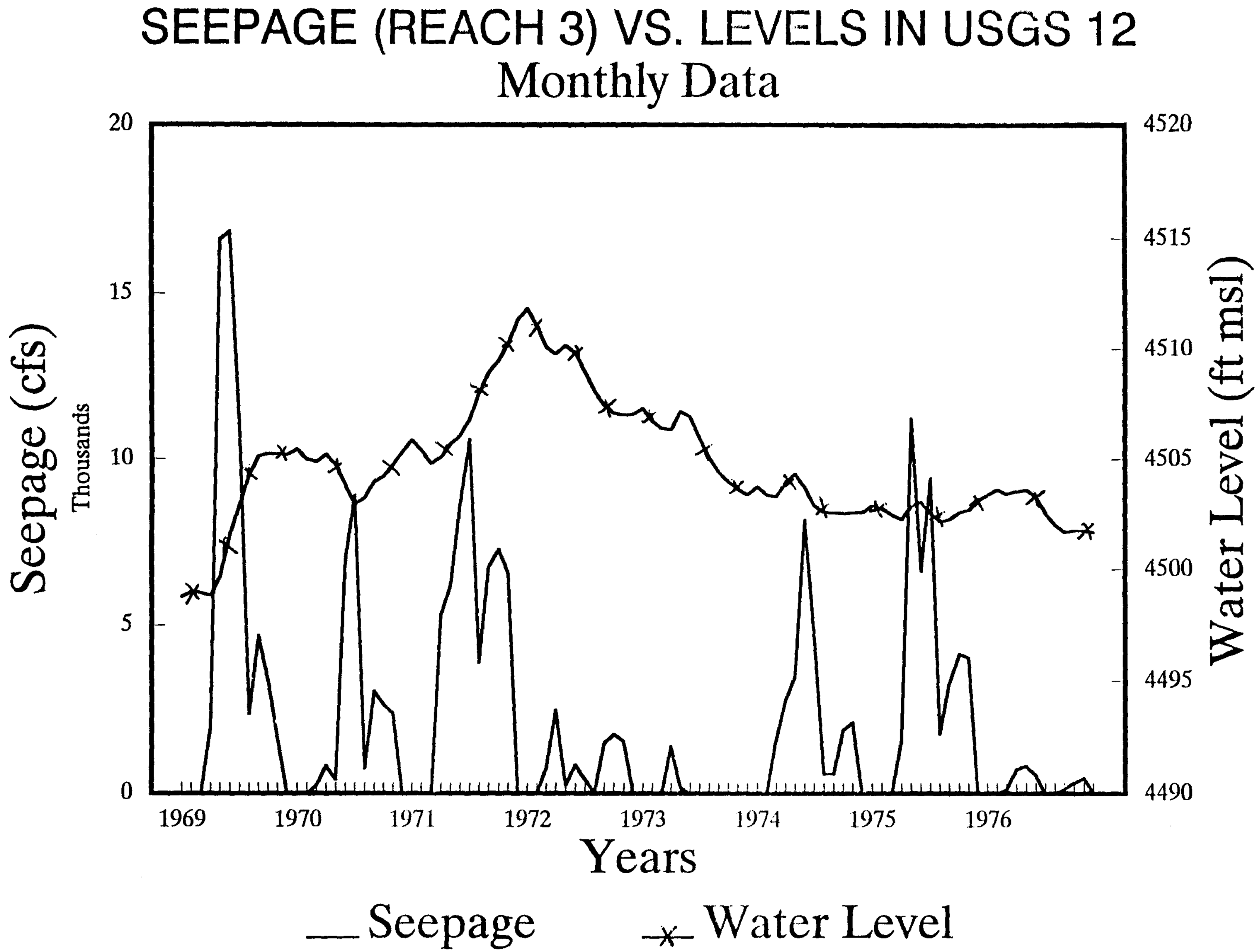
SEEPAGE (REACH 3) VS. LEVELS IN USGS 12
Monthly Data
Seepage (cfs)
Thousands
Water Level (ft msl)
20
15
10
5
0
4520
4515
4510
4505
4500
4495
4490
1969
1970
1971
1972
1973
1974
1975
1976
Years
Seepage
Water Level

FIGURE 5

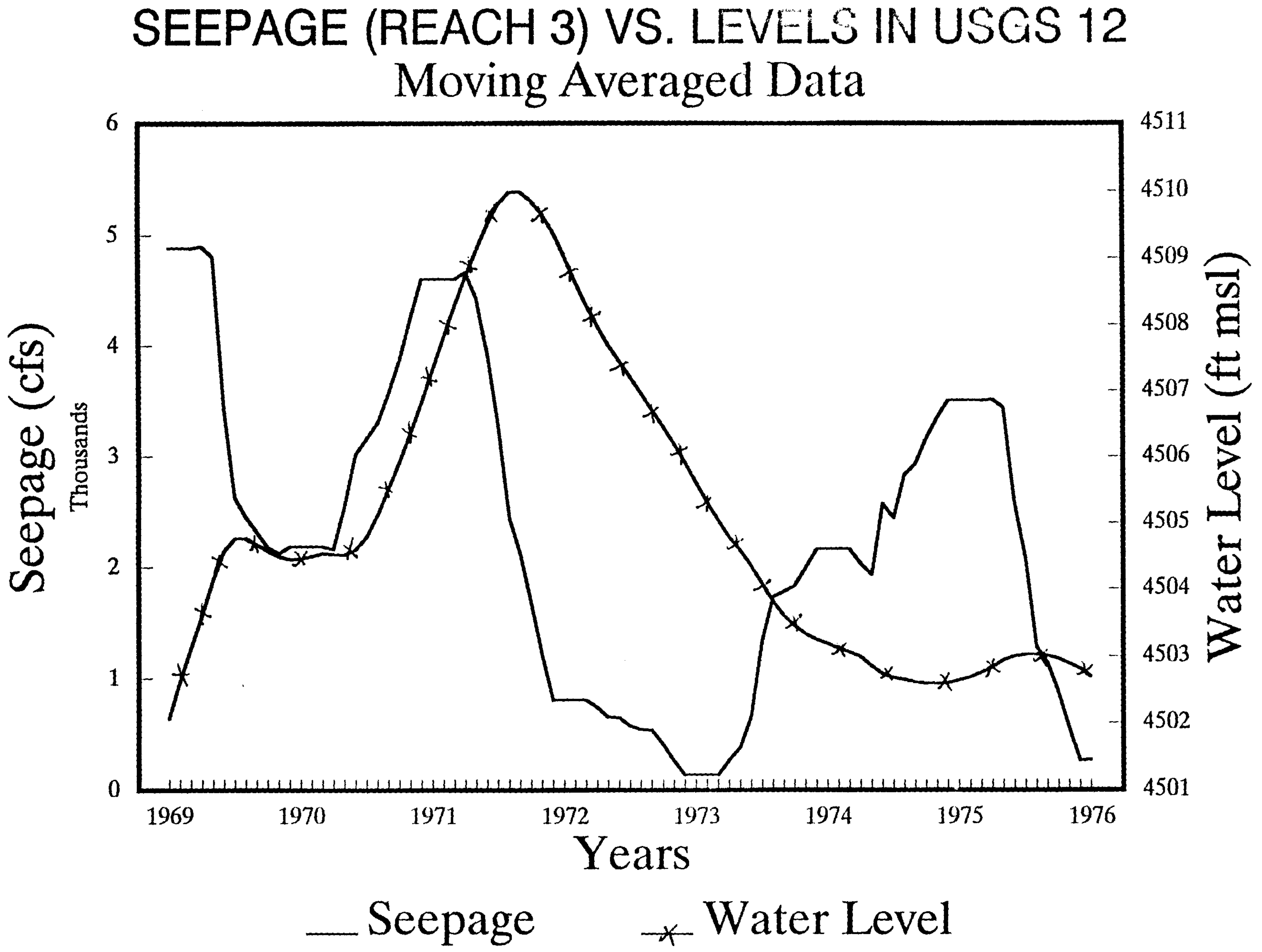
SEEPAGE (REACH 3) VS. LEVELS IN USGS 12
Moving Averaged Data
Seepage (cfs)
Thousands
0
1
2
3
4
5
6
Water Level (ft msl)
4501
4502
4503
4504
4505
4506
4507
4508
4509
4510
4511
1969
1970
1971
1972
1973
1974
1975
1976
Years
Seepage
Water Level

FIGURE 6

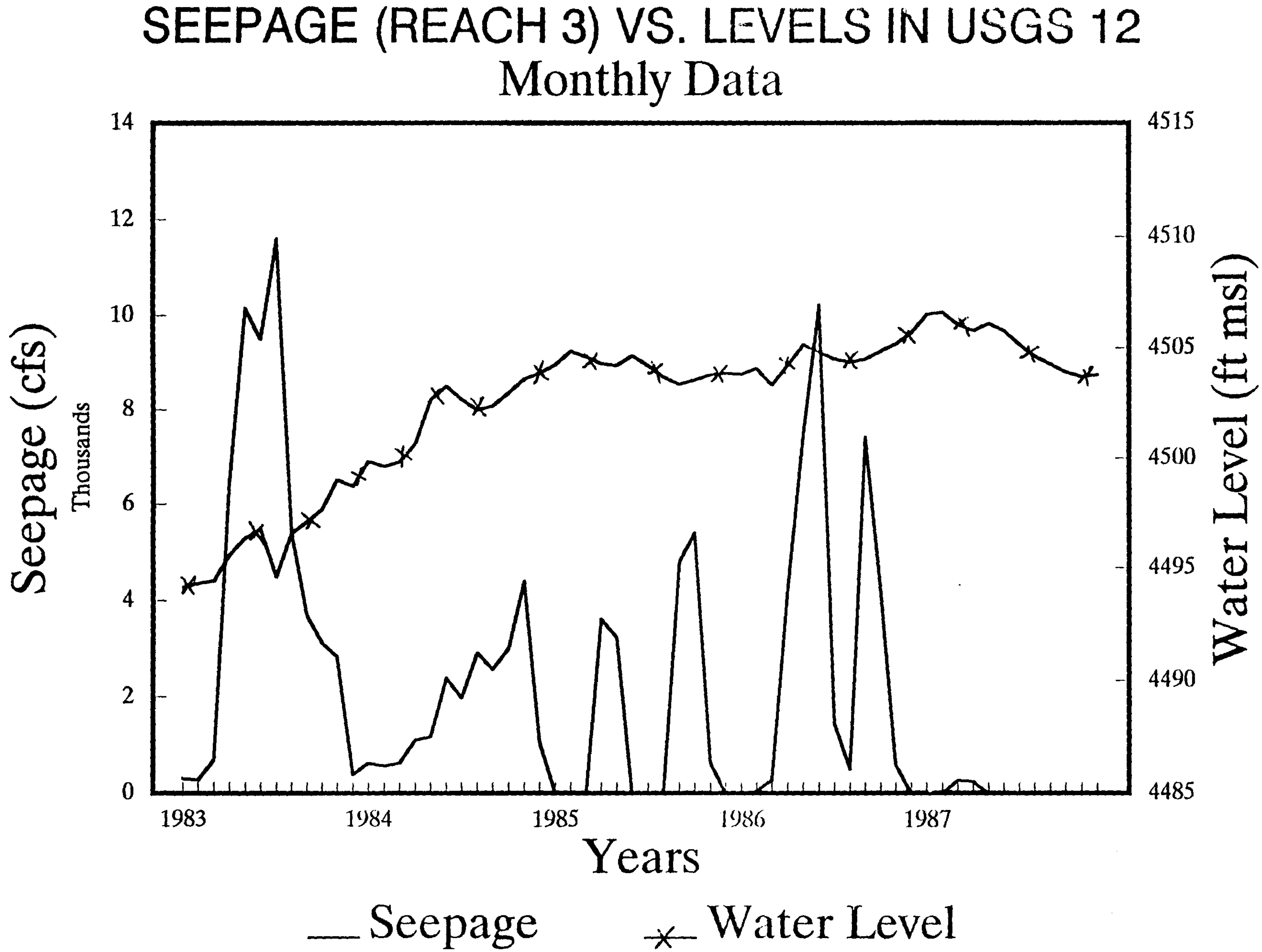
SEEPAGE (REACH 3) VS. LEVELS IN USGS 12
Monthly Data
Seepage (cfs)
Thousands
Water Level (ft msl)
14
12
10
8
6
4
2
0
4515
4510
4505
4500
4495
4490
4485
1983
1984
1985
1986
1987
Years
Seepage
Water Level

FIGURE 7

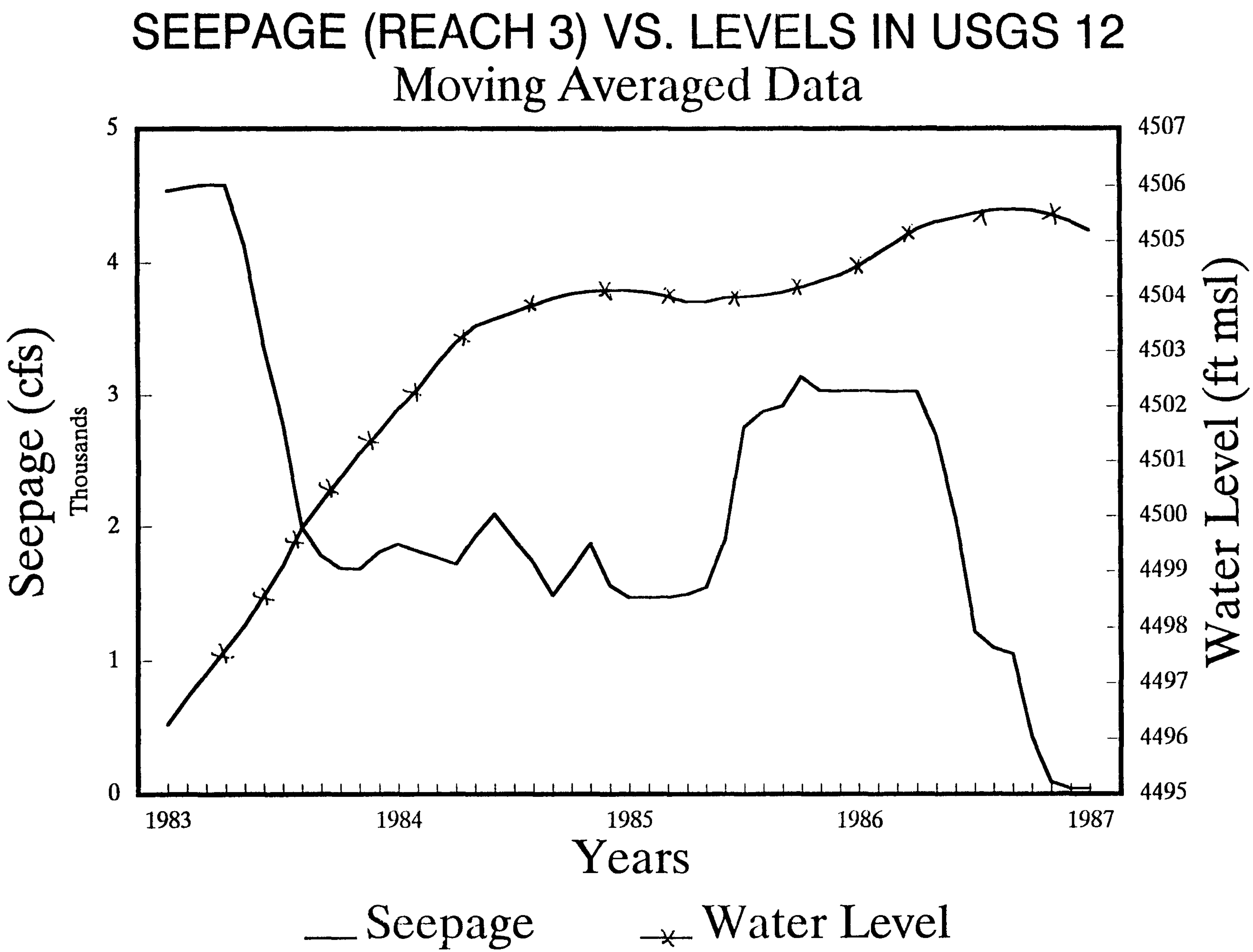
SEEPAGE (REACH 3) VS. LEVELS IN USGS 12
Moving Averaged Data
Seepage (cfs)
Thousands
Water Level (ft msl)
5
4
3
2
1
0
4507
4506
4505
4504
4503
4502
4501
4500
4499
4498
4497
4496
4495
1983
1984
1985
1986
1987
Years
Seepage
Water Level

FIGURE 8

and 0.04 (log of n = 1 month) for Reach 2. The correlation was even worse during the second time period.

Conclusions:

Unfortunately, these results did not yield any additional insight into the seepage/groundwater correlation in the vicinity of the Big Lost River. In fact, the previous analyses presented by Coasts, in comparing Reach 3 seepage with water levels in USGS 18, showed a far stronger correlation ($r^2 = 0.61$ at n = 7 months) that this supplemental evaluation of well USGS 12. For Reach 2, both studies indicated almost no correlation between the estimated Reach 2 seepage and water levels in USGS 9 or 84.

There are no definitive explanations for these results, although, in his thesis (pgs. 92-99), Coats provided several possible suggestions for lack of correlation in Reaches 2 and 3. Other possibilities may include a very uneven spatial distribution of fracture zones, with a more direct linkage between surface and groundwater in the vicinity of Reach 1 and the spreading areas; or regional flow patterns that overshadow the seepage impacts from these downstream reaches.

Perhaps, with further studies of other wells near these reaches, a more complete picture could be obtained of the timing of the groundwater response to river seepage. It is doubtful that any additional monitoring wells are necessary near the river, as long as those in place continue to be measured at least once each month. During any future periods of high flow in the Big Lost River, it would be appropriate to measure groundwater levels on a weekly basis while seepage is occurring from the reaches or spreading areas.

Data Base and Analysis Procedures:

To provide the INEL Oversight Committee with the information to continue with further analyses such as these, the data used in the thesis by Coats, and the supplemental study by Horn, have all been included as part of this Final Report. A total of 38 data files have been stored on three 3 1/2" floppy disks, and the following table lists each file and directory name, along with a description of the data contained.

These files are all in a QUATTRO PRO spreadsheet format, and it is assumed that INEL Oversight staff will have access to this software. Other proprietary spreadsheet software (such as EXCEL) permits QUATTRO PRO files to be translated into other spreadsheet formats. The regression and correlation analyses that were performed as part of these studies were done using the spreadsheet regression commands, simply identifying the appropriate dependent and independent variables, lagged in time as necessary. The software users manual will provide the details of these commands for any inexperienced user.

LIST OF QUATTRO FILES PROVIDED ON DISKS

Disk 1:	
File Name	Description
2500.WQ1	Avg. monthly flow at Gage 13132500
2510.WQ1	Avg. monthly flow at Gage 13132510
2513.WQ1	Avg. monthly flow at Gage 13132513
2520.WQ1	Avg. monthly flow at Gage 13132520
2535.WQ1	Avg. monthly flow at Gage 13132535
GS182535.WQ1	Avg. monthly flows and moving averages for Gage 13132535; avg. monthly water levels and moving averages for USGS Well 18
GS82500.WQ1	Avg. monthly flows and moving averages for Gage 13132500; avg. monthly water levels and moving averages for USGS Well 8
GS82510.WQ1	Avg. monthly flows and moving averages for Gage 13132510; avg. monthly water levels and moving averages for USGS Well 8
GS92500.WQ1	Avg. monthly flows and moving averages for Gage 13132500; avg. monthly water levels and moving averages for USGS Well 9
GS92510.WQ1	Avg. monthly flows and moving averages for Gage 13132510; avg. monthly water levels and moving averages for USGS Well 9
GS92535.WQ1	Avg. monthly flows and moving averages for Gage 13132535; avg. monthly water levels and moving averages for USGS Well 9
\GAGES\2500.WQ1	Raw data from Gage 13132500
\GAGES\2513.WQ1	Raw data from Gage 13132513
\GAGES\2515.WQ1	Raw data from Gage 13132515
\GAGES\2520.WQ1	Raw data from Gage 13132520
\GAGES\2535.WQ1	Raw data from Gage 13132535
\GAGES\2580.WQ1	Raw data from Gage 13132580
\SEEP-MON\R1-SEEP.WQ1	Total monthly seepage, Reach 1

Disk 1:	
File Name	Description
\SEEP-MON\SP-SEEP.WQ1	Total monthly seepage, Spreading Areas
\SEEP-MON\R2-SEEP.WQ1	Total monthly seepage, Reach 2; average monthly water levels for well USGS 84
\SEEP-MON\R3-SEEP.WQ1	Total monthly seepage, Reach 3; average monthly water levels for well USGS 12
Disk 2:	
File Name	Description
\SEEP-DAY\6976SEEP.WQ1	Daily seepage from spreading areas (1969-1976)
\SEEP-DAY\8387SEEP.WQ1	Daily seepage from spreading areas (1983-1987)
\SEEP-DAY\DAYSEEP1.WQ1	Daily seepage from Reach 1 (both time periods)
\SEEP-DAY\DAYSEEP2.WQ1	Daily seepage from Reach 2 (both time periods)
\SEEP-DAY\DAYSEEP3.WQ1	Daily seepage from Reach 3 (both time periods)
\WELLS\USGS-12.WQ1	Raw data from USGS Well 12
\WELLS\USGS-17.WQ1	Raw data from USGS Well 7
\WELLS\USGS-8.WQ1	Raw data from USGS Well 8
\WELLS\USGS-86.WQ1	Raw data from USGS Well 86
\WELLS\USGS-9.WQ1	Raw data from USGS Well 9
\WELLS\USGS-18.WQ1	Raw data from USGS Well 18
\WELLS\USGS-78.WQ1	Raw data from USGS Well 78
Disk 3:	
File Name	Description
2500COL.WQ1	Daily flow data in a single column (13132500)
2510COL.WQ1	Daily flow data in a single column (13132510)
2513COL.WQ1	Daily flow data in a single column (13132513)
2520COL.WQ1	Daily flow data in a single column (13132520)
2535COL.WQ1	Daily flow data in a single column (13132535)

APPENDIX

AUTHORIZATION TO SUBMIT

THESIS

This thesis of Erik R. Coats, submitted for the degree of Masters of Science in Civil Engineering and titled "Seepage Rates from the Big Lost River and Effects on the Groundwater at the Idaho National Engineering Laboratory," has been reviewed in final form, as indicated by the signatures and dates given below. Permission is now granted to submit final copies to the College of Graduate Studies for approval.

Major Professor ____________________ Date 1/10/94
Dennis Horn

Committe Members John Finnie ____________________ Date __________
John Finnie

George Bloomsburg ____________________ Date __________
George Bloomsburg

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Administrator ____________________ Date __________
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Discipline's
College Dean ____________________ Date __________
Richard Jacobsen

Final Approval and Acceptance by the College of Graduate Studies

____________________ Date __________
Jean'ne M. Shreeve

SEEPAGE RATES FROM THE BIG LOST RIVER AND EFFECTS ON THE GROUNDWATER AT THE IDAHO NATIONAL ENGINEERING LABORATORY (1969-1976, 1983-1987)

ABSTRACT

by Erik R. Coats, M.S.
University of Idaho, 1993

Chairman: Howard Peavy

A seepage-groundwater study was performed for the Idaho National Engineering Laboratory (INEL) in southeastern Idaho to determine if the vast quantities of seepage from the Big Lost River had a measurable and predictable impact on the local groundwater basin beneath the INEL. Daily seepage quantities from the Big Lost River were estimated using both the inflow-outflow method and an empirical technique developed during the research. Total monthly seepage values were then calculated and compared against average monthly water levels for selected wells on the site. The hypothesis was that a time lag would exist between seepage events and changes in groundwater levels.

A simple linear regression was applied to the data, with groundwater levels dependent on seepage, in an effort to determine a time lag. Regressions were performed on both the raw monthly data and one-year moving averages of the data. Five different comparisons were examined, using both raw monthly data and moving averaged data, and three demonstrated very strong correlation. The three were: seepage from reach 1 versus groundwater levels in USGS 8, seepage from the spreading areas versus USGS 9, and the combined seepage from

reach 1 and the spreading areas versus USGS 9. For reach 1 and USGS 9, correlation coefficients for the raw monthly data and the moving averaged data were 0.359 at a lag of seven months and 0.884 at a lag of six months, respectively. The spreading areas versus USGS 9 comparison yielded coefficients of 0.683 and 0.937 (both with a time lag of five months), respectively, and for the combined seepage and USGS 9, coefficients were 0.94 and 0.68 at a time lag of five months.

ACKNOWLEDGEMENTS

The author wishes to sincerely acknowledge his appreciation to Professor Dennis Horn for his guidance and assistance while acting as major professor and administering the project and assistantship that made the author's education possible.

Gratitude is also expressed to Dr. John I. Finnie for taking time to listen to concerns and answer questions.

Thanks are due to Dr. Howard Peavy, Head of the Department of Civil Engineering, for his review of the thesis.

Thanks are also due to the Idaho National Engineering Laboratory for their support. The documents and data provided were critical to the research, and without their cooperation this project could not have completed.

Finally, a very special thanks to my parents and my wife, Kristine. Their support was both abundant and unending.

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CHAPTER I

Introduction

The Idaho National Engineering Laboratory (INEL) encompasses about 890 square miles of the eastern Snake River Plain in southeastern Idaho (figure 1.1). Established in 1949 to build, test, and operate nuclear reactors, INEL's primary activities have included testing of various types of nuclear reactors and reactor fuel cells, the processing, consolidation, and temporary storage of nuclear wastes, and various environmental research projects. As a consequence of these operations, tritium, strontium-90, iodine-129, nitrate, sodium, and chloride have been disposed to or have migrated downward to the Snake River Plain aquifer, which is the major aquifer underlying the Snake River Plain (Bennett, 1990). Seepage from the Big Lost River, which flows onto the INEL, can greatly affect the concentration and distribution of these contaminants in the aquifer.

The Big Lost River begins in the Pioneer Mountains and the Lost River Range and flows southeast towards the INEL site, past Arco, and to its terminus (known as the playas) in the northern portion of the site. Flow becomes intermittent past Arco; intermittent because, depending on the magnitude of the flow in the river, sometimes water will reach the playas and other times it will infiltrate well in advance of them. Two causes can be attributed to this phenomenom. First, the channel is lined with highly permeable alluvial

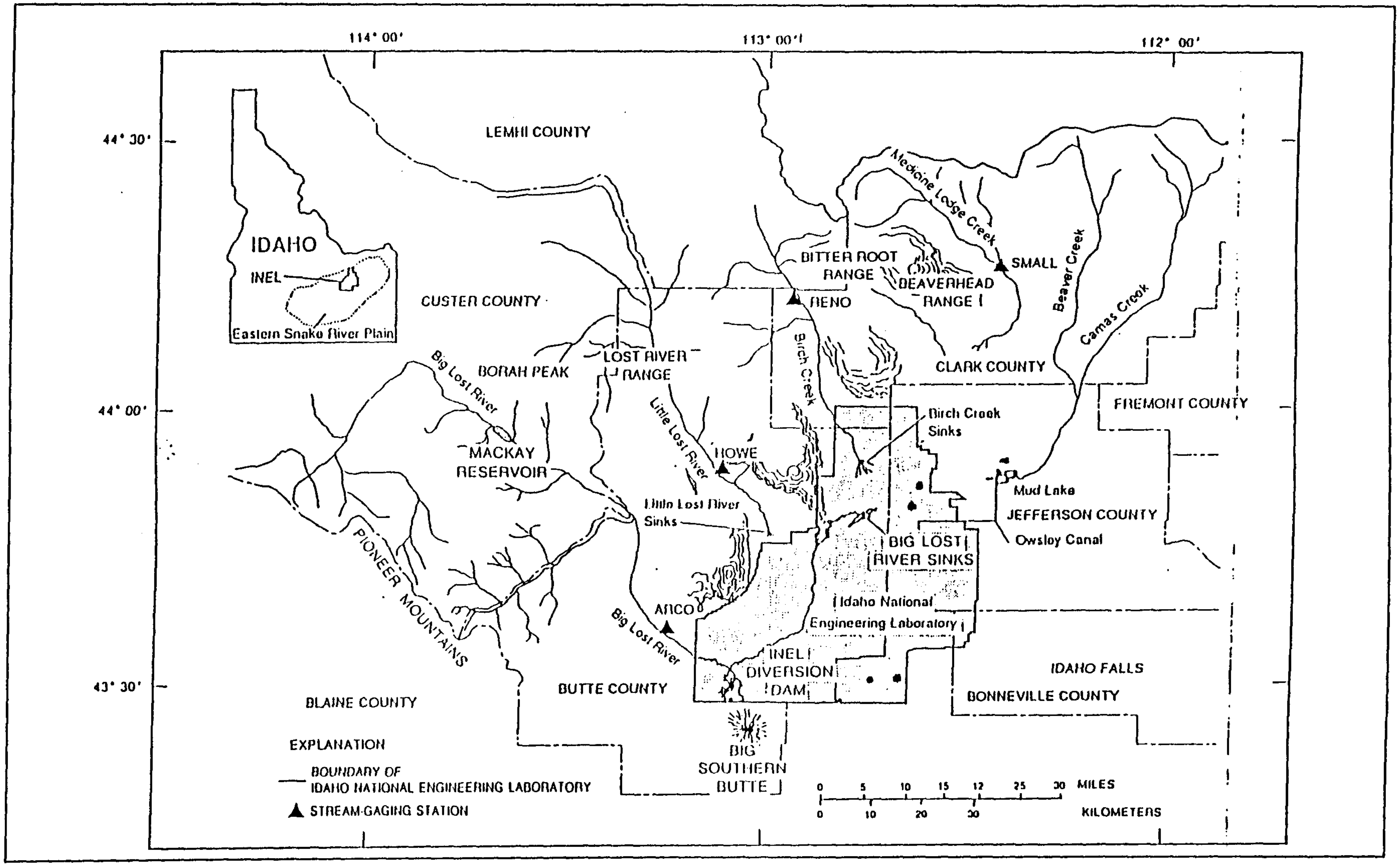

Figure 1.1 - Location of the Idaho National Engineering Laboratory with the Big Lost River

deposits, which allow for high seepage rates. Second, the Snake River Plain aquifer is at great depths (greater than 200 feet below land). Therefore, seepage from the river feeds the groundwater, with no return flow to the river. Since the Big Lost River is highly prone to lose large quantities of water through seepage, and since seepage can have such a large impact on the groundwater, it is important to identify high seepage areas and quantify the amount of seepage that can be expected. To complete the study, the correlation between seepage and groundwater should be examined because of its potential impact on contaminant migration.

Purpose

There are two primary objectives of this thesis. First is to study seepage rates along the Big Lost River basin from Arco through the INEL to the playas, determine seepage functions for selected reaches based on daily flows, and develop a daily seepage record for these reaches. Second is to study the effects of seepage on the regional groundwater system and develop relationships between seepage and groundwater levels. The years included in the study are 1969-1976 and 1983-1987.

Two similar studies have been performed on the Big Lost River and Snake River Plain aquifer beneath the INEL, but they both had limitations. Bennett (1990) used monthly streamflow data to obtain seepage functions, and did not attempt to develop equations relating seepage to groundwater levels. Nace and Barraclough (1952) did not develop any equations for seepage or seepage-groundwater correlation, and because substantial groundwater data were not available, only stated general conclusions regarding flow in the river and corresponding groundwater levels. This study has substantially more data than available to Nace and Barraclough, and uses daily rather than monthly flows.

Objectives

The specific objectives of this thesis are as follows:

1. Collect streamflow data for the Big Lost River and groundwater level data for the INEL.

2. Determine daily seepage rates for the Big Lost River and seepage equations for river reaches using regression analyses.

3. Correlate seepage with groundwater levels.

CHAPTER II

Big Lost River Basin

The Big Lost River flows out of the Pioneer Mountains and the Lost River Range onto the eastern Snake River Plain near Arco, Idaho, draining about 1500 square miles (see figure 1.1). Flow in the river is controlled by Mackay Dam, an irrigation reservoir 30 miles upstream of Arco near the town of Mackay, Idaho. Between Mackay Dam and Arco there are numerous irrigation diversions that operate between April and September. Much of the flow in the river is diverted for irrigation upstream of Arco, and therefore water reaches the Snake River Plain only during higher water years or large flood events.

During higher water years, water will flow past Arco and eventually across the western boundary of the INEL. Here the river flows out of a narrow canyon and into a channel that is 200- to 300-feet wide and cut into the plain less than 20 feet deep. This is in contrast to the 60 foot cut just downstream of the Arco gage. Approximately 6.5 miles downstream from the INEL boundary the river is split by the INEL flood diversion system, which was constructed in 1958 and enlarged in 1984 (Bennett, 1990). Here an earth dam diverts flow from the river into four spreading areas, A, B, C, and D (figure 2.1), where water is allowed to both seep into the ground and evaporate. Gates in the dam permit undiverted flow to continue onto the site.

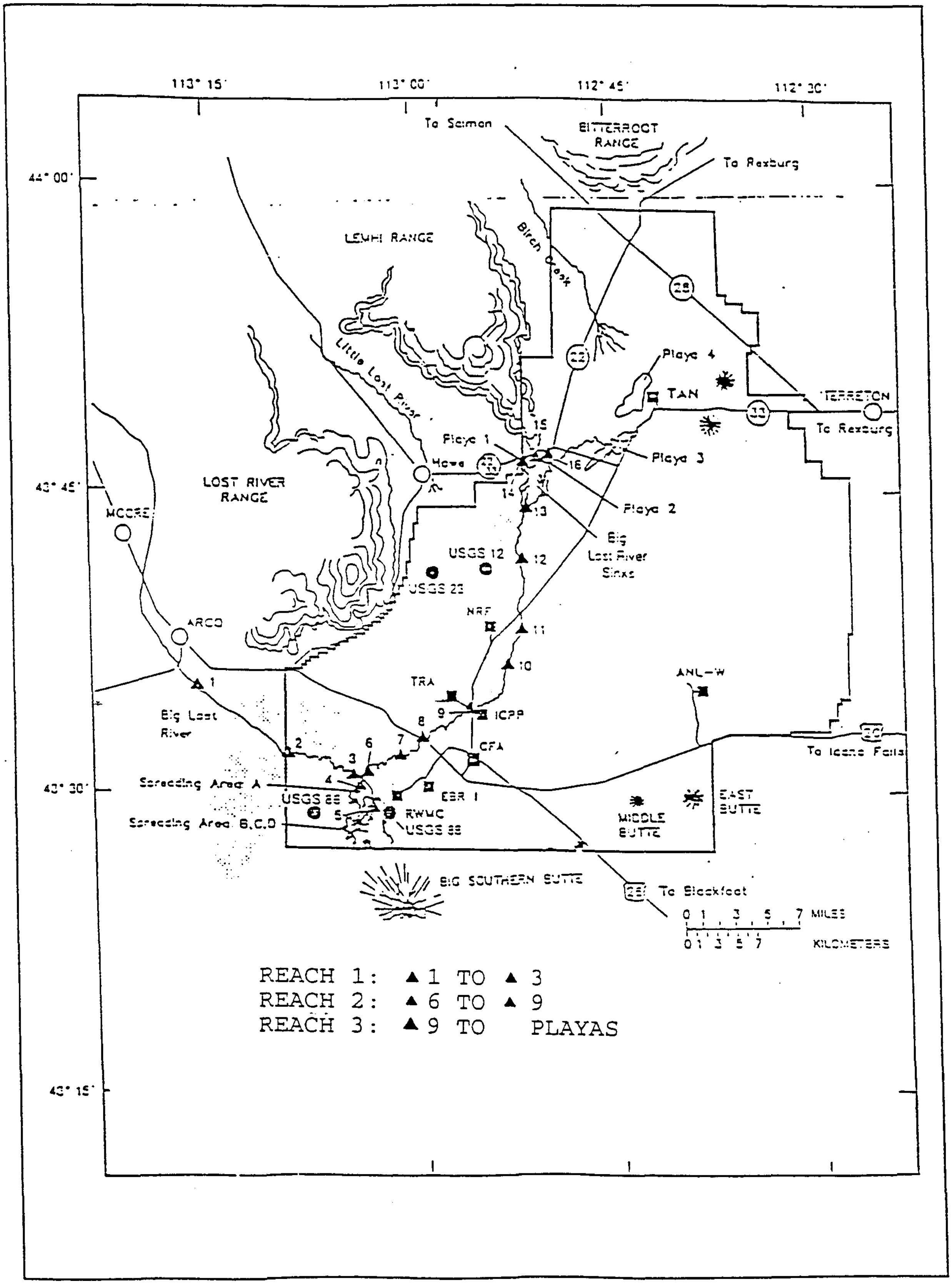

Figure 2.1 - Location of the Spreading Areas on the Idaho National Engineering Laboratory

As the river flows northward, the channel continues to be less incised into the plain. Near highway 20, 6 miles downstream of the diversion dam, the channel depth is less than 10 feet, and downstream of highway 20 the river settles into a floodplain 1 to 4 miles wide.

Finally, as the river nears the playas, flow splits into a number of small channels that lead to the terminus. The playas (figure 2.1), named simply 1, 2, 3, and 4, have areas of 350, 110, 1000, and 1350 acres, respectively. Only during extremely high flow events does flow reach the playas.

Spreading Areas

In the early 1950s flooding occurred both at the Test Reactor Area (TRA) and Idaho Chemical Processing Plant (ICPP) (McKinney, 1985). These incidences, along with other research and investigation, prompted the need for flood protection for the INEL site.

The original flood detention system was built in 1958. It was designed to divert 1000 cfs out of the main river channel into the spreading areas (McKinney, 1985). As already stated, there are four cells to the system: spreading areas A, B, C, and D. Water first flows into A, then progressively passes through the next three as each basin fills.

A large runoff event in the spring of 1965 was approximately double the flood diversion design event (55 years, as later determined). All four basins nearly filled up, and it took about a month for the levels to subside

(McKinney, 1985). This event proved the need for the diversion areas, as well as the need to expand them. It also showed there was the need to detain the water for several months, rather than a few days as the system was designed for. In 1966 some minor work (riprapping dikes 1 and 2 and increasing the back slope to 3:1) was performed to the system to ensure it could detain flows for longer periods of time (McKinney, 1985).

Much research has been performed on the Big Lost River system. In 1969, Lamke determined stage-discharge relationships for both the flood diversion system and the Big Lost River. A report in 1972 summarized P.H. Carrigan's results for a study on the probability of exceeding the flood diversion system. He determined that it would be exceeded on the average once every 55 years, and that if the capacity of the diversion channels were doubled the system would take a 300 year storm event.

In late 1983 and early 1984 there was another flood threat on the site, and again the flood diversion system was tested. Details of the event are very lengthy (see McKinney, 1985), but in summary, air temperatures dropped well below 0°F and ice formation in the diversion channel nearly caused overtopping of the dike. An extensive work force eventually got the problem under control without too much damage to the system. Due to this near-catastrophe, the detention system was upgraded in 1984 to handle a peak flow of 5300 cfs.

Snake River Plain Aquifer and Geology

General

The eastern Snake River Plain is a structural basin 200 miles long and 50 to 70 miles wide. The INEL lies in the west-central portion of the plain, and is underlain by Tertiary and Quaternary volcanic rocks interbedded with sediment deposits that include clay, silt, sand and gravel. The sequences of rock and sediments are greater than 10,000 feet thick (Rightmire and others, 1987).

The Snake River Plain aquifer is a groundwater reservoir that may contain more than 1 billion acre-feet of water (Barraclough and others, 1981). The aquifer can be very productive, producing several thousand gallons per minute from the basalt-sediment sequences with little drawdown. Transmissivities range from 134,000 to 13,400,000 ft^2 per day (Robertson and others, 1974, p. 12). Depth to water is 200 feet in the north and 900 feet in the south, with a regional groundwater flow direction from northeast to southwest. The effective base of the aquifer likely coincides with the top of a thick and widespread sequence of clay, silt, sand and basalt (Anderson, 1990). For well INEL-1, a very deep observation well on the site, this sequence is found to begin at 1220 feet below the land surface (Mann, 1986). In other wells on site it ranges from depths of 800 to 1500 feet. This suggests the effective aquifer thickness varies from 600 to 800 feet in most places, based on the fact that the assumed base of the aquifer slopes from northeast to southwest, nearly parallel with the slope of the water table (Anderson, 1991).

RWMC, ICPP and TRA

The Radioactive Waste Management Complex (RWMC) is a facility for the storage of radioactive and chemical wastes. Low-level and transuranic wastes are buried in shallow pits and trenches. The Idaho Chemical Processing Plant (ICPP) has been used for reprocessing of spent nuclear fuel rods, and the Test Reactor Area (TRA) for nuclear research.

All three sites and their immediate surroundings are underlain by many basalt flows, basalt-flow groups, and basalt-flow units. A basalt flow is a solidified body of rock that was formed by a lateral, surficial outpouring of molten lava from a vent or fissure (Bates and Jackson, 1980). A basalt-flow unit is a separate, distinct lobe of lava that issues from the main body of a lava flow (Bates and Jackson, 1980). A basalt-flow group is a sequence of one or more petrographically similar flows or flow units that are extruded from the same vent or magma source within the course of a single eruption or multiple eruptions during a relatively short interval of time (Kuntz and others, 1980). There are many basalt groups, with thicknesses up to 114 feet, that either lie directly over older groups or are separated by a sediment bed, which may have been deposited during volcanic inactivity. There are some major sediment beds, ranging up to 50 feet in thickness and containing poorly to well sorted layers of clay, silt, sand and gravel.

Depth to water at the TRA and ICPP sites ranges from 430 to 480 feet, whereas it is approximately 600 feet at the RWMC. Zones of perched water are found at all sites. At the TRA and ICPP sites they are a result of seepage from percolation ponds and at the RWMC they are caused by seepage from the diversion ponds, which are south of the RWMC.

Hydrologic Implications

The many basalt flows all are fractured to some degree which allows water to move vertically and horizontally through them. Sediment beds facilitate or retard the movement of water, depending on the sorting and grain sizes. When water becomes perched, it may move horizontally until finding either a more permeable zone, a vertical fracture or a well open to deeper depths.

The area just north and east of the TRA and ICPP has experienced structural uplift, which has caused fracturing of the sediments and basalts. As a result, the hydraulic conductivity has increased and groundwater responds more rapidly to recharge. Also, many of the older flow groups throughout the site have experienced tilting, folding and thus fracturing. All of this complicates the mechanisms of unsaturated flow, and makes it difficult to predict the flow of water from the ground surface to the groundwater.

<u>Climate</u>

In general, the climate at the INEL is semiarid. Average annual precipitation is 9.07 inches, with maximum 24-hour and 1-hour values of 2 inches and 1 inch, respectively.

Snow does fall on the site from mid-November to mid-April, and the average annual snowfall is 26 inches. The largest depth ever measured at the site is 27 inches. Average monthly maximum temperatures range from 87°F in July to 28°F in January, and average minimums range from 49°F in July to 4°F in January.

CHAPTER III

Groundwater Recharge

Many models have been developed to determine groundwater recharge. A basic approach is presented by Freeze and Cherry (1979). They state that if we limit ourselves to watersheds in which the surface water divides and groundwater divides coincide, and for which there are no external inflows or outflows of groundwater, the water balance equation for an annual period would take the form:

$$P = Q + E + \delta S_s + \delta S_G$$

where:

P = precipitation,
Q = runoff,
E = evapotranspiration,
δS_s = change in storage of the surface-water reservoir,
δS_G = change in storage of the groundwater reservoir (both saturated and unsaturated).

This is a very simple approach to the interactions between groundwater and surface water. Other, more complex models have been developed based on this equation. Most primarily examine recharge from agriculture activities, although some include canal and stream losses. Also, some models consider just the saturated zone while others integrate both the unsaturated and saturated zone. Most significant and reliable models include both processes.

Freeze (1969) developed a one-dimensional, vertical, unsteady, unsaturated flow model for groundwater recharge. The model calculates runoff and the amount of infiltrated water after irrigation and precipitation. It then relates the unsaturated zone processes of infiltration and evaporation to the saturated zone processes of recharge and discharge. Freeze defines these processes as follows. Infiltration is the entry into the soil of water made available at the ground surface, together with the associated downward flow. Evaporation is the removal of water from the soil at the ground surface, together with the associated upward flow. Recharge is the entry into the saturated zone of water made available at the water table surface, together with the associated flow away from the water table within the saturated zone. Discharge is the removal of water from the saturated zone across the water-table surface, together with the associated flow toward the water table within the saturated zone.

As stated, water table fluctuations occur when a change in recharge or discharge is not compensated by a change in infiltration or evaporation. Controlling parameters in the model include: rate of rainfall or evaporation, duration of rainfall or evaporation, soil type, antecedent soil moisture conditions, groundwater recharge or discharge rate, depth to the water table, and depth of ponding. This model requires extensive data and time to set up and run.

Burrell (1987) developed a computer model for the Oakley Fan area of southern Idaho. The goal was to determine the amount of recharge to the groundwater system from deep percolation and canal seepage in the irrigated portions of the study area (Burrell, 1987). Recharge was calculated for grids of one-half mile square. The model included the effects of evapotranspiration, change in soil moisture, deep percolation, and canal and stream seepage losses, and was based on a monthly timestep. Deep percolation is calculated using the net irrigation application plus precipitation minus evapotranspiration, and requires that hydraulic conductivity be calculated if flow is unsaturated. The Brooks-Corey relationship is used to calculate the unsaturated conductivity, and it requires the displacement pressure and the capillary pressure in each soil layer.

Only two examples of unsaturated-saturated recharge models are discussed here, but they demonstrate the complexity involved when including the unsaturated zone. Hydraulic conductivity must be calculated, and since it varies with capillary pressure, measurement probes need to be installed at different locations within the unsaturated zone. Soil moisture is also needed and must be measured similarly to capillary pressure. These are only a few of the parameters required for the unsaturated zone, but they demonstrate how extensive the unsaturated zone must be monitored to run an unsaturated - saturated recharge model.

Seepage Estimation Methods

Three primary methods are used to measure seepage from a river or canal. They are the ponding method, the inflow-outflow water balance method, and the seepage meter method. There are advantages and disadvantages to each method depending on the area of study and the type of information available.

The ponding method requires construction of temporary bulkheads across each end of a channel to impound water for measurement. Once they are in place, the experimenter then monitors the change in water depth in the impoundment. Seepage is calculated by determining the total volume of water that leaked out during the monitoring period.

This method can only be used under special circumstances because impoundment of a stream or canal cannot easily be done, and bulkheads are very expensive and take time to install. Also, results from this method have some drawbacks. If the reach impounded is too long, the measured seepage rate is only an average, and any high rate seepage areas are not located. Also, impoundments allow for greater sedimentation, which can seal, to a degree, the wetted perimeter. Therefore the seepage rates measured could be lower than if influenced by currents. Seepage is also controlled by the wetted surface area and hydraulic head. Ponding can increase water depth and the wetted area, so calculated seepage rates could be skewed higher than exist naturally. Increased head forces more water into the soil, which speeds up and increases the depth of soil saturation. When a soil becomes saturated, water flows more readily through the strata because the soil-water tension has been decreased and voids have been filled. Increased wetted area provides more surface area for the water to escape through.

Despite the problems associated with the results from this method, it does yield the most reliable results for average reach seepage, compared to other methods, because all inputs and outputs can be accurately measured.

A simpler approach to seepage estimation is the inflow-outflow method, which measures seepage using a water balance approach. All inflows and outflows from the experimental reach are recorded, and seepage becomes inflow minus outflow. In-stream flow must be measured as well as any diversions, return flows, leaks, and spills from the watercourse. Accuracy for this method relies entirely on the accuracy of the flow measurements. To minimize any inaccuracies in measurements, long reaches should be used. Seepage from the watercourse then outweighs the errors in measurement.

The inflow-outflow method is best applied when only average reach seepage rates are needed. The data, inflows and outflows, are generally available, provided most inflows and outflows are already gaged, so little extra setup of measuring equipment is required. However, it cannot be used to determine high or low loss areas in a reach because measurements are not that accurate in short reaches.

A third seepage estimation method is seepage metering, which consists of monitoring seepage meters installed in the bed of a watercourse. With such meters, seepage can be measured for small areas. An advantage is that seepage can be measured throughout the year because the meters require no special operating conditions, as does the ponding method. This technique cannot be used in rocky areas because the bed material must seal around the cup. Also, to obtain a reliable average value of loss for a river reach, measurements must be made at many locations. This method is considered to provide good quantitative results when applied correctly, but is used primarily to locate high and low seepage areas rather than average reach seepage rates.

Netz (1980) applied the inflow-outflow method to determine seepage from canals in southeastern Idaho. Flow measurements were broken into two groups. Either they met all specified criteria set by the author, and were "prime time measurements" (Netz, 1980), or they violated one or more criteria. These criteria were adopted to eliminate some of the errors of seepage measurements, and they were as follows (Netz, 1980):

1. Water measurement conditions are such that no more than ±5 percent error in flow rates can be expected.
2. The canal stage is low and fluctuating no more than 0.02 feet during the time the measurements are being made.
3. The reach of the canal is long enough to assure that the accumulated error due to water measurement will not be over ±75 percent of the measured outflow due to seepage.

All collected flow data were separated into their respective groups and analyzed statistically to determine the reliability of the measurements. Statistics were calculated using a computer program called Statistical Analysis System. In this program a general linear model was applied to the data, with seepage dependent on canal bottom type, soil type, and season of measurement.

Results of the study showed that the "prime time measurements" gave the best results, and that the inflow-outflow method was best applied during very low flow periods when the "prime time measurement" criteria could be met. Also, Netz determined that the factors used did not show a high enough significance to be predictive parameters of seepage, and that the study did not suggest that a mathematical model could be applied to the study area to predict seepage. Netz noted that groundwater was high in some areas of study and could have had a large impact on canal flow.

Previous Site Research

In 1990, C.M. Bennett, then employed by the USGS, researched streamflow losses for the Big Lost River from Arco through the INEL site and wrote a report detailing his results. The period of time he examined was July 1972 to July 1978 and July 1981 to July 1985. Seepage losses in acre-feet per month were calculated for river reaches between streamflow gages using the inflow-outflow method and monthly flow data from the river gages. Seepage was defined simply as the monthly flow at the upstream gage minus monthly flow at the downstream gage. Four reaches, or areas, were examined: the Arco gage to the INEL diversion dam, the diversion spreading areas, below the diversion dam to Lincoln boulevard, and Lincoln boulevard to the playas. Equations to predict flow at downstream gages based on flow at the next upstream gage were also developed for the reaches from the

Arco gage to INEL diversion dam, and the INEL diversion dam to Lincoln Boulevard. Monthly flows were plotted with the upstream gage on the x-axis and the downstream gage on the y-axis. A regression analysis was run on these data with the upstream gage as the independent variable, and regression equations were developed. The resulting equations were nearly linear, with r-squared values of 0.990 and 0.987, respectively. Regression analyses were not run on the other two reaches because there either were not enough data available or regression analysis did not apply.

Four wells were used to examine the relationship between flow in the river and groundwater level changes. Well hydrographs were presented and compared to high and low river flow periods. No attempt was made to relate the flow in the river and groundwater levels by quantitative equations; only general discussion was made.

Nace and Barraclough (1952) also studied recharge from the Big Lost River. During the second half of 1951 there was very high runoff in the Big Lost River, which permitted a study of seepage from the Big Lost River. At that time only one gage was permanent (the Arco gage), so ten temporary measuring sites downstream of Arco were established. Daily flow measurements were taken at Arco and periodic measurements (3 to 5 times between the months of August and November) were taken at the other sites during the periods of high flow. River reaches were defined as reaches between measuring stations, and seepage rates for each reach were calculated on days when measurements were made at all stations. Units of seepage were in cubic feet per day per square foot, using river cross sections and stadia measurements to obtain cross sectional area. Groundwater levels were monitored, but since the event was so short-lived no direct conclusions could be drawn. Based on the streamflow at the Arco gage, the total daily and annual amounts of recharge, minus an assumed two percent loss for evapotranspiration, were estimated by assuming all flow past Arco either sank into the ground or evaporated.

CHAPTER IV - SEEPAGE ESTIMATION

Introduction

The selection of appropriate seepage estimation methods for the Big Lost River was a critical step in meeting the objectives of this study. Although different methods were reviewed for application, factors such as data availability and accuracy limited the potential methods to those which would yield "lumped", or average daily, seepage rates for the different reaches along the river. A traditional technique, the inflow-outflow method (previously described), was selected for each river reach where flow rates were available at both ends. For the spreading areas, where the inflow was known but the time-rate of outflow (seepage and evaporation) was not measured, other, less accurate methods were explored, and will be explained in further detail later.

Results from this portion of the study included the development of seepage equations and the calculation of daily seepage values for the entire period of study for all the defined river reaches and spreading area. These daily seepage values were then later used to compare seepage and groundwater levels, as determined from selected well data.

To estimate seepage from the river, reaches had to be defined and corresponding streamflow data obtained. All available streamflow gage data for the Big Lost River were collected and reviewed. Time was spent studying Bennett's report (1990) since his work was recent and similar to the research in this study. It seemed reasonable to consider using similar study reaches. Primary requirements in the delineation of reaches were that all flow into or out of each reach must have been measured, and records of these flows be available for extended periods of time. Extensive prior records were necessary for two reasons. First, there was no time to perform field flow measurements, and most important, due to extended drought conditions, there had been little or no flow in the Big Lost River below Arco in recent history. Also, to develop justifiable equations and results, a long period of record was necessary because streamflow is highly variable, and short periods of time do not fully demonstrate the possible variations in flow or seepage conditions.

Study reaches were apparent after a study of gage locations and Bennett's report. Gages with extended periods of record were located at Arco, in the entrance to the diversion areas, just below the diversion dam, and at Lincoln Boulevard. In addition, a gage record could be synthesized upstream of the diversion dam by summing flows from the gages below the dam and into the diversion areas. Since these were the only gages available below Arco, their location defined study reaches, and they were the same as those used by Bennett. This permits the results of this study to be compared with those obtained previously.

In summary, the reaches used in this study were as follows: reach 1 was from Arco to upstream of the diversion dam, reach 2 was from below the diversion dam to Lincoln Boulevard bridge, and reach 3 was from Lincoln Boulevard bridge to the playas. Figure 2.1 shows the study area, including the river gage locations, reaches, spreading areas and the approximate locations of the wells used to study seepage/groundwater relationships.

For each river reach defined, inflow and outflow data were available, with the exception of some irrigation return flow just below Arco and any surface runoff. However, irrigation return flow is apparently minimal. Bennett stated (1990) that miscellaneous measurements indicate that return flow during the irrigation season probably is less than 1 cfs. There were times during the period of study when return flow and/or surface runoff caused flow to increase in a reach, but it was not often (13.8% of the time in reach 1 and 11.2% of the time in reach 2), and as will be explained later, seepage for these days was determined differently than for other days.

<u>Seepage Estimates for River Reaches</u>

Using the inflow-outflow method, seepage in a reach can be defined as:

$$S = Q_1 - Q_2 + \Sigma q$$

where S is the average daily seepage, Q_1 and Q_2 are the mean daily upstream and downstream flow rates, respectively, and Σq represents the sum of all other inflow (runoff, return flow) and outflows (evaporation, water withdrawals). With no data available to estimate Σq (termed "local inflow") with any accuracy, and with the results of other studies indicating that most of the time local inflow is probably small, it was therefore assumed that as long as Q_1 exceeded Q_2, Σq could be considered equal to zero. However, for those days when Q_2 was greater than Q_1, this seepage equation could not be applied without resulting in a negative seepage rate (outflow from groundwater), which is unrealistic, given the depth to groundwater. It therefore became necessary to use another method of estimating seepage for these days, since an entire set of daily seepage values was necessary for the second portion of the study, comparing seepage and groundwater levels.

After examining various alternatives, it was eventually decided to use regression analysis to develop, for each reach, an equation that would estimate the daily seepage rate that might be expected if the Σq term in the seepage equation was indeed equal to zero. This equation could then be used in lieu of the inflow-outflow method for those 10-15% of the days with apparent negative seepage.

As an initial attempt to develop these seepage equations, the relationship between Q_1 and Q_2 for each reach was closely examined. Previously, Bennett had ascertained there was a linear relationship between monthly upstream and downstream flows, both in reaches 1 and 2. Scatter plots of daily flows for the same reaches, upstream gage against downstream gage, demonstrated a similar linear relationship. It was therefore postulated that a linear regression in the form of $Q_2 = f(Q_1)$ could be used to estimate downstream flow, given the upstream flow, and this estimated value then used in the previous seepage equation in place of the measured Q_2.

Initially, it was assumed that all daily flows should be used in developing these regression equations, including those days when downstream flow exceeded upstream flow. As hypothesized, if all flows were included, then the equations would be more accurate. A linear regression was therefore performed using the data for each reach, with downstream flow dependent on upstream flow. Linear regression, rather than nonlinear, was selected because only two variables were involved in the analysis, and a linear relationship had already been suggested by both the scatter plots of the data and by Bennett's study.

To evaluate the results of these regressions, the r-squared (r^2) value was used to indicate the fraction of the total variation in the dependent variable that is explained by the independent variable. The closer r^2 is to one (r^2 can be any number between 0 and 1), the more successful the linear regression model is in predicting values of the dependent variable. In other words, the higher the r^2 the more linear the correlation between the two variables. As might be expected, the regressions for both reach 1 and 2 indicated strong linear correlation between upstream and downstream flows, with r^2 values of 0.98 for both reaches. Scatter plots of the data for both reaches 1 and 2 are shown in figures 4.1 and 4.2.

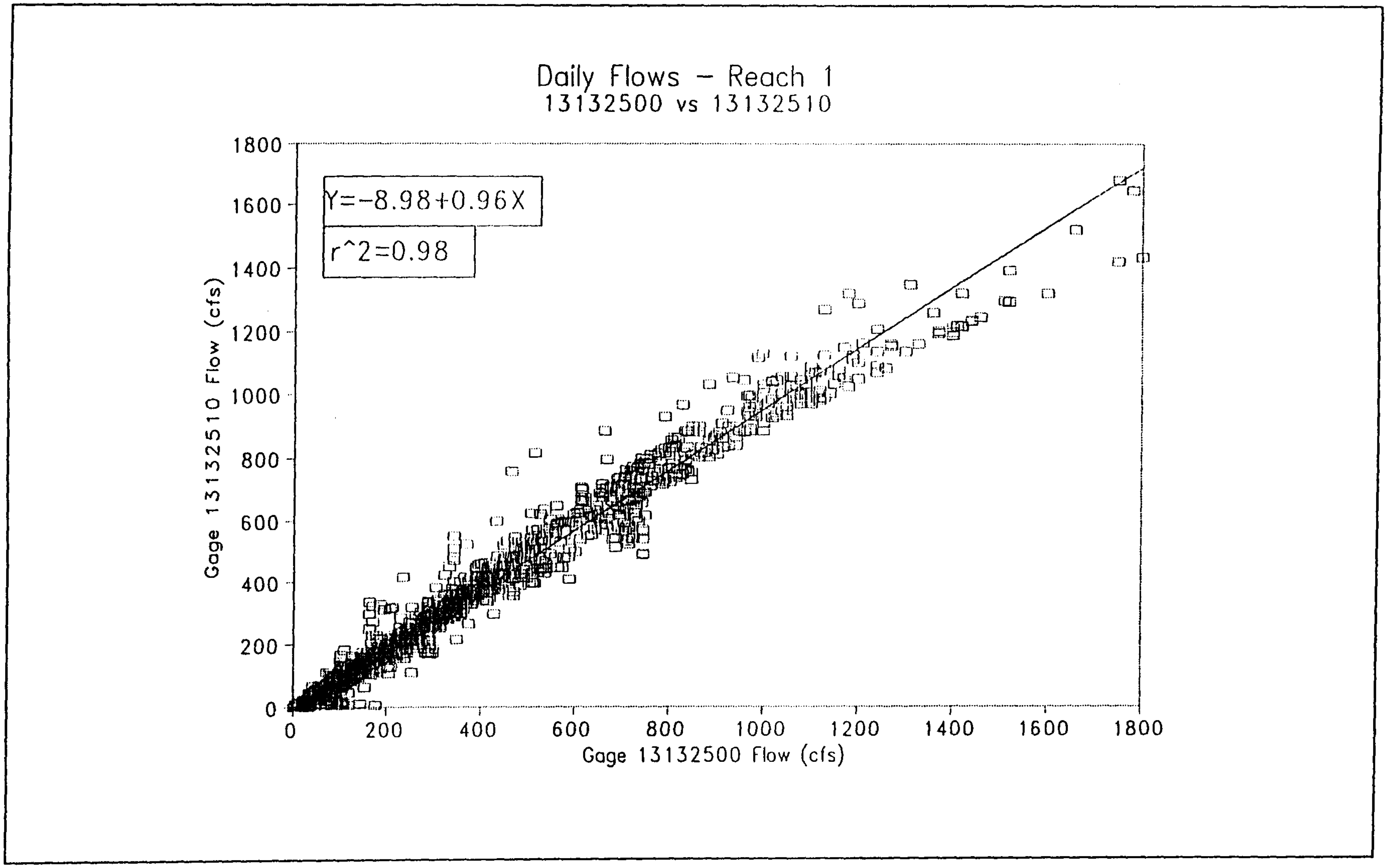

Figure 4.1 - Scatter Plot for Daily Flow Readings, Reach 1

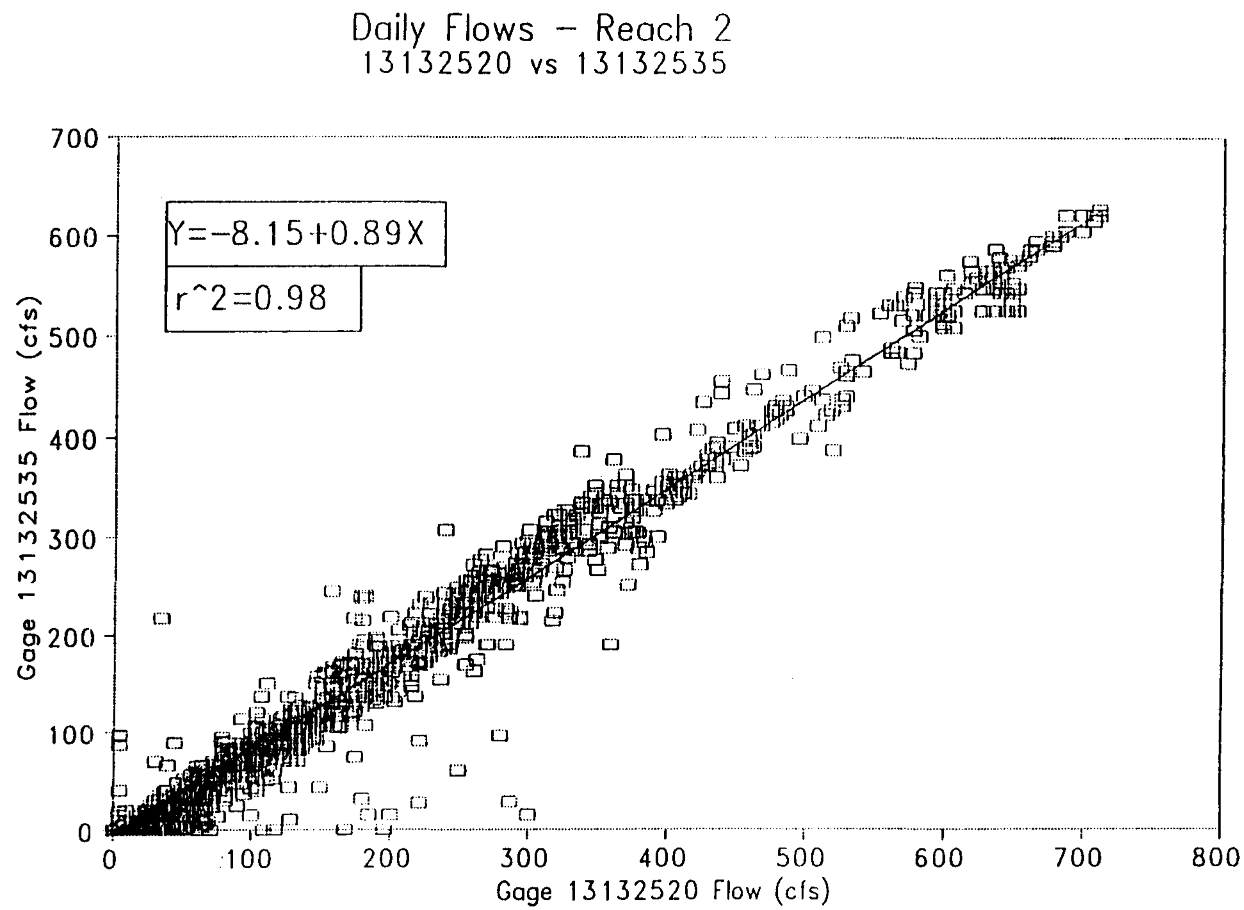

Figure 4.2 - Scatter Plot for Daily Flow Readings, Reach 2

After evaluating the results of the regression, and examining the application of the equations to estimate seepage, it was concluded that there were at least two problems with the selected approach. The use of all data values in the regression analysis, including those days when downstream flows exceeded upstream flows, introduced two separate populations of the independent variable: one in which there was no local inflow into the reach, and one which included local inflow. Since the objective eventually was to estimate seepage for days with Q_2 greater than Q_1 by predicting Q_2 without local inflow, it was decided to eliminate all daily data with Q_2 greater than Q_1 from subsequent regression analyses.

The second problem with the completed analyses was disclosed after examining numerous instances where the upstream flow was fairly constant, but the downstream flow fluctuated significantly. With the proposed approach, a constant value of Q_1 would always yield the same value of seepage, since Q_2 is based only on the value of Q_1. Therefore, attempting to correlate only downstream flow with upstream flow provided useful insight for the study, but not necessarily useful results. It should be noted here that with this much data and river conditions such as these, with no significant inflow from tributaries or groundwater, a regression of this sort performed on most river reaches should produce similar results because the flow would always decrease downstream due to seepage.

Since the first approach was an indirect attempt to develop a seepage equation, it was decided, after further study, that a more direct approach was necessary. Seepage is obviously a function of flow in the river reach, but not necessarily only upstream flow. It was concluded that a more reasonable choice might be the average flow in the reach. Larger average flows mean increased depths throughout the reach, and increased depth means more wetted area, so seepage would increase with flow. When small local inflows create increased flows downstream (although still less than upstream flow), seepage would be greater than if overland flow was zero because the average flow in the reach would be greater. Therefore, a new model of the seepage process was adopted:

$$S = f(\bar{Q})$$

where $\bar{Q}$ represents the average daily flow in the reach, calculated by averaging each day's flow at the upstream and downstream gages (excluding those days when downstream exceeded upstream), and S represents the difference between the upstream and downstream flows. Using this model, a linear regression of S on $\bar{Q}$ results in the following equation:

$$S = \beta_0 + \beta_1 * \bar{Q}$$

where:

$\bar{Q}$= average flow in the reach, cfs
S = seepage in the reach, cfs
β_0, β_1 = regression coefficients, unitless

The drawback to this approach is that both the dependent and independent variables (S, $\bar{Q}$) are a function of the two other variables (Q_1, Q_2), which are in turn strongly correlated. This results in a regression analysis that does not meet all statistical regression criteria, and may introduce spurious correlation. However, the model was nevertheless believed to be a valid representation of the seepage process, defined by the inflow-outflow method. Since the ultimate use of the equation was to estimate seepage for less than 14% of all the daily values, the overall validity of the total seepage data set should not be significantly impacted by errors from this approach.

Results: Reach 1

Using the previous regression equation applied to the data for reach 1 resulted in the following regression coefficients:

$$r = 0.63$$
$$r^2 = 0.40$$
$$\beta_0 = 10.201$$
$$\beta_1 = 0.080$$

A scatter plot of the data, S versus $\bar{Q}$, is presented in figure 4.3, indicating that although there is some observable linearity, it is significantly less than the observed relationship between Q_1 and Q_2 (figure 4.1). Out of the total of 4,089 plotted points in figure 4.3, there are a few dozen obviously apparent outliers, which deserve a brief discussion.

The Big Lost River is aptly named because flow sinks and returns quite often as it approaches Arco. Below Arco the groundwater reservoir drops to much greater depths, depths to which it can no longer feed the river. Because of this, and

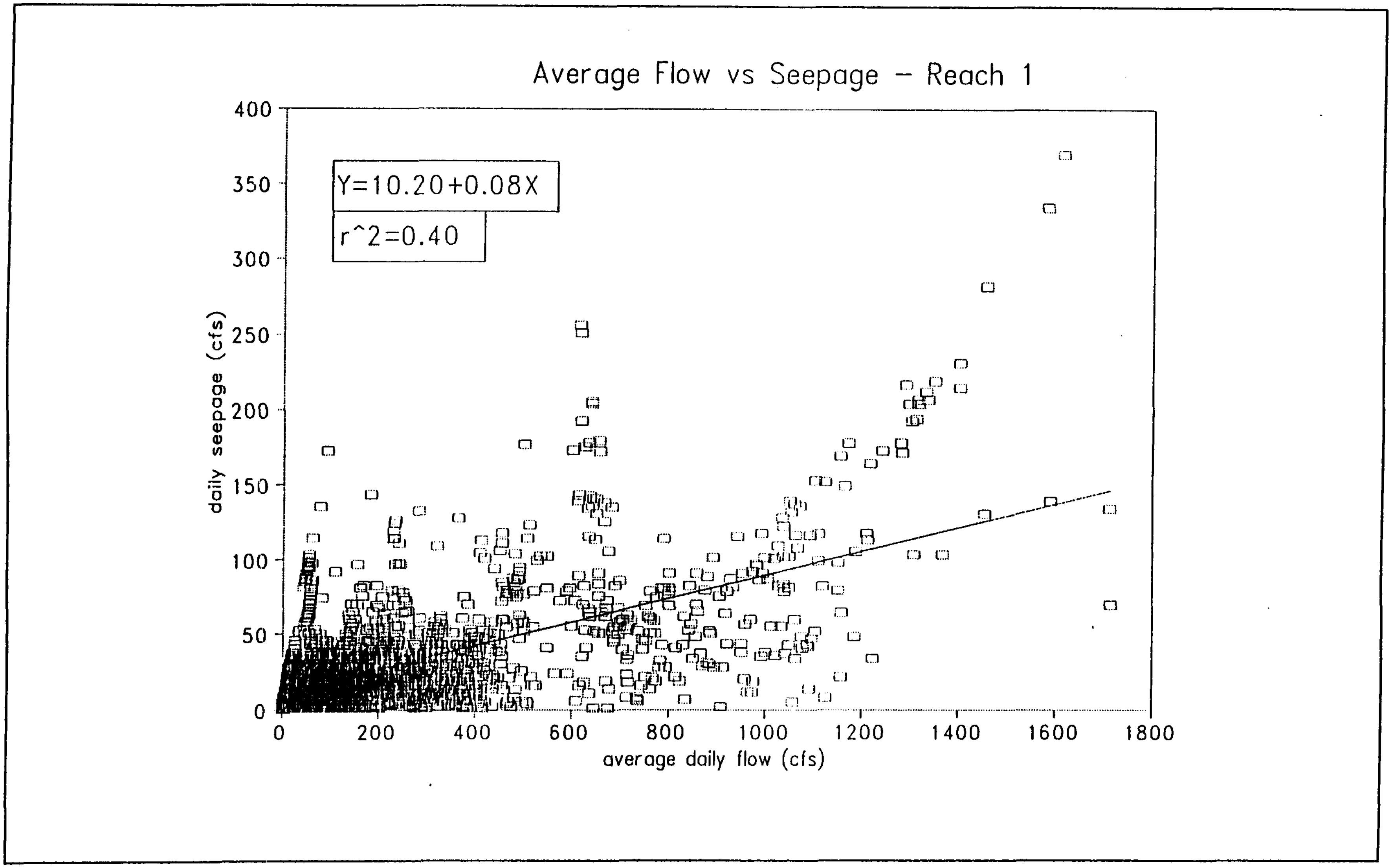

Figure 4.3 - Scatter Plot of Average Flow vs Seepage, Reach 1

depending on the amount of flow in the river, there are times when all the water passing Arco sinks before it reaches the INEL diversion dam. As a result of this situation, in the plot of average daily flow versus daily seepage, it can be noticed that seepage at times exceeds average daily flow. The explanation of this phenomenon is obvious. If 400 cfs passes Arco but completely sinks completely soon after, average daily flow in the reach is 200 cfs but seepage is 400 cfs. This is the cause of most of the extreme outliers seen in figure 4.3. To perhaps resolve this problem, another approach considered for the study was to perform the same analysis on only those days when flow was registered at both upstream and downstream gages. However, it was decided that seepage is seepage, whether all the flow or some of the flow seeps into the ground, and it is debatable whether this other approach would have given a more linear plot, or a more representative model of the system.

It must also be kept in mind that the data in figure 4.3, and the associated regression equation, do not include those days when downstream flow exceeded upstream flow. The equation was applied, however, to those days to calculate the daily seepage. This resulted in a complete data set of seepage values for reach 1, enabling the seepage-groundwater correlation analyses to then be performed (described in Chapter V). While this approach cannot be considered entirely accurate, the equation generated was only applied to 13.8% of the days in reach 1.

Results: Reach 2

For reach 2, seepage versus average daily flow is plotted in figure 4.4. As with reach 1, the linearity in the plot is obvious, especially if the few dozen outlier points are eliminated (4,212 points are plotted here). The cause of these outliers has been previously explained. Regression coefficients for this reach were:

$$r = 0.62$$
$$r^2 = 0.38$$
$$\beta_0 = 11.310$$
$$\beta_1 = .106$$

using the same form of regression equation previously discussed. With this equation applied to the 11.2% of the days in which downstream flow exceeded upstream flow, a complete set of daily seepage values was again obtained.

Results: Reach 3

No analysis was appropriate for this reach, since all the water seeped into the ground between Lincoln boulevard and the playas, and no streamflow gage data were available for the playas prior to 1985. This made it impossible to perform an analysis similar to reaches one and two. Therefore, for the seepage-groundwater analyses, seepage was equal to the flow recorded at Lincoln boulevard. This was not the most accurate method to estimate seepage, since at times in the past large volumes of water have ponded in the playas, making the situation similar to that of the spreading basins, but it was the best that could be offered.

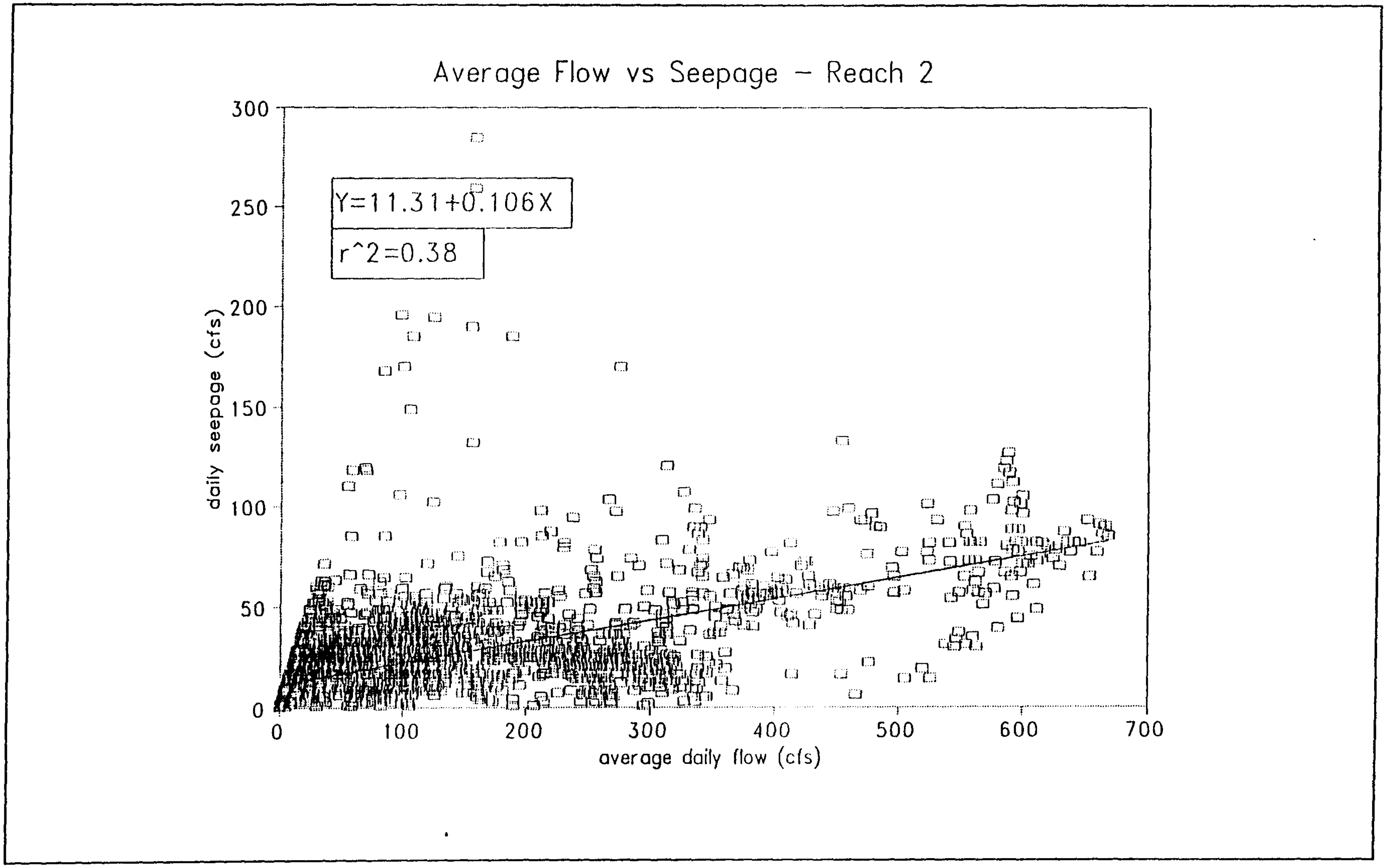

Figure 4.4 - Scatter Plot of Average Flow vs Seepage, Reach 2

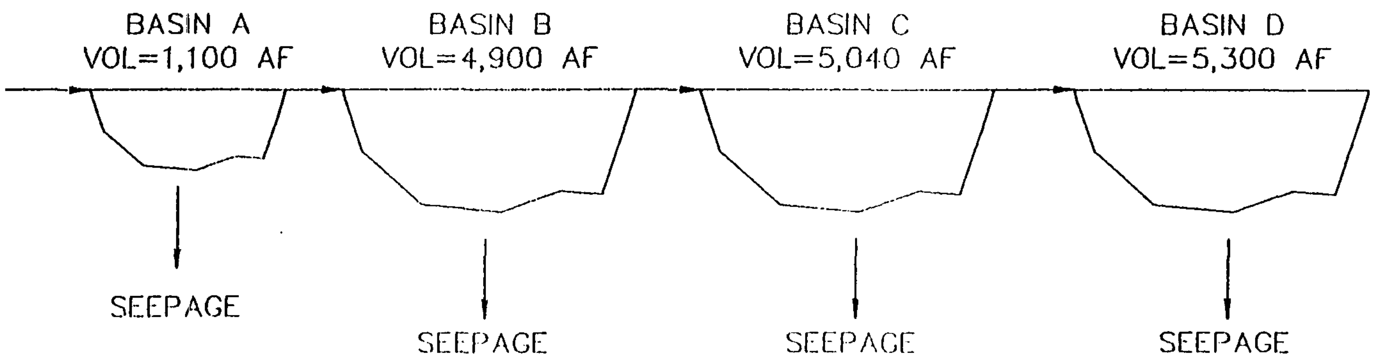

Figure 4.5 - Schematic Section of Spreading Areas and the respective volumes

Seepage Estimation for the Spreading Areas

A schematic of the spreading areas, including the storage volumes estimated for each basin, is provided in figure 4.5. The seepage beneath these spreading areas is important to an overall understanding of the groundwater recharge at INEL. Because large volumes of water may be diverted from the Big Lost River and permitted to enter the ground in a limited geographic area, the potential to affect critical sites such as the RWMC (in close proximity to the spreading areas) is significant. During periods of high flows on the river, much of the seepage within the INEL boundaries may take place in these basins, and correlation studies between seepage and groundwater should reflect this. Therefore, seepage equations for the spreading areas are necessary if these seepage quantities are to be estimated.

Unfortunately, determining seepage in the spreading areas is a more difficult task compared to that for the river reaches. A simple inflow-outflow approach could not be applied to this area since water ponds as storage volumes, rather than flowing continuously through. Instead, daily flows were routed through each basin, seepage equations were used to calculate daily seepage per basin, and total daily seepage was then estimated as the sum from all the basins. To accomplish this procedure of flow routing throughout the system of spreading basins, several prior studies were carefully examined to develop routing and seepage parameters.

P.H. Carrigan (1972) developed a computer program to route flows down the Big Lost River to the diversion dam, through the spreading areas, and also on down to the playas. His report studied the probability of exceeding the capacity of the spreading area flood-control system. To properly route flows through the spreading area he developed seepage equations for each of the four basins. These equations were extracted from his computer code and an attempt was made to apply them to this study. In his equations, seepage was a function of both a unit seepage-evaporation loss rate determined by Carrigan and depth in the basins. For some unknown reason his equations produced questionable results when applied to the data in this study. After further review of Carrigan's report, it was noticed that once flow into a basin stopped, water in each basin appeared to seep instantly, signifying no lag time for seepage. This indicated that possibly these equations were developed primarily for the purpose of his flood-evaluation study, and in this application seepage was not important once flow into a basin ceased.

Since Carrigan's equations did not seem to work, another INEL report, a study of the 1983-1984 flood threat by J.D. McKinney (1985), was examined. In it McKinney stated that, by observation, it took one month for the spreading areas to drain once they were completely filled (McKinney, 1985, p. 6). Using this estimated time lag and the approximate volumes of the spreading areas from McKinney's report

(McKinney, 1985, p B-3), seepage rates in acre-feet per month were calculated by dividing the volume of each pond by one month (31 days). Rates were then converted to cubic feet per second (cfs) per day. The seepage rates calculated for each basin were applied with the flow routing, and a set of daily seepage values for the entire period of study was then determined. It should be noted that the one month time for the basins to empty was during the summer, and therefore could have been overestimated. Because of this possibility, a sensitivity analysis on the seepage time was performed by doubling it to two months. These results will be discussed later.

Results: Spreading Areas

To perform the routing and seepage analyses for the spreading areas, spreadsheet software, QUATTRO 4.0, was used to handle all of the data. Daily inflows were obtained from the gage located near the diversion dam, at the inlet to the spreading basin system. Water was routed into the first basin until it filled, and then sequentially routed into the remaining basins. For each basin, the daily storage changes were calculated by subtracting a seepage rate from the daily inflow rate, and these storage changes then were used to determine the new storage volume for that basin. This procedure was performed continuously for both periods of record (1969-76, 1983-87), with a daily seepage record tallied at the end for all the basins.

The seepage rates used in each basin were initially based on the 31-day emptying time observed by McKinney, and held constant regardless of the depth of water, or storage volume, in the basin. To test the sensitivity of the resulting total daily seepage amounts to this 31-day assumption, a second simulation was performed using a 62-day time for calculating the seepage rates for each basin. The results showed that the drain time was consistent with the time-based method used to calculate the seepage rates, and it increased to two months. This should have been expected, since drain time and seepage rates were dependent on one another. Despite the failure of this sensitivity analysis, seepage rates for the analysis were kept dependent on the one-month drain time. There is a lack of information on seepage rates from the spreading areas, and it was felt that, without any actual field measurements, the estimated one-month drain time was the best estimate available. The shortcomings associated with the seepage estimation technique used in this analysis will be discussed further in chapter VI.

Concluding Remarks

The accomplishments of the seepage estimation process included: determination of study reaches, which were a basis for the entire study; seepage equations for reaches 1 and 2 and the spreading areas (with limited applicability for those pertaining to reaches 1 and 2); and, most importantly, a daily seepage record for the entire period of study. As a final comparison for the seepage analysis, the average seepage rate in reaches 1 and 2 and the spreading areas for the entire study period are listed below:

Average seepage in reach 1 = 782.3 cfs
Average seepage in reach 2 = 574.7 cfs
Average seepage in spreading areas = 3688.5 cfs

The importance of these differences in seepage will be seen in the impact on the groundwater, and therefore the next analysis, seepage versus groundwater levels.

CHAPTER V

Seepage-Groundwater Correlation

OVERVIEW

Seepage or recharge from the Big Lost River does not directly affect all facilities on the INEL site because some are located too far from the river. However, areas near the river channel or spreading basins may experience temporary saturation of the underlying porous media, including localized groundwater mounding. As has already been discussed, radioactive and other wastes have been allowed to infiltrate into the Snake River Plain aquifer through percolation ponds at the TRA and other sites. Also, radioactive waste is buried at the RWMC. If any of these contaminants are migrating through the unsaturated zone and into the aquifer, any significant changes in groundwater levels or the saturated zones could alter the concentration, path, and timing of the transport process. For this reason, a clear understanding of the relationship between seepage and groundwater response is essential.

Therefore, the ultimate objective of this study was to examine the time series of both seepage and groundwater levels near the river, and to determine whether a consistent relationship exists between the two time series. It was hypothesized that if groundwater levels directly responded to changes in seepage rates, there would be a time lag evident in this response. Such a lag, if it could be identified and quantified, would provide a clue to the migration time of surface water through the unsaturated zone and into the aquifer.

APPROACH

To initiate this part of the study, wells and their locations on the site first had to be identified and researched to determine which to use in examining the seepage-groundwater relationship. Maps of well locations on the INEL site were obtained, and from a review of these maps a number of potential candidate wells were selected. Although the maps indicated both private and USGS wells, it was decided, for several reasons, to work only with USGS well data. The data from most of the private wells are proprietary, and therefore may have been difficult to obtain without a lengthy permission process. On the other hand, the USGS well data are in the public domain, the wells are in excellent locations near the study reaches, and their descriptions and behavior have been well-documented in a variety of USGS site-hydrology reports.

This prior hydrologic research performed by the USGS was carefully reviewed. Many reports included brief sections that discussed various USGS wells and their response to flows in the river, with an identification of those wells that appeared to respond most dramatically to changes in flow. Bennett (1990) included in his report a discussion about USGS wells and compared hydrographs with flow in the river. Earlier USGS hydrology reports did the same. An inventory of wells used in these reports was compiled and compared against the initial, more comprehensive list of wells. Many of the wells initially chosen were geographically grouped together and at most only one per group was necessary for the study. Wells located in close proximity to the river and spreading areas were considered most important because of their prime location.

From all of these evaluations, a list of the most responsive wells was assembled, and it was then finally narrowed down to the choices of USGS wells 8, 9 and 18. USGS 8 is located just outside the western boundary of the site and near the Arco river gage. It provided a good location for correlation with seepage in reach one. USGS 9 is located near the spreading areas and provided information for groundwater beneath the RWMC. USGS 18 is located in the northern portion of the site and was compared against seepage from reach three. All water level data available from these wells were entered into a QUATTRO spreadsheet for use in the correlation analysis.

Flow and Water Levels

To begin the correlation analysis, a broad approach was initially taken by simply comparing flows in the river with groundwater levels in the selected wells. This exercise was performed to determine if any correlation appeared to exist with these wells before too much time was spent in detailed comparisons of seepage and groundwater levels.

A consistent time basis was necessary to compare well hydrographs with streamflow hydrographs. All the river gages were monitored on a continuous basis, but the same was not true for the wells. Water levels were checked sporadically, most often once or twice a month. Therefore, average monthly water levels were compared to average monthly flows. To obtain averages for the wells, all measurements taken each month were averaged to obtain monthly water levels. Average monthly flows were simply the means of the daily flows for the entire month.

In the analysis, well hydrographs were first graphically, then mathematically, compared against flow hydrographs in different reaches, depending on the well locations with respect to reaches. USGS 8 records were compared against both the gages at Arco (gage 13132500) and above the diversion dam (synthesized gage 13132510); USGS 9 was compared against the flows above the diversion dam and at Lincoln boulevard (gage 13132535), and USGS 18 was compared against the gage at Lincoln boulevard. These graphs are presented in figures 5.1 through 5.10. As is illustrated with these graphs, the most notable fact is that a time lag appears to exist, with groundwater levels responding consistently after significant flow events.

To study this apparent time lag phenomenon further, the hydrographs were first smoothed to be more readable and easier to compare. One statistical procedure to do this is termed "moving average", and is often used to reduce short-term variability of data to more easily detect underlying longer-term trends. A moving average is established by first

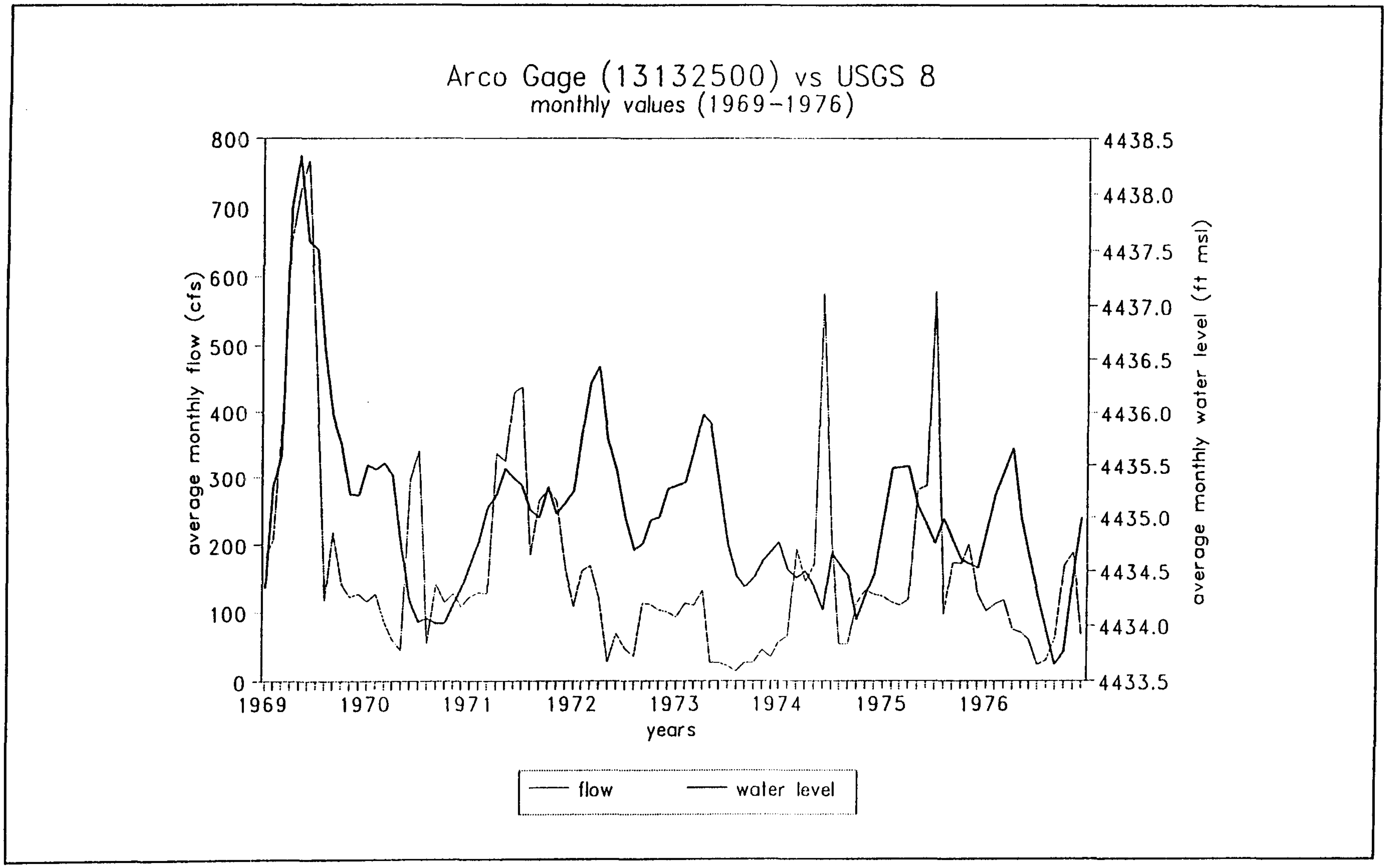

Figure 5.1 - Hydrograph Comparisons, Arco Gage vs USGS 8 (1969 to 1976)

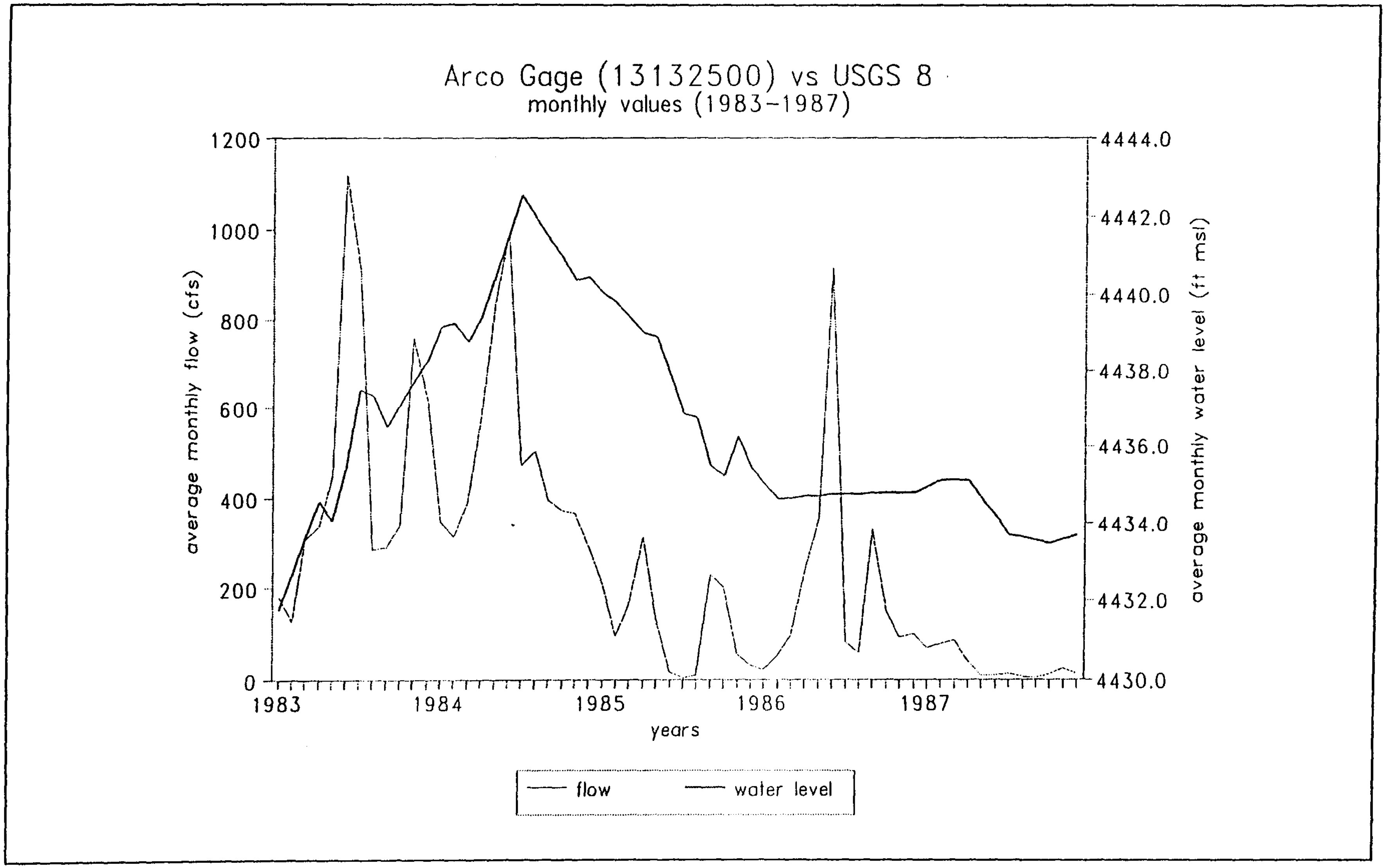

Figure 5.2 - Hydrograph Comparison, Arco Gage vs USGS 8 (1983 to 1987)

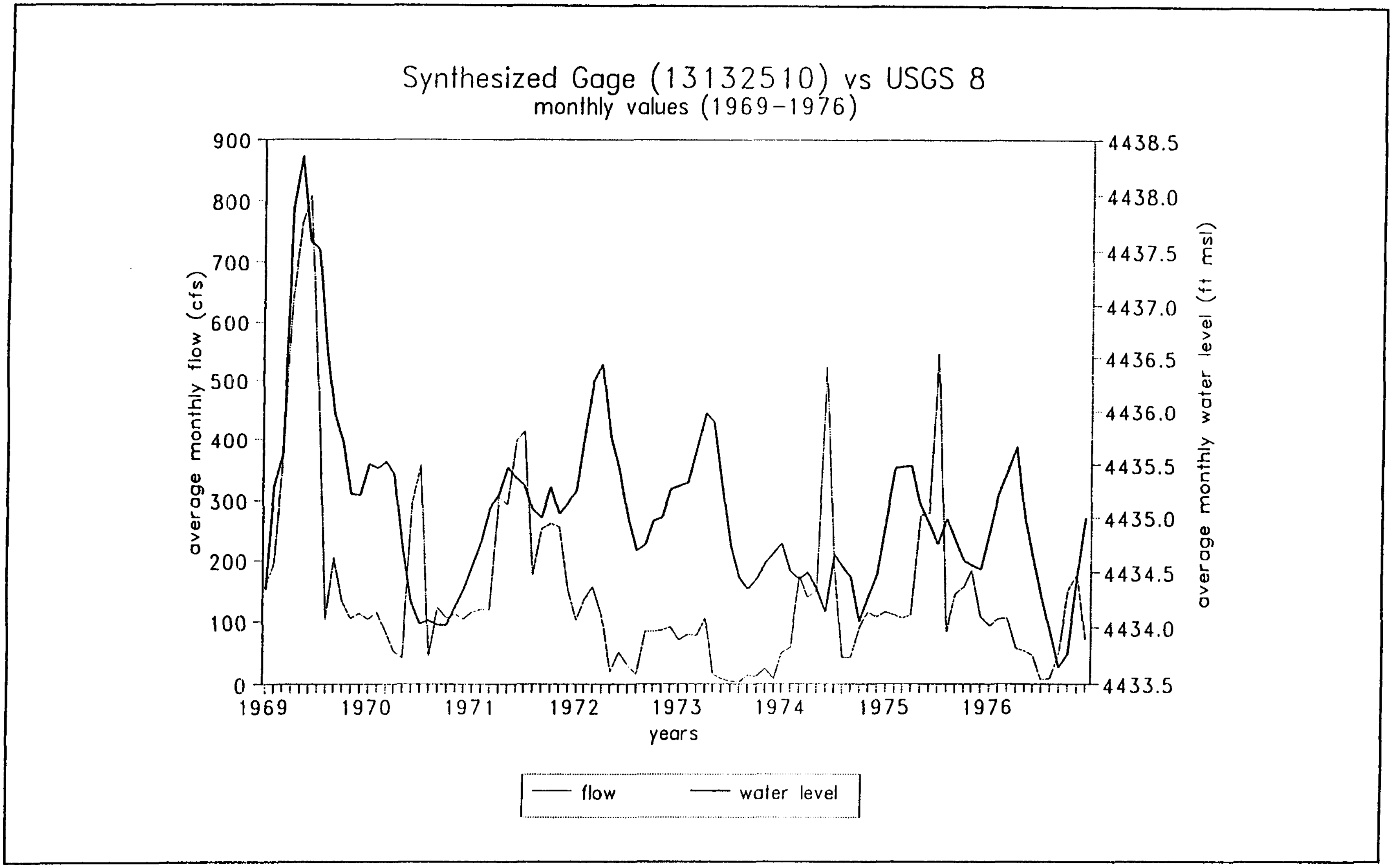

Figure 5.3 - Hydrograph Comparisons, Synthesized Gage vs USGS 8 (1969 to 1976)

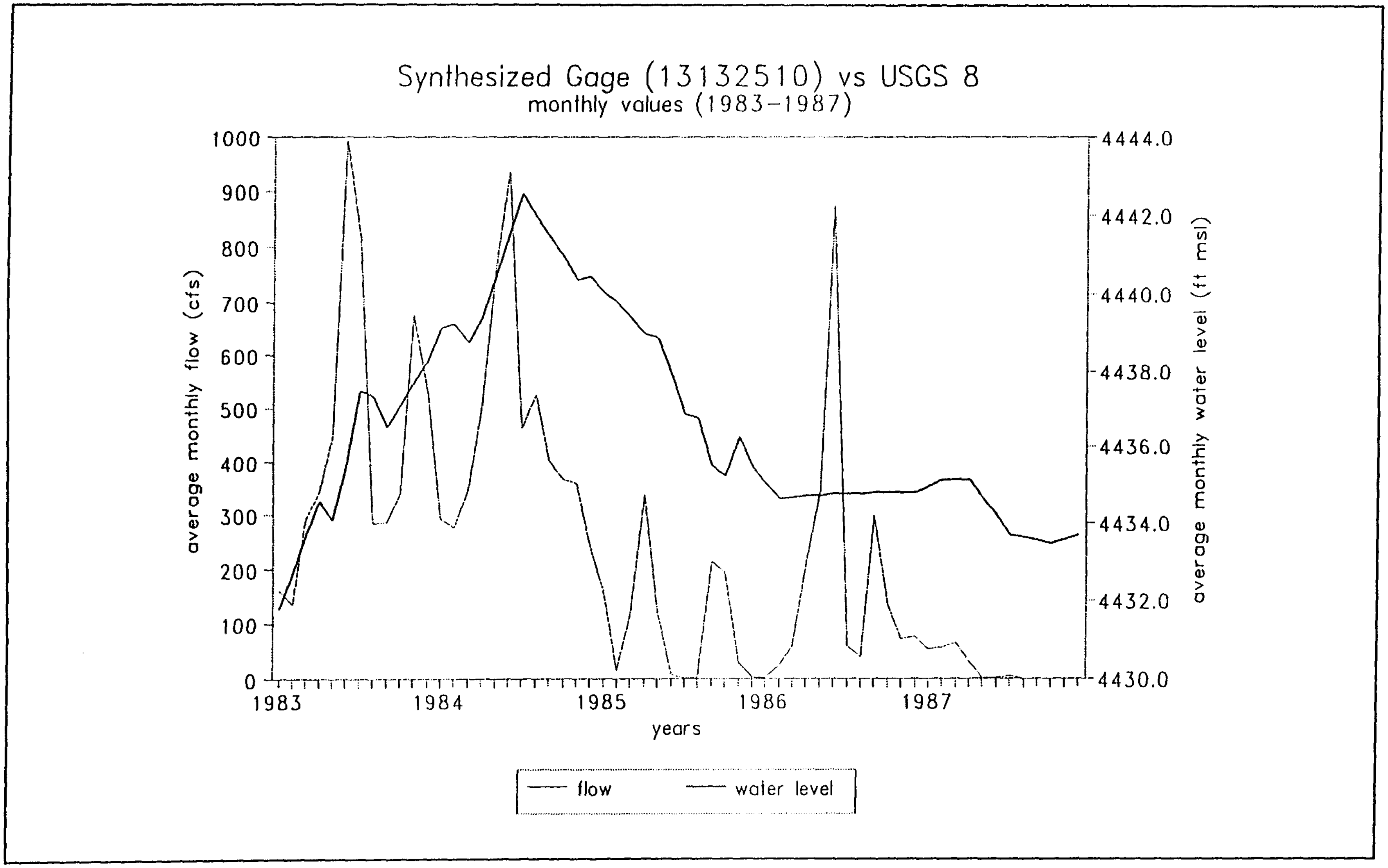

Figure 5.4 - Hydrograph Comparisons, Synthesized Gage vs USGS 8 (1983 to 1987)

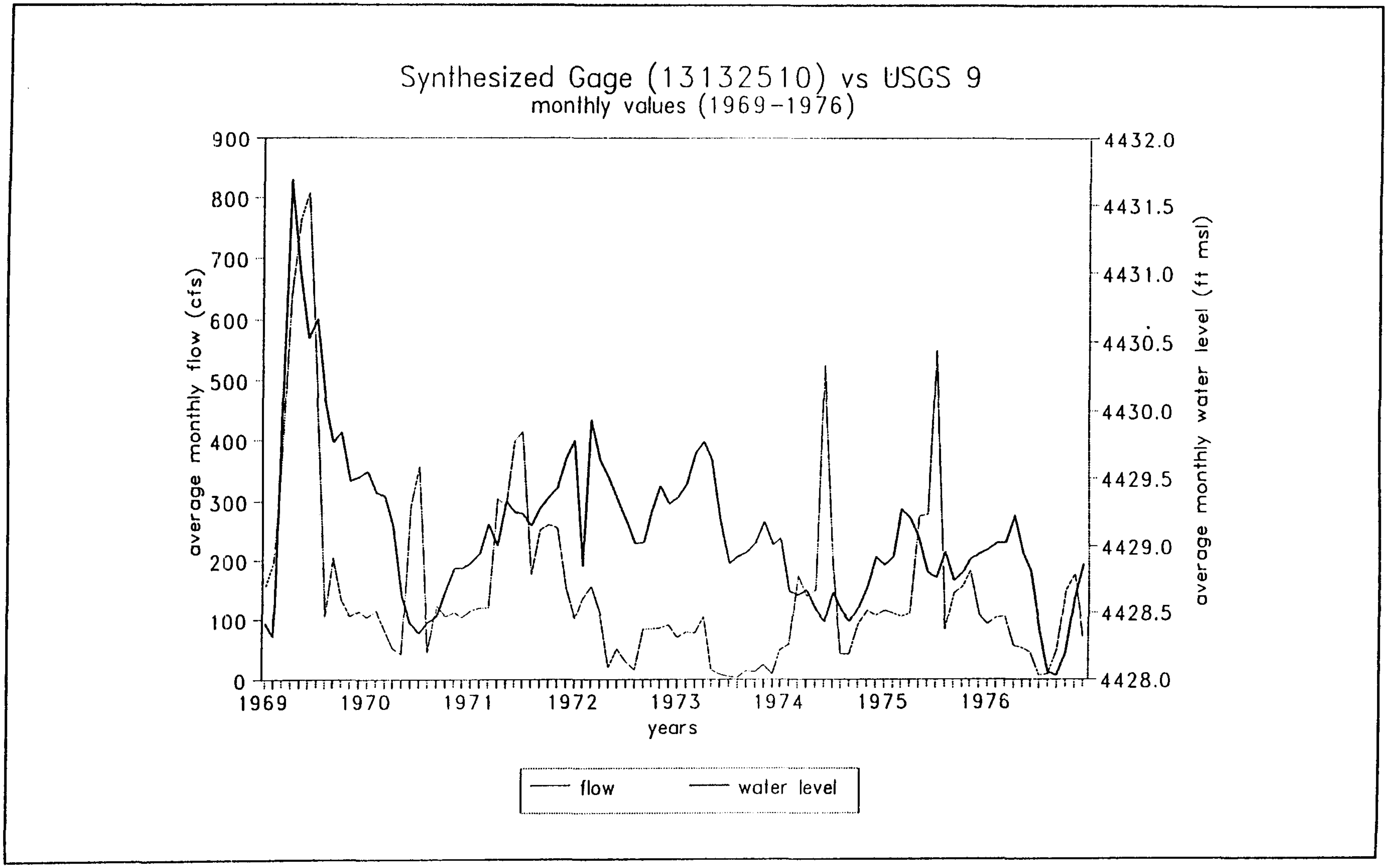

Figure 5.5 - Hydrograph Comparisons, Synthesized Gage vs USGS 9 (1969 to 1976)

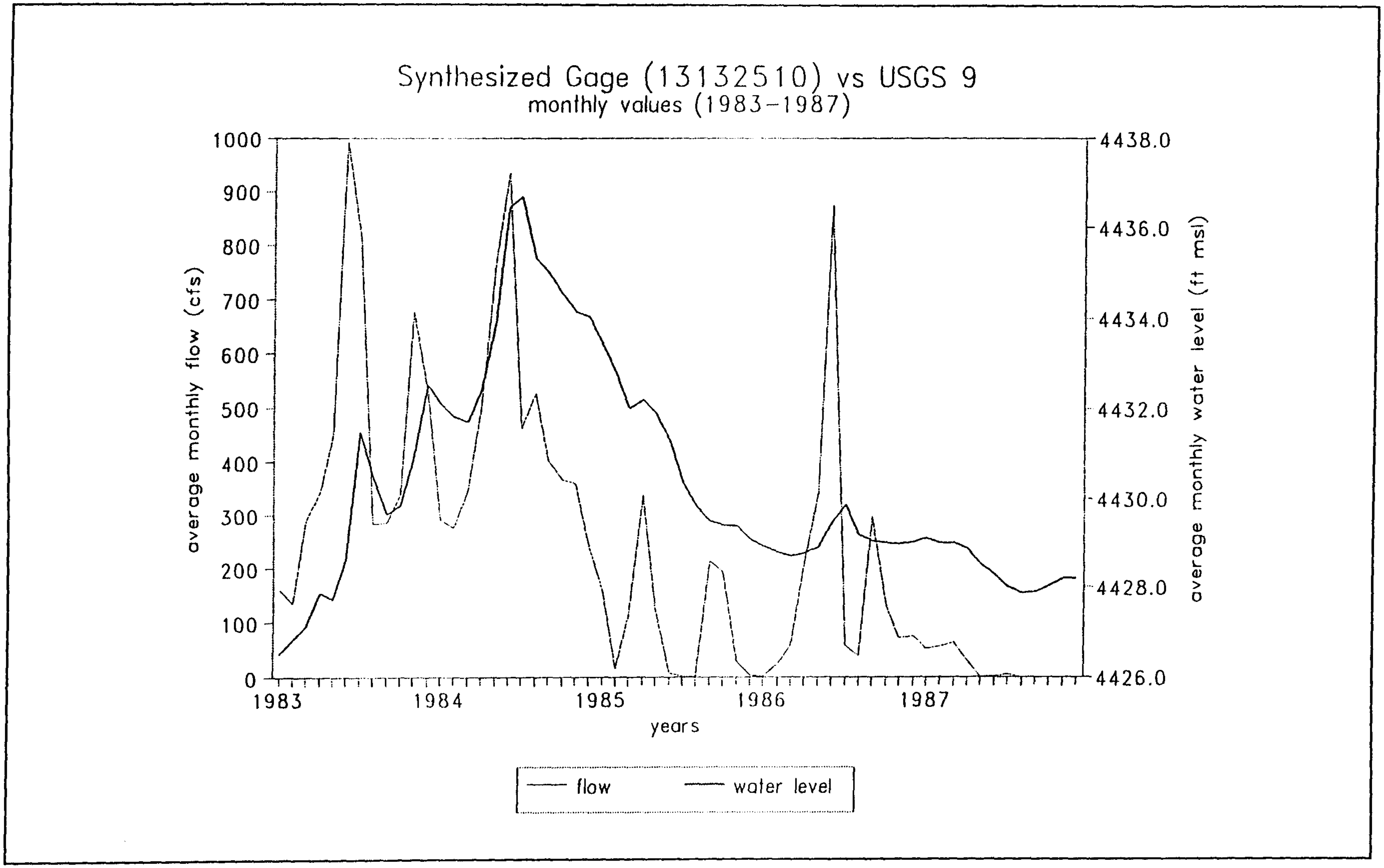

Figure 5.6 - Hydrograph Comparisons, Synthesized Gage vs USGS 9 (1983 to 1987)

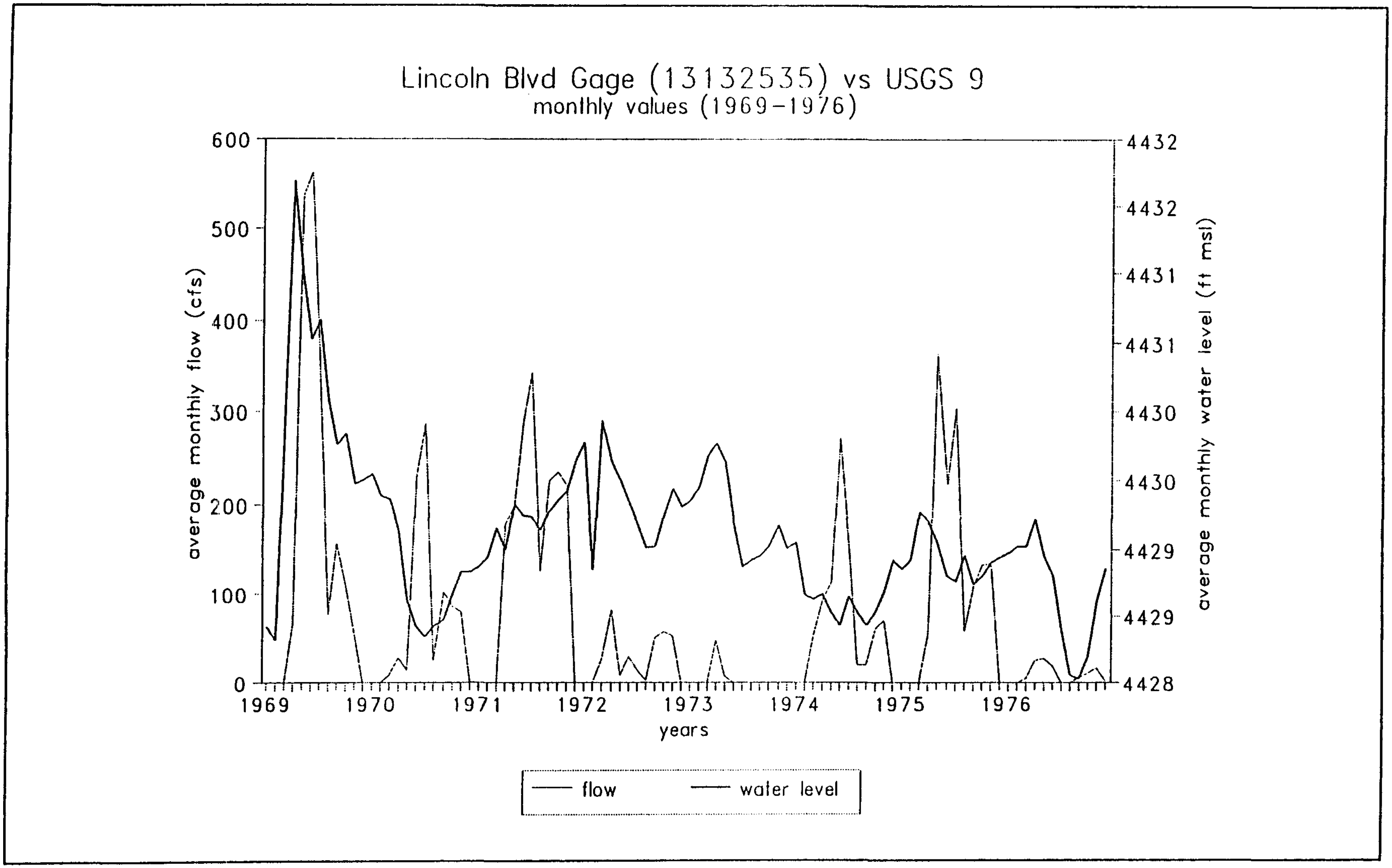

Figure 5.7 - Hydrograph Comparisons, Lincoln Boulevard Gage vs USGS 9 (1969 to 1976)

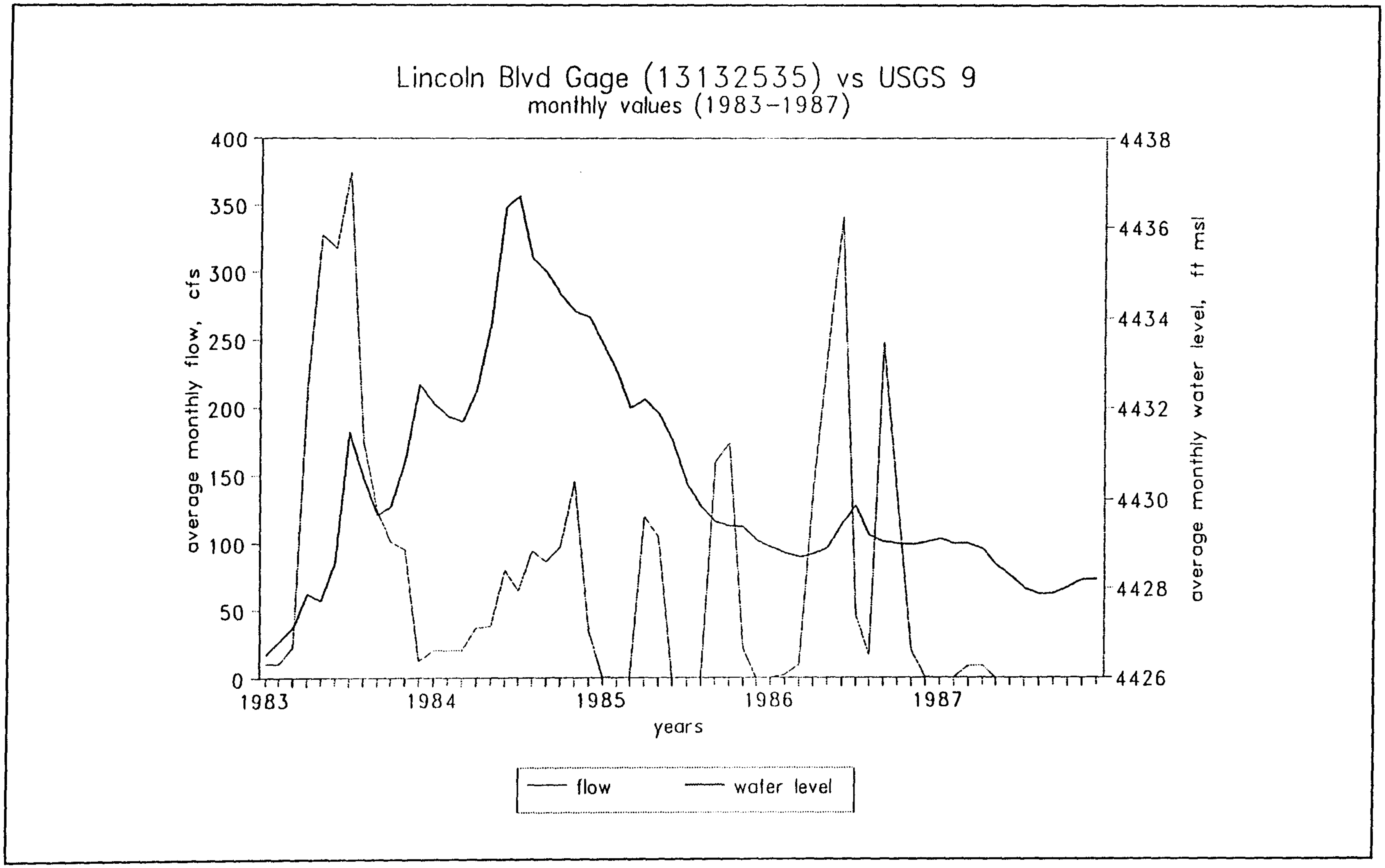

Figure 5.8 - Hydrograph Comparisons, Lincoln Boulevard Gage vs USGS 9 (1983 to 1987)

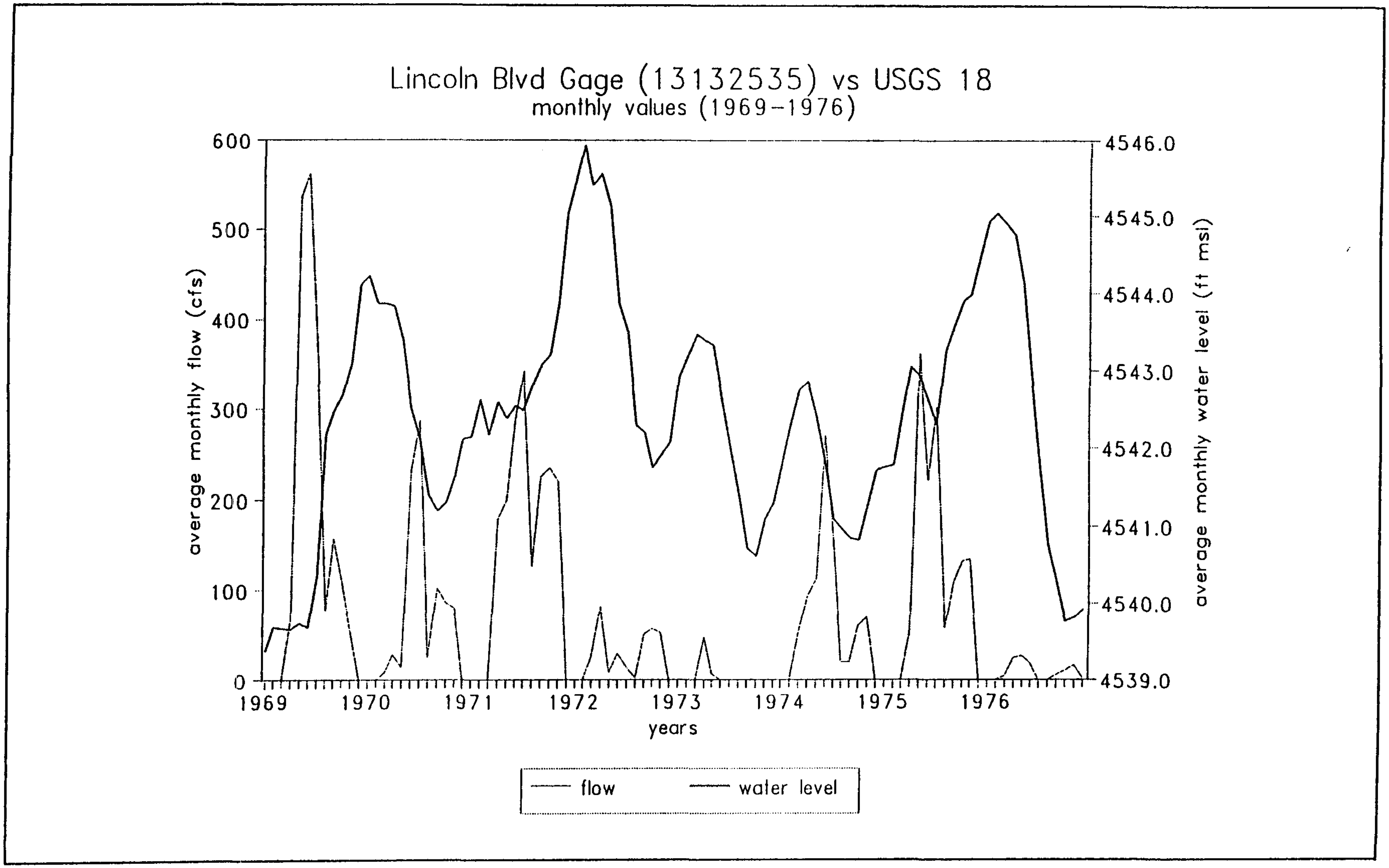

Figure 5.9 - Hydrograph Comparisons, Lincoln Boulevard Gage vs USGS 18 (1969 to 1976)

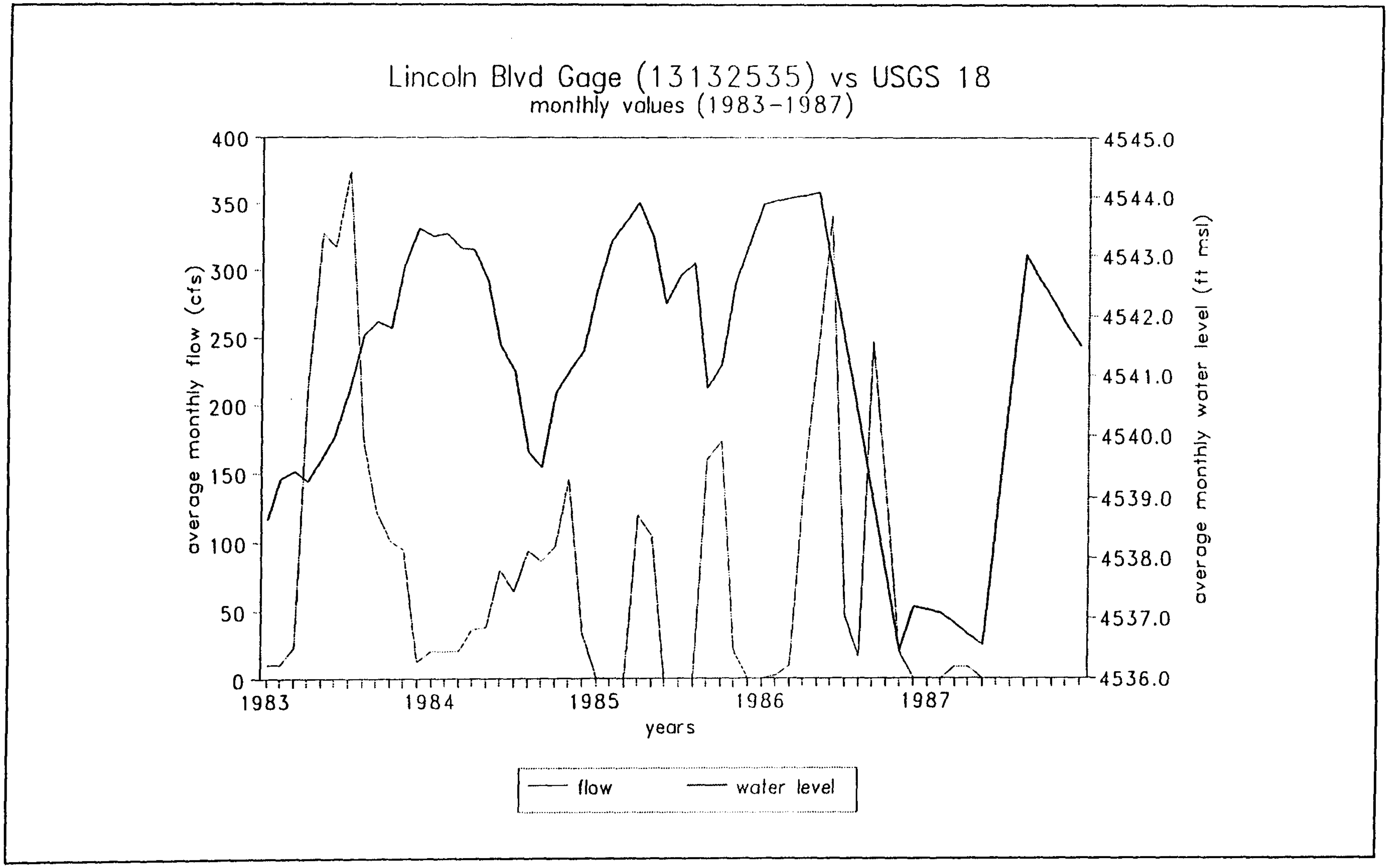

Figure 5.10 - Hydrograph Comparisons, Lincoln Boulevard Gage vs USGS 18 (1983 to 1987)

taking an average over a sequential set of numbers, X, of a total data set, Y, beginning with the first number, i. Then another average is taken over the same amount of numbers, X,but beginning at i+1. It ends when the moving average includes the last number of the total data set, Y. In this case the moving average was over a period of time (ie: months, years).

A number of different time period moving averages, starting at two months, were examined and it was eventually concluded that a moving average based on a 12-month period provided the best smoothed representation of the data sets. This was obtained by taking the average of water levels and flows from January to December, then February to January, March to February, etc., until the entire periods of record (1969-76 and 1983-87) had been averaged.

Obviously, this averaging process causes significant distortion to the time series by including future as well as prior values in the calculation of each point. However, since the two time series (flow and groundwater levels) are both averaged in the same way, these distortions do not adversely affect the comparisons that can be made between them. At this point in the study, the primary objective was simply to determine whether, visually, there appeared to be a consistent relationship between streamflow and groundwater levels. If so, a more detailed evaluation of the seepage-groundwater relationship is indicated.

After performing the necessary calculations, the 12-month moving averages of flow and groundwater levels were plotted, and can be seen in figures 5.11 through 5.20. An examination of these graphs demonstrates that there is an obvious time lag relationship between flows in the river and groundwater levels in the selected wells. By visual inspection of these figures, it appears that this time lag is between four and seven months. This first approach therefore confirms that the wells chosen were hydrologically connected to the river and that a time-lagged correlation did exist between flow rates and groundwater levels.

Seepage and Water Levels

The final step in the study was to compare total monthly seepage with average monthly groundwater levels, since it is the seepage, not flow, that impacts the groundwater in a cause-effect relationship. In these analyses, seepage from reach one was compared against water levels in USGS 8, seepage from the spreading areas and reach two were compared against USGS 9, seepage from reach three was compared against USGS 18, and the combined seepage from reach one and the spreading areas was compared against USGS 9. Again it was analyzed on a monthly basis, using total monthly seepage and average monthly water levels. Hydrographs were graphed together for comparison, with seepage and water levels on the y-axis and time on the x-axis.

REACH 1 AND USGS 8

Figures 5.21 and 5.22 show the plots of total monthly seepage and average monthly water levels against time, for the two separate time periods (1969-76 and 1983-87). It was noted that the graphs displayed considerable short-term variability, indicating that a moving average may be beneficial in comparing the time series. However, as with the previous analysis, there was some obvious similarity between graphs, with water levels lagging after seepage. A 12-month moving average was then applied to the data, and the resulting graphs are shown in figures 5.23 and 5.24. The time lag is more noticeable in these plots, with the average monthly water levels following consistently in time the total monthly seepage, increasing or decreasing in the same manner, but later in time. Peaks and valleys do not precisely match, but this was not unexpected since there are many other unexplained factors involved in the seepage-groundwater interaction, including time of year, amount of flow in the river, and position of the groundwater table. In the original hypothesis, time lag was assumed independent of these other factors, simply because insufficient data were available to adequately evaluate their effects. To determine if a significant time-lagged correlation existed between the monthly seepage and average monthly water levels, regression analyses were performed on the data. With groundwater levels as the dependent variable, monthly seepage values (the independent variable in the regression) were lagged by time

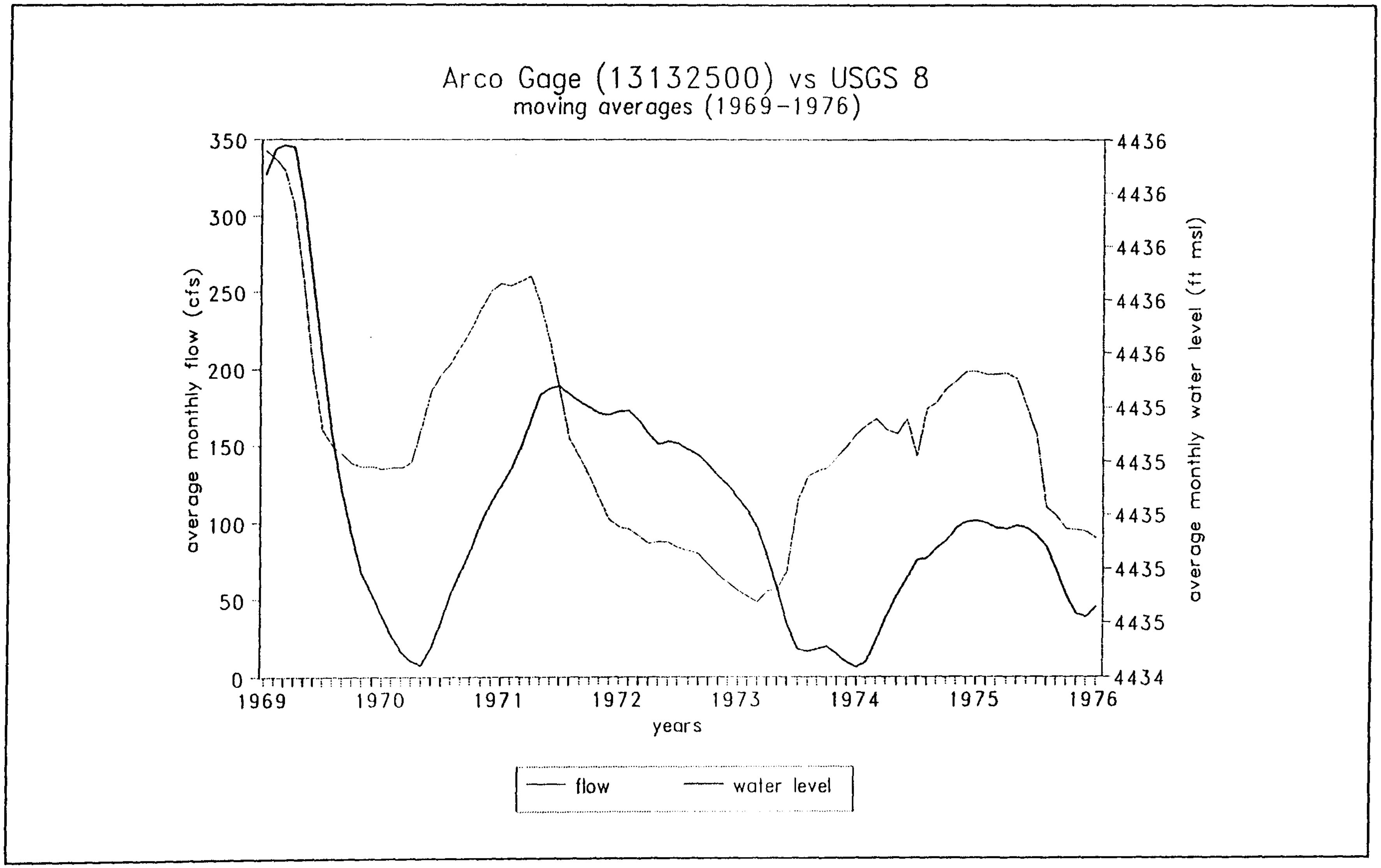

Figure 5.11 - Moving Average Hydrographs, Arco Gage vs USGS 8 (1969 to 1976)

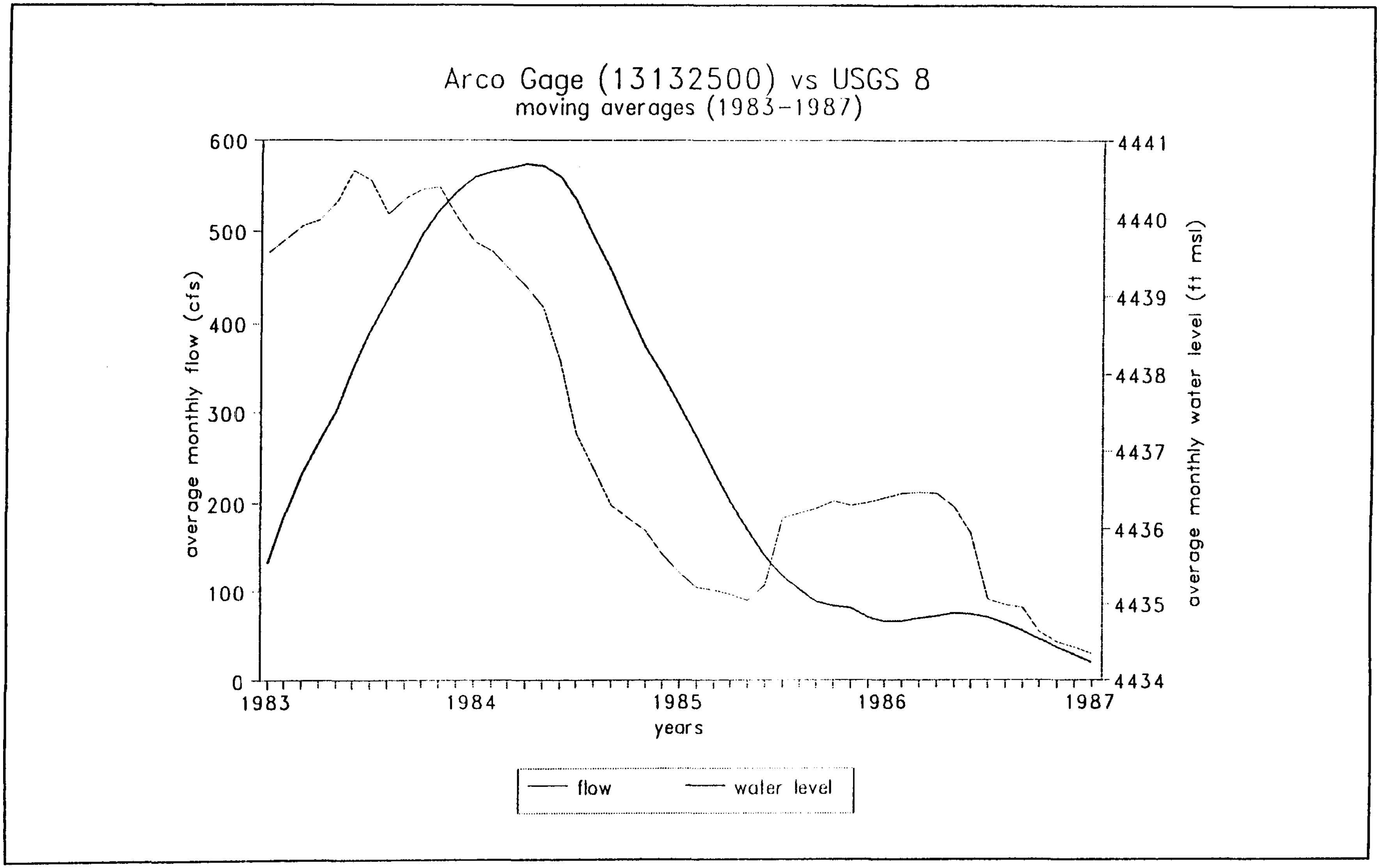

Figure 5.12 - Moving Average Hydrographs, Arco Gage vs USGS 8 (1983 to 1987)

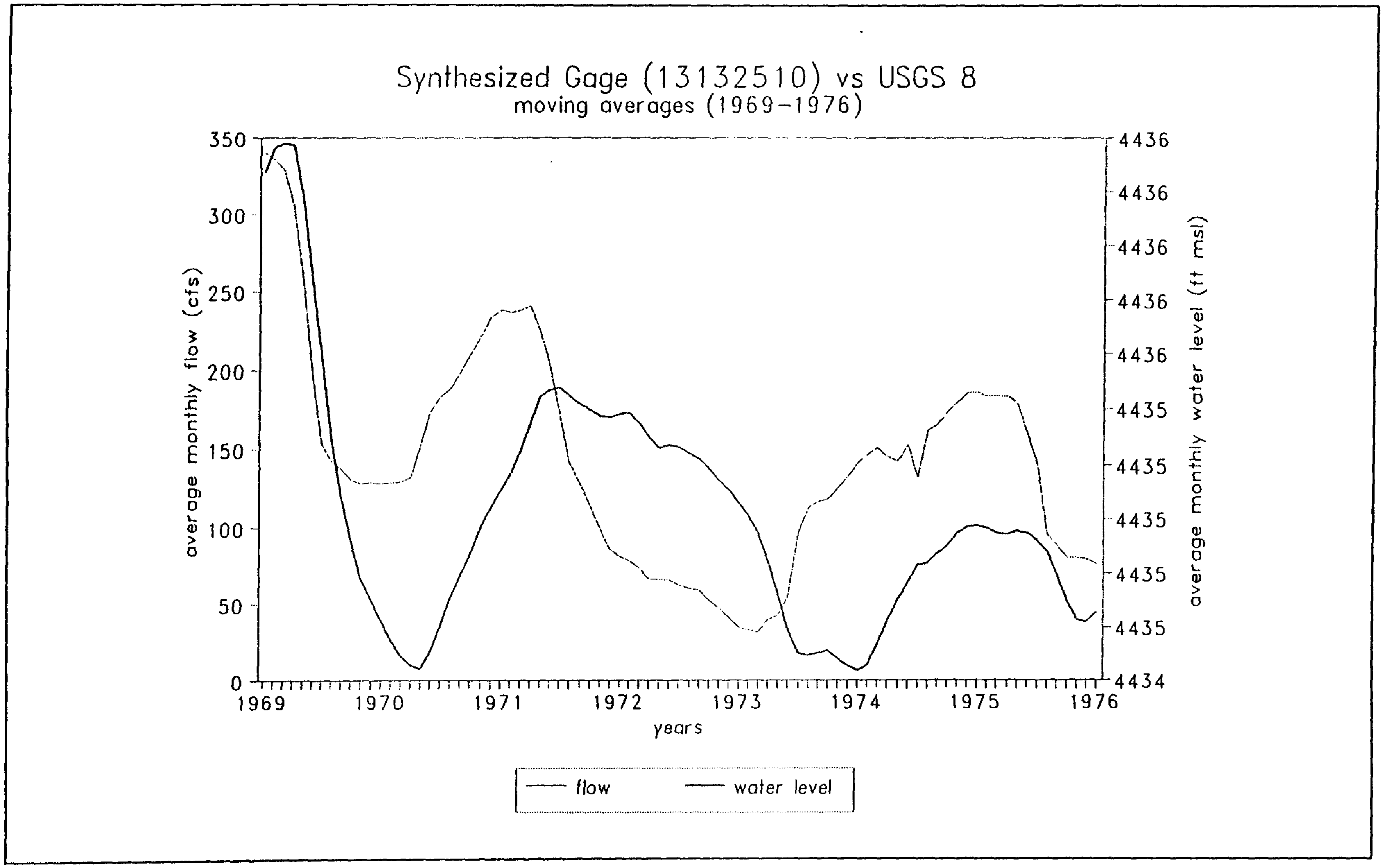

Figure 5.13 - Moving Average Hydrographs, Synthesized Gage vs USGS 8 (1969 to 1976)

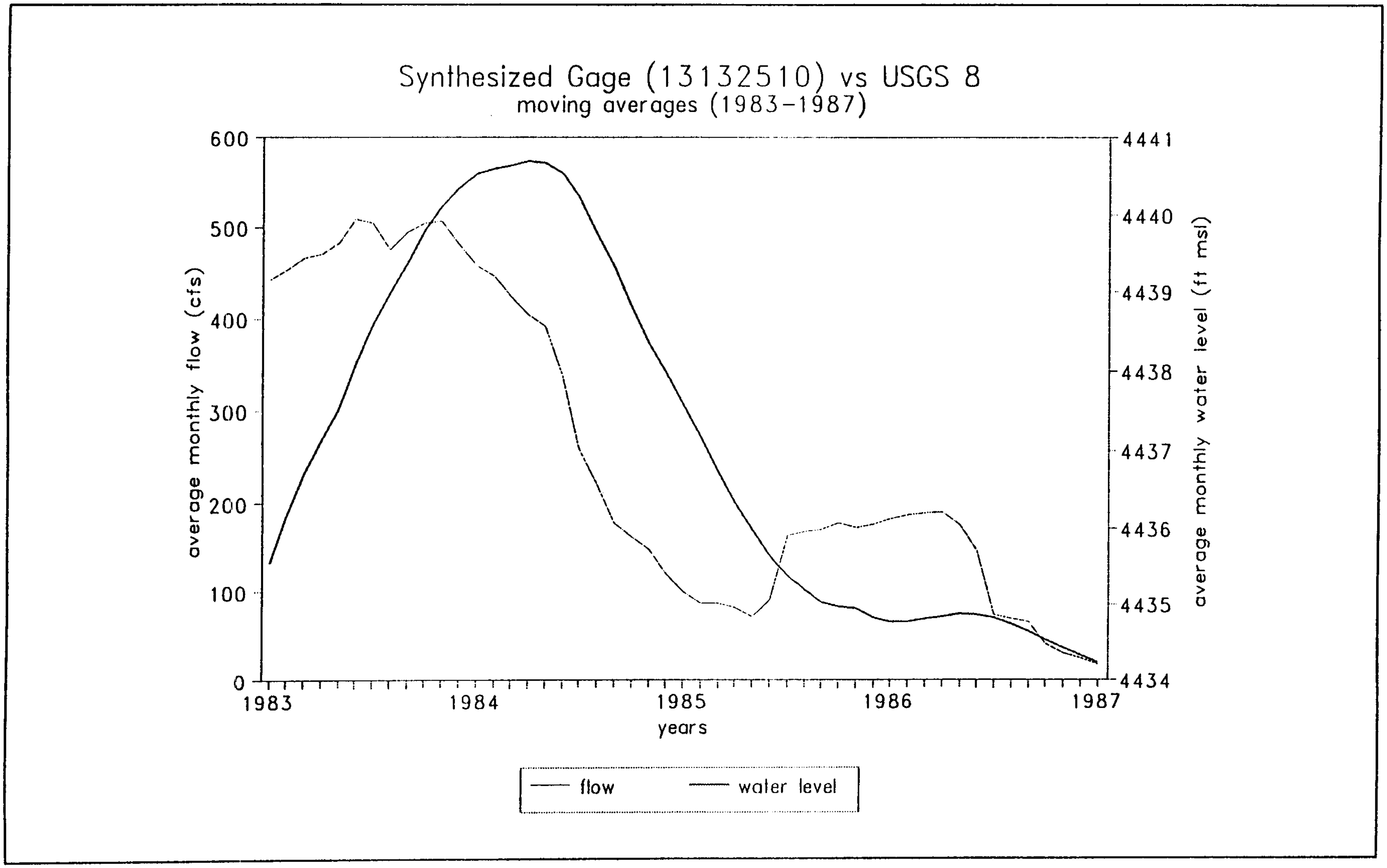

Figure 5.14 - Moving Average Hydrographs, Synthesized Gage vs USGS 8 (1983 to 1987)

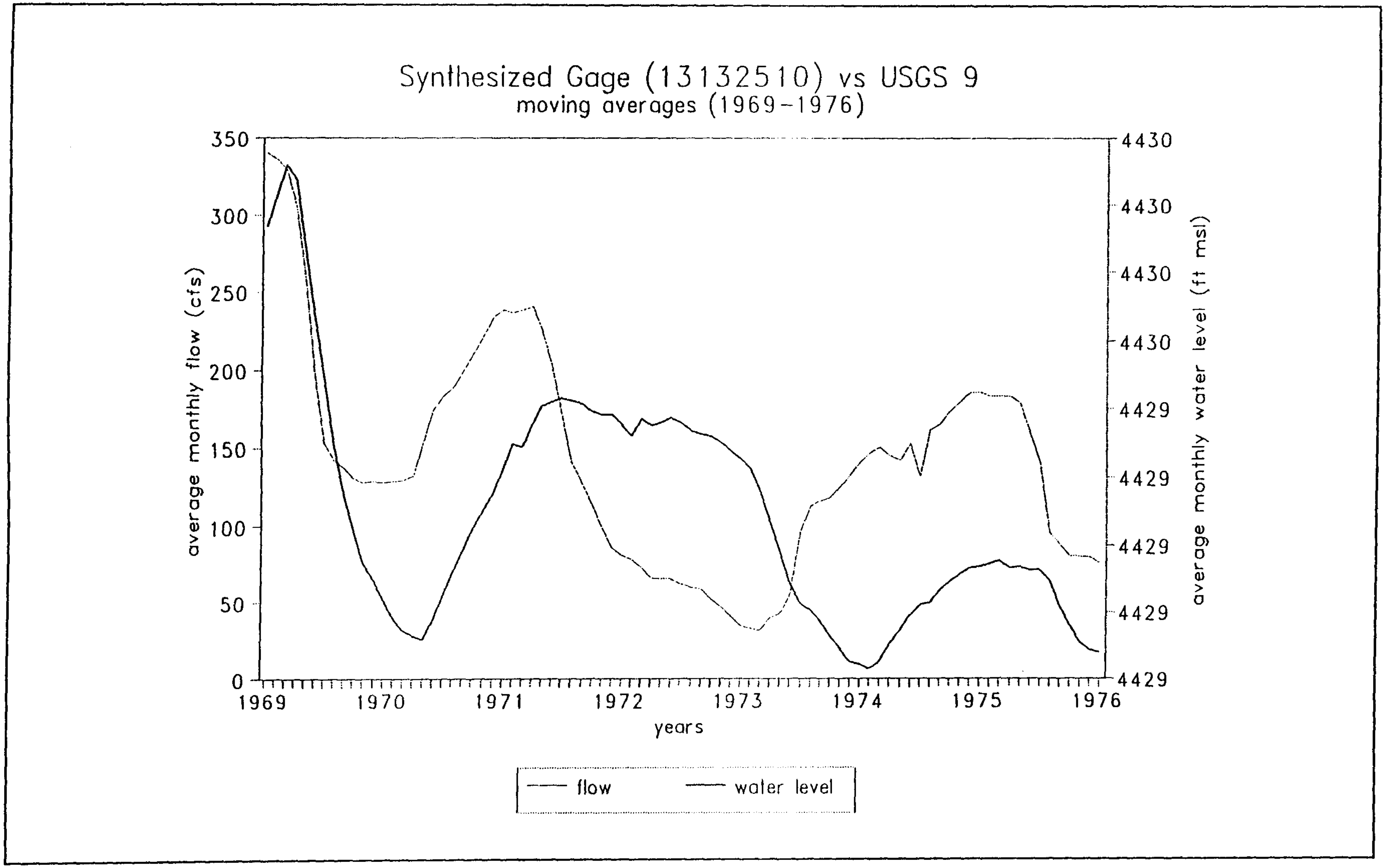

Figure 5.15 - Moving Average Hydrographs, Synthesized Gage vs USGS 9 (1969 to 1976)

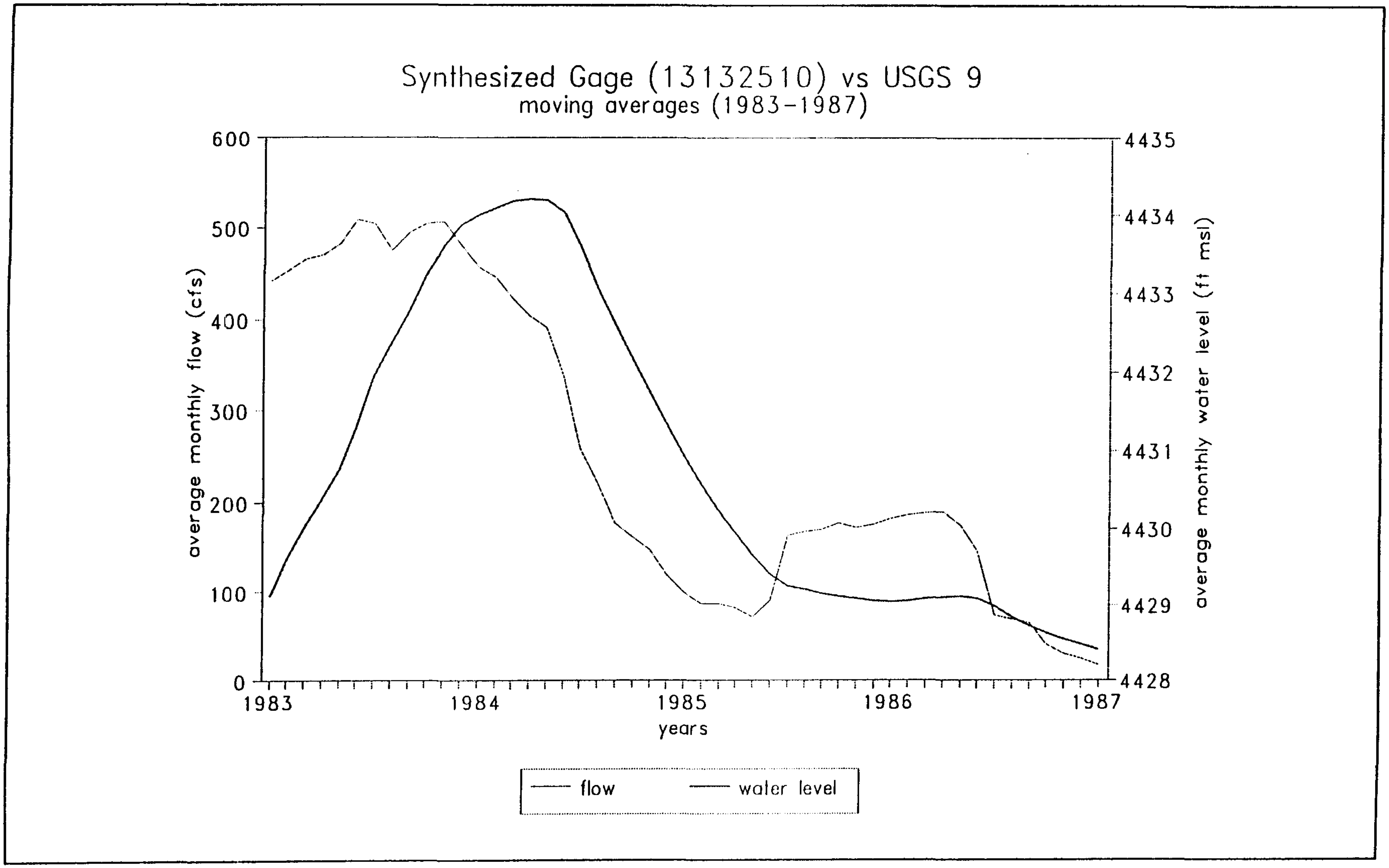

Figure 5.16 - Moving Average Hydrographs, Synthesized Gage vs USGS 9 (1983 to 1987)

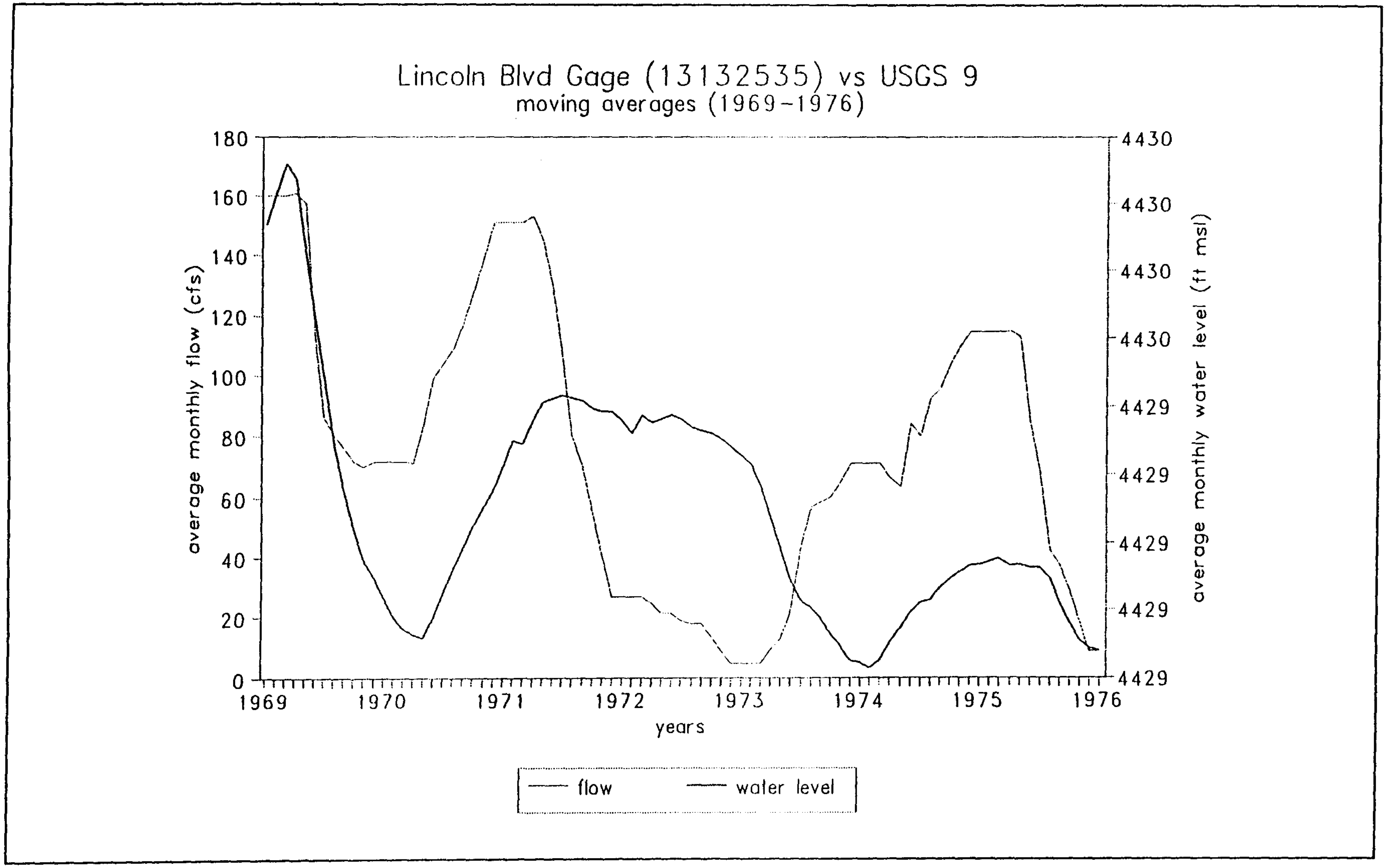

Figure 5.17 - Moving Average Hydrographs, Lincoln Boulevard Gage vs USGS 9 (1969 to 1976)

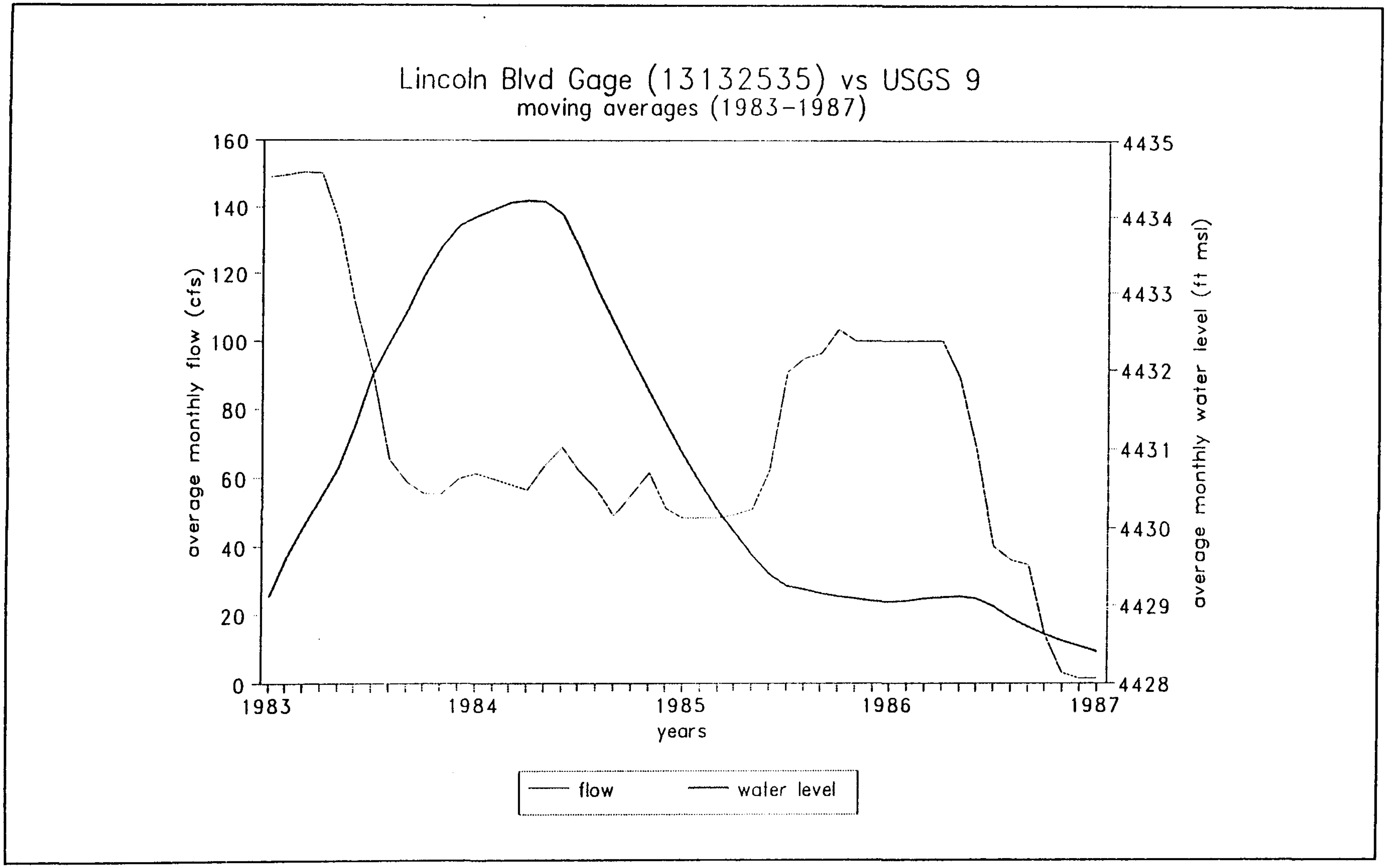

Figure 5.18 - Moving Average Hydrographs, Lincoln Boulevard Gage vs USGS 9 (1983 to 1987)

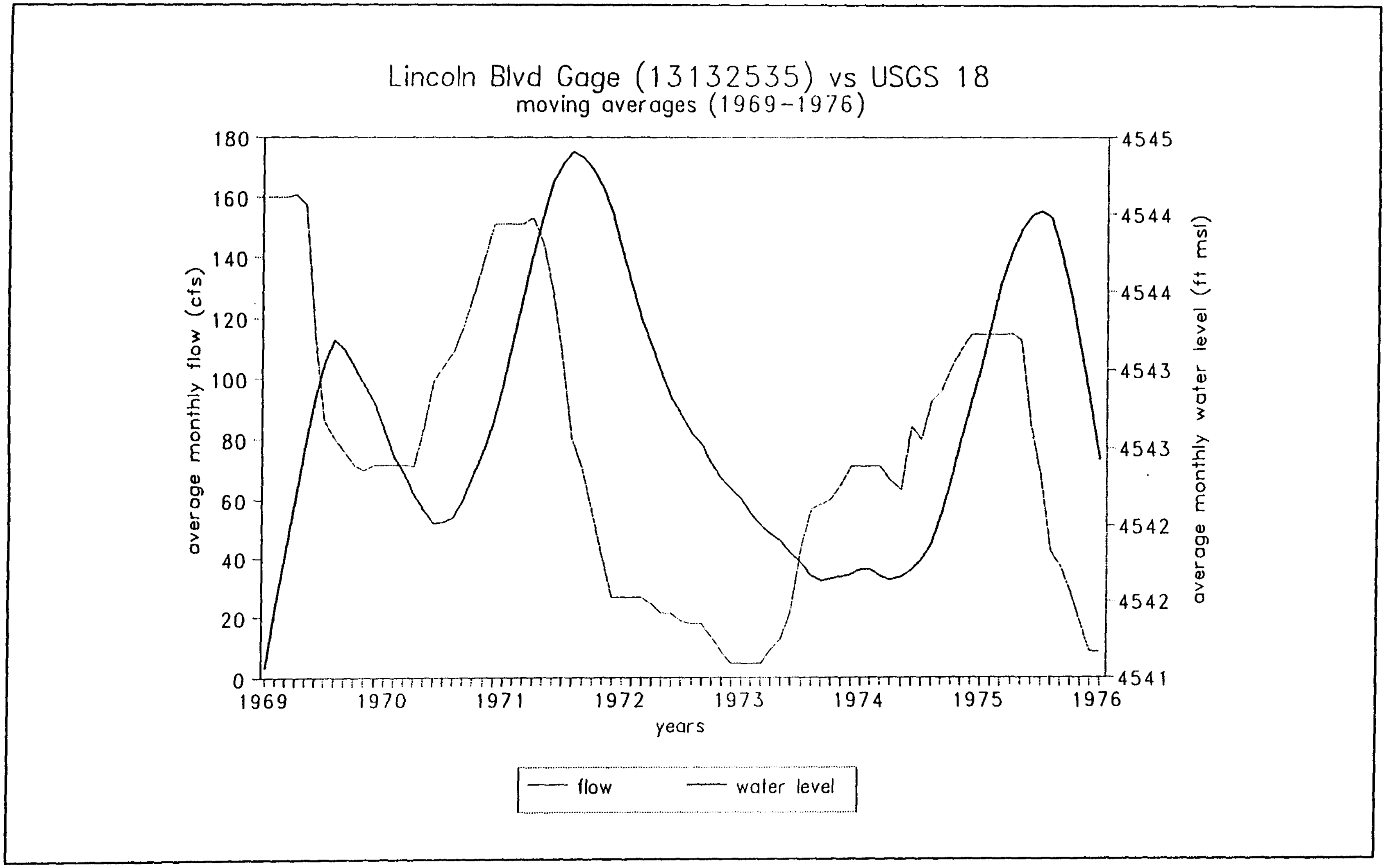

Figure 5.19 - Moving Average Hydrographs, Lincoln Boulevard Gage vs USGS 18 (1969 to 1976)

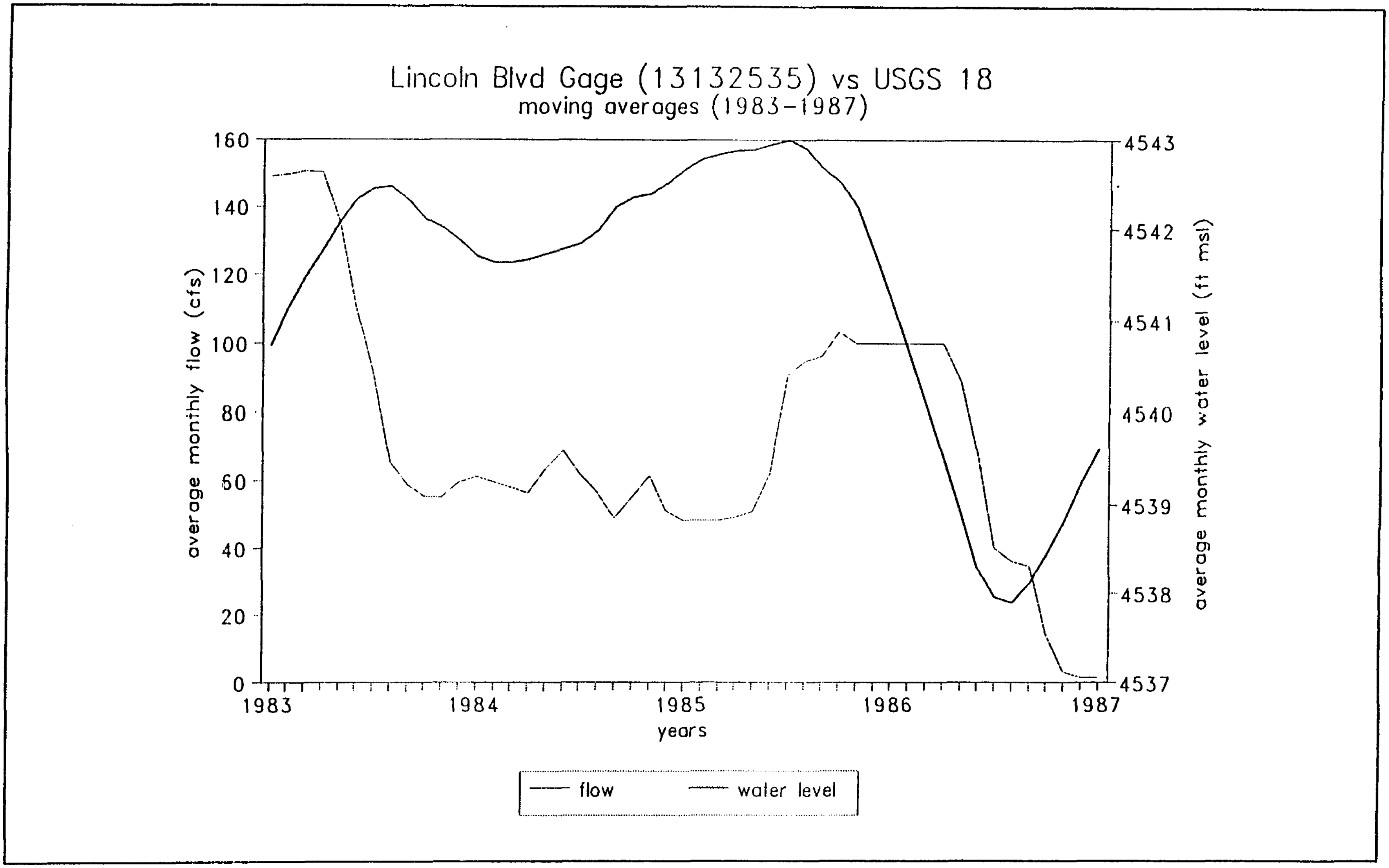

Figure 5.20 - Moving Average Hydrographs, Lincoln Boulevard Gage vs USGS 18 (1983 to 1987)

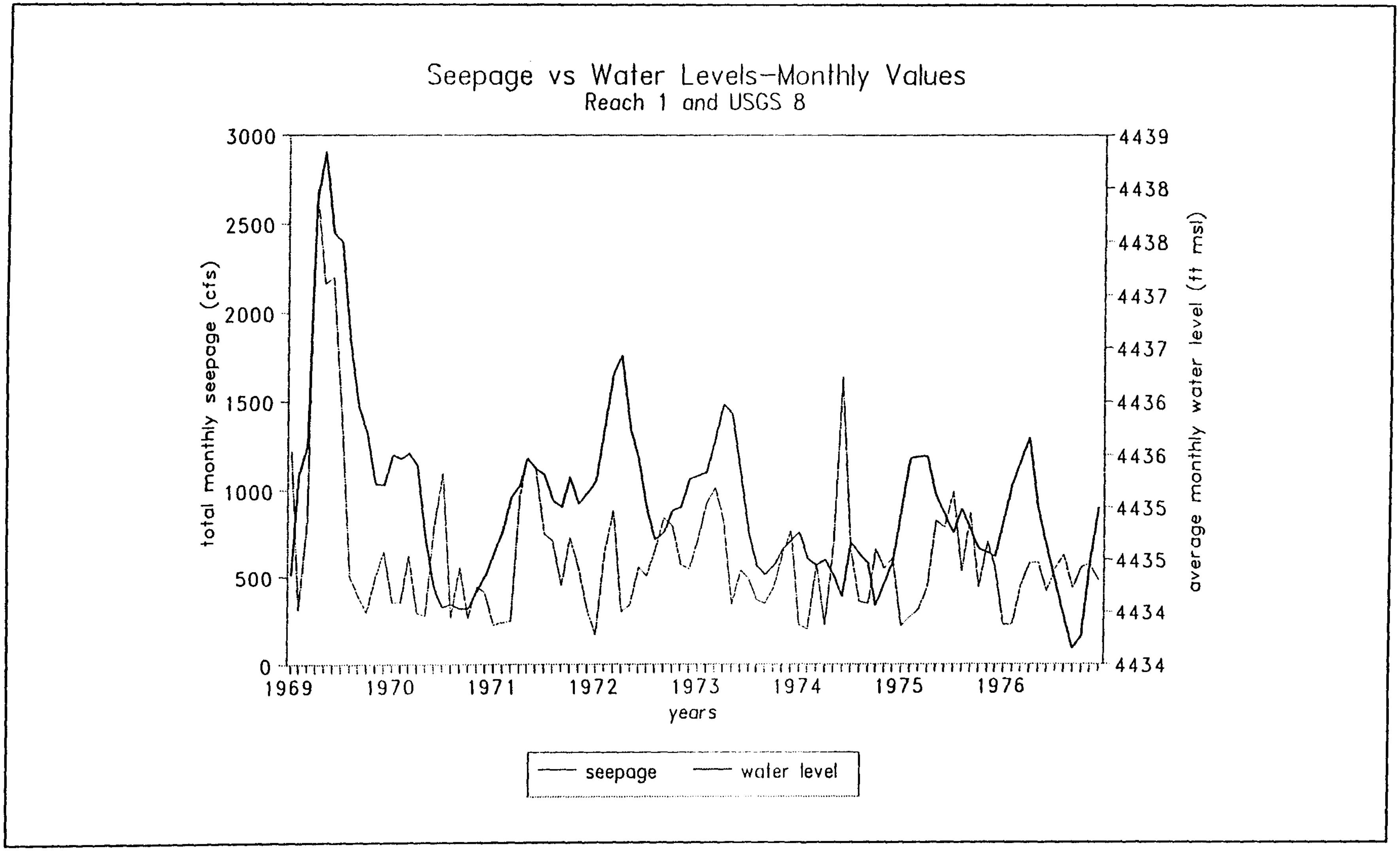

Figure 5.21 - Monthly Seepage and Water Level Hydrographs, Reach 1 and USGS 8 (1969 to 1976)

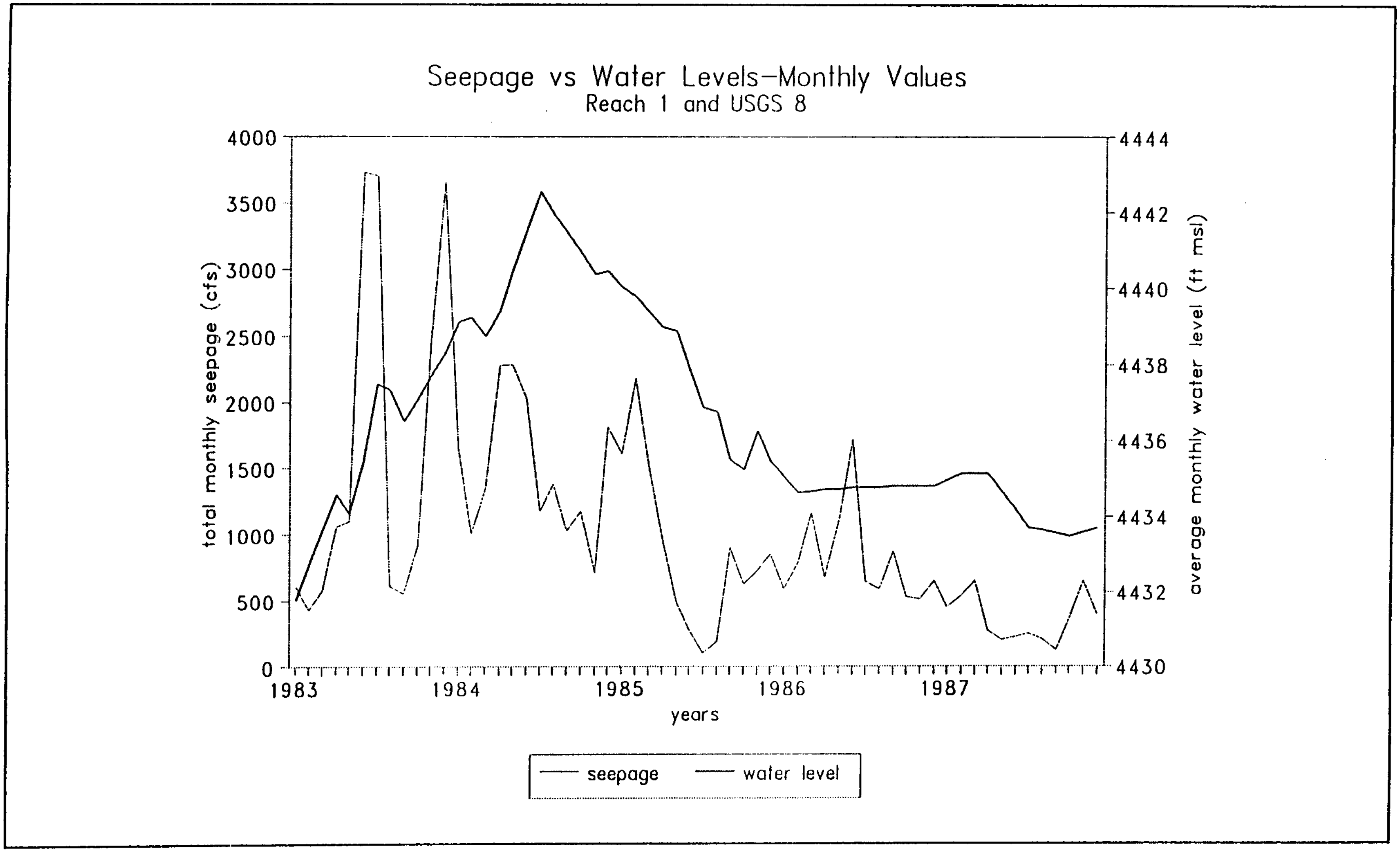

Figure 5.22 - Monthly Seepage and Water Level Hydrographs, Reach 1 and USGS 8 (1983 to 1987)

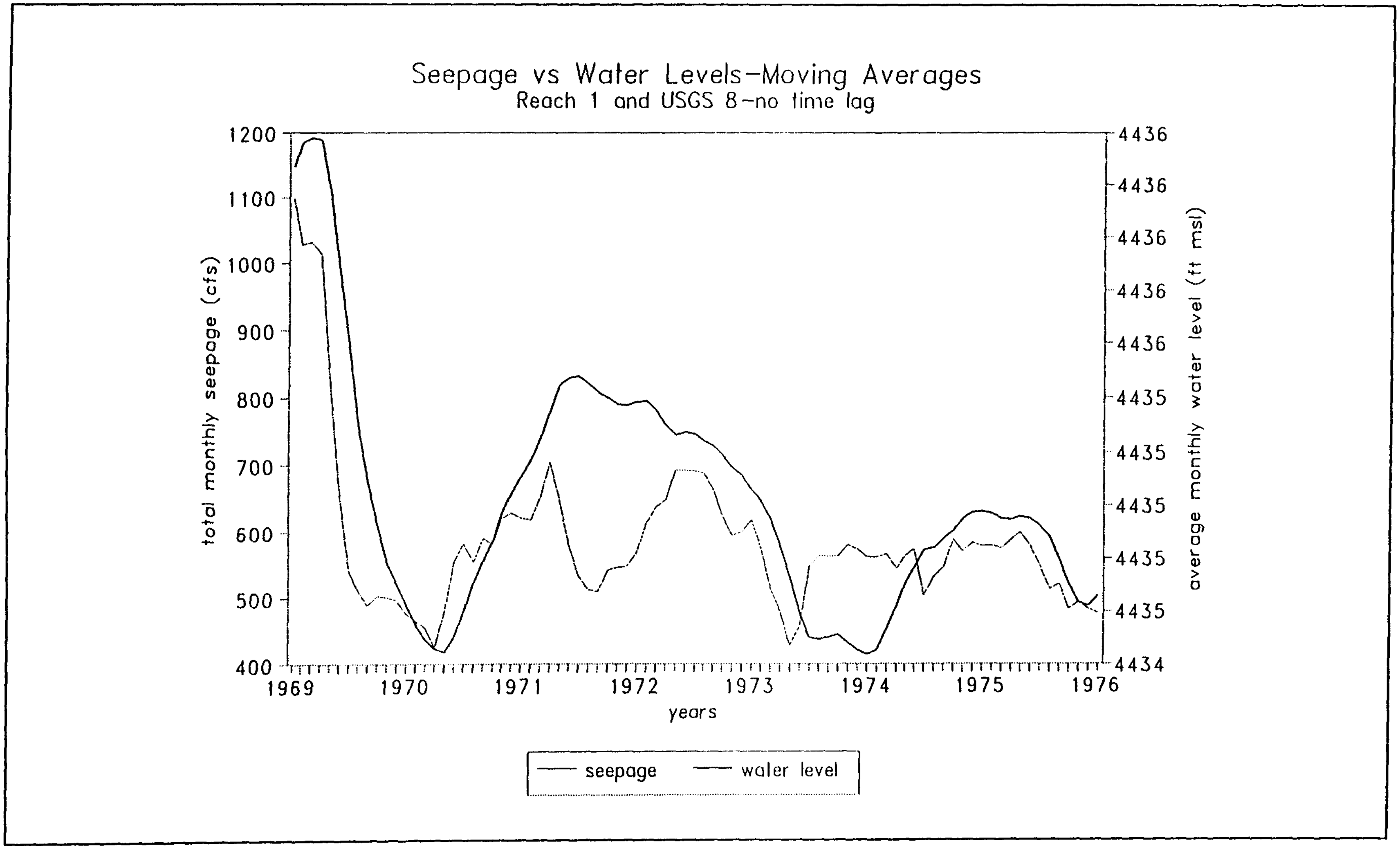

Figure 5.23 - Moving Average Seepage and Water Level Hydrographs, Reach 1 and USGS 8 (1969 to 1976)

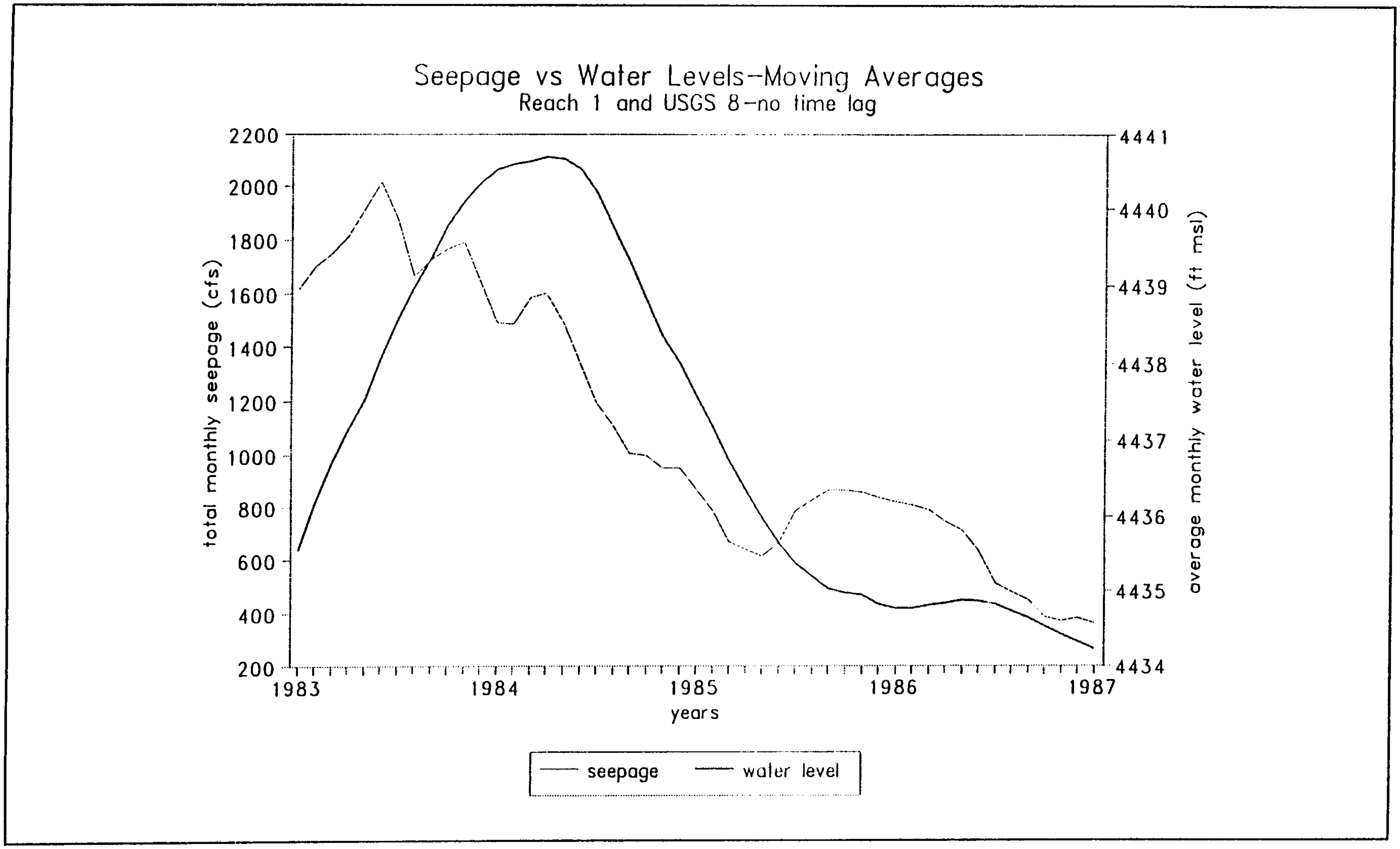

Figure 5.24 - Moving Average Seepage and Water Level Hydrographs, Reach 1 and USGS 8 (1983 to 1987)

periods ranging from two to seven months according to the relationship:

$$G_i = \beta_0 + \beta_1 * S_{i-n}$$

G_i represents the average groundwater level at time i, S_{i-n} is the total monthly seepage n months prior to i, and β_0 and β_1 are regression coefficients.

Initially, these regressions used the 12-month moving averages of the data to obtain a preliminary assessment of the time lag behavior. Since these moving averages have much of the original data variability removed by the averaging process, it was anticipated that the regressions should display a stronger correlation between the variables than the raw monthly data would yield. It was also anticipated that if a consistent time lag, n, existed, the r^2 value for that value of n would be larger than for any other n. For the values of n tested, the following table presents the r^2 values obtained by the regression analyses:

TABLE I: time lag vs r^2,
Moving Averaged Data

TIME LAG, n, MONTHS	DETERMINATION COEFFICIENT, r^2
2	0.795
3	0.839
4	0.868
5	0.882
6	0.884
7	0.873
8	0.854

The highest resulting determination coefficient, r^2, was 0.884 at an n of six months, demonstrating very strong correlation between the two time series. However, the r^2 at five months was 0.882. There is negligible difference between the two, but it shows that the lag peaks near six months. A regression using more frequent measurements (weekly, daily) would better identify the lag. Figure 5.25 shows a scatter plot of the moving averages of total monthly seepage against average monthly water level for the six month time lag. It illustrates the linearity and correlation between the two variables that the regression analyses suggested.

Following these preliminary assessments, regression analyses were then applied to the raw monthly data, using the same time lag sequence, beginning at n equal to two months. Table II presents the values of r^2 for these regressions.

This approach yielded a seven month time lag as the best correlation, with an r^2 of 0.359. For the same analysis at six months the r^2 was 0.327. Although it was known that the r^2 values for the raw data would be lower, since the data had not been averaged statistically, a different time lag had not been expected. However, a review of Table II indicates that there originally had been very little difference between the r^2 values for n=6 (r^2=0.884) and n=7 (r^2=0.873). Figure 5.26 presents a scatter plot of the moving averaged data with n=7, for comparison with figure 5.25.

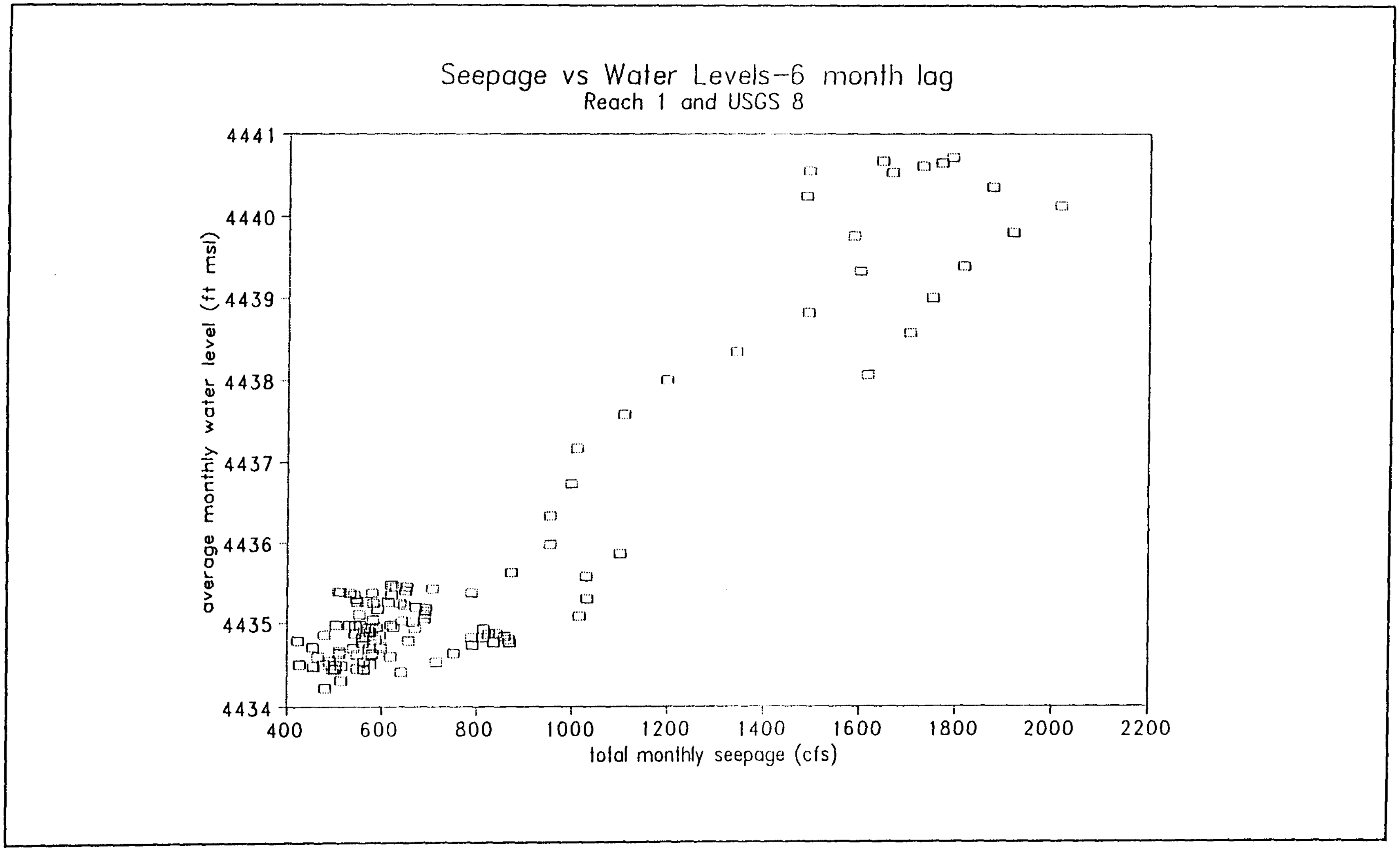

Figure 5.25 - Reach 1 and USGS 8: Seepage vs Water Levels Scatter Plot with a 6 Month Time Lag

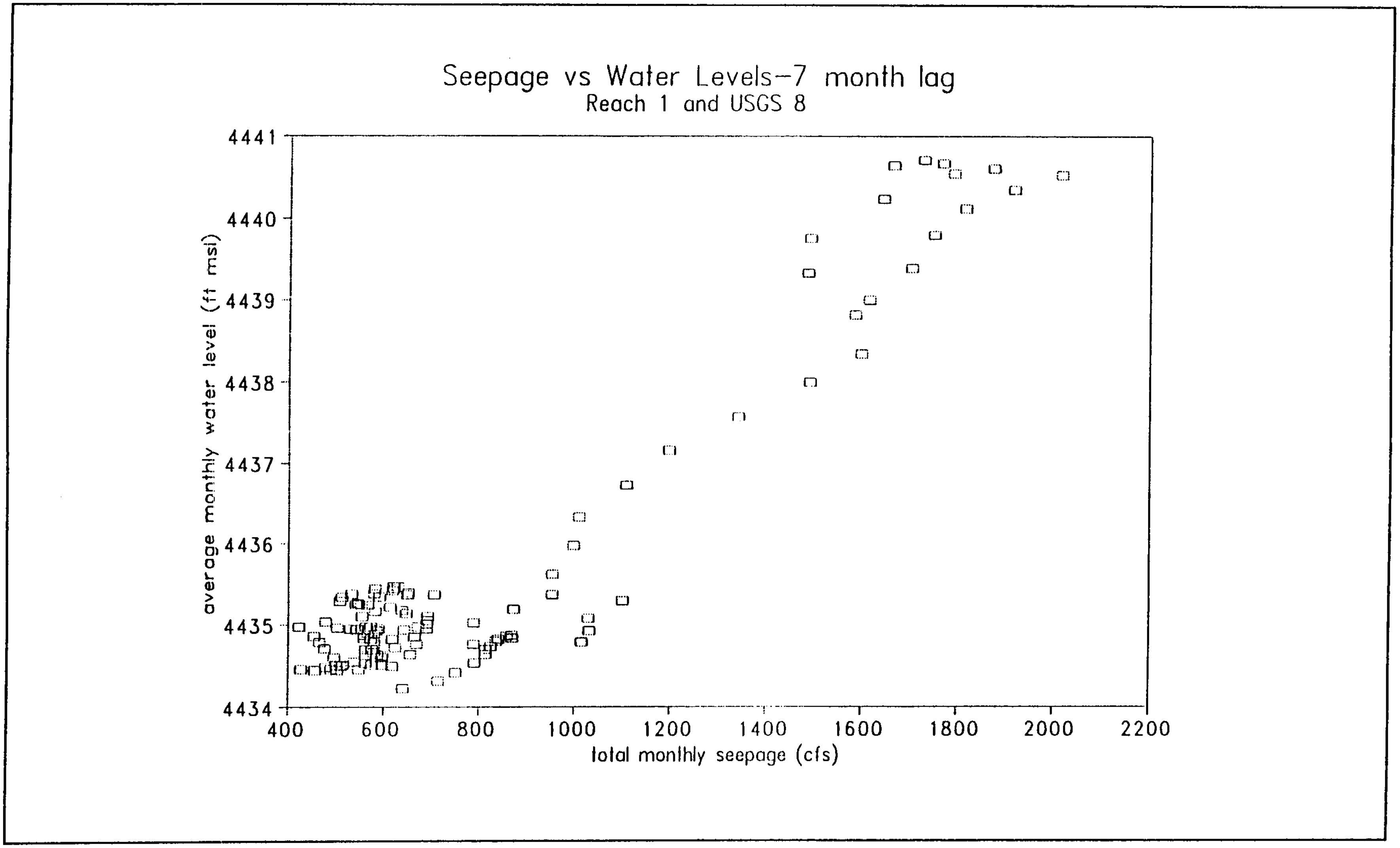

Figure 5.26 - Reach 1 and USGS 8: Seepage vs Water Levels Scatter Plot with a 7 Month Time Lag

The results from these analyses indicate that although the use of moving averaged data makes the graphical presentation of the time series easier to read and compare, there is time lag sensitivity lost in the process. However, the regression on both the moving averaged and raw data showed good correlation at both a six-month and seven-month time lag, with reasonably consistent results. From this it was assumed that the lag was was approximately six to seven months.

TABLE II: time lag vs r^2, Raw Monthly Data

TIME LAG, n, MONTHS	DETERMINATION COEFFICIENT, r^2
2	0.344
3	0.312
4	0.294
5	0.294
6	0.327
7	0.359
8	0.347

SPREADING AREAS AND USGS 9

The seepage process in the spreading areas is more direct than in the river reaches, since almost all of the water entering the basins is lost through infiltration and percolation. In addition, the seepage is confined to a relatively small area, with the selected well located in very close proximity. Therefore, the correlation between the two was anticipated to be high.

The plots of monthly values of seepage and water level against time can be seen in figures 5.27 and 5.28. Again, the time lag and similarity between graphs is quite noticeable. Figures 5.29 and 5.30 show the 12-month moving averages of the data. Regressions on the moving averaged data yielded an r^2 of 0.937 at five months as the best correlation. A regression on the monthly data gave an r^2 of 0.683 at five months. For both analyses, the five month correlation was the highest. Again, for the moving averaged data, the loss in sensitivity is demonstrated with the small difference between time lags of four and five months. Table III presents the r^2 values versus n for the moving averaged and the raw data.

Scatter plots of the two regressions with n=5 months can be seen in figures 5.31 and 5.32. The relationship in figure 5.31 is very nearly linear and visually demonstrates why the r^2 for the moving averaged data was so high. Since the raw

TABLE III: time lags vs r^2,
Moving Average and Raw Data

TIME LAG, n, MONTHS	MOVING AVERAGE r^2	RAW DATA r^2
2	0.873	0.611
3	0.915	0.635
4	0.936	0.666
5	0.937	0.683
6	0.917	0.679
7	0.883	0.644

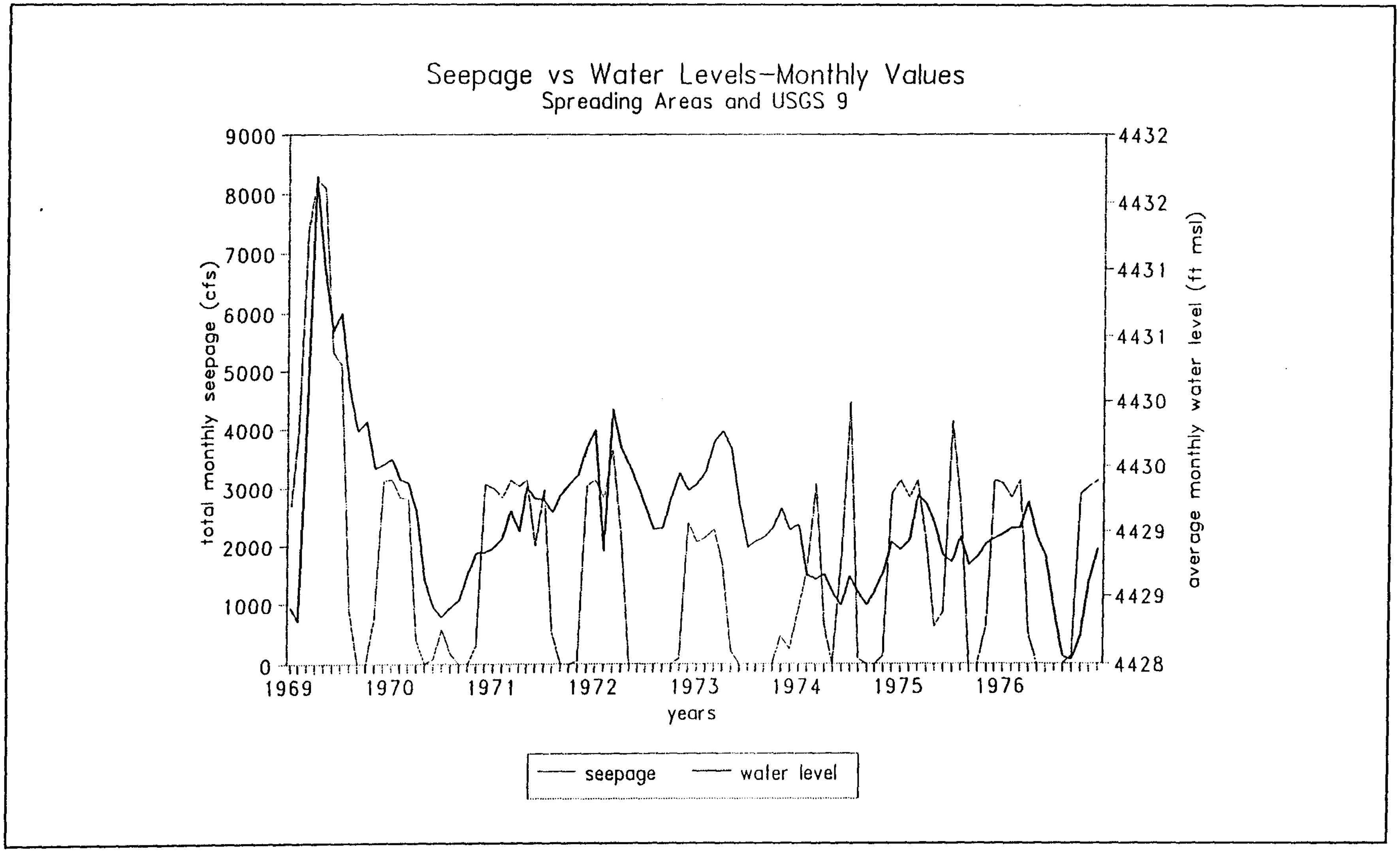

Figure 5.27 - Monthly Seepage and Water Level Hydrographs, Spreading Areas and USGS 9 (1969 to 1976)

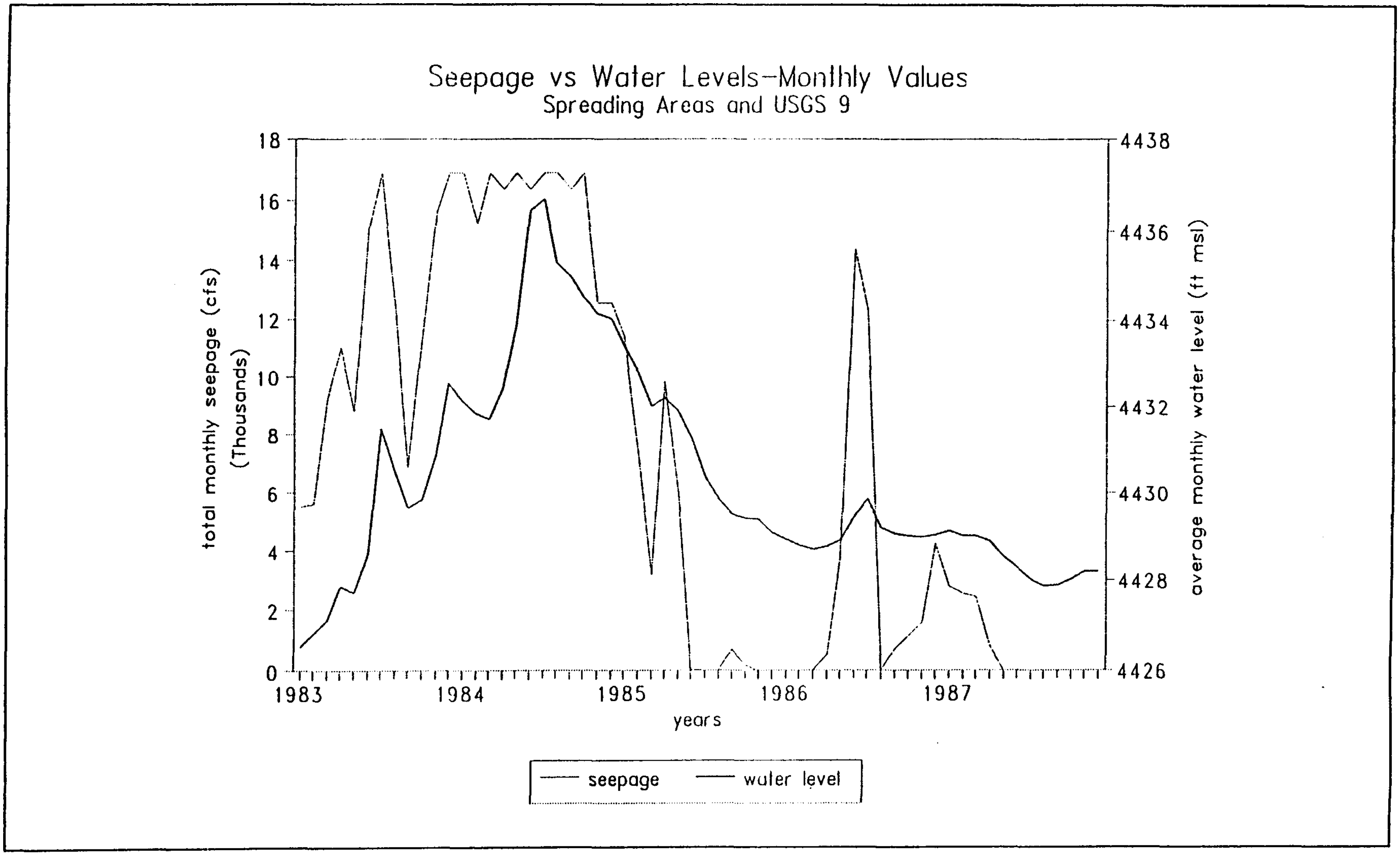

Figure 5.28 - Monthly Seepage and Water Level Hydrographs, Spreading Areas and USGS 9 (1983 to 1987)

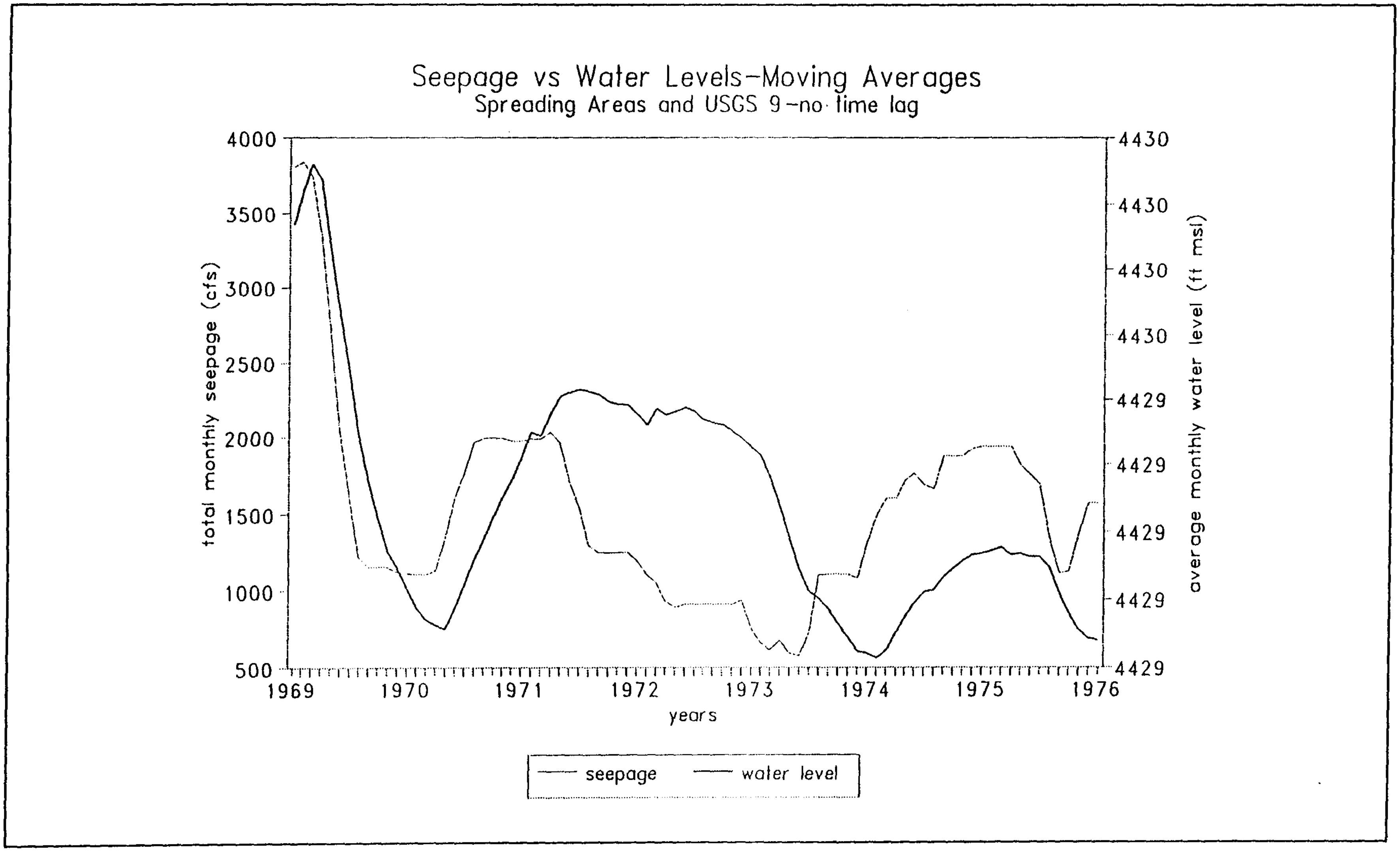

Figure 5.29 - Moving Average Seepage and Water Level Hydrographs, Spreading Areas and USGS 9 (1969 to 1976)

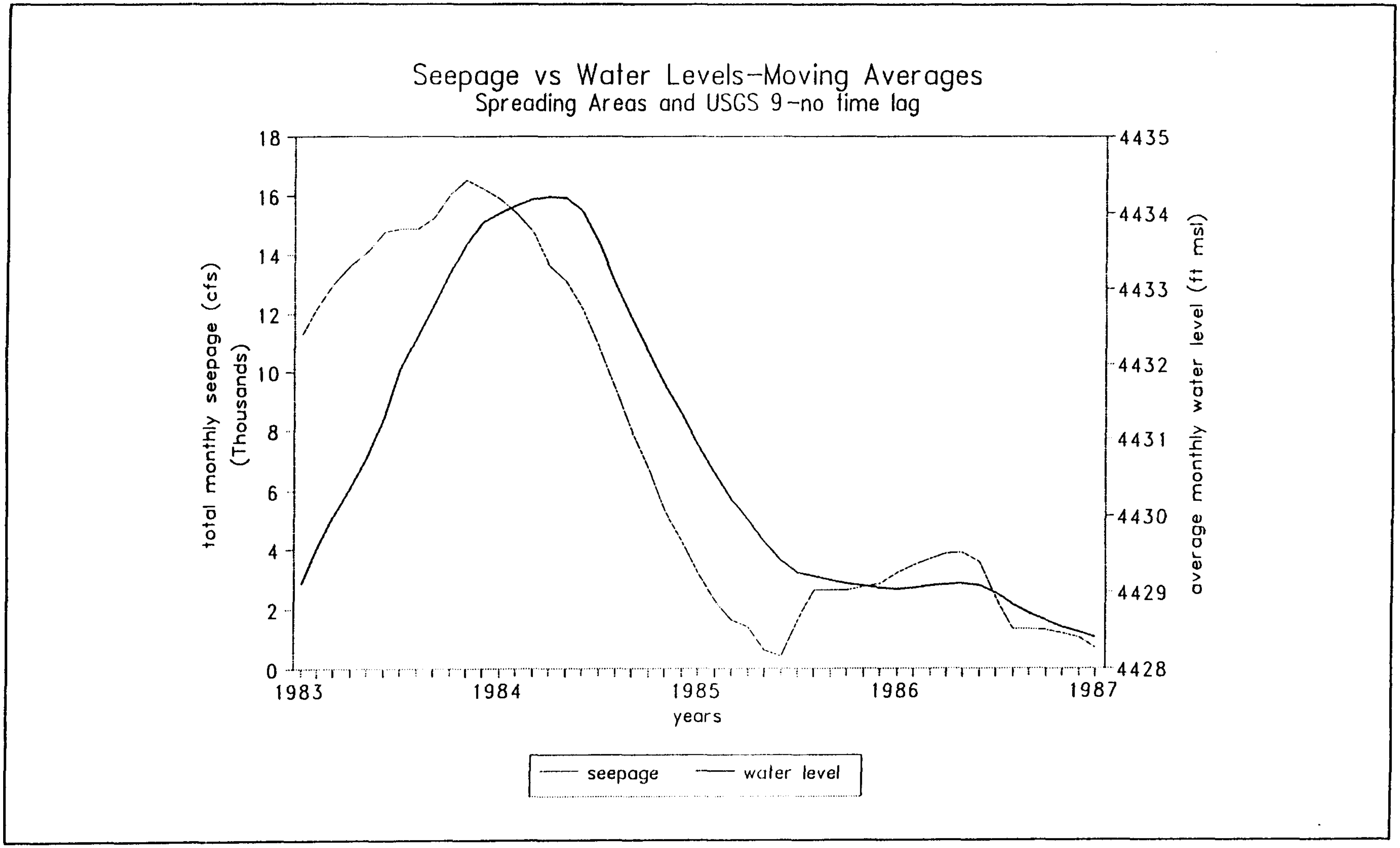

Figure 5.30 - Moving Average Seepage and Water Level Hydrographs, Spreading Areas and USGS 9 (1983 to 1987)

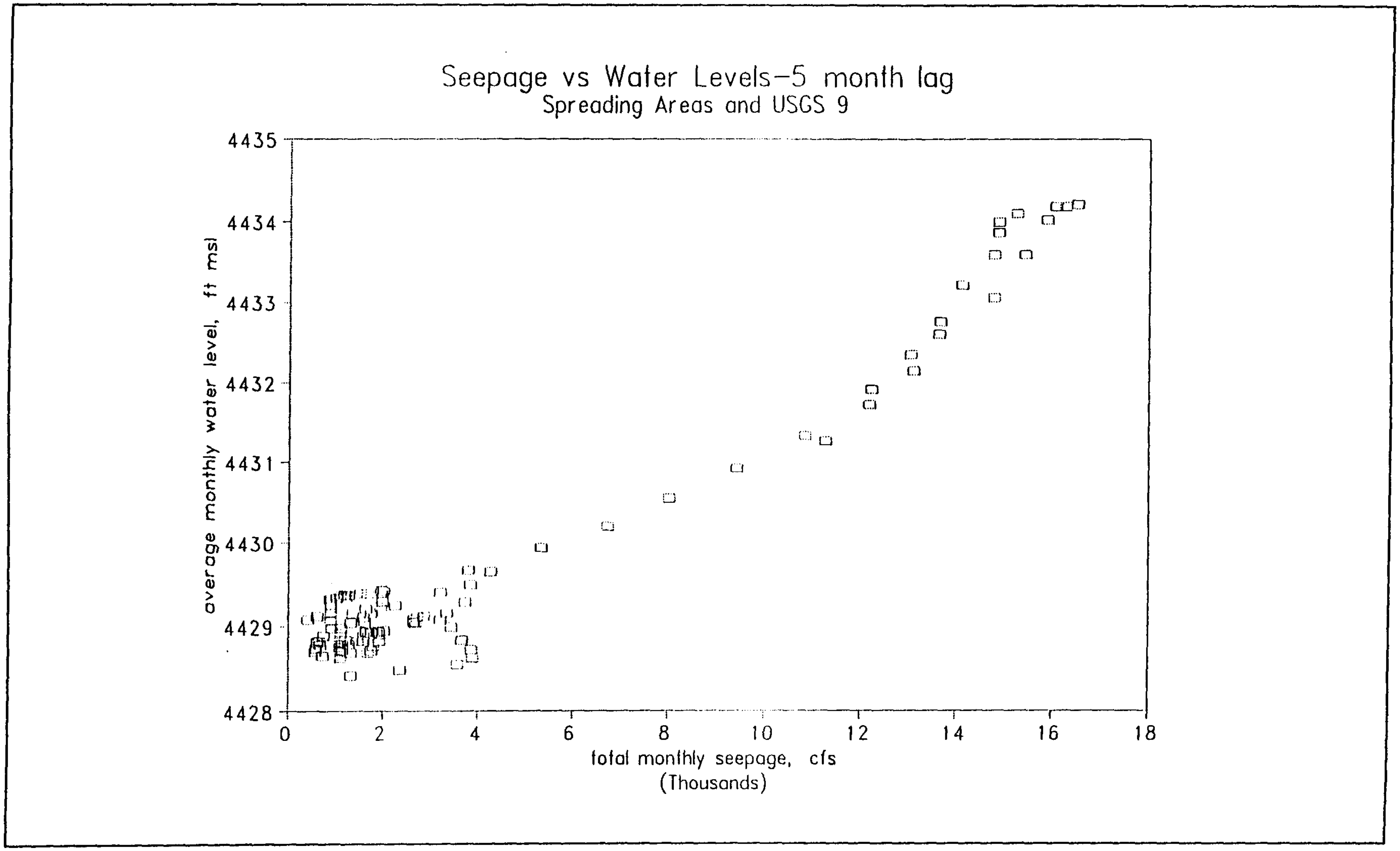

Figure 5.31 - Spreading Areas and USGS 9: Seepage vs Water Levels Scatter Plot of Moving Averages with a 5 Month Time Lag

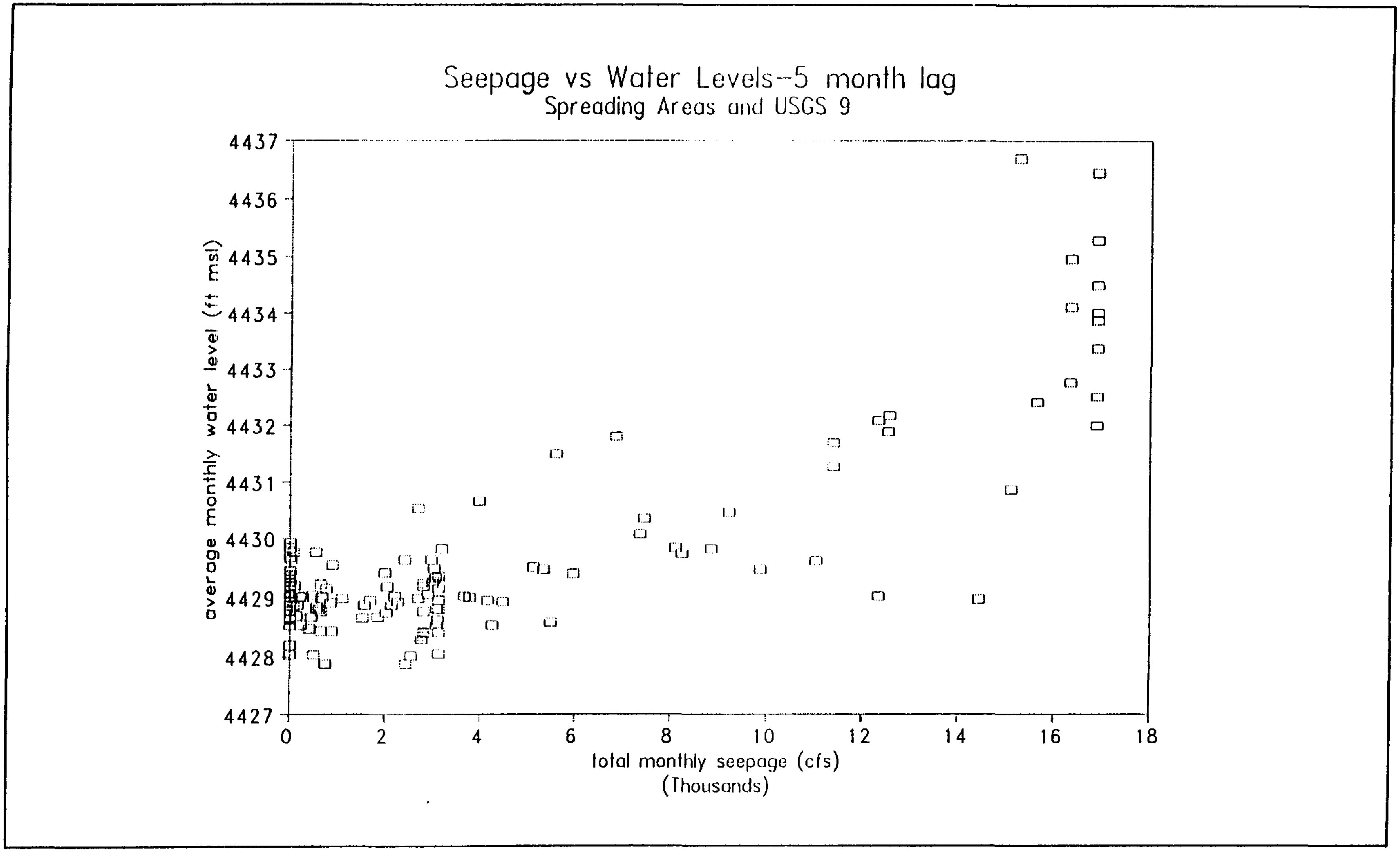

Figure 5.32 - Spreading Areas and USGS 9: Seepage vs Water Levels Scatter Plot of Monthly Values with a 5 Month Time Lag

data have considerably greater variability, their scatter plot, figure 5.32, does not demonstrate this degree of linearity. Compared to the previous reach 1 analysis, the correlation coefficients for the spreading areas were considerably higher, especially for the raw data. It was concluded that the results were better than those for reach 1, for the reason previously stated.

REACH 1, SPREADING AREAS, AND USGS 9

Plots for this comparison can be seen in figures 5.33 through 5.38. Seepage here was defined as the sum of the seepage from reach 1 and the spreading areas, and was correlated again to the water levels in USGS 9. This analysis was done to determine if seepage from reach 1 would have any additional impact on the time lag calculated between seepage from the spreading areas and USGS 9. Regression analyses using the combined data sets yielded an r^2 of 0.94 at a five-month time lag for moving averages, and an r^2 of 0.68 at five months for the raw monthly data. These numbers did not differ from the comparison of the spreading areas alone and USGS 9, primarily because the volume of seepage from the spreading areas far outweighed that from reach one. (For the periods of time in the study, the average seepage rate from the ponds was 3699.5 cfs, and 782.3 cfs from reach 1.)

REACH 2 AND USGS 9

Reach 2 is located north and east of USGS 9 (greater than 5 miles), and it was believed that a good correlation

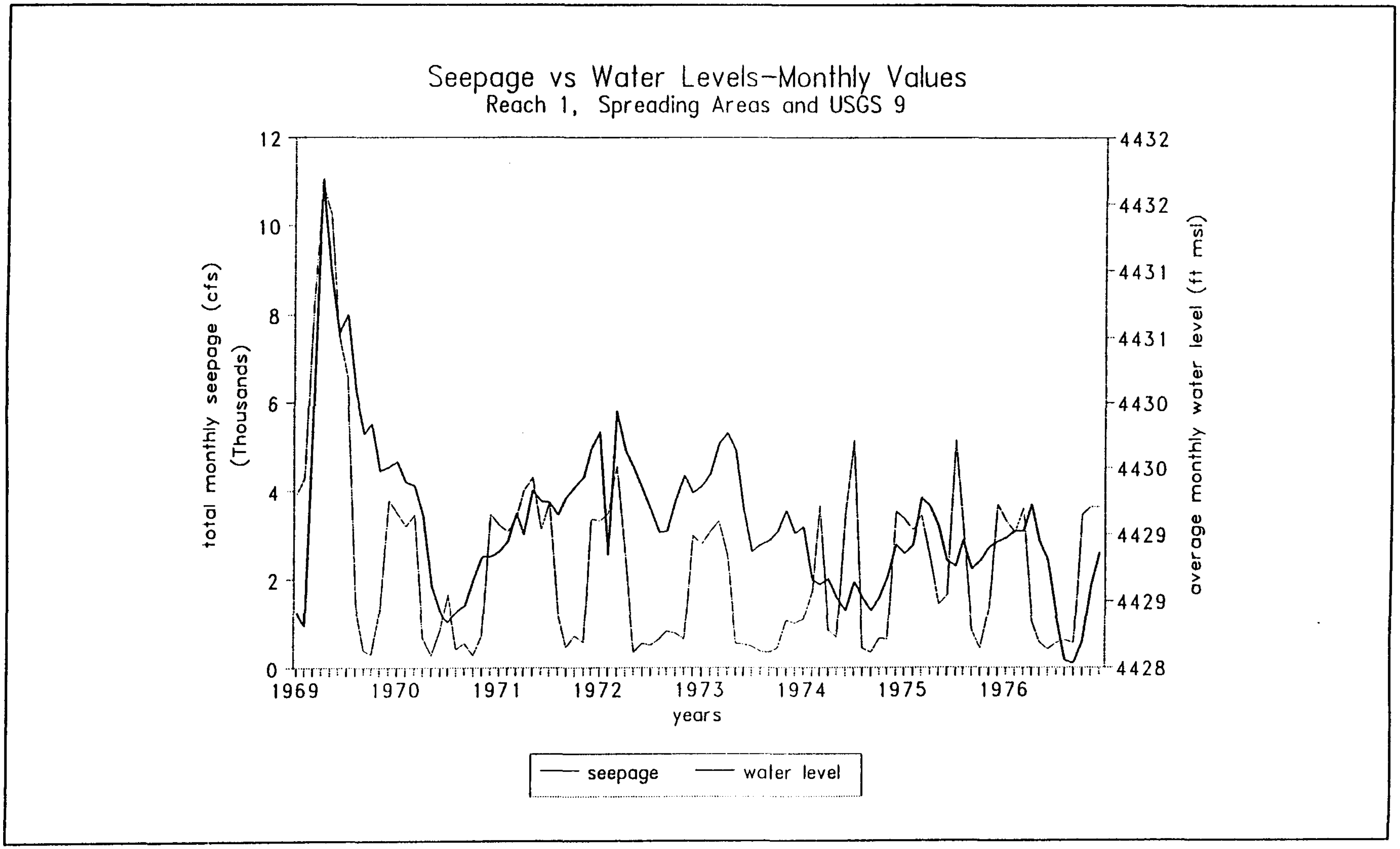

Figure 5.33 - Monthly Seepage and Water Level Hydrographs, Reach 1, Spreading Areas and USGS 9 (1969 to 1976)

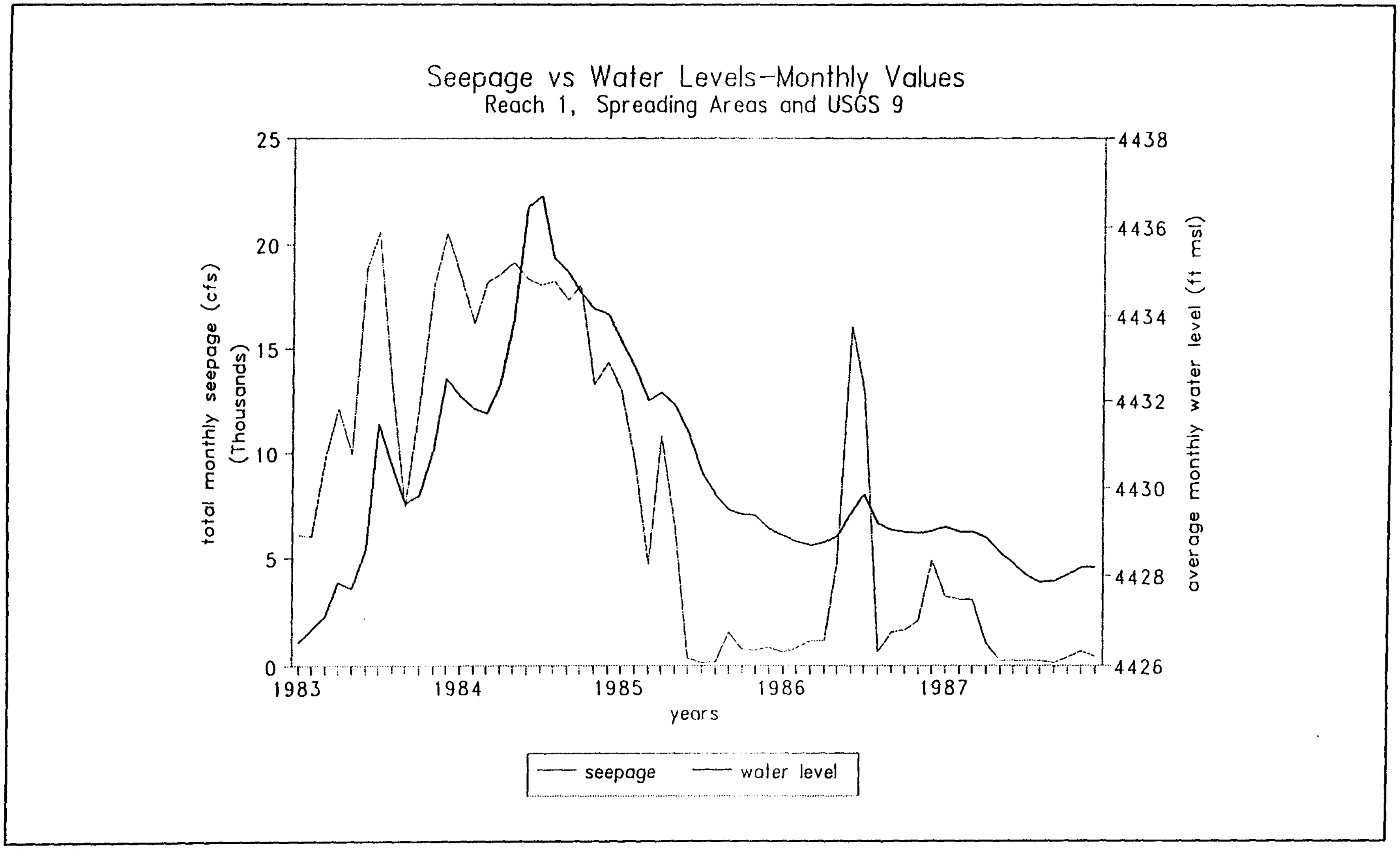

Figure 5.34 - Monthly Seepage and Water Level Hydrographs, Reach 1, Spreading Areas and USGS 9 (1983 to 1987)

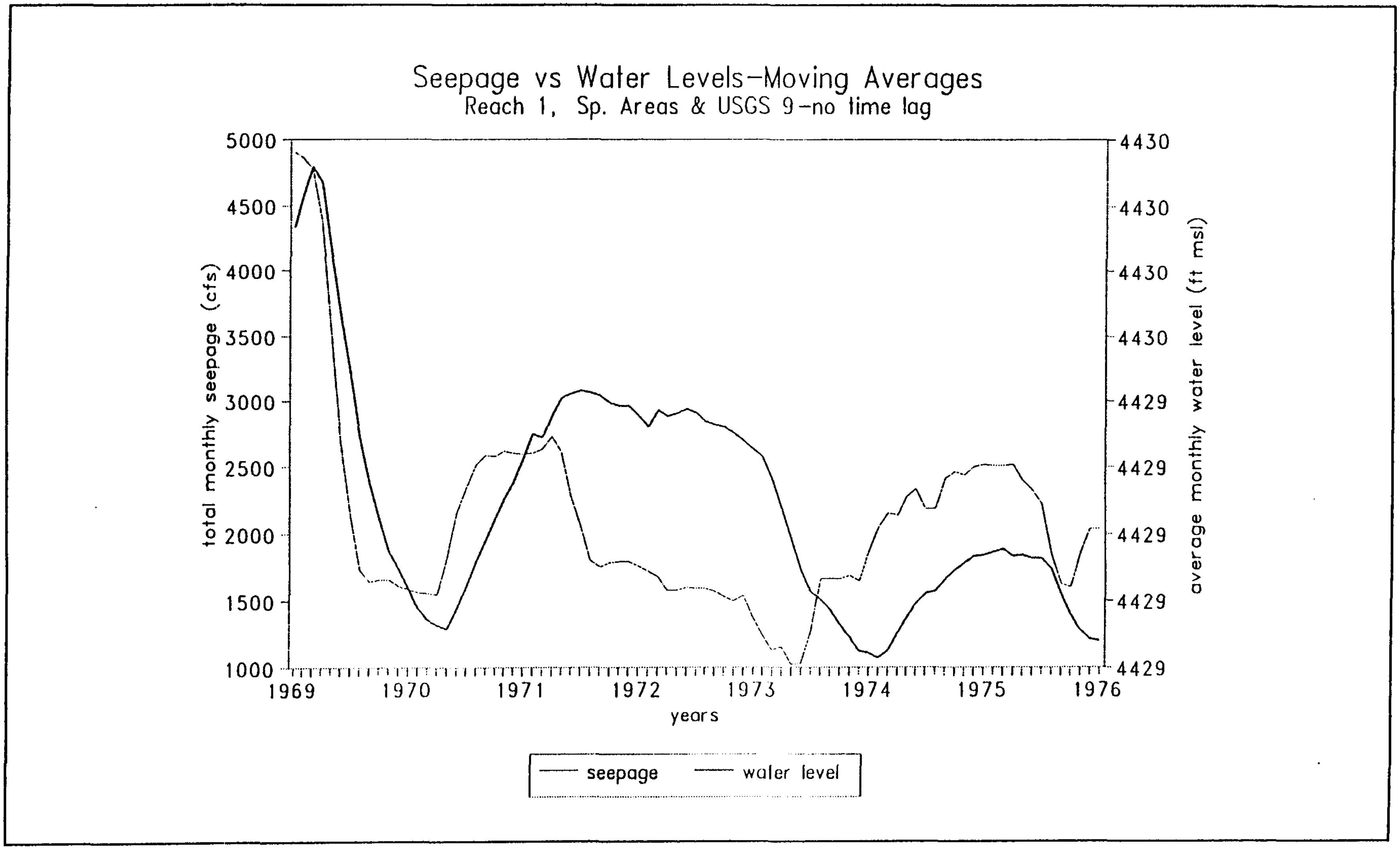

Figure 5.35 - Moving Average Seepage and Water Level Hydrographs, Reach 1, Spreading Areas and USGS 9 (1969 to 1976)

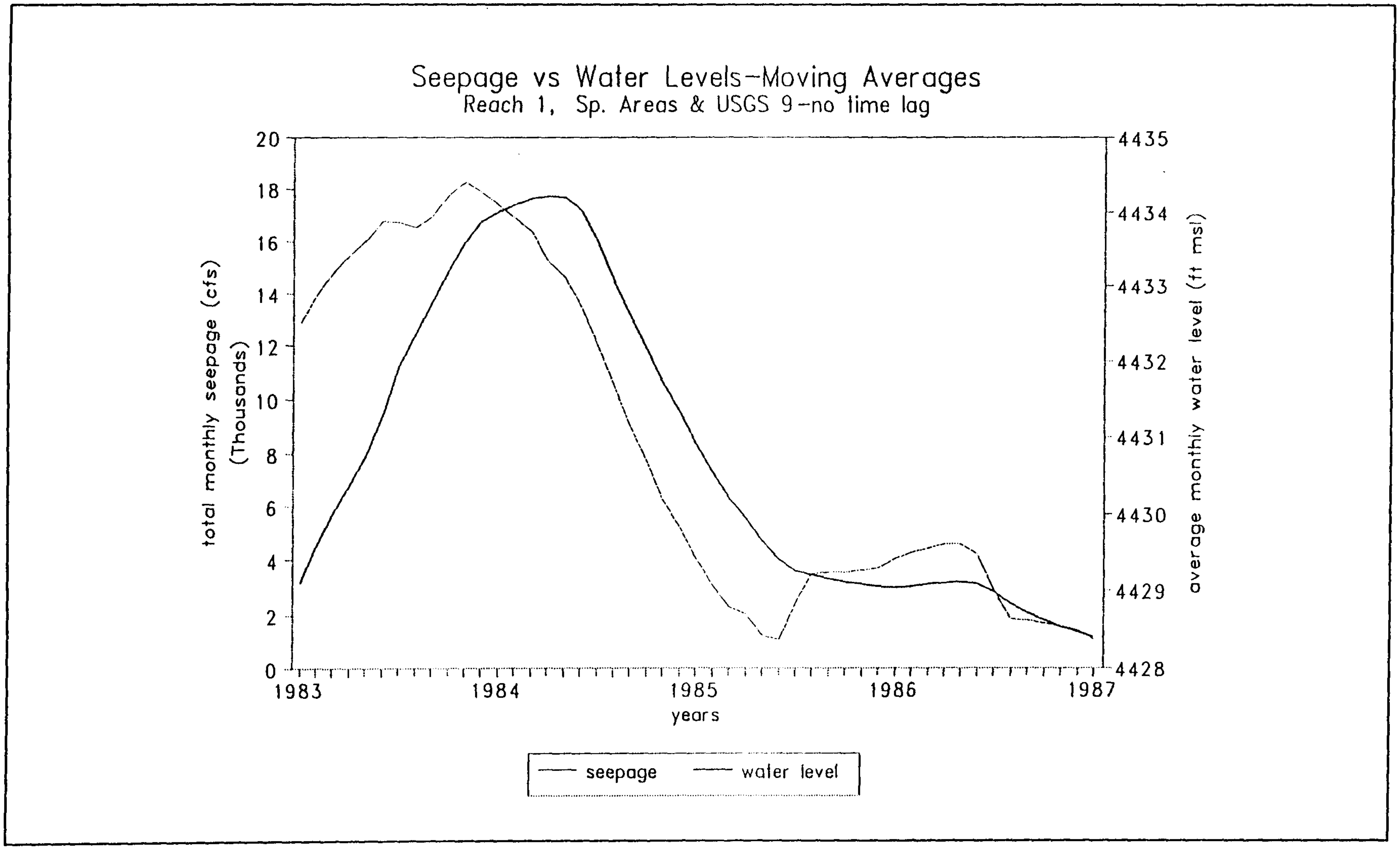

Figure 5.36 - Moving Average Seepage and Water Level Hydrographs, Reach 1, Spreading Areas and USGS 9 (1983 to 1987)

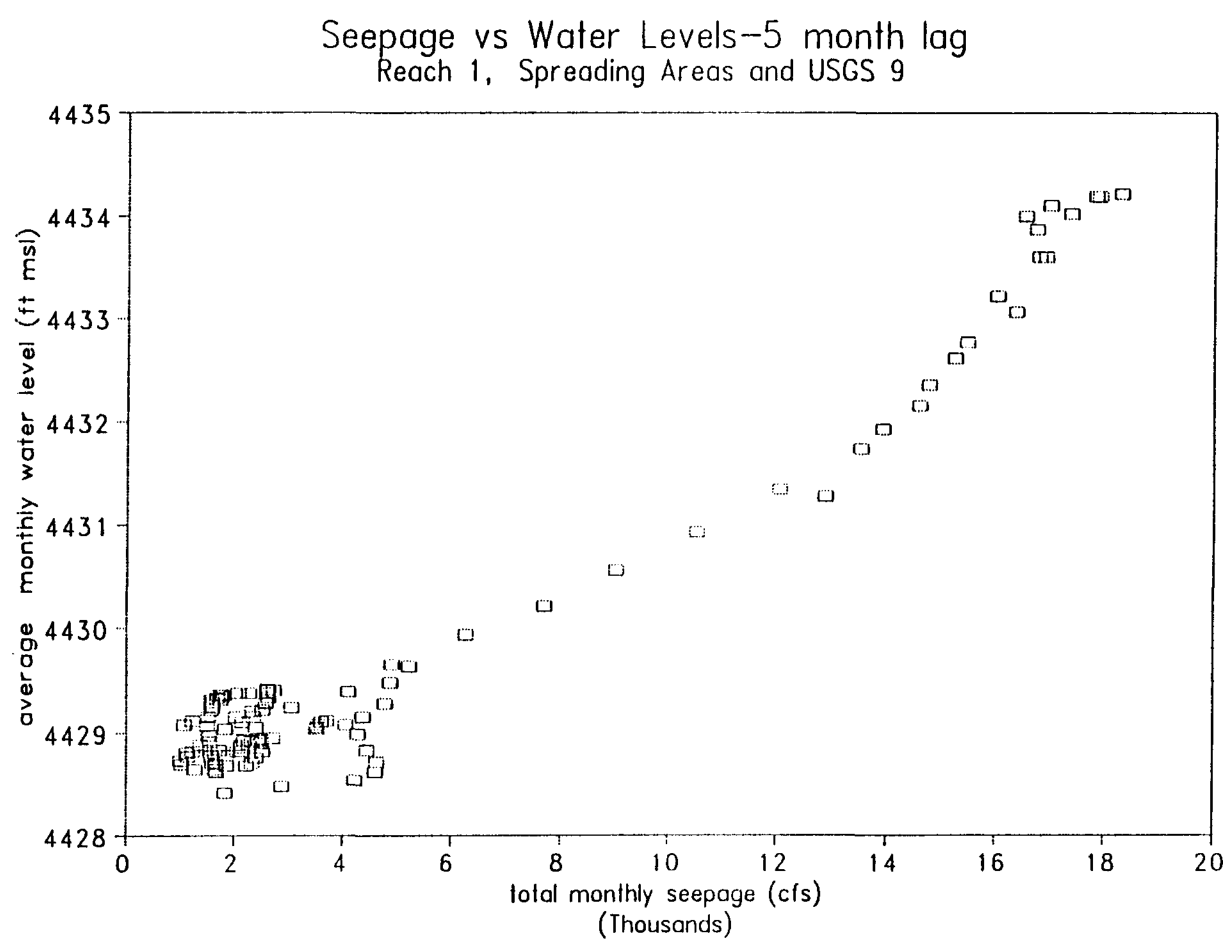

Figure 5.37 - Reach 1, Spreading Areas and USGS 9: Seepage vs Water Levels Scatter Plot of Moving Averages with a 5 Month Time Lag

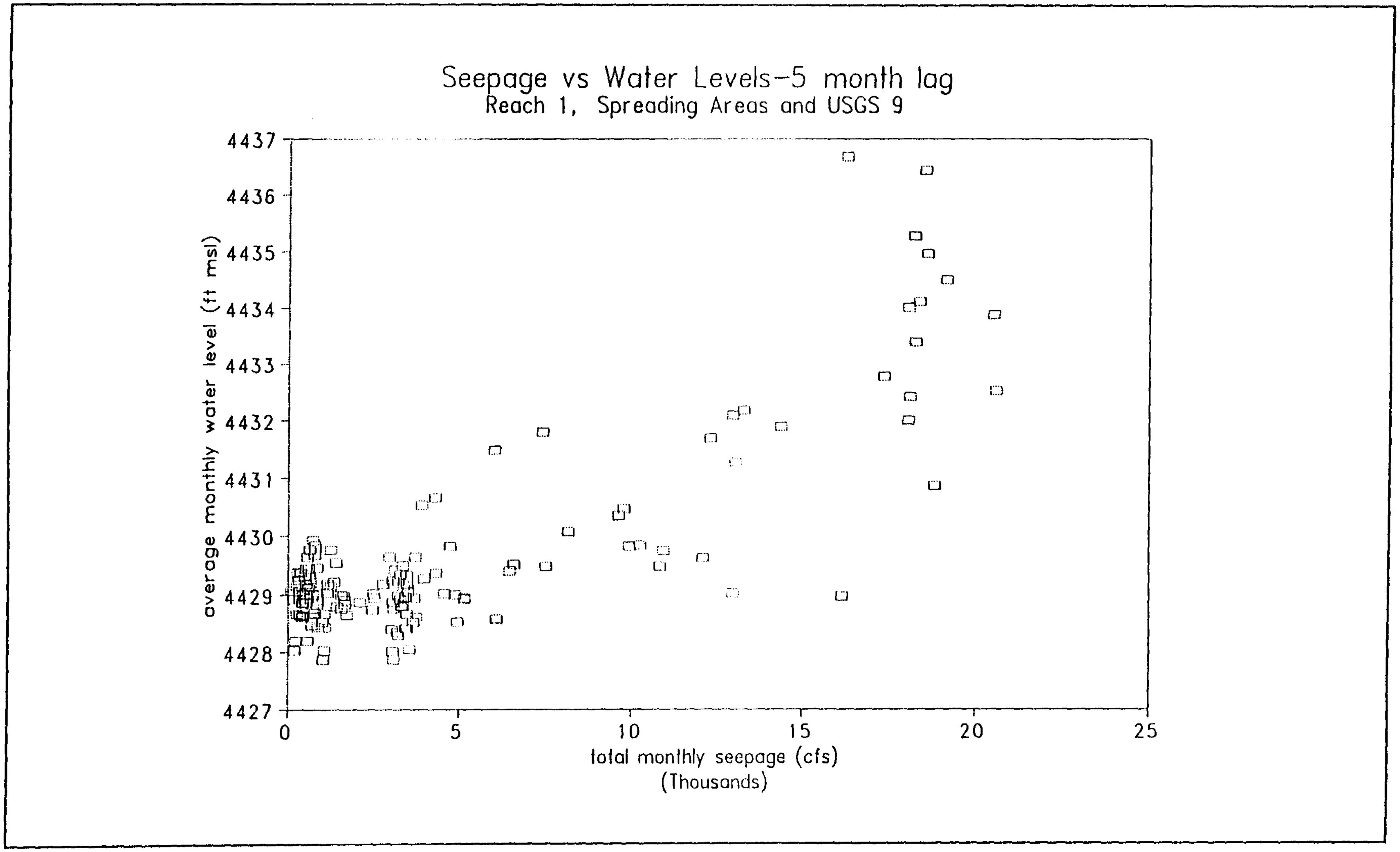

Figure 5.38 - Reach 1, Spreading Areas and USGS 9: Seepage vs Water Levels Scatter Plot of Monthly Values with a 5 month time lag

would exist between seepage from reach 2 and water levels in the well since regional groundwater flow is to the southwest. Again, the same correlation procedures used previously were applied, and the 12-month moving averages were plotted (see figures 5.39 and 5.40). As is demonstrated in the plots, there was little correlation, especially in the period from 1983-1987. There appeared to be some correlation during the first period, but a regression on the data yielded a maximum r^2 of 0.017 at n=2 months. For the second period of study, the strongest correlation was r^2=0.05 at n=6 months. Both of these determination coefficients indicate little or no correlation since they are nearly zero.

Only one theory was proposed as to why no correlation existed between seepage from reach 2 and water levels in USGS 9. The original hypothesis, suggesting that there would be a correlation because the reach was northeast of the well and groundwater flow is southwest, was wrong. It is possible that seepage does impact the well, but not in any measurable manner using the methods in this study.

This attempted correlation was the only one performed in the study for reach 2. USGS 9 was the closest well to reach 2 of those chosen, and the obvious choice for study. Given the distance to the other two wells and the flow direction of groundwater, there were no other feasible choices for comparison. However, there are many other wells located much closer to reach 2, and further study might reveal a strong correlation exists with some of them.

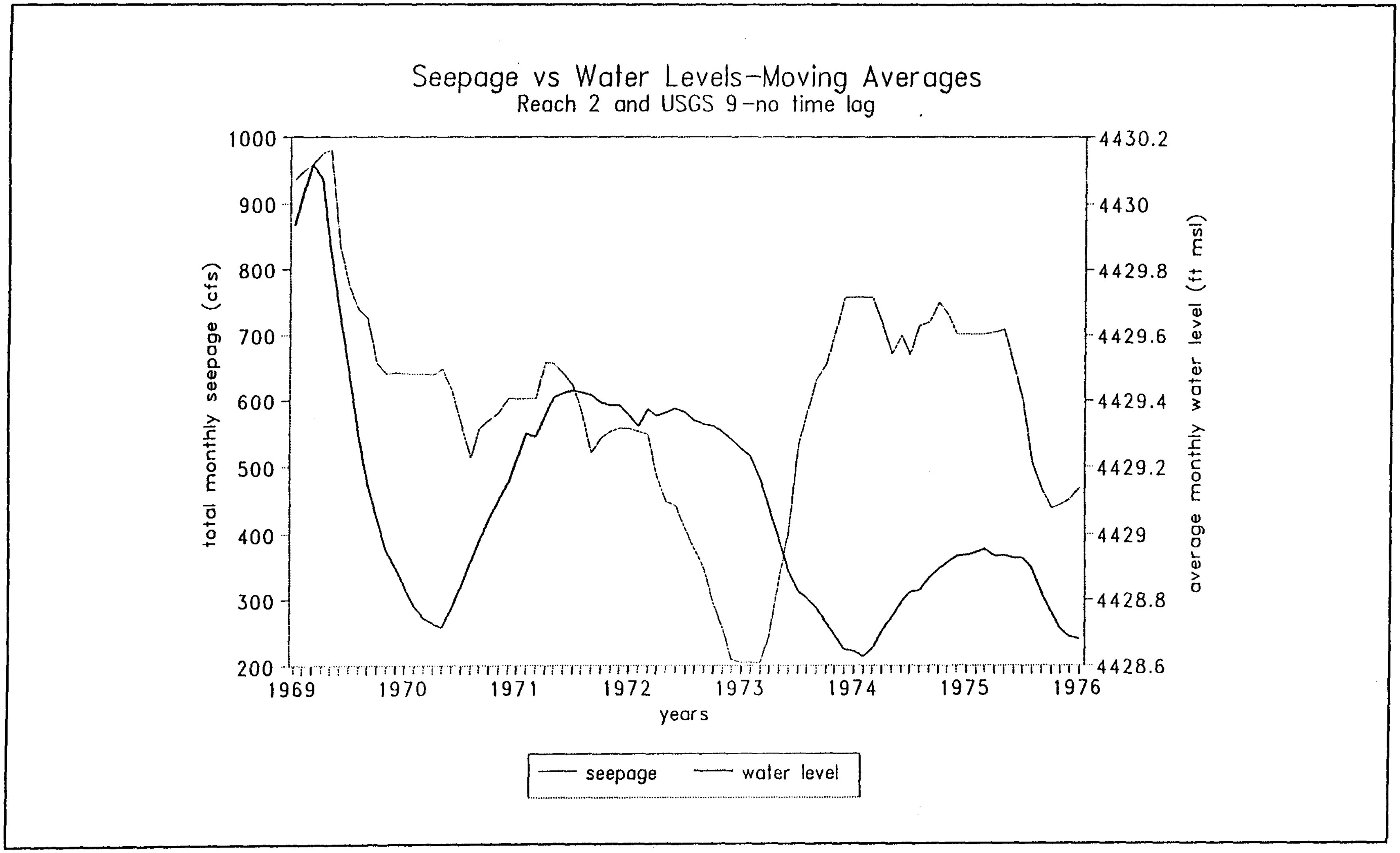

Figure 5.39 - Moving Average Seepage and Water Level Hydrographs, Reach 2 and USGS 9 (1969 to 1976)

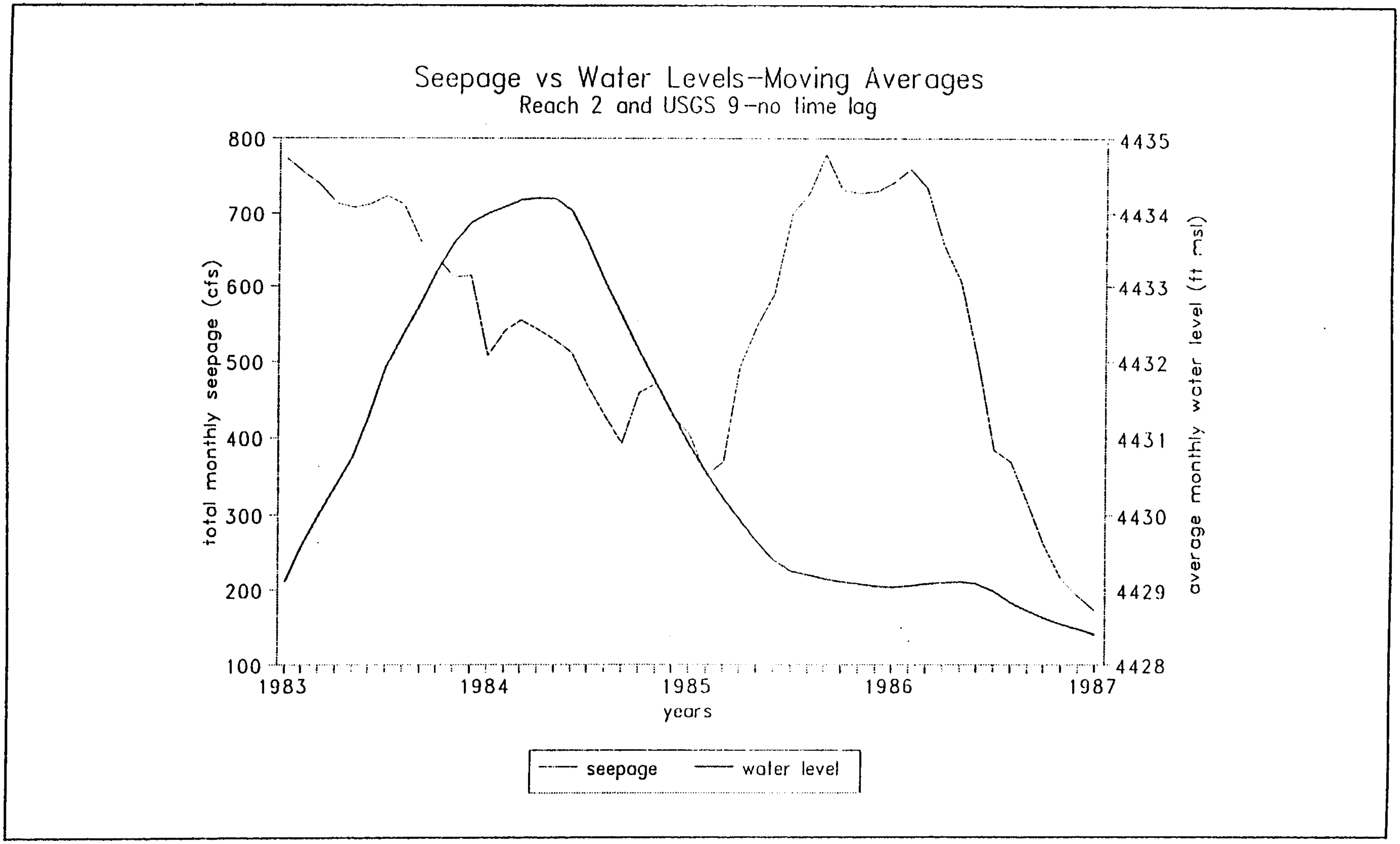

Figure 5.40 - Moving Average Seepage and Water Level Hydrographs, Reach 2 and USGS 9 (1983 to 1987)

REACH 3 AND USGS 18

When the study wells were selected for these analyses, it was believed that the comparison of reach 3 and USGS 18 would adequately represent the system in the northern portion of the site. Using the same procedures previously described, the seepage and well data were averaged, with a 12-month moving average, and these moving average graphs are shown in figures 5.41 and 5.42. Although figure 5.41 depicts some peak-to-peak correlation during the first period of study, 1969-76, the regression analyses for this period yielded only a maximum r^2 of 0.61 for n=7. There was no significant correlation at all during the second period of study, 1983-87 (the maximum r^2 was 0.14 for n=8).

One theory was postulated as to why there was at least a weak correlation during the first period but not the second. Regional groundwater flow is naturally to the southwest, but it has been documented as altering to the northeast under higher river flow conditions. USGS 18 is located at the far northeast end of reach three. The first period of time, 1969-76, was a high flow period, and thus seepage could have impacted the well by changing the groundwater flow direction. The second period, 1983-87, was a below average period, so groundwater flow most likely continued to the southwest and seepage had no impact, with correspondingly low correlation. From this analysis, it was presumed that seepage only affects the northern portion of the site during high flow conditions.

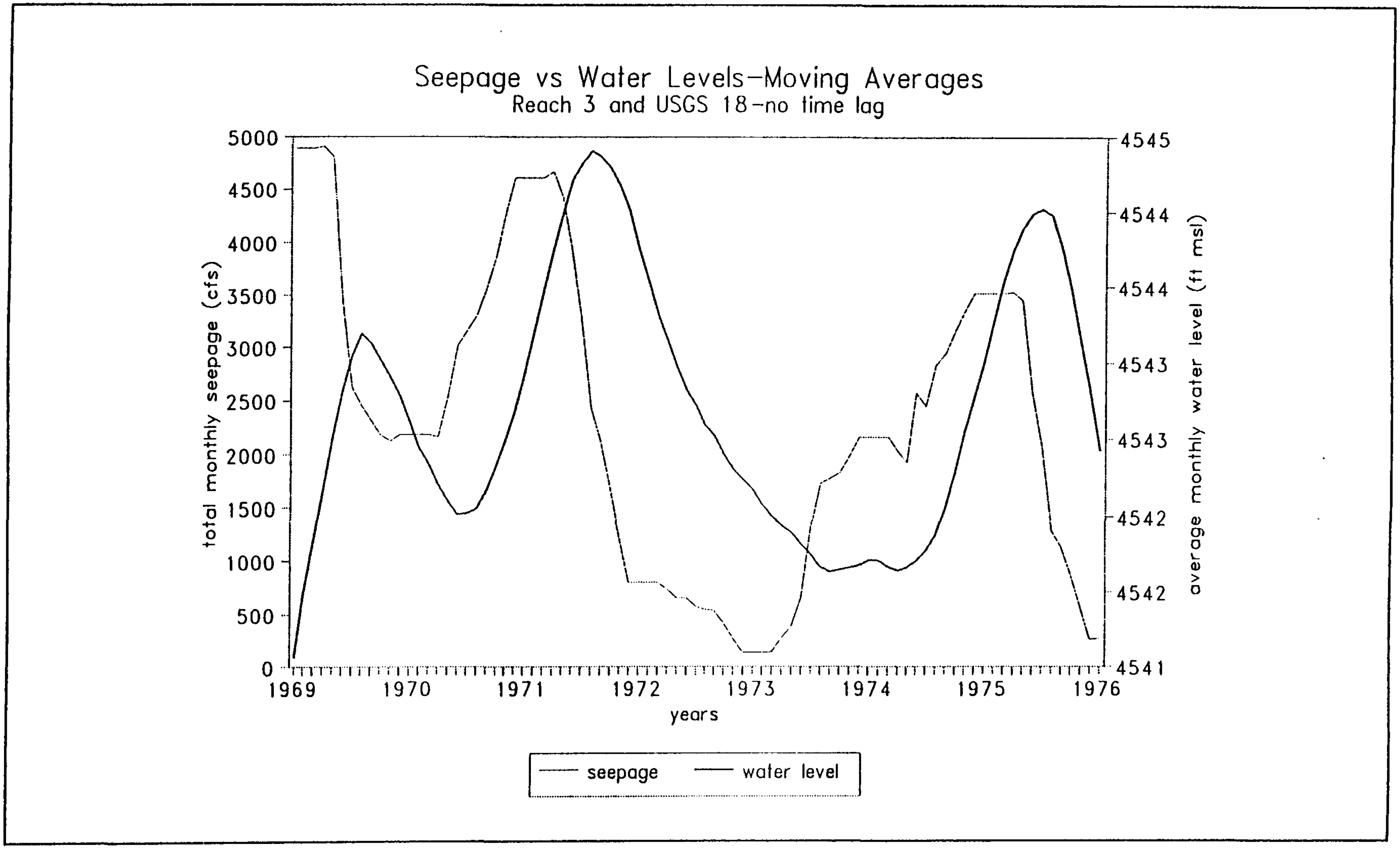

Figure 5.41 - Moving Average Seepage and Water Level Hydrographs, Reach 3 and USGS 18 (1969 to 1976)

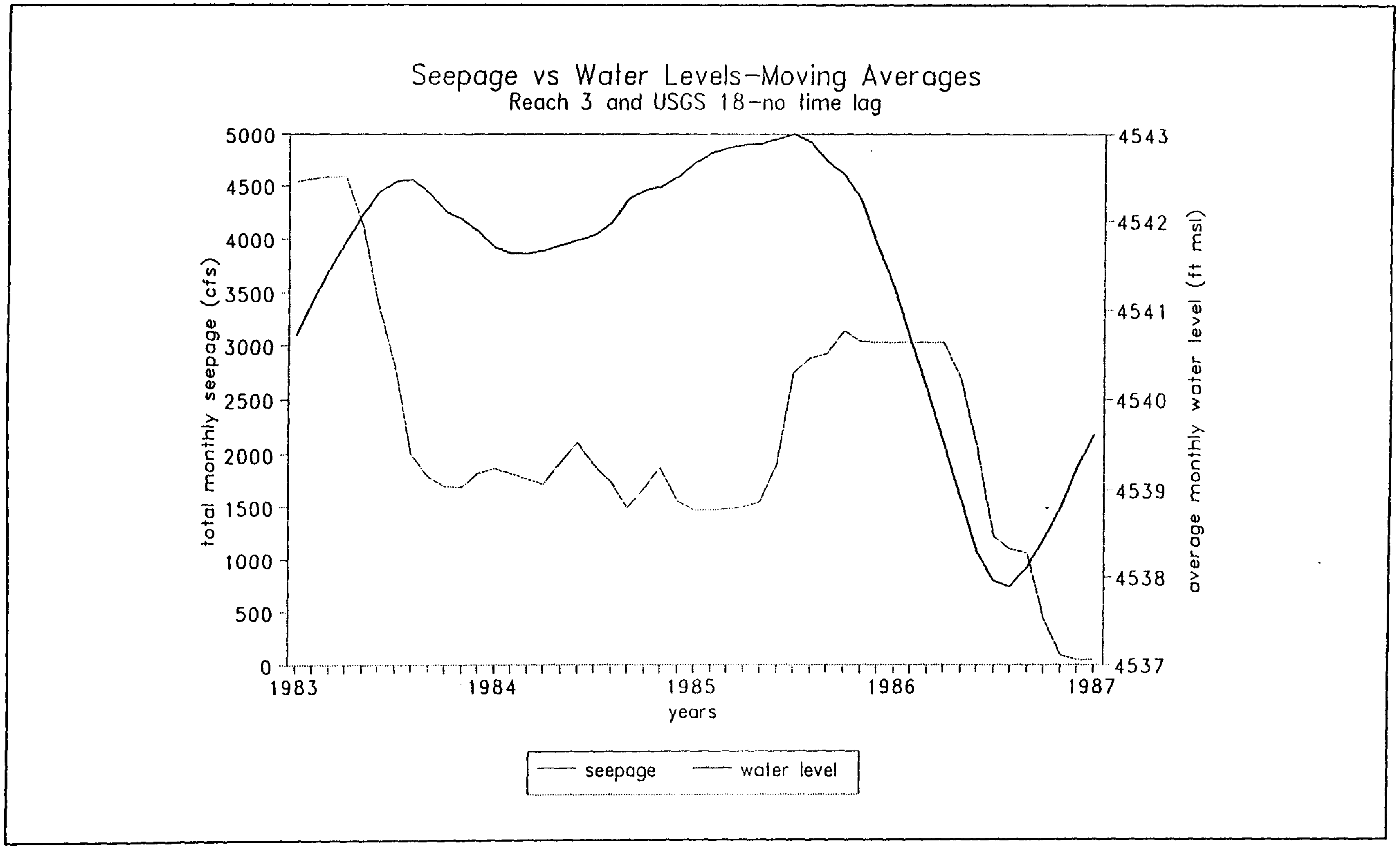

Figure 5.42 - Moving Average Seepage and Water Level Hydrographs, Reach 3 and USGS 9 (1983 to 1987)

Summary

These regression analyses demonstrated that a distinguishable time lag does exist between seepage from the Big Lost River and groundwater levels in the Snake River Plain aquifer. Using moving averaged and raw data, time lags for each reach vs well comparison were determined, and, except for Reach 1 vs USGS 8, there was always a single time lag for both analyses that had the highest correlation coefficient. However, for all analyses the r^2 did not vary too greatly from the highest for different values of n. To justify selecting a single time lag, all comparisons were plotted and visually inspected, and it is believed through that process the conclusions made here are still valid.

Comparison with Previous Work

No direct comparison between this study and others was possible since this was the first study of its kind for the site. However, results could be indirectly compared with Bennett's (1990) to determine if the same conclusions were reached regarding the connection between flow in the Big Lost River and groundwater levels. Nace and Barraclough (1952) did not compare seepage or flow to groundwater levels in any wells, as discussed earlier, and therefore no comparison can be made with their study.

Bennett (1990) noted that for his two study periods (July 1972 to July 1978 and July 1981 to July 1985), the first one experienced a net decline in groundwater levels and the second one a net increase. Although the present study used slightly different study periods, Bennett's respective decreases and increases can be seen on all of the well hydrographs presented here. As has already been studied in this report, and was noted by Bennett also, groundwater levels fluctuate consistently with changes in flow in the river. Bennett summarized his analysis of the correlation between groundwater and surface water by stating that two areas on the INEL site appeared to be most significantly affected by recharge from the river: just north of the Naval Reactors Facility (NRF) and southwest of the RWMC.

The current study again researched the area near the RWMC (USGS 9) and concluded that there is a strong seepage-groundwater level correlation for the area. The well selected for analysis near the NRF (USGS 18) was different than the two Bennett used (USGS 12 and 23), and more to the northeast. Results of the analyses showed some correlation during the first period of study but not during the second, possibly due to the fact that flow was practically nonexistent from 1983-1987 and thus would not affect groundwater. Bennett's second period of study was earlier than this study's and was during high river flow, which would affect groundwater much more. Bennett also noted that prominent groundwater peaks are seen in 1967, 1969, 1983 and 1984, years when very high flows were found in the river. These peaks can also be seen in the hydrographs presented in this report.

Chapter VI

Summary

Seepage from the Big Lost River can cause large fluctuations in both groundwater levels and the direction of groundwater flow in the Snake River Plain aquifer beneath the INEL. In turn, the paths and concentrations of radioactive waste in the groundwater can be altered. Some researchers have investigated seepage rates along the Big Lost River, and others have spent time monitoring groundwater in the Snake River Plain aquifer. There has even been limited research into the relationship between flow in the river and groundwater levels. However, no specific research has examined the correlation between seepage from the river and groundwater.

Research for this thesis initiated the investigation into the relationship between seepage and groundwater levels. Two parts were involved in the study. Seepage losses for both the spreading areas and reaches of the river were studied in part one. Part two then studied the correlation between seepage and groundwater levels in selected USGS wells.

Part one was the most time intensive and difficult part of the research. Seepage estimation techniques had to be researched and appropriate models selected to generate daily seepage data sets. Available data for use in the models were limited, and field measurements could not be made to supplement the data. Therefore, all models had to rely on historical information and the accuracy accompanying it.

Although there was some uncertainty regarding the accuracy of the estimation techniques for both the river and spreading areas, the results were considered reasonable representations of daily seepage. The primary objective in developing a seepage record was for part two of the study, in which the time-patterns of the seepage and groundwater levels were to be correlated. For such correlation studies, the timing of the peaks and troughs is more important than the precise estimations of their amplitudes. Correlation models, such as regression analysis, search for consistent patterns between data sets. The seepage hydrograph patterns obtained from the analyses were believed to adequately represent the seepage process, and therefore would characterize well any correlation that may exist.

Part two of the study was not as difficult or time consuming. The approach was based on standard statistical methods, and it was a matter of applying those methods to determine the strongest correlations between seepage and groundwater levels. Strong correlations were found between seepage and groundwater levels at different locations around the INEL site. The regression analyses did not produce any predictive models, but they did identify an area of research that has implications regarding the future operations at the site.

Conclusions

This study confirmed Bennett's (1990) theory that a strong linear relationship existed between upstream and downstream flow along the Big Lost River on both a monthly and daily basis. It was also determined that a relationship appeared to exist between seepage and average reach flow, although more data collection, research, and analyses would be required to determine the validity of the hypothesis. In estimating seepage, none of the methods applied in the research could be considered entirely accurate, although the inflow-outflow method provided acceptable results. Seepage from the spreading areas was most predictable, but given the limited information available for this study, the accuracy of the results was left in doubt. Nevertheless, the seepage methods applied here served the necessary purpose for the study.

The seepage-groundwater level analysis demonstrated that there is a strong time lag of 5-7 months between seepage events and groundwater level movement. This lag could represent travel time through the unsaturated zone, it could be a pressure response, or, most likely, it could be a combination of the two. However, if it does represent a travel time, then it is much shorter than estimated by site researchers. Correlations were greatest in the spreading areas, primarily due to such large quantities of water sinking in a small geographic area. This is also the area where seepage can most impact site activities. Results also demonstrated that seepage most strongly impacts the areas in the vicinity of and downgradient of the river. The correlation between reach 3 and USGS 18 illustrated that only under extreme flow conditions does seepage impact areas upgradient of the river.

Recommendations

Research for this thesis was the first to examine the correlation between seepage and groundwater levels. It is the hope of those involved in this study that future researchers can use and expand on the results obtained here. At the conclusion of the study thought was given towards methods to improve the results.

Seepage Analysis

Seepage analyses on the river reaches produced acceptable results when the inflow-outflow method was applied, but less so when the empirical estimation technique was required (days with downstream flows in excess of upstream). Review of the study results and methods applied produced two techniques to improve the seepage-average flow correlation.

The correlation between seepage and average flow might have been stronger and more representative if only those days when flow was present at both the upstream and downstream gage had been included in the analysis (the only criterion for this research was flow at the upstream gage). As a further advancement on that theory, the average flows and respective seepage values could be divided into percentile groups (for example, the lower 10 percent of average flows into one group, the second lowest 10 percent into another group, etc.), regressions performed on the separate groups, and regression equations developed (10 in this example). Average flow could then be determined for a reach, and a specific equation applied to calculate seepage.

The equations developed in the seepage-average flow analysis were adequate for completing the seepage record for this study, but no matter the degree of finessing, they do not adequately model all of the physical conditions affecting seepage. Average flow does represent flow depth and velocity, but not other physical factors such as varying stream bank hydraulic conductivities and changing degrees of saturation. One of two methods might be used to improve the river seepage results.

A method described in chapter III, the seepage metering method, would be an acceptable technique to estimate seepage throughout the river reaches. Meters could be installed and monitored at locations along the river to determine average reach seepage rates. Such a task would, however, require many measurements under a number of flow conditions. The method used in this study, the inflow-outflow method, would also be an accurate procedure if additional flow measurements were obtained. It would require increased flow monitoring throughout the reaches, including any point sources of irrigation diversions, return flow, or local inflow. Although this is a good idea in theory, considering that flow data have been limited in recent history, it is the less appealing technique. Both methods would require a large outlay of time and money for further field analyses, resources that were not available for this research. It is debatable whether such an undertaking should be considered. Time and money are limited, the seepage analysis used in this study is not altogether inadequate, and seepage from the spreading areas appears to be more critical.

Increasing the accuracy of seepage estimates in the spreading areas would be an easier task and potentially more productive. Results from this study showed that seepage from the spreading areas does appear to have a much larger impact on the site. Under higher flow conditions, measured flow could be diverted into the basins and the ponding method for estimating seepage applied. A time record of water stages in the basins would also be necessary to accurately measure the seepage rates throughout the year. If conditions did not permit such a study, seepage meter tests could be taken in the field at predetermined locations throughout the basins to obtain average seepage values for each basin. Either of these techniques would considerably improve the seepage estimate approach used in this study.

Seepage-Groundwater Correlation

Future seepage-groundwater analyses should be examined on a smaller time interval basis. Some accuracy was lost by studying the correlation on a monthly basis, and a smaller time basis might provide more accurate results and better identify the time lag. More extensive well records, the limiting parameter in this analysis, may exist, but could not be located or were not available for this research. Assuming such records are not available, it is recommended that further monitoring of wells focus on expanding the monitoring periods to a daily or weekly basis.

Seepage from the spreading areas has a large impact on the groundwater, as was demonstrated with the strong correlation with groundwater levels in well USGS 9. Either another well should be located to correlate better with seepage from the spreading areas, or it is recommended that this well be monitored more frequently. Based on the results obtained in this study, this is an area that requires further study, since seepage could have a large impact on the buried waste at the RWMC.

The correlation between reach one and USGS 8 was also strong, but primarily with the moving seepage averaged data. A smaller time basis might increase the correlation for the raw data. However, it is recommended that another well be located, one on site and closer to site facilities. Since interest in this study is focused on the impact seepage can have on potential waste transport, it makes sense to study a well closer to the areas of concern. USGS 8 was an adequate choice, and the best that could be identified at the time.

Although this study attempted to identify those wells that would directly respond to seepage from the river, USGS well 18 was, in retrospect, not a good selection for correlation with reach two. Previous studies reported that it did correlate with flow in the Big Lost River, and although this may be true, results from this study showed that the correlation was not strong. Future research with reach two should examine other wells, focusing on areas closer to the TRA/ICPP sites. Those areas were initially identified for study in this thesis, but no well with a extensive record fitting the study period could be found. Researchers on site would be in better position to locate and obtain data on such wells.

Some valuable site information was learned in this study, and it is hoped that the results of this study provide more of an understanding about the hydrologic system on the INEL site and furnish future researchers a base from which to continue study.

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